UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

- - -

HYDRAULIC LABORATORY REPORT NO. 1.3

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Technical Memorandum No. 322

- - -

HYDRAULIC MODEL EXPERIMENTS FOR THE DESIGN OF THE HOOVER DAM
BOOK 1
RESULTS OF VISUAL TESTS ON PRELIMINARY SPILLWAY TYPES

By

E. W. LANE, RESEARCH ENGINEER

- - -

Denver, Colorado
Jan. 15, 1933

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UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: HYDRAULIC MODEL EXPERIMENTS FOR THE DESIGN OF THE HOOVER DAM

BOOK 1

RESULTS OF VISUAL TESTS ON PRELIMINARY SPILLWAY TYPES

By E. W. LANE, RESEARCH ENGINEER

Under direction of

J. L. SAVAGE, CHIEF DESIGNING ENGINEER

TECHNICAL MEMORANDUM No. 322

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MEMORANDUM TO CHIEF DESIGNING ENGINEER

By E. W. Lane, Research Engineer

Subject: Hydraulic Model Experiments for the Design of the Hoover Dam - Book 1 - Results of Visual Tests on Preliminary Spillway Types.

The unprecedented height of the Hoover Dam necessitated thorough study of many phases of its design to insure absolutely the safety of the structure. The spillways were one feature which demanded especially careful attention. It was found necessary to provide for a flow of 400,000 second-feet, which with the fall of 500 feet represents an energy of 22,700,000 horsepower, or about seven times the energy of the water passing over the falls at Niagara. Moreover, the velocity of the water would reach 175 feet per second, or two miles a minute, which considerably exceeds that in any similar structure so far constructed. It is obvious, therefore, that the greatest possible care was necessary to insure that these spillways would act exactly as expected, in order that no damage might result.

History of the Spillway Model Tests

To discharge so great a quantity of water over the top of the dam, permitting it to fall down the face to the river below involved great danger to the security of the dam foundations. The large tunnels which were necessary to divert the river during the construction of the foundation of the dam provided a possible outlet for the water and would discharge it back into the river far enough downstream to insure the safety of the dam. Spillway plans were therefore developed utilizing these tunnels. The first drawings of the project using the tunnels included one spillway of the shaft or glory hole type on each side of the river, but this was recognized to be only a preliminary layout. When funds for extensive study of the project became available, all practical forms of spillways were laid out and estimated of them prepared. These included glory holes, side channels with and without gates and various combinations of these forms. Spillways using an entirely separate tunnel system were also planned and estimated. It was evident that extensive model tests would be necessary to secure an economical and safe solution of this problem and these experiments were carried out along with the design and cost estimates. The necessity of providing a spillway form which would discharge without undesirable effects through the long tunnels proved to be unusually exacting, and the spillways as finally evolved represent the result of a process of design, estimation, experimentation and elimination, covering a period of more than two years of intensive study.

The hydraulic model tests were begun at the hydraulic laboratory of the Colorado Agricultural College at Ft. Collins, Colorado. The first model was of the glory hole type on a 1:60

scale designed for a capacity corresponding to 100,000 second-feet in the prototype. As a result of the speeding up of the Hoover Dam Project as an employment relief measure, it was necessary to make a decision on the general type of spillway without the extensive tests on this model as at first contemplated.

The second model was of the side channel type without crest gates also on a 1:60 scale designed for a prototype capacity of 100,000 second-feet at elevation 1234.5 and 140,000 second-feet at the top of the dam. Before the experiments on this model were completed, a decision was made to increase the spillway capacity to 200,000 second-feet. The next was a 1:60 model of the side channel type with a Stoney gate at one end. While experiments on this model were being conducted a large 1:20 scale model of the side channel type with drum gates on the crest was built at a specially constructed laboratory at Montrose, Colorado.

The Stoney gate spillway did not prove satisfactory but tests were conducted on the model of it to throw additional light on certain difficulties which had developed on the large drum gate crest model. From the results of the tests on both these models a new 1:60 drum gate design was prepared and tested at the Colorado Agricultural College. After many changes were made a satisfactory form of spillway was developed and a 1:20 scale model of this form was built at the Montrose laboratory for purposes of comparison.

Experiments were then undertaken to develop the form of spillway crest giving the greatest discharge capacity. This work was carried on at the Colorado Agricultural College laboratory. From these experiments a more efficient form of crest was developed and useful data was obtained for the design of the crest gates. Forms of crest suitable for other forms of gate and for free crests were also developed.

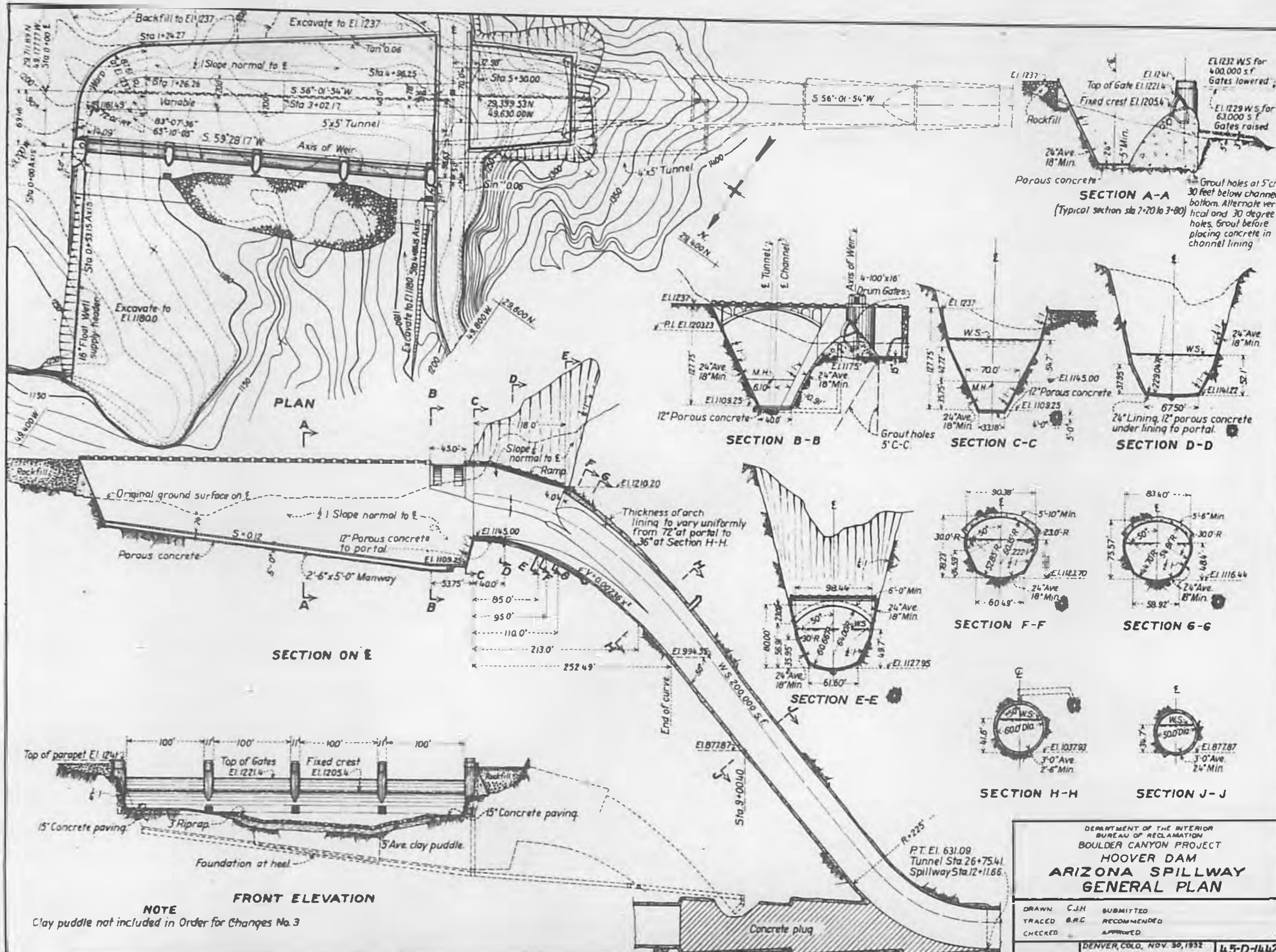
As a result of these studies, the crest of the side channel model previously developed was altered to increase its discharge. Models of this ~~final~~ design were constructed on scales of 1:100, ~~1:60~~ and 1:20; comparisons of the results of these models indicated that substantially similar results would be obtained on the prototype.

The spillway model testing work at the Colorado Agricultural College laboratory was begun in October, 1930, and carried on nearly continuously until April, 1932, a large part of the time using two shifts per day. Tests on the 1:60 and 1:100 models of the final design were carried on intermittently from September, 1932, through January, 1933. The plans for the laboratory at Montrose were started in February, 1931, and it was completed in July of that year. Tests at this laboratory were carried on continuously from July until closed down by freezing weather in December. Work was resumed in August, 1932, and carried on for about four months. Including two small models not previously mentioned, a total of eleven models were tested, many of them with a large number of variations, using linear scales from 1:100 to 1:20 involving discharges of 2 to 112 second-feet.

Throughout the entire series of tests, close contact was maintained with the designing department in order that the plans developed should be sound not only from the hydraulic but also from the structural and construction standpoint.

Results of the Spillway Model Tests

The results of the model tests were highly satisfactory. The form developed is shown by the drawings of the spillways on the Arizona and Nevada sides of the Colorado River shown on Figures 1 and 2 respectively. The location of the spillways with respect to the dam and other features of the development is shown on Figure 3. These spillways amply satisfy the exacting requirements resulting from the unprecedented magnitude of the Hoover Dam, at a reasonable cost and with the assurance of absolute safety. Without these experiments, the same degree of security could only have been obtained by the use of plans costing several million dollars more than those adopted, and the savings made possible by these tests have unquestionably justified the extensive studies which were carried out. The benefit of the tests, however, will not be confined to the Hoover Dam alone. Ideas have been investigated and principles and data have been developed which will be of great benefit in the design of future installations of many types. In order that information may be used on the future spillway problems of the Bureau of Reclamation and



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
ARIZONA SPILLWAY
GENERAL PLAN
DRAWN C.J.H. SUBMITTED
TRACED B.R.C. RECOMMENDED
CHECKED APPROVED
DENVER, COLO., NOV. 30, 1932
45-D-1442
PLAN
Backfill to El. 1237
Excavate to El. 1237
Sta 1+24.27
Sta 1+26.26
Sta 4+96.25
Sta 3+02.17
S. 56°-01'-54" W
S. 59°28'17" W
Axis of Weir
5'x5' Tunnel
4'x5' Tunnel
Sta 5+50.00
29,399.53 N
49,630.00 W
Excavate to El. 1180.0
18" Float Well supply header
Excavate to El. 1180
SECTION ON ℄
Original ground surface on ℄
½:1 Slope normal to ℄
S = 0.12
Porous concrete
12" Porous concrete to portal
El. 1109.25
2'-6"x5'-0" Manway
El. 1145.00
El. 1210.20
Thickness of arch lining to vary uniformly from 72" at portal to 36" at Section H-H.
W.S. 200,000 s.f.
End of curve
El. 994.35
El. 877.87
Sta. 9+00.40
R=225'
P.T. El. 631.09
Tunnel Sta. 26+75.41
Spillway Sta. 12+11.66
Concrete plug
SECTION A-A
(Typical section sta 2+20 to 3+80)
El. 1237
El. 1241
Top of Gate El. 1221.4
Fixed crest El. 1205.4
El. 1232 W.S. for 400,000 s.f. Gates lowered
El. 1229 W.S. for 63,000 s.f. Gates raised
Rockfill
24" Ave. 18" Min.
Porous concrete
Grout holes at 5' crs. 30 feet below channel bottom. Alternate vertical and 30 degree holes. Grout before placing concrete in channel lining
SECTION B-B
4-100'x16' Drum Gates
P.I. El. 1203.23
El. 1109.25
12" Porous concrete
Grout holes 5' C-C
SECTION C-C
El. 1145.00
12" Porous concrete
El. 1108.25
SECTION D-D
24" Lining, 12" porous concrete under lining to portal.
El. 1141.22
SECTION E-E
El. 1127.95
SECTION F-F
El. 1123.70
SECTION G-G
El. 1116.44
SECTION H-H
El. 1037.93
60.0' Dia.
3'-0" Ave. 2'-6" Min.
SECTION J-J
50.0' Dia.
El. 877.87
3'-0" Ave. 24" Min.
FRONT ELEVATION
Top of parapet El. 1241
Top of Gates El. 1221.4
Fixed crest El. 1205.4
Rockfill
15" Concrete paving
3' Riprap
5' Ave. clay puddle
Foundation at heel
NOTE
Clay puddle not included in Order for Changes No. 3

FIG. 1

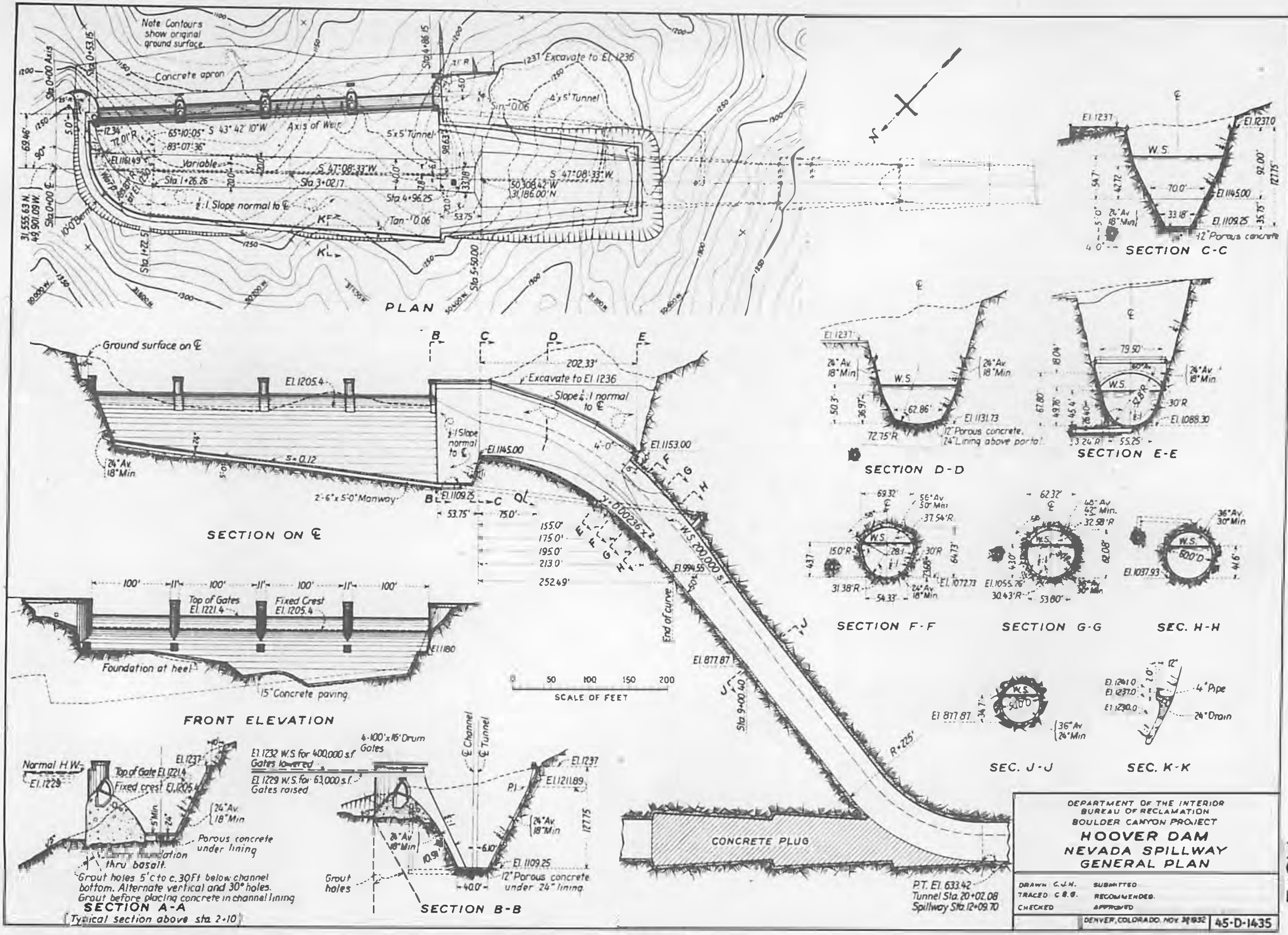
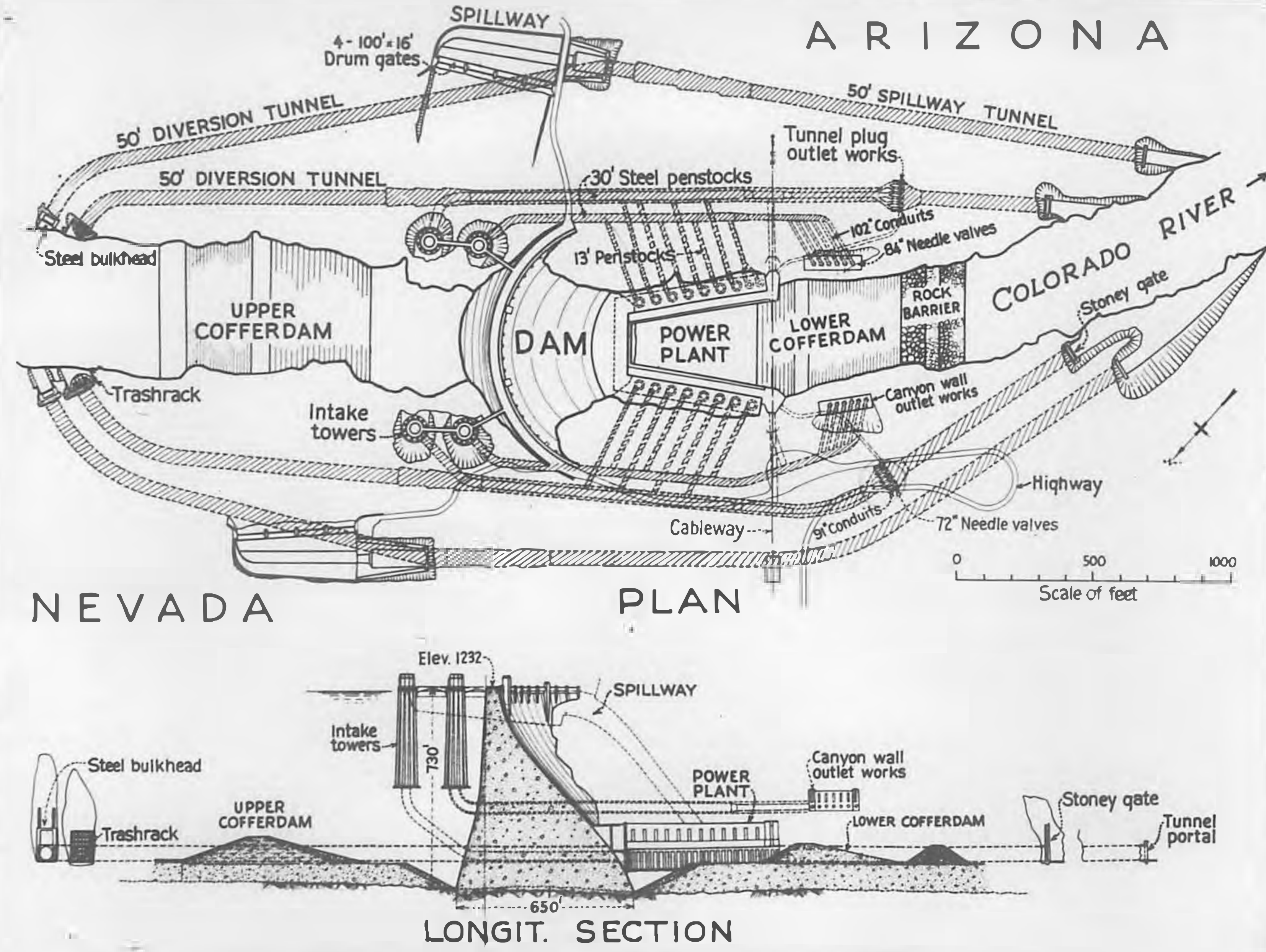

PLAN
SECTION ON ℄
FRONT ELEVATION
SECTION A-A
Typical section above sta. 2+10
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
SEC. H-H
SEC. J-J
SEC. K-K
CONCRETE PLUG
SCALE OF FEET
Note Contours show original ground surface.
Concrete apron
Axis of Weir
Excavate to El. 1236
4'x5' Tunnel
5'x5' Tunnel
Ground surface on ℄
Foundation at heel
15" Concrete paving
Top of Gates El. 1221.4
Fixed Crest El. 1205.4
2'-6"x5'-0" Manway
End of curve
Normal H.W. El. 1229
Porous concrete under lining
Grout holes 5' c to c. 30 Ft below channel bottom. Alternate vertical and 30° holes. Grout before placing concrete in channel lining
4-100'x16' Drum Gates
El. 1232 W.S. for 400,000 s.f. Gates lowered
El. 1229 W.S. for 63,000 s.f. Gates raised
12" Porous concrete under 24" lining
P.T. El. 633.42
Tunnel Sta. 20+02.08
Spillway Sta. 12+09.70
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
NEVADA SPILLWAY
GENERAL PLAN
DRAWN C.J.H.
TRACED C.B.B.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO, NOV. 30, 1932
45-D-1435

FIG. 2



HOOVER DAM AND APPURTENANT WORKS

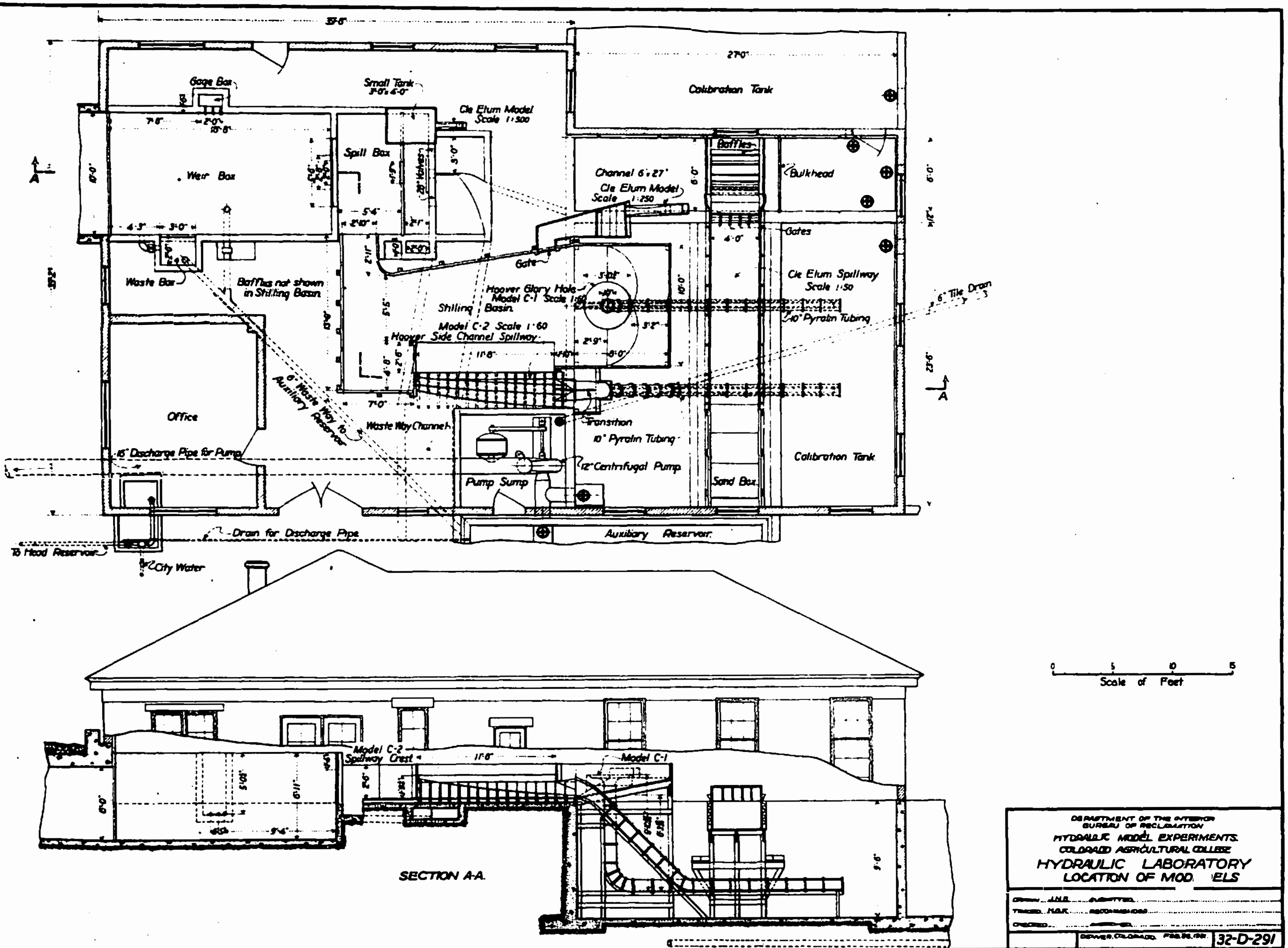
FIG. 3

other engineering agencies, the following report of the tests has been prepared.

Hydraulic Laboratory of the Colorado Agricultural College

By the generous permission of the Colorado Agricultural College, the small scale experiments for the design of Hoover Dam spillways were carried out in their hydraulic laboratory (Plate 1-A) which has been described in detail in the Engineering News, page 602, Vol. 70, October 2, 1913. The facilities of the laboratory were adequate for tests of models of Hoover spillway as large as 1:60 scale. The permission to use the laboratory and the hearty cooperation of the college staff and of the staff of the U. S. Bureau of Agricultural Engineering, who ordinarily use the laboratory, has been greatly appreciated.

Figure 4 is a drawing of the laboratory showing the location of the models first constructed. The flow was obtained from a reservoir of 30,000 cubic feet capacity located upon a hill behind the laboratory (Plate 1-B). The flow out of the reservoir was controlled by hand-operated gates. From these gates the discharge passed into a weir box 19.5 feet long, 10 feet wide, and 7 feet deep. In the side of this box 13 feet upstream from the weir was a bypass which was controlled by a movable crest, and another of smaller discharge controlled by a valve. Fine adjustments of the quantity discharged through the model were made by varying the flow through these bypasses. The head on the weir was observed by means of a float gage

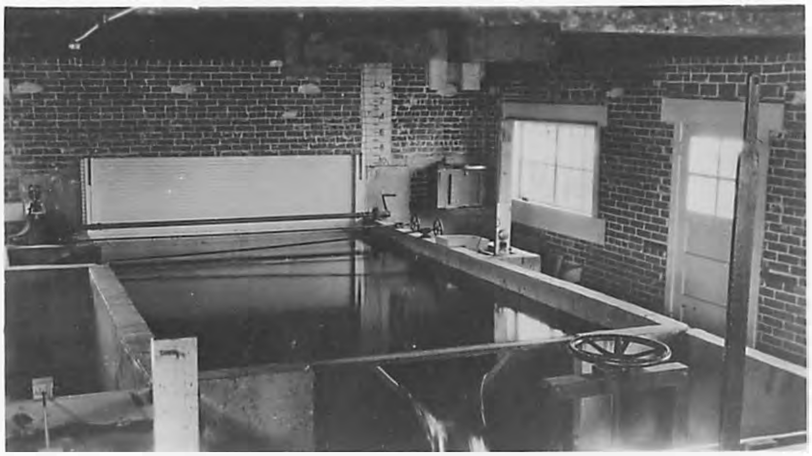


DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HYDRAULIC MODEL EXPERIMENTS
COLORADO AGRICULTURAL COLLEGE
HYDRAULIC LABORATORY
LOCATION OF MOD ELS
DRAWN J.N.B. SUBMITTED
TRACED H.A.K. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO FEB. 26, 1931 32-D-291

FIG. 4

A-THE LABORATORY

B-THE SUPPLY RESERVOIR FOR
THE LABORATORY



C THE WEIR BOX AND MEASURING WEIR

THE HYDRAULIC LABORATORY OF
THE COLORADO AGRICULTURAL COLLEGE

similar to that developed at the Cornell University.* The gage was

*Trans., Am. Soc., C. E., p. 1154, Vol. 83, 1920.

located in a stilling-pool connected with a main channel by a pipe, as shown in Figure 4.

The discharges for all of the tests except the series to determine the best crest shape were measured by means of either a 90° V-notch weir (Plate 1-C) or a 2' Cipolletti weir, both of which had been volumetrically calibrated. For the experiments on the drum gate model, the crest of the Cipolletti weir was somewhat submerged. To determine the discharge for this condition an extensive set of volumetric calibration tests was also made. The location of the glory hole and first side channel models are shown on Figure 4.

The Montrose Laboratory

Because of the tremendous power of the water to be handled by these spillways it was believed to be necessary to take every reasonable precaution to insure that they would function in the manner desired. It was therefore decided to build a model on as large a scale as was reasonably possible, in order to indicate any discrepancies between the action of the model and prototype which might arise from the smaller size of the model. A model scale of 1:20 was decided upon for these tests. This required a discharge of 112 second-feet and a fall within the model of approximately 30 feet. As no laboratory was available which provided satisfactory facilities it was decided to build one on the Uncompahgre Irrigation Project at Montrose in southwestern Colorado. At this point a drop in the South Canal provided a fall of approximately 50 feet and a flow of 200 second-feet or more throughout the irrigation season.

Figure 5 is a drawing showing the layout of the laboratory constructed at this point and Plates 2-A and B show the appearance when completed. The water was taken out of the ditch above the chute through two 48" circular gates shown on Plates 2-C and 3-A. A check controlled by 4"x4" needles (Plates 3-B and C) was installed below the gates to raise the water in the ditch sufficiently in times of low flow to permit the diversion of the desired quantity through the laboratory. From the intake gates the water passed through the two barrels of the intake gate structure into the upper end of the weir channel. This channel was of wood frame construction, 12 feet wide with walls 8 feet high. The total length of this channel upstream from the weir was 80 feet. At the upper end were located three sets of baffles to quiet the water approaching the weir (Plate 4-A). These baffles were built with panels of cross slats. The spacing of the slats was varied as a result of experiments until a very uniform distribution of the velocities was obtained in the channel approaching the weir.

At the lower end of the wooden channel was located the weir box and measuring weir (Figure 5). In order that the elevation

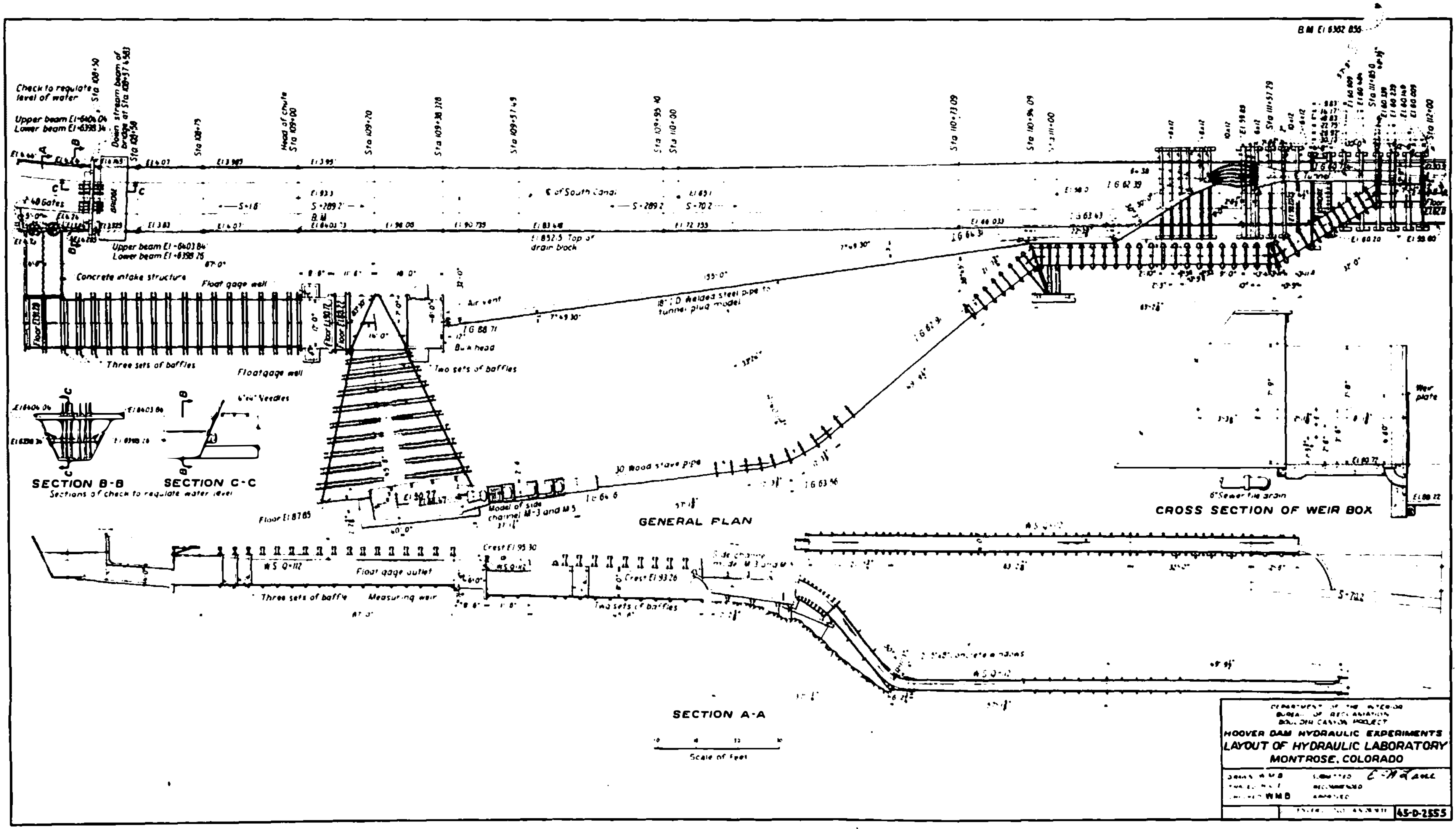
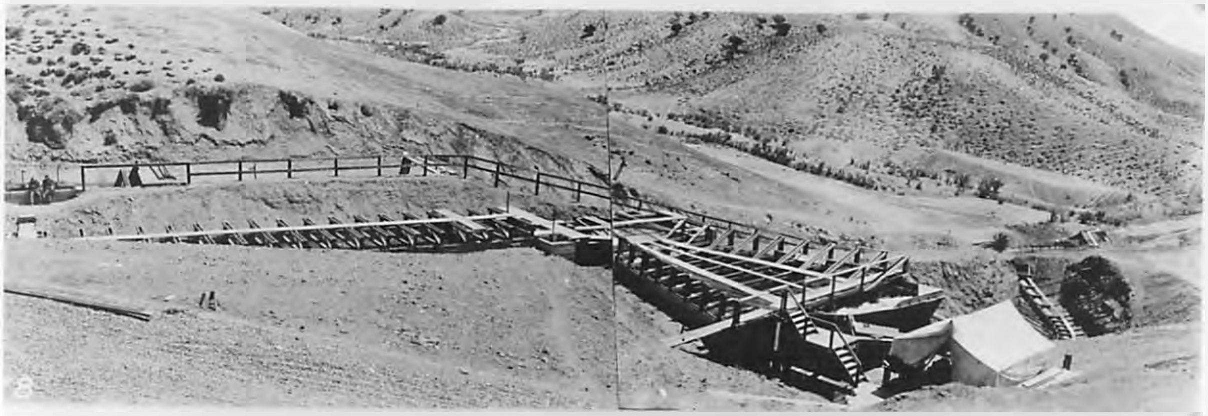

Check to regulate level of water
Upper beam El 6404.04
Lower beam El 6398.34
Down stream beam of bridge at Sta 109+57.6583
Head of chute Sta 109+00
Concrete intake structure
Float gage well
Three sets of baffles
Floatgage well
Two sets of baffles
Air vent
Bulk head
18" I.D. Welded steel pipe to tunnel plug model
30" Wood stave pipe
℄ of South Canal
Upper beam El 6403.84
Lower beam El 6398.75
El 852.5 Top of drain block
Model of side channel M-3 and M-5
Floor El 87.65
SECTION B-B
Sections of check to regulate water level
SECTION C-C
GENERAL PLAN
CROSS SECTION OF WEIR BOX
Weir plate
6" Sewer tile drain
B.M. El 6302.856
Three sets of baffle
Measuring weir
Float gage outlet
Two sets of baffles
Crest El 95.30
Crest El 93.26
SECTION A-A
Scale of Feet
HOOVER DAM HYDRAULIC EXPERIMENTS
LAYOUT OF HYDRAULIC LABORATORY
MONTROSE, COLORADO
45-D-2555

FIG. 5



A-PANORAMA OF MONTROSE LABORATORY

B- ANOTHER VIEW OF MONTROSE LABORATORY

C-HEADGATES OF MONTROSE LABORATORY

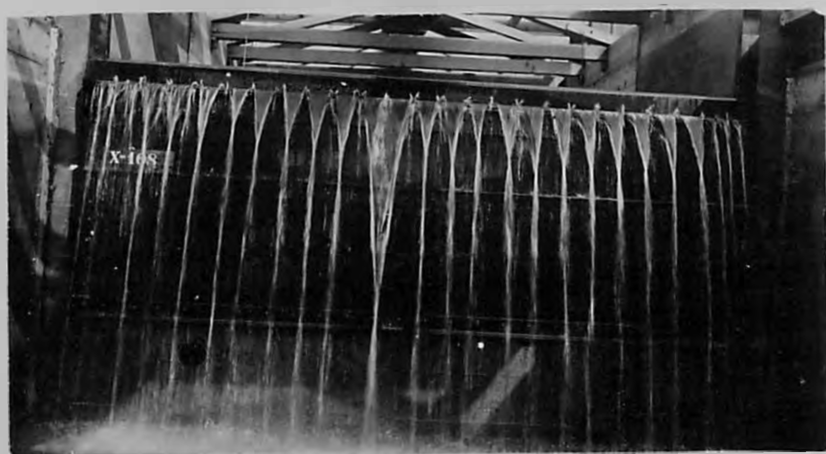
VIEWS OF MONTROSE LABORATORY

A-HEADGATES OF MONTROSE LABORATORY

B-CHECK IN DITCH, WATER FLOWING

C-CHECK IN DITCH, NO WATER FLOWING

A- WEIR CHANNEL BAFFLES



B-DOWNSTREAM SIDE OF MEASURING WEIR

C-WATER DISCHARGING OVER MEASURING WEIR

VIEWS OF MONTROSE LABORATORY

of the weir crest might not be subject to change, the weir box was constructed of concrete and rested on the shale bedrock. The side walls and floor were smooth. Two drains extended through the floor of the weir box by means of which the weir tank could be drained. The weir was a duplicate of the Francis weir with suppressed contractions, except that the length was 12 feet instead of the 10 feet which Francis used. The crest was 4.6 feet above the floor and the walls were offset downstream from the crest to permit aeration of the nappe. The weir plate was built up of steel plate and structural steel shapes (Plate 4-B). The crest was formed of a steel angle with horizontal crest 1/4" wide. Care was taken to keep the upstream face of the weir smooth in the vicinity of the crest and to keep the upstream corner true and sharp. Very smooth conditions of flow over the weir were obtained (Plate 4-C). The head on the weir was observed on float gages of the Cornell type set in concrete pits on either side of the weir box (Plate 5-A). The scales were held in metal frames attached to the concrete in order that the zeros might not be changed by swelling of the wood. The float wells were connected through a short section of rubber garden hose to piezometer openings in the weir box walls, which opened 6 feet upstream and 1.0 feet below the weir crest, as on the Francis weir.

After passing over the weir the water turned at right angles, passed through another set of baffles (Plate 5-B) into an expanding forebay at the end of which was located the spillway model. The

A-FLOAT GAGE FOR INDICATING HEAD ON WEIR

B-FOREBAY BAFFLES

C-WATER FROM LABORATORY RETURNING TO CANAL

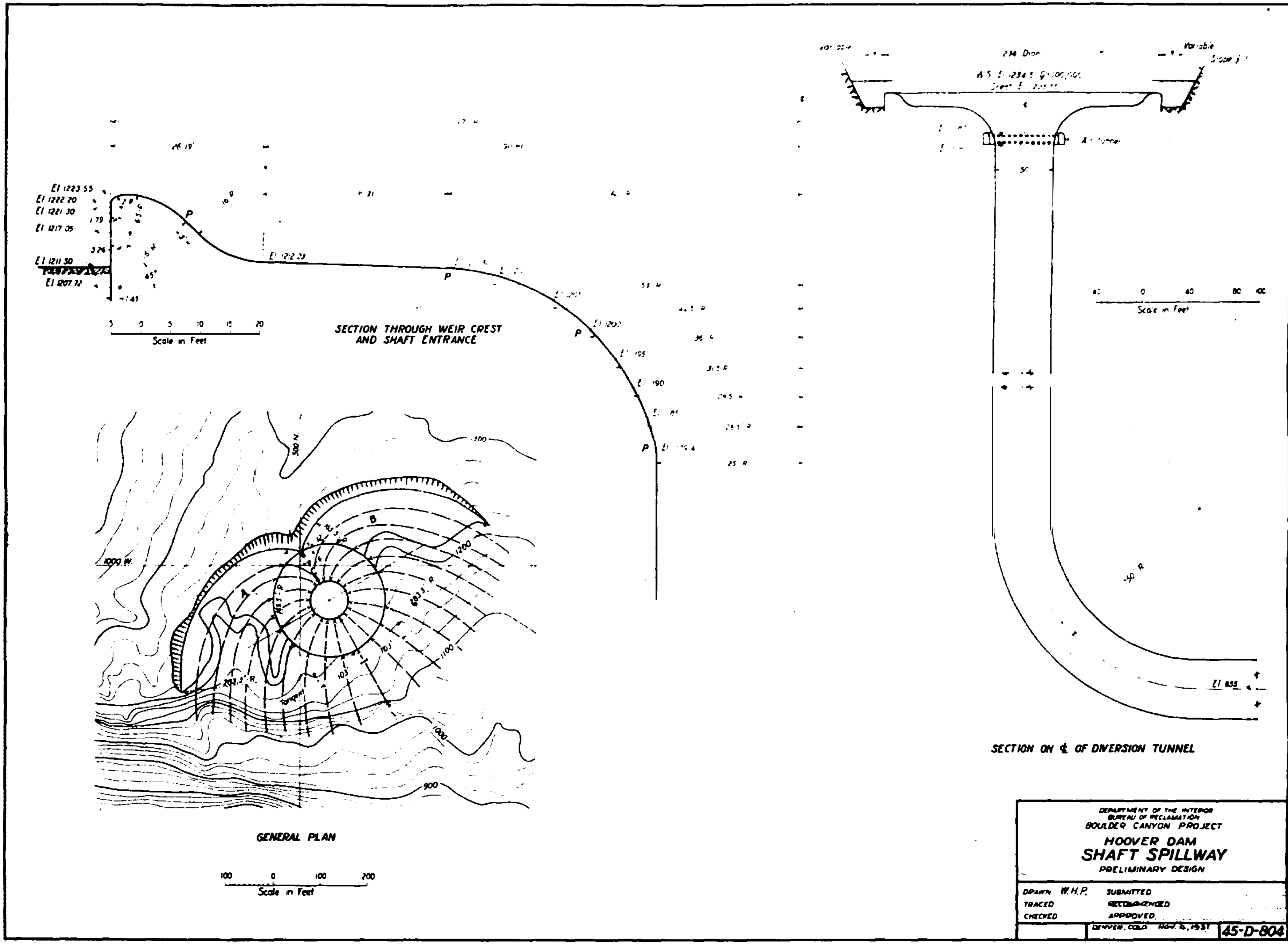
VIEWS OF MONTROSE LABORATORY

peculiar shape of the laboratory was dictated by the necessity of having the water approach the model at right angles to the spillway crest. After passing through the spillway and tunnel model, the water was discharged into a rectangular wooden flume which empties back into the irrigation canal (Plate 5-C).

THE GLORY HOLE SPILLWAY EXPERIMENTS - MODEL C-1

The preliminary drawings of the Hoover Dam were made with spillways of the glory hole or shaft type, but this was merely a tentative design, as it was expected to make extensive model experiments and base the selection on the results obtained from them. The design of this type of spillway contemplated a discharge of 100,000 second-foot for each shaft, with the water surface in the reservoir at El. 1234.5, half-way up the parapet wall or 2-1/2 feet above the top of the dam. In order to meet the condition specified for the Hoover Dam Project, that the discharge from a flood as large as that in 1884 should be carried safely in the river channel below the dam, the two spillways had to limit the discharge to 62,500 second-feet (not including power house discharge) at El. 1229.

The spillway designed to meet these two requirements with the prototype dimensions is shown on Figure 6. It consisted of a circular crest 234 feet in diameter of ogee cross section below which was a morning glory shaped funnel leading to the 50-foot diameter shaft. The purpose of the ogee crest was to produce a high discharge



El 1223.55
El 1222.20
El 1221.30
El 1217.05
El 1211.50
El 1207.72
El 1212.23
P
Scale in Feet
SECTION THROUGH WEIR CREST AND SHAFT ENTRANCE
GENERAL PLAN
Scale in Feet
A
B
1000 W.
1200
1100
1000
900
Tangent
Variable
Variable
Slope 1/4:1
234 Diam.
Air Tunnel
Scale in Feet
El 855
SECTION ON ℄ OF DIVERSION TUNNEL
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
SHAFT SPILLWAY
PRELIMINARY DESIGN
DRAWN W.H.P.
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO.
45-D-804

FIG. 6

coefficient, in order to give a large discharge per foot of crest length, as this would enable a shorter crest to be used, resulting in a smaller, cheaper spillway.

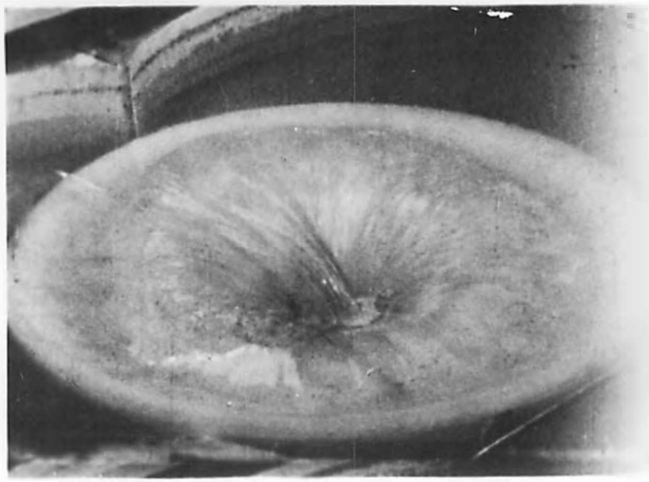
The model of this spillway was tested in the hydraulic laboratory of the Colorado Agricultural College at Ft. Collins, on a scale of 1:60 or 1" = 5'. The relation of the quantities in the model to those in the prototype for a 1:60 scale are given in Table I.

The model of the glory hole for convenience was designated as the C-1 model, the C indicating Collins, and the 1 that it was the first model tested there. Thus M-4 indicates the fourth model tested at Montrose. As the tests described in this report were carried on with models of several different scales, each of which had different discharge and other relations to the prototype, a great deal of confusion will be avoided if the corresponding dimensions and discharges of the prototype are used rather than the actual quantities observed on the model. This will enable comparisons to be made between the performance of models of different scales without the necessity of reducing the values on one to the corresponding values on the other. Moreover, it will enable one to visualize and appreciate the true significance of the effects as they would appear in the actual spillway in a way that the expressions in terms of the actual model size could not do without a great deal of computation. Throughout this report, therefore, all quantities such as lengths, elevations and discharges, will, unless otherwise stated, be given in terms of the corresponding quantities on the prototype rather than the actual quantities observed on the model.

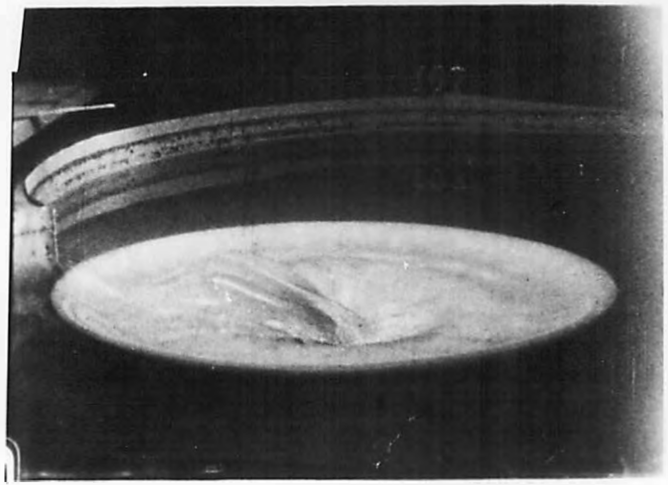
The discharge through the model was measured on a 2-foot Cipolletti weir. The water was led to the spillway through a series of baffles, which produced still water upstream from the structure. The crest section of the spillway was built of paraffin. The paraffin block was first cast approximately to the shape desired and then cut accurately to dimensions by a template in the form of a knife or screed which was revolved about an axis in the center of the spillway shaft as shown on Plate 6-A. The completed crest section is shown on Plate 6-B. Although paraffin made a rather soft model, easily scarred or nicked, it was easily patched and for this type of model is believed to have been as easy to build and satisfactory in its operation as could have been secured with any other material. Some checking occurred, but this could be easily remedied, the only permanent change being to the appearance. The vertical tunnel, vertical bend and horizontal tunnel were formed of transparent pyralin, a material similar to celluloid, in order that the action of the water within them could be observed. The various sections of the bend were made of pyralin sheets 1/8 inch thick, formed in a hydraulic press. The straight sections were about 18 inches long, flanged at the ends and with a longitudinal butt joint with outside strap. The 90-degree vertical bend was made up of four $22\frac{1}{2}$-degree flanged bends, each bend being divided into two similar flanged sections on the plane

A-CONSTRUCTION OF PARAFFIN CREST OF GLORY HOLE MODEL

B-COMPLETED CREST OF GLORY HOLE MODEL



C-GLORY HOLE DISCHARGING HALF CAPACITY



D-GLORY HOLE DISCHARGING FULL CAPACITY

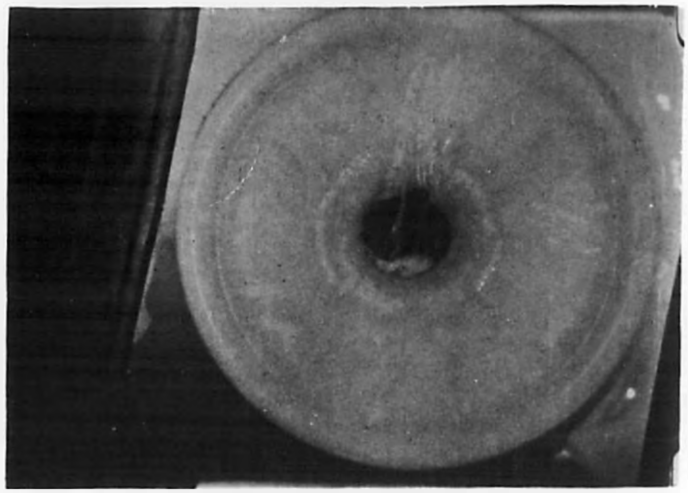
passing through the center line of the bend. The 90-degree bend was made in sections in order to facilitate construction and to enable experiments to be made with $22\frac{1}{2}$-degree, 45-degree, or $67\frac{1}{2}$-degree bends also.

Results of Tests of Glory Hole Model

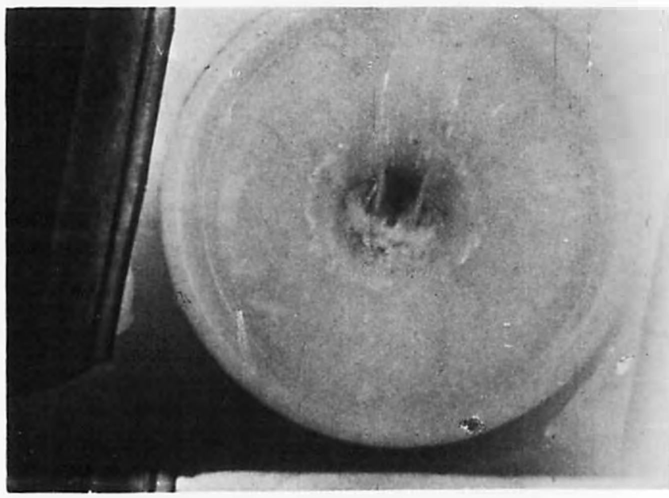
As a result of the speeding up of the Hoover Dam Project as an employment relief measure, it was necessary to make a decision on the general type of spillway without the extensive tests at first contemplated. In order not to delay work on the side channel models, experiments on the glory hole type consisted only of visual observations and photographs.

In the design of this form of spillway it was assumed that the water would flow over the crest and toward the spillway shaft in a radial direction, which would make equal depths at all points equally distant from the axis of the shaft. This would produce a smooth condition of flow in the crest section and down the vertical shaft. The model showed, however, that this condition would not be obtained in the spillway. A distinct concentration of flow occurred along the radial line from the center to the point at the back side of the spillway formed by the ends of the two approach channels. This concentration took the form of a ridge, across the intake section as shown on Plate 6, C and D. As a result, the water did not flow down the shaft equally distributed around the walls, but in a concentrated stream that jumped across the top of the shaft from the canyon wall to the lake side as shown on Plate 7, and dropped down the shaft with a very irregular distribution. For very large flows this ridge or jet tended to seal the top of the shaft and cause a suction and pulsation which was very undesirable. This action passed through a cycle of three stages as shown on Plate 8. In the first stage, view A, the stream flowing across the top of the shaft sealed the top and a suction resulted which drew the water down into the shaft as shown by view B. When the vacuum in the shaft was broken by the entrance of air from above, a distinct piling up of the water occurred over the top of the shaft as shown in view C. This was followed by the condition shown in view A and a repetition of the cycle.

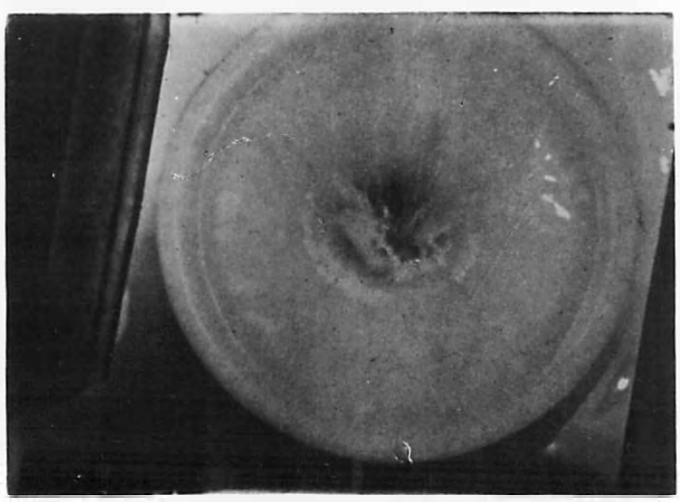
Studies were undertaken to obtain a more equal distribution of flow in the inlet section of the spillway. The concentration of flow was found to be due to the fact that the water did not enter the intake section in a radial direction. To bring the cost of the spillway to a reasonable figure, it was necessary to set the structure partially in an excavation in the side of the canyon and about half of the water passing over the crest is carried to the crest in the excavated channels A and B, in which the direction of flow is not at right angles to the crest but follows the paths shown in Figure 6. The water approaching the crest, therefore, has a component in the direction of the crest, and did not flow over it radially, but took an inclined direction as shown by the stream lines of the figure.



A-DISCHARGING ONE-QUARTER CAPACITY



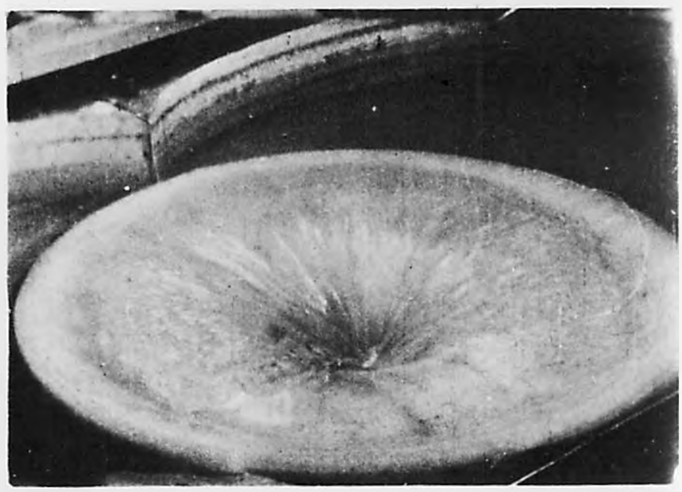
B-DISCHARGING ONE-HALF CAPACITY



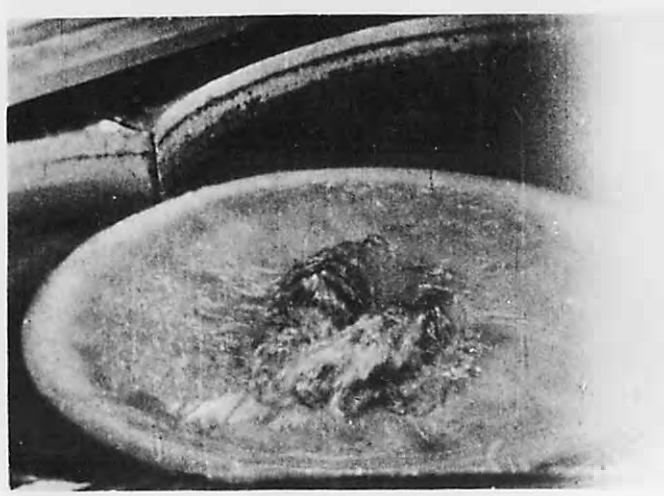
C-DISCHARGING FULL CAPACITY

LOOKING DOWN ON GLORY HOLE CREST- MODEL C-1

A-NORMAL CONDITION



B-SUCTION FROM SHAFT-ACTING



C-SUCTION BROKEN

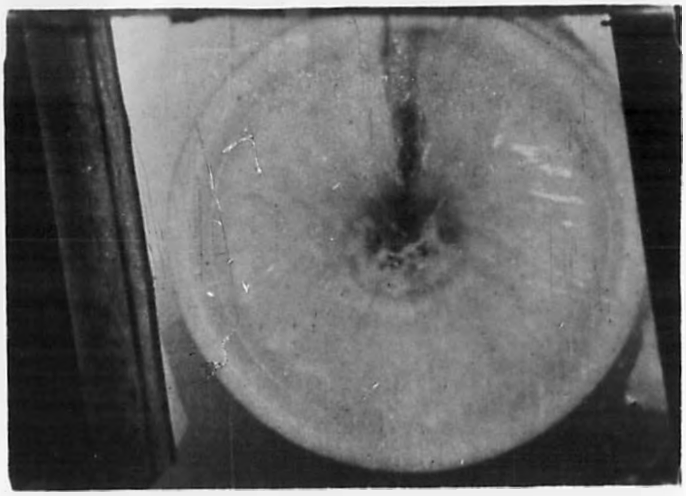
THREE PHASES OF ACTION OF GLORY HOLE
WITH DISCHARGE ABOVE DESIGNED CAPACITY-MODEL C-1

As the water approached the center of the morning glory shaped section, more of it tended to concentrate at C and enter the shaft on that side, causing a disturbed condition of flow at that point. The directions of these currents were determined by inserting coloring matter into the water as shown on Plate 9. View A shows the direction of the stream along the ridge of concentration leading from the ends of the two approach channels. This was radial, as shown by the black streak representing the path of the colored water, since the flow from the two approach channels which were inclined in opposite directions and balanced each other. At 90° to this line the flow was not radial, but was nearly tangent to the edge of the shaft as shown on view B. It was the concentration of these non-radial flows which forms the ridge.

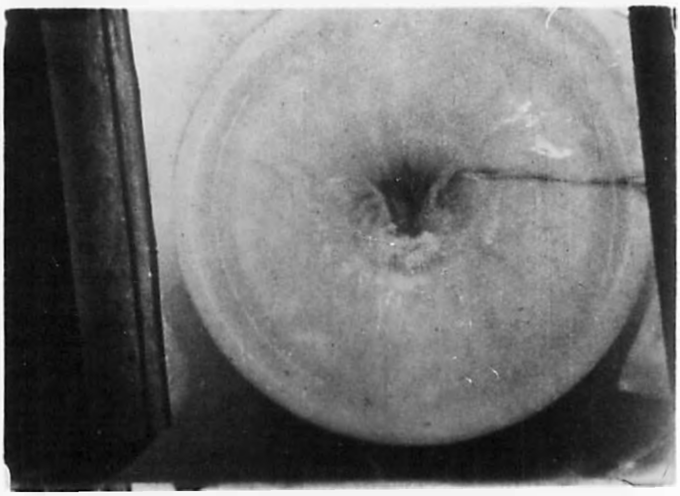
It is believed that a more even distribution of the flow entering the spillway shaft could have been secured with radial piers on the crest and on the morning glory shaped section, similar to those used on the Davis Bridge spillway.* It was intended to test this out,

*The Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam, and Hydraulic Test on Working Models, by Ford Kurtz, Trans., A. S., C. E., p. 1, Vol. 88, 1925.

and a somewhat similar set in the approach channel, as shown on Plate 9-C were tried out, with somewhat favorable results. However, the effect of the impact of the water at the bottom of the vertical shaft would obviously be so much more severe even with a perfect distribution



A- DIRECTION OF STREAM FROM ENDS OF APPROACH CHANNEL



B-DIRECTION OF STREAM 90° FROM ENDS OF APPROACH CHANNELS

C-FLOW CONDITIONS WITH GUIDE VANES

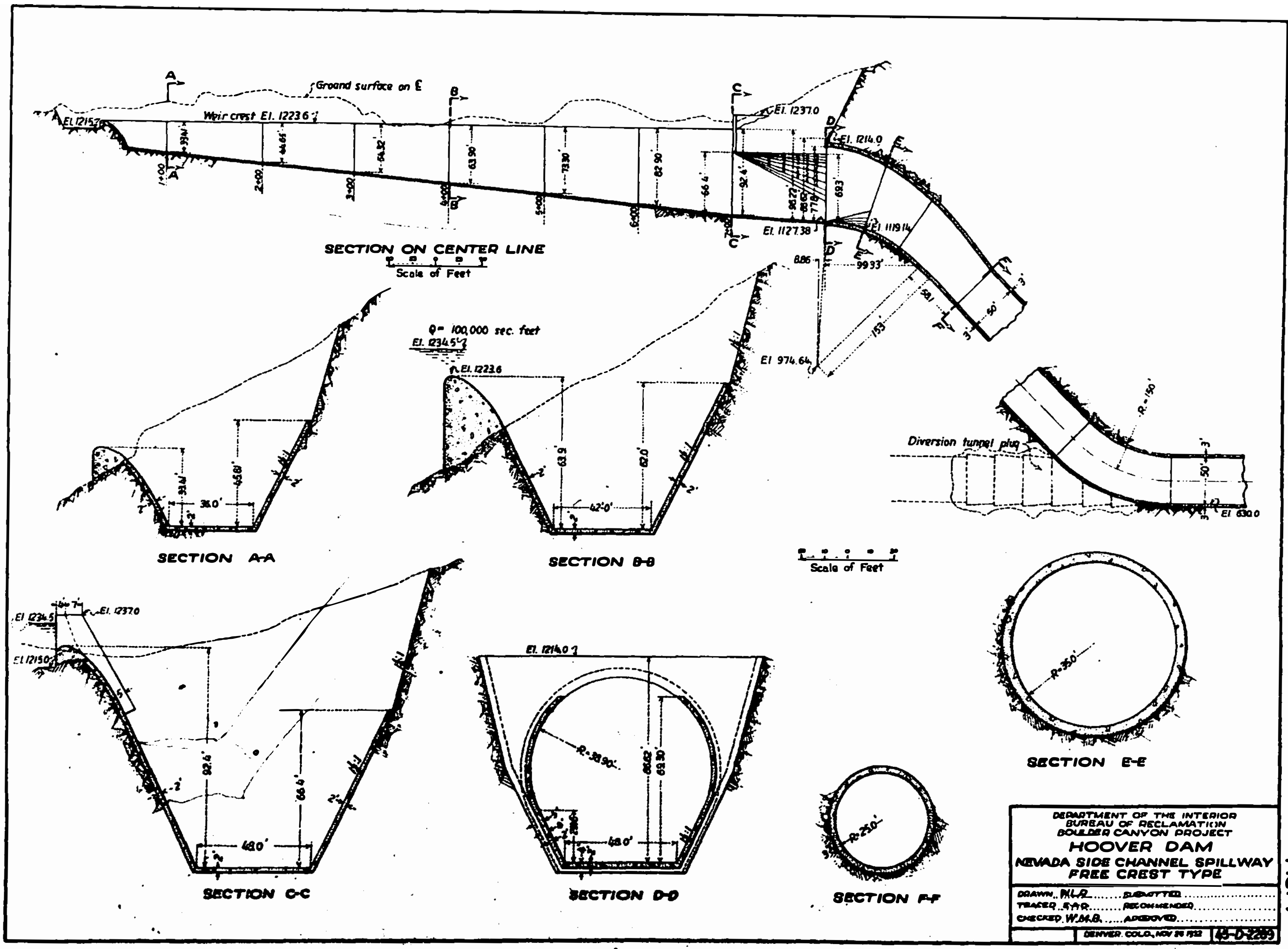
GLORY HOLE OR SHAFT SPILLWAY - MODEL C-1 of flow than after sliding down the inclined shaft from the side channel; with the unprecedented velocities resulting from the 600-foot drop at the Hoover Dam it was believed to be safer to use the side channel type, and as an immediate decision on the work on the glory hole general type was necessary in order not to delay the work, the side channel type was adopted and the model discontinued.

SIDE CHANNEL SPILLWAY EXPERIMENTS - FREE CREST TYPE - MODEL C-2

The first side channel plan for the Hoover Dam contemplated a plain ogee crest without gates. The design of this spillway is shown in Figure 7. This crest had a discharge capacity of 31,250 second-feet at El. 1229 and 100,000 second-feet at El. 1234.5, the same as the glory hole type, but the spillway side channel and the tunnel had sufficient capacity to take care of the flow over the crest up to the top of the dam, or approximately 140,000 second-feet. The model of this structure was also erected in the Colorado Agricultural College laboratory on a 1:60 scale. The position of the apparatus is shown on Figure 4. The water measurement apparatus was the same as that previously described.

The Construction of the Model

The side channel of this model was constructed of galvanized sheet iron supported on sheet metal buttresses. It was expected that several sizes of channel would be tested and the buttresses were designed so that various cross sections of channel could be obtained by

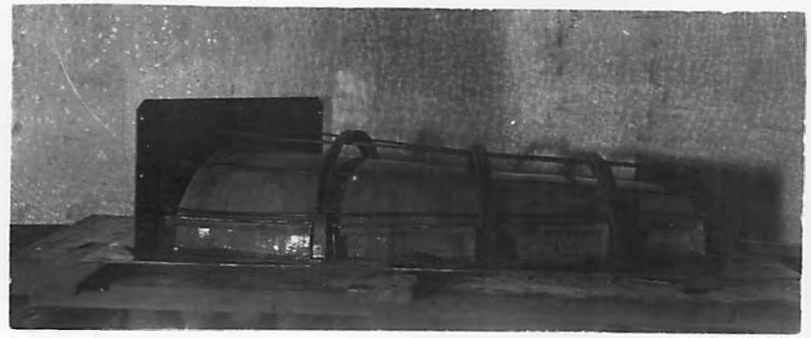


Ground surface on ℄
Weir crest El. 1223.6
El. 1237.0
El. 1214.0
El. 1127.38
El. 1119.14
El. 974.64
SECTION ON CENTER LINE
Scale of Feet
Q = 100,000 sec. feet
El. 1234.5
El. 1223.6
SECTION A-A
SECTION B-B
Diversion tunnel plug
R = 150'
El. 630.0
Scale of Feet
El. 1234.5
El. 1237.0
El. 1215.0
El. 1214.0
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
NEVADA SIDE CHANNEL SPILLWAY
FREE CREST TYPE
DRAWN W.L.R. SUBMITTED
TRACED E.A.R. RECOMMENDED
CHECKED W.M.B. APPROVED
DENVER, COLO., NOV 28 1932
45-D-2289

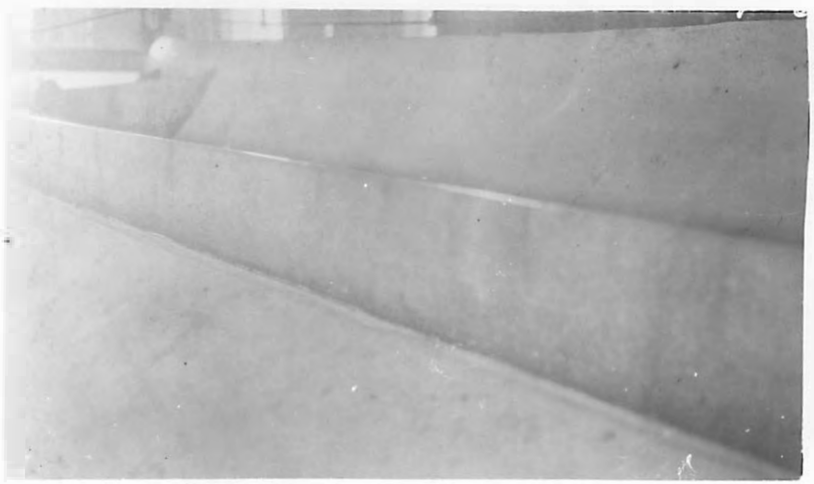
FIG. 7

slight changes of the position of the buttresses. The ogee crest of the structure was made of paraffin (Plate 10-D). A sloping floor, simulating the sloping mountain side intersecting the vertical face of the spillway crest 35 feet below the top, was placed in the approach channel to the spillway (Plate 10-C). The transition from the trapezoidal section of the side channel to the circular 50-foot diameter section of the tunnel was constructed of reinforced concrete. It was built in halves, divided by a vertical plane along the center line. Plaster of Paris cores for the opening were cast around accurately spaced templates, shown on Plate 10-A. A reinforcing of rods was built around the core (Plate 10-B) and the transition cast around these using Lumnite cement. When the cement had set the core was removed and the two halves were bolted together. Three windows were cast into the top through which the flow in the transition could be observed. These windows could be closed with concrete plugs accurately cast to fit the shape of the top of the transition. The same pyralin tube which formed the model of the vertical shaft and tunnel for the glory hole model was used for the inclined and horizontal tunnels of this model. The slope of the inclined tunnel was 45 degrees with the horizontal and the bend between the two sections of tunnel was formed of two of the $22\frac{1}{2}$-degree bends.

A-TEMPLATES FOR CORE OF TRANSITION



REINFORCING FOR TRANSITION-CORE IN PLACE

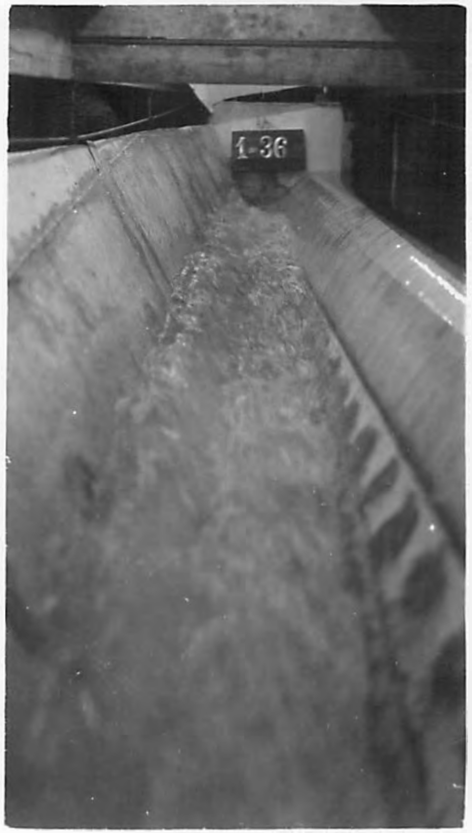


C-SLOPING APRON AND PARAFFIN CREST

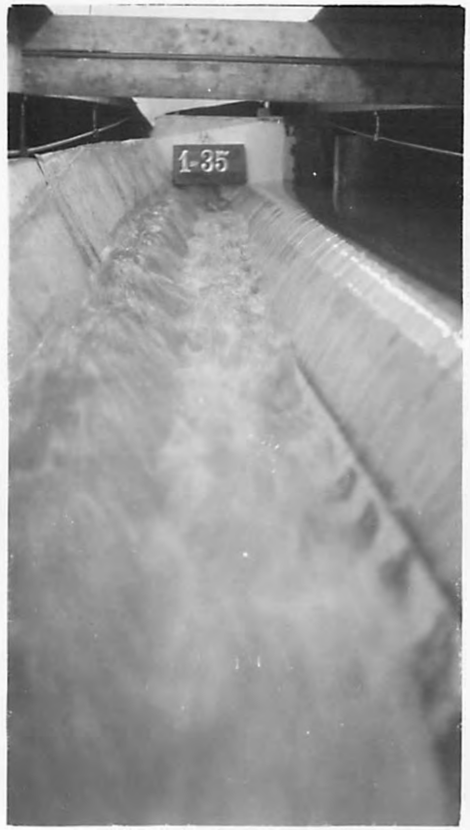
D-PARAFFIN CREST AND SIDE CHANNEL

Action of the Free Crest Model C-2

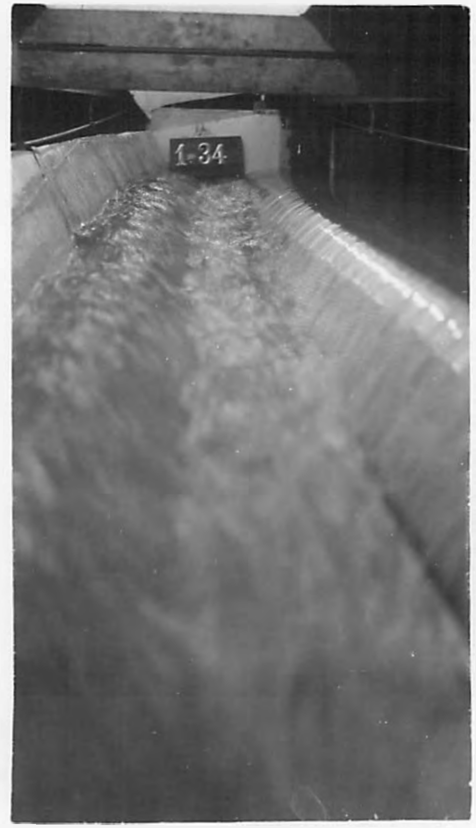
The action of the spillway channel for flows of 50,000, 100,000 and 150,000 second-feet is shown on Plate 11. A flow of 150,000 second-feet entering the tunnel portal is shown on Plate 12-C. For these flows the nappe passing over the ogee crest plunges under the water in the trough and causes the water in the trough to flow with a spiral motion, the motion at the bottom of the channel being from the weir side across diagonally to the opposite side. Under favorable conditions this spiral motion is so pronounced that the centrifugal force tends to move the solid water to the outside of the spiral and the lighter air bubbles are forced to the center, forming a core of air in the center of the spiral for the full length of the channel. When the discharge was raised sufficiently above that for which the spillway was designed, the depth of flow at the upper end of the trough is raised until the nappe no longer dives under the water in the trough, but passes over the top, as shown in Plate 12-A. This tends to cause a spiral in the trough in the opposite direction from that formed when the nappe dives under. Further down the trough the water level is lower and the nappe still dives under, with the result that the spiral at the upper end tends to turn in a clockwise direction looking downstream and that at the lower end in a counterclockwise direction. At the junction of these two motions there is an unstable condition set up and the point where the nappe changes from a counterclockwise to a clockwise condition



A-50.000 SECOND FEET DISCHARGE

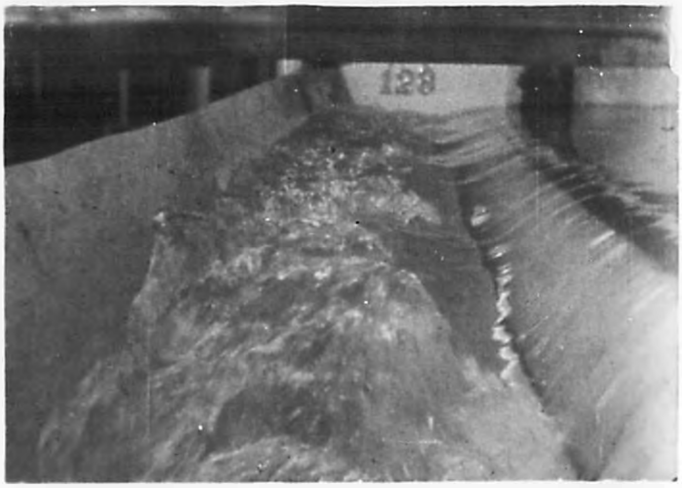


B - 100.000 SECOND FEET DISCHARGE



C -150000 SECOND FEET DISCHARGE

PLATE 11



A-PULSATING FLOW CONDITION - LOOKING UPSTREAM

B-PULSATING FLOW CONDITION-LOOKING DOWNSTREAM

C-FLOW AT PORTAL ENTRANCE

SIDE CHANNEL SPILLWAY-MODEL C-2

moves up and down the channel over a considerable range, giving rise to a sort of pulsating condition of flow in the channel. The division point between the counterclockwise and clockwise rotation is clearly shown on Plate 12-B.

Scope of Quantitative Experiments

Soon after this model was completed it was decided to increase the capacity of each spillway from 100,000 second-feet to 200,000 second-feet. This necessitated a redesign of the spillway and while the office studies to develop the most economic design for this size were under way an extensive series of experiments were carried out on this model to check the theory of the side channel spillway and to investigate a number of ways of reducing its cost.

The basic theory of the side channel spillway was developed by Mr. Julian Hinds* and is based on the assumption that all the energy of the water falling over the spillway crest is dissipated as heat and the flow down the spillway trough is caused only by the water surface slope in this channel itself.

There were four series of tests, the conditions of each being as follows:

I The side channel with trapezoidal cross section as designed, with various discharges.

*Side Channel Spillways, Julian Hinds, Trans., Am. Soc., C. E., p. 881, Vol. 89, 1926.

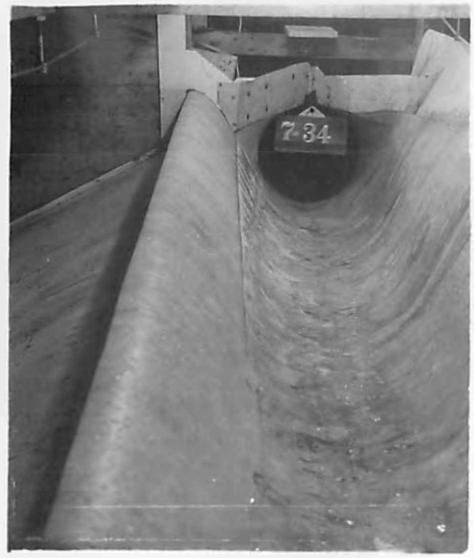
II Various discharges with submerged weirs across the downstream end of the spillway channel to change the conditions of flow in it.

III A channel with the same sides as the original design but with a flat floor at a higher elevation, formed by a false bottom.

IV A channel with practically the same sides as the original design but with the bottom formed by a circular arc tangent to the sides (Plates 13-A and B). The bottom of the channel of this model was slightly higher than the original design.

Hydraulically, the side channel spillway is a very inefficient device, as so much of the energy of the water which falls over the crest is used up in heat and does not cause flow down the side channel. If part of the energy could be used in producing flow along the side channel, a greater velocity in it would result and a smaller and cheaper channel could therefore be used. To a certain extent this can be accomplished by means of guide vanes or deflectors which direct the flowing water after passing over the crest into a direction more or less parallel to the center line of the channel. Different types and combinations of deflector vanes were tried to determine how much could be accomplished in reducing the spillway cost by this means. One type with vanes on the crest is shown on Plates 13-C and D.

One of the uncertainties of spillway design is the allowance which must be made for the air which is entrained in the side channel, due to the turbulent conditions of flow existing therein. The theory



A-LOOKING UPSTREAM

B-LOOKING DOWNSTREAM

SIDE CHANNEL WITH ROUNDED BOTTOM

C-NO DISCHARGE

D-DISCHARGING

SIDE CHANNEL WITH VANES ON THE CREST

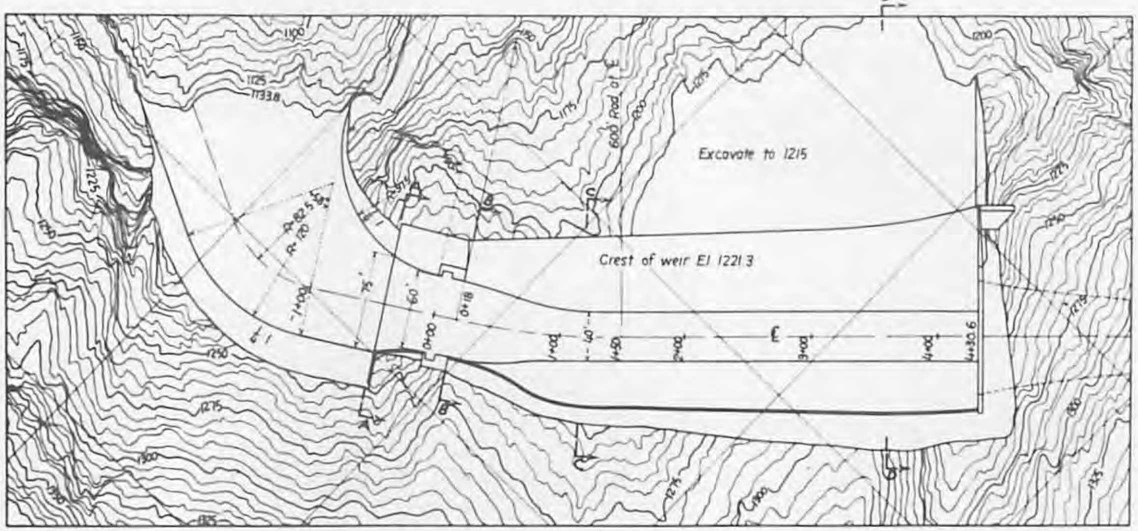
SIDE CHANNEL SPILLWAY-MODEL C-2

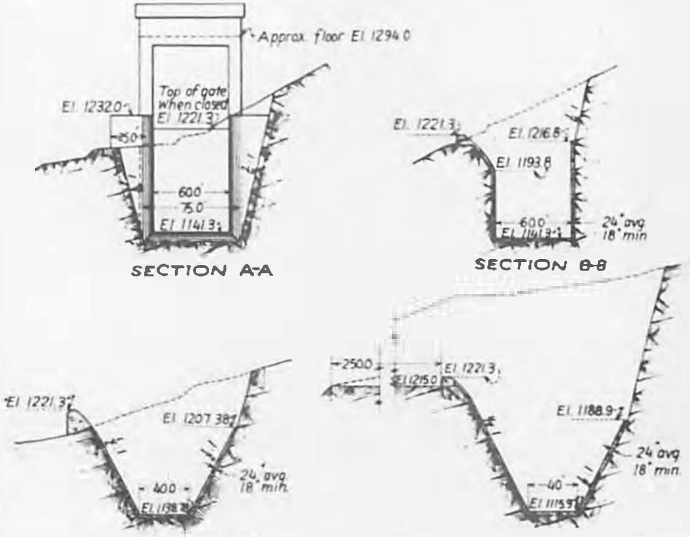
as developed by Hinds considers only solid water, but if a quantity of air is entrained in this water, its volume is correspondingly increased and the size of the channel necessary to carry it is presumably increased also. Measurements were therefore made of the air entrained in the flow through the model.

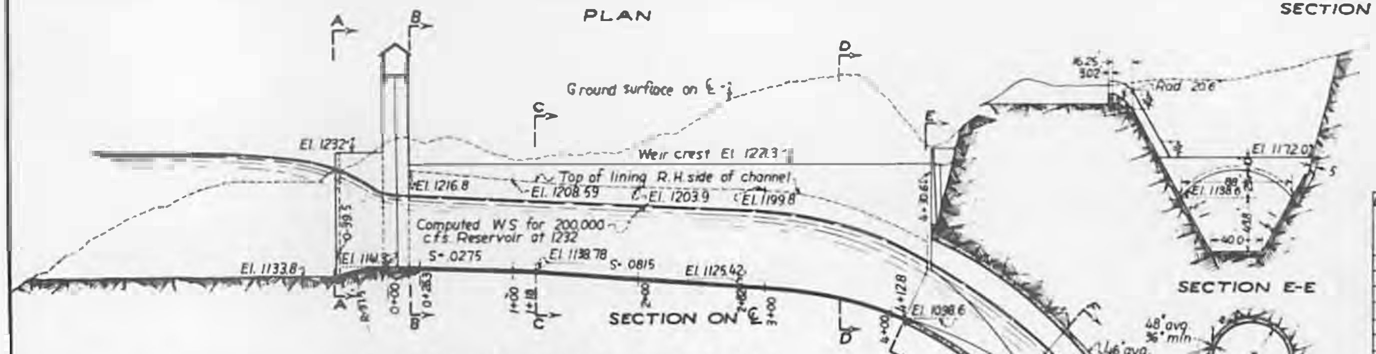
The results of these tests were largely negative from the standpoint of producing much smoother conditions of flow in the spillway. They are of interest, however, from a hydraulic standpoint and will be described in detail in the later portions of this report.

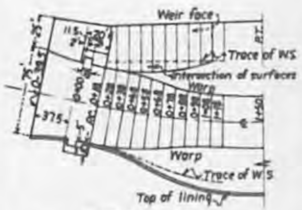
THE STONEY GATE SIDE CHANNEL SPILLWAY

When the decision was made to build for a total spillway capacity of 400,000 second-feet instead of 200,000 second-feet, the first design worked out to accomplish this increase in capacity consisted of a side channel spillway with crest length of 700 feet discharging 125,000 second-feet and a 50-foot by 50-foot Stoney gate at the end, with a capacity of 75,000 second-feet. This design was included in the contract drawings (drawing No. 45-D-940). Further studies showed that a reduction in cost could be obtained by reducing the length of the spillway and increasing the size of the Stoney gate. The design shown on Figure 8 was developed which used a 400-foot crest without gates, under a 10.7-foot head, giving a discharge of 50,000 second-feet and a Stoney gate 60 feet wide and 80 feet high with a discharge capacity of 150,000 second-feet. The top of the Stoney gate





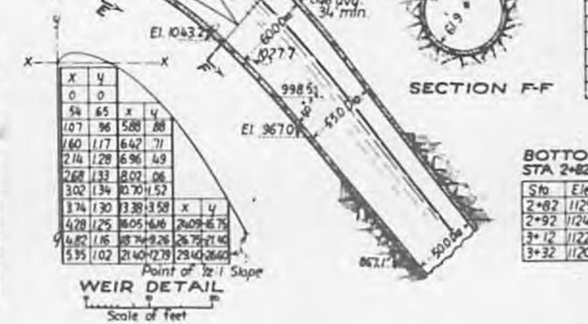




DETAIL OF CHANNEL ENTRANCE

TRANSITION STA. 0+18 TO STA. 1+18

Station on ℄	Bottom Elev.	Bottom Width	Side slope	Intersection of warped surface with weir face: Elev.	Distance To ℄	Distance To weir	Station on weir	Top of lining R.H. side: Elev.	Distance to ℄
0+18	1141.3	60	Vert.	1193.80	30.00	30.19	-0.6	1216.8	30
0+28	1141.24	60	Vert.	1189.00	30.00	31.60	+0.7	1216.8	30
0+38	1140.97	59	.026:1	1185.94	30.70	33.23	10.1	1215.97	31.45
0+48	1140.70	57	.082:1	1184.48	32.10	36.05	19.7	1214.40	34.54
0+58	1140.42	54.3	.152:1	1183.72	33.80	34.20	29.3	1213.02	38.19
0+68	1140.15	50.4	.232:1	1183.25	35.25	34.50	38.8	1211.85	41.84
0+78	1139.88	46.5	.313:1	1182.98	36.75	34.65	48.4	1210.48	45.35
0+88	1139.60	43.1	.388:1	1183.16	38.48	34.50	57.9	1209.50	48.65
0+98	1139.33	41.4	.442:1	1187.93	42.20	32.10	67.6	1208.73	51.45
1+08	1139.06	40.4	.477:1	1193.36	46.05	29.40	77.4	1208.01	53.15
1+18	1138.78	40.0	.5:1	1193.36	47.29	29.40	86.7	1207.38	54.30
1+50	Radial line 78.01 ℄ to weir face								



x	y	x	y	x	y
0	0				
.54	.65				
1.07	.96	5.88	.88		
1.60	1.17	6.42	.71		
2.14	1.28	6.96	.49		
2.68	1.33	8.02	.06		
3.02	1.34	10.70	-1.52		
3.74	1.30	13.38	-3.58		
4.28	1.25	16.05	-6.16	24.09	-16.75
4.82	1.16	18.74	-9.26	26.75	-21.40
5.35	1.02	21.40	-12.78	29.40	-26.40

Point of ½:1 Slope

WEIR DETAIL

Scale of feet

TYPICAL TRANSITION SECTION
TRANSITION – OPEN CHANNEL TO 50 FT ϕ TUNNEL

Distance from portal along invert	Invert Elev.	H	Top arch h	r_T	C_T	R_T	Floor arch r_F	C_F	R_F	h_S	$\phi°$
0	1098.6	65.8	43.8	22.0	88.0	55.0	Flat			43.8	24
10'	1094.2	65.1	41.1	24.0	84.75	49.45	1.5	41.19	142.08	39.6	26½
25'	1087.3	64.3	37.8	26.5	80.74	44.0	4.0	43.56	61.25	33.8	30
37.5'	1080.8	63.8	34.1	29.7	76.53	39.4	6.0	45.62	46.4	30.9	32
50'	1074.0	63.3	31.0	32.3	73.10	36.76	8.0	47.8	39.4	23.0	34¼
62.5'	1066.7	63.0	27.5	35.5	69.13	34.56	10.0	49.88	36.1	17.5	36½
75'	1059.2	62.4	24.0	38.4	64.90	32.9	12.0	51.7	33.86	12.0	37¾
87.5'	1051.4	62.1	20.2	41.9	60.22	31.78	14.0	53.4	32.45	6.2	39¾
100.0'	1043.2	61.9	16.0	45.9	54.25	30.95	16.0	54.25	30.95	0	41½
122.9'	1027.7	60 foot ϕ circle									44¼
203.9'	967.0	55 foot ϕ circle									50
334.2'	867.1	50 foot ϕ circle									50

BOTTOM GRADES
STA. 2+82 TO STA. 4+12.8

Sta.	Elev.	Sta.	Elev.
2+82	1125.42	3+52	1117.7
2+92	1124.60	3+72	1113.2
3+12	1122.90	3+92	1106.8
3+32	1120.80	4+12.8	1098.6

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

HOOVER DAM
NEVADA SPILLWAY
ONE – 80' x 80' STONEY GATE AND 400' WEIR CREST

DRAWN W.L.R. — SUBMITTED
TRACED E.A.O. — RECOMMENDED
CHECKED W.F.B. — APPROVED

DENVER COLO., MAY 14, 1932 — 45-D-2527

FIG. 8

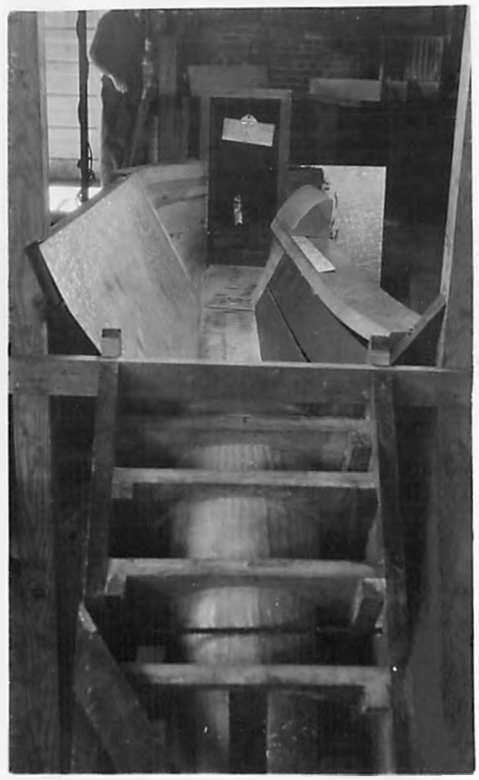
when closed was at the same elevation as the crest of the weir, and under certain conditions water passed over the top of the gate.

Stoney Gate Model C-3

A model of the Stoney gate spillway shown on Figure 8 was built on a scale of 1:60 or 1 inch = 5 feet at the Colorado Agricultural laboratory. Drawings of this model are given on Figures 9 and 10. With this scale the model of the spillway, not including the tunnel, was about 7 feet long and 1½ feet high and the discharge through the model was approximately 7.2 second-feet. The relation between the model and prototype quantities is the same as that for the glory hole model or a linear ratio of 1:60 (see Table I).

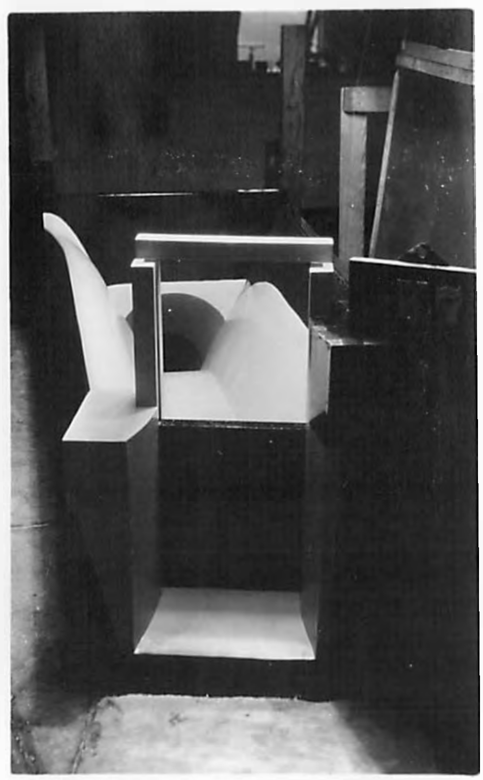
The trough of the model was constructed of a wooden frame covered with galvanized iron as shown on Plate 14. The ogee crest and the warped section below the gates were of wood. After assembling the portions of the model the surfaces representing the concrete were painted with gray Duco and the remainder red enamel, to contrast in the photographs. The topography of the canyon side in the vicinity of the spillway was of a lamellar wood construction with boards of a thickness corresponding to 4 feet on the prototype, which were cut along the contour lines. The transition from the trapezoidal side channel to the circular inclined tunnel was constructed of wood staves held in place by collars at intervals as shown on Plate 14-B. The staves were smoothed down and painted on

A-INCOMPLETE MODEL.
LOOKING DOWNSTREAM



B-INCOMPLETE MODEL.
LOOKING UPSTREAM

C-INCOMPLETE MODEL.
LOOKING DOWNSTREAM



D- COMPLETE MODEL.
LOOKING DOWNSTREAM

STONEY GATE SIDE CHANNEL SPILLWAY - MODEL C-3

TABLE I

Relation of Quantities in Model and Prototype

Quantities	In Terms of N*	Ratio of Prototype to Model				
		For N = 20	For N = 60	For N = 64	For N = 100	For N = 10
Length	N	20	60	64	100	106
Area	N^2	400	3,600	4,096	10,000	11,23
Volume	N^3	8,000	216,000	262,144	1,000,000	1,191,01
Velocity	$N^{0.5}$	4.472	7.746	8	10	10.2
Discharge	$N^{2.5}$	1,788.8	27,885	32,768	100,000	115,68
Time	$N^{0.5}$	4.472	7.746	8	10	10.29
Energy	N^4	160,000	12,960,000	16,777,216	100,000,000	126,247,69
Impulse	$N^{3.5}$	35,777	1,673,136	2,097,152	10,000,000	12,262,22

* N = Ratio of lineal dimensions of the prototype to those of the model.

The model ratio is usually expressed in terms of 1:N.

the inside and the outside coated with a thick layer of paraffin to prevent swelling. The model with the adjacent topography is shown on Plate 15-A.

The flow through the model was measured on the 2-foot Cipolletti weir previously described. The water surface in the channel was observed by means of a point gage extending down from level parallel bars above the channel as shown on Plate 15-B. A large number of piezometers was installed in the vicinity of the gate transition to observe the pressure there. The location of the piezometer openings is shown on Figures 9 and 10 and the glass tube gages with which the pressures were observed as shown on Plate 15-C.

Outline of Tests

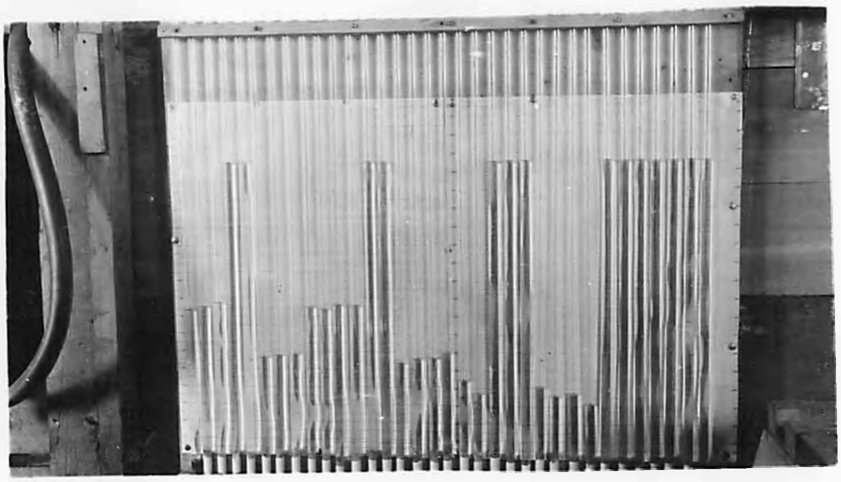
A large number of tests were made on the Stoney gate model to determine whether reasonably smooth flow conditions could be obtained in it, and whether it would have the discharge capacity for which it was designed. A number of changes were made in the model to improve the flow conditions, the results of which were also recorded. The following is a brief outline of the observations which were made in these tests:

A. The Original Design.

1. Flow for probable conditions of operation.
2. Flow over spillway crest only.
3. Flow through gate only.

A-COMPLETED MODEL WITH TOPOGRAPHY

B-OBSERVING WATER SURFACE ELEVATION WITH POINT GAGE



C-PIEZOMETER BOARD FOR OBSERVING PRESSURES

STONEY GATE SIDE CHANNEL SPILLWAY - MODEL C-3

B. Alternatives in the Original Design.

1. Effect of dentated baffles on channel floor.
2. Flow conditions with various types of Stoney gate piers.
3. Effect of alterations in gate approach channel.
4. Result of lengthening the transition between the rectangular gate section and the trapezoidal section of the channel.
5. Effect of guide walls at ends of ogee crest.
6. Flow conditions with raised floor (to obtain data for drum gate design).

C. Determination of Discharge Coefficients.

1. Determination of crest discharge coefficient.
2. Determination of Stoney gate discharge coefficient.
3. Investigation of effect of Stoney gate draw down on discharge over weir crest.

In the following paragraphs the action of the spillway and changes made to try to improve it will be described. The results of the quantitative hydraulic experiments will be discussed in a later portion of this report.

Flow under Normal Operating Conditions

In the spillway shown on Figure 8 the top of the Stoney gate, when closed, is at the same elevation as the top of the ogee crest. Under normal operating conditions, if the water rose in the reservoir, it would flow over both the ogee crest and the top of the

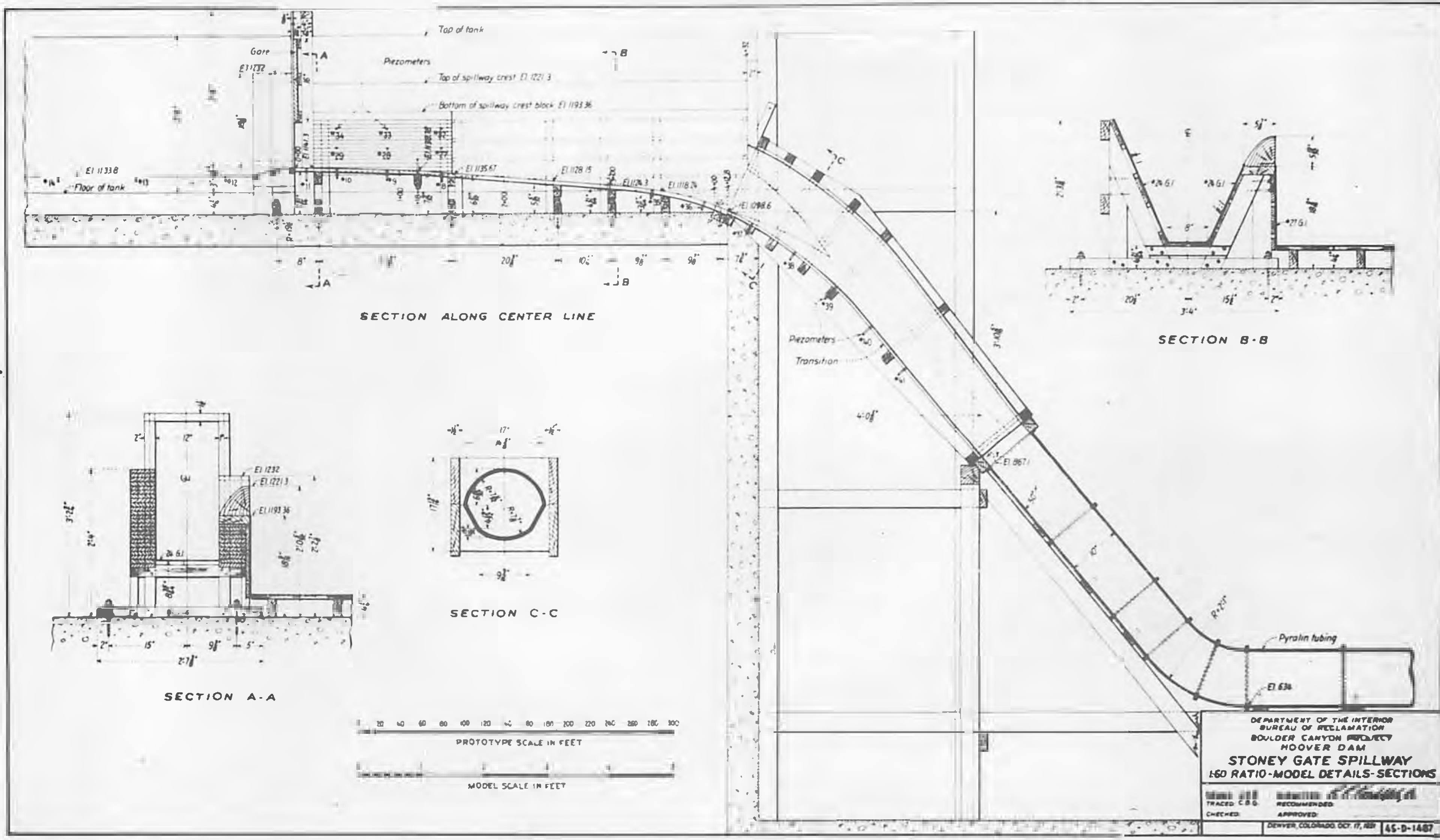

Top of tank
Gate
Piezometers
Top of spillway crest El. 1221.3
Bottom of spillway crest block El. 1193.36
Floor of tank
El. 1133.8
El. 1135.67
El. 1128.15
El. 1098.6
SECTION ALONG CENTER LINE
Piezometers
Transition
El. 867.1
Pyralin tubing
El. 634
SECTION B-B
El. 1232
El. 1221.3
El. 1193.36
SECTION A-A
SECTION C-C
PROTOTYPE SCALE IN FEET
MODEL SCALE IN FEET
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
STONEY GATE SPILLWAY
1:60 RATIO-MODEL DETAILS-SECTIONS
TRACED
RECOMMENDED
CHECKED
APPROVED
DENVER, COLORADO, OCT. 17, 1931
45-D-1487

FIG. 10

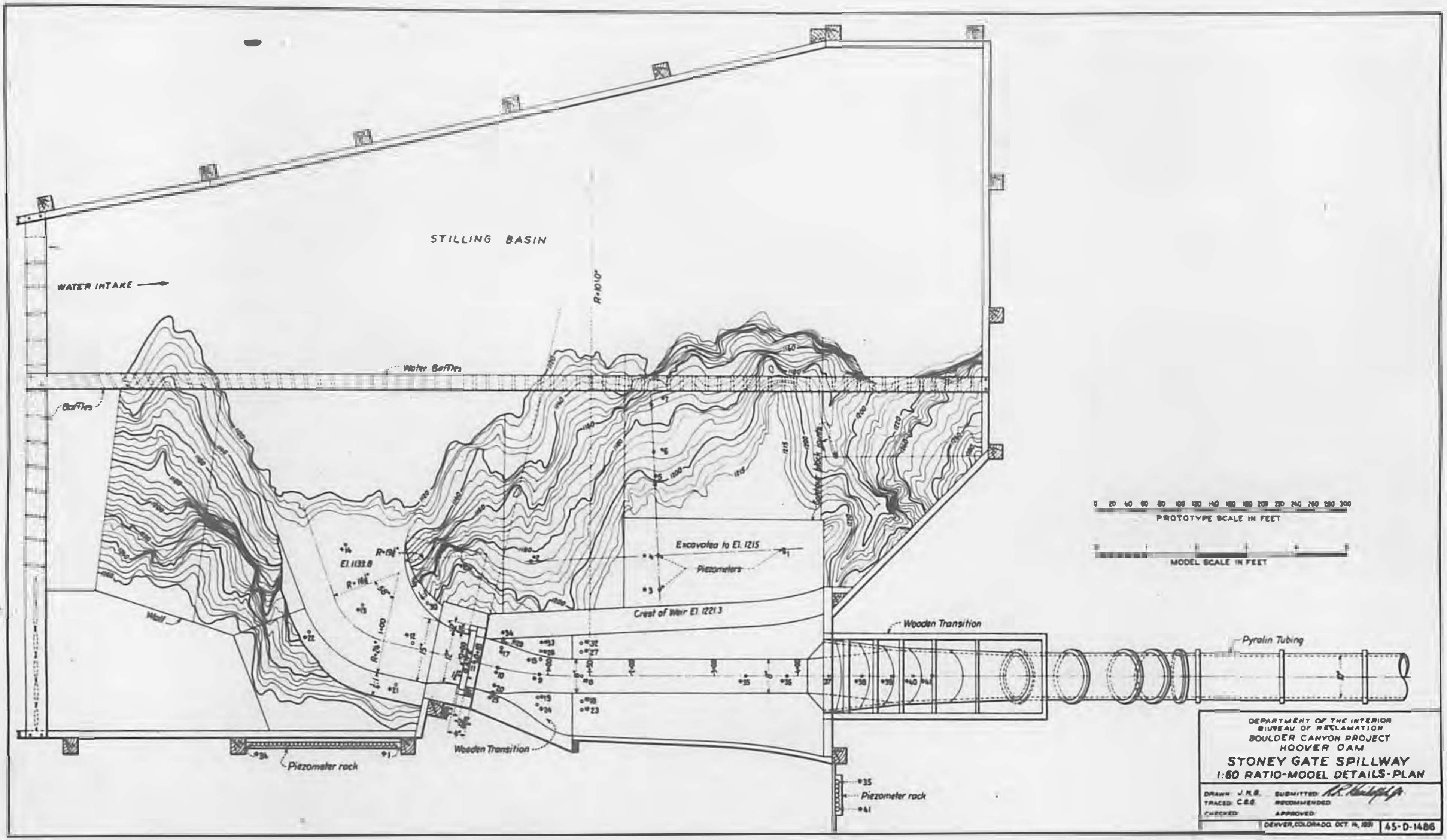
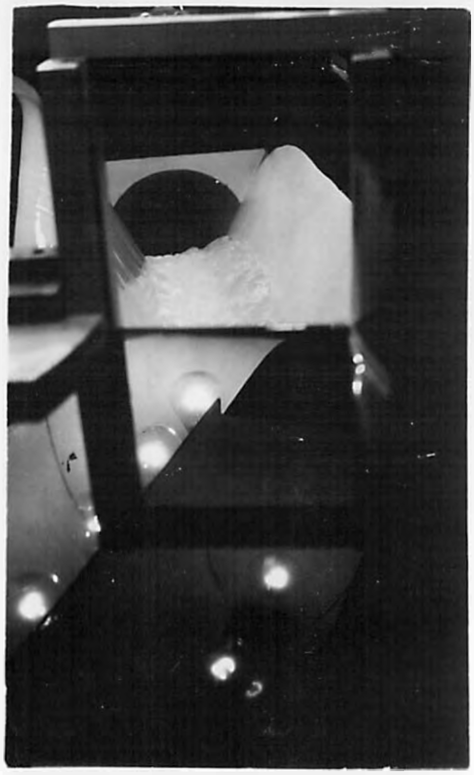

STILLING BASIN
WATER INTAKE
Water Baffles
Baffles
Wall
R=1010'
Excavated to El. 1215
Piezometers
El. 1132.8
Crest of Weir El. 1221.3
Wooden Transition
Wooden Transition
Pyralin Tubing
Piezometer rack
Piezometer rack
PROTOTYPE SCALE IN FEET
MODEL SCALE IN FEET
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
STONEY GATE SPILLWAY
1:60 RATIO-MODEL DETAILS-PLAN
DRAWN J.N.B.
TRACED C.B.B.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO, OCT. 14, 1931
45-D-1486

FIG. 9

gate until the discharge corresponded to approximately 60,000 second-feet, being the flow which would pass when the reservoir reached the maximum flow line or Elevation 1232. If the reservoir tended to rise further, the Stoney gate would be opened enough to prevent the reservoir rising above elevation 1232. For flows between 60,000 and slightly over 80,000 second-feet the gate would not open sufficiently to bring its top above the water level in the reservoir and there would be a discharge over as well as under the gate. For greater discharges the top of the gate would be above the reservoir water level and all the flow would pass beneath it.

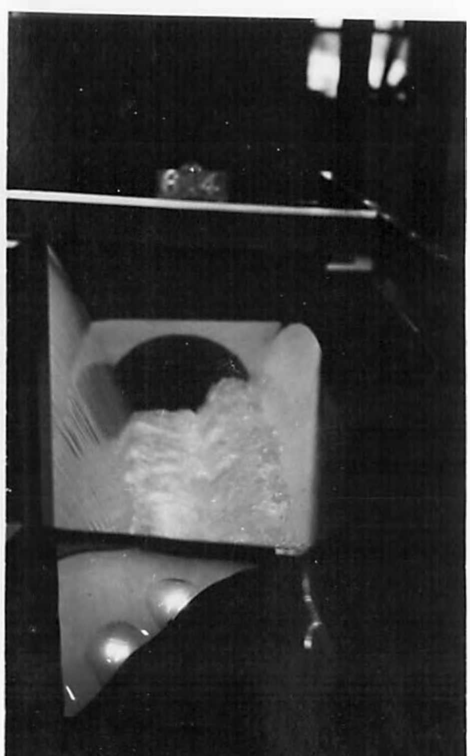
The conditions of flow in the model for each multiple of 20,000 second-feet discharge are shown on Plates 16 to 20 inclusive. With the lower discharges most of the flow entered the spillway over the ogee crest and the condition in the channel was similar to that in the free crest side channel type previously described. In the bottom of the channel it piled up on the side opposite the crest to a higher elevation than on the side adjacent to the crest. At about 60,000 second-feet discharge, there was a distinct stationary wave against the back side of the channel just below the gate as shown on Plate 17. As the flow was increased this wave became higher and moved somewhat downstream. The flow in the channel was very turbulent and much higher on the back side of the channel. As the discharge approached the design capacity the flow conditions became



A - LOOKING DOWNSTREAM

B - LOOKING UPSTREAM

DISCHARGE 20000 SEC FT

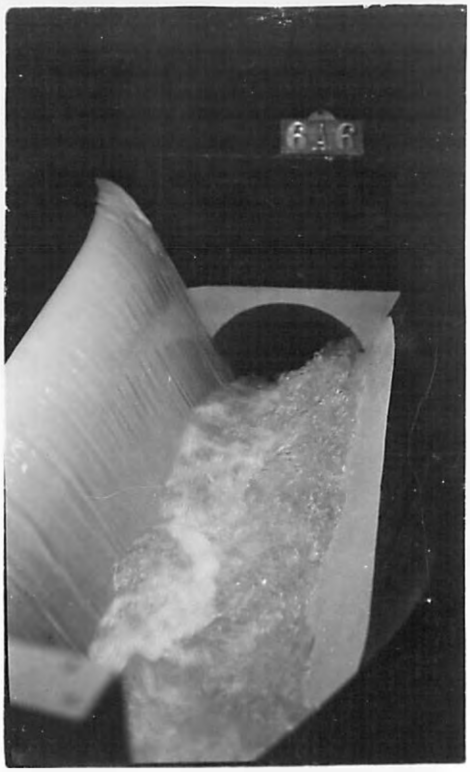


C - LOOKING DOWNSTREAM

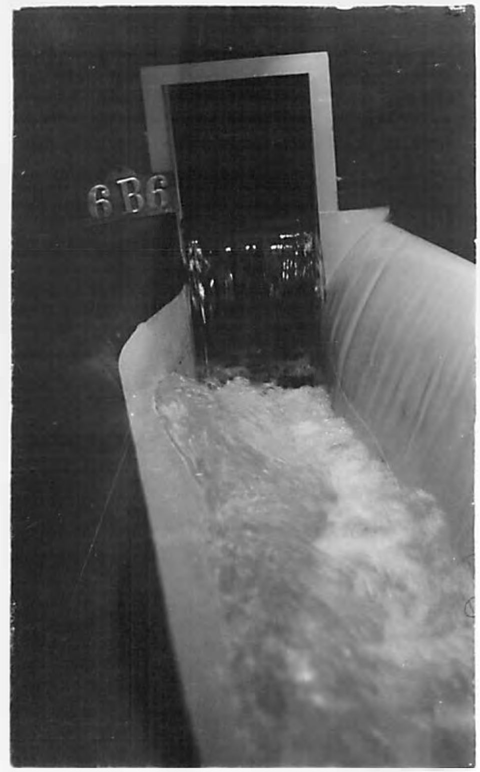
D - LOOKING UPSTREAM

DISCHARGE 40.000 SEC FT

STONEY GATE SIDE CHANNEL SPILLWAY - MODEL C-3

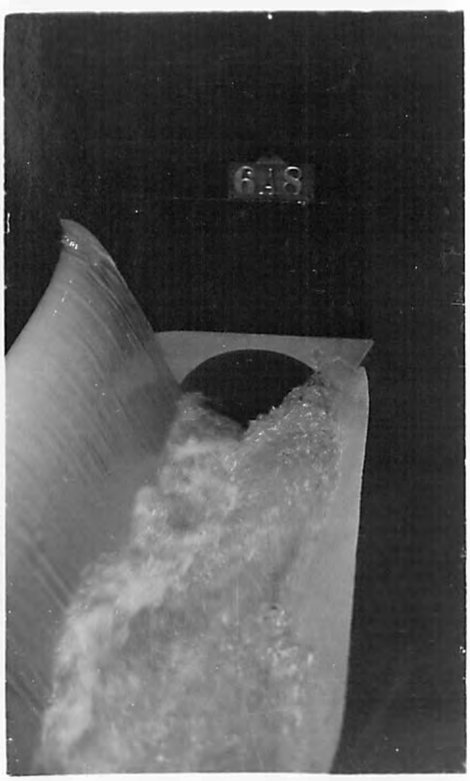


A- LOOKING DOWNSTREAM



B - LOOKING UPSTREAM

DISCHARGE 60,000 SEC FT

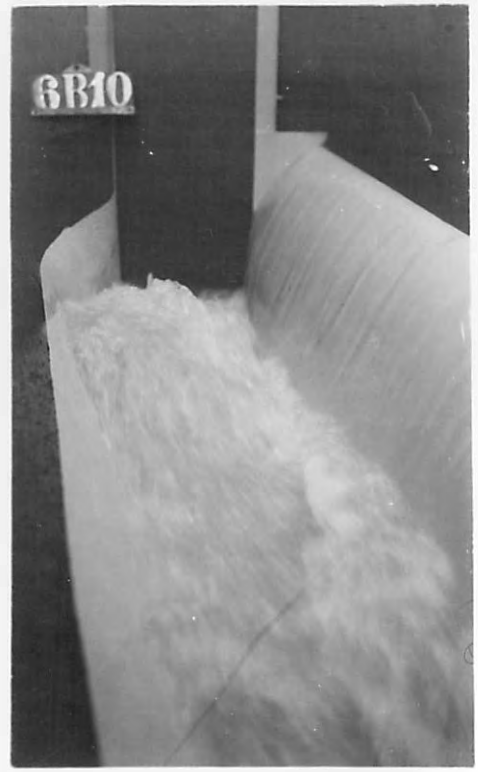


C - LOOKING DOWNSTREAM

D - LOOKING UPSTREAM

DISCHARGE 80,000 SEC FT

STONEY GATE SIDE CHANNEL SPILLWAY - MODEL C-3



A - LOOKING DOWNSTREAM

B - LOOKING UPSTREAM

DISCHARGE 100,000 SEC. FT.

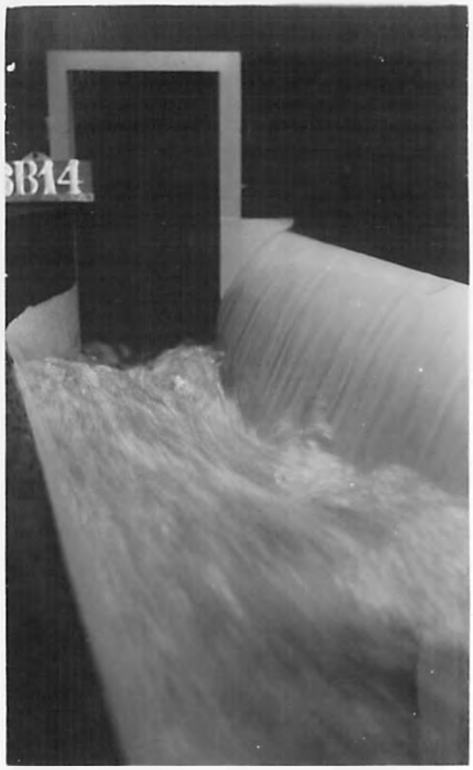
C - LOOKING DOWNSTREAM

D - LOOKING UPSTREAM

DISCHARGE 120,000 SEC. FT.

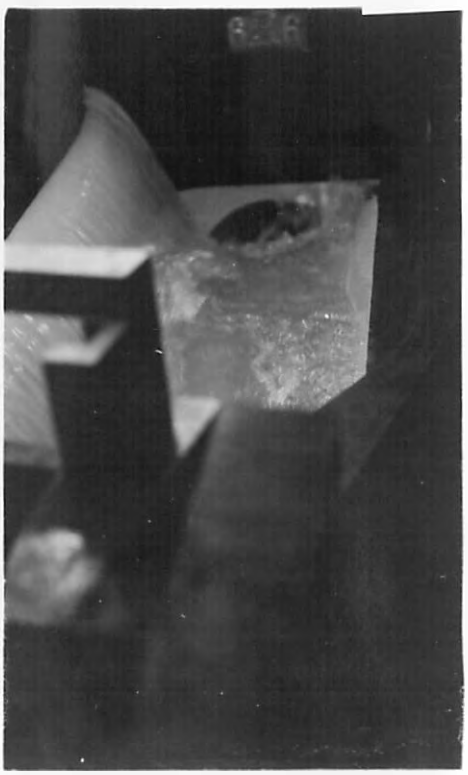
STONEY GATE SIDE CHANNEL SPILLWAY - MODEL

A-LOOKING DOWNSTREAM



B-LOOKING UPSTREAM

DISCHARGE 140.000 SEC FT

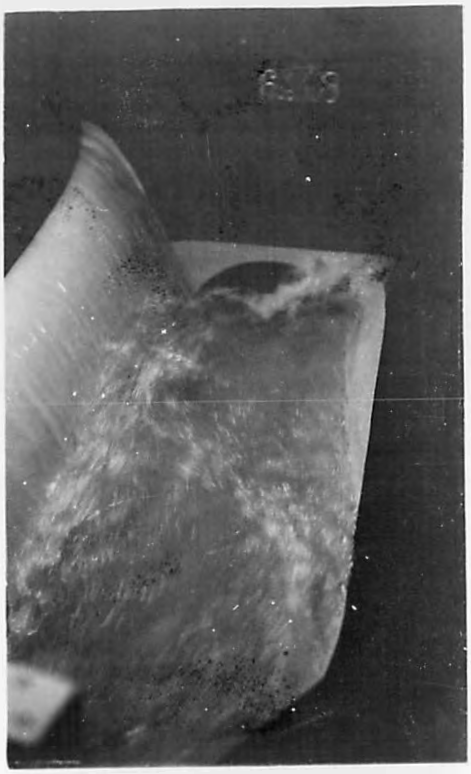


C-LOOKING DOWNSTREAM

D-LOOKING UPSTREAM

DISCHARGE 160.000 SEC FT

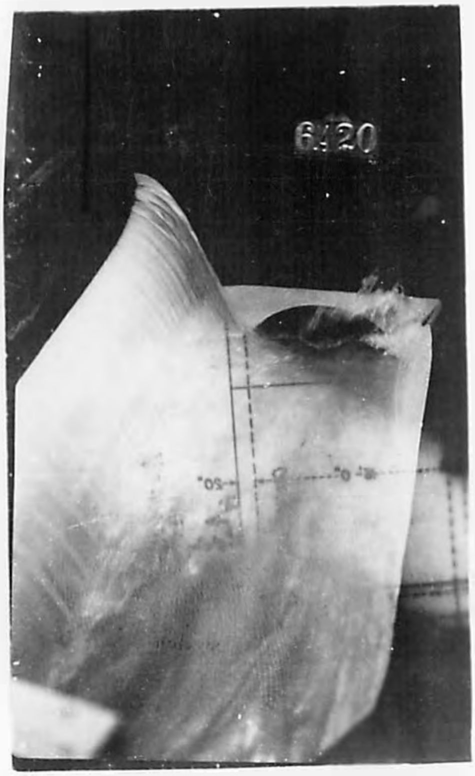
STONEY GATE SIDE CHANNEL SPILLWAY - MODEL C-3

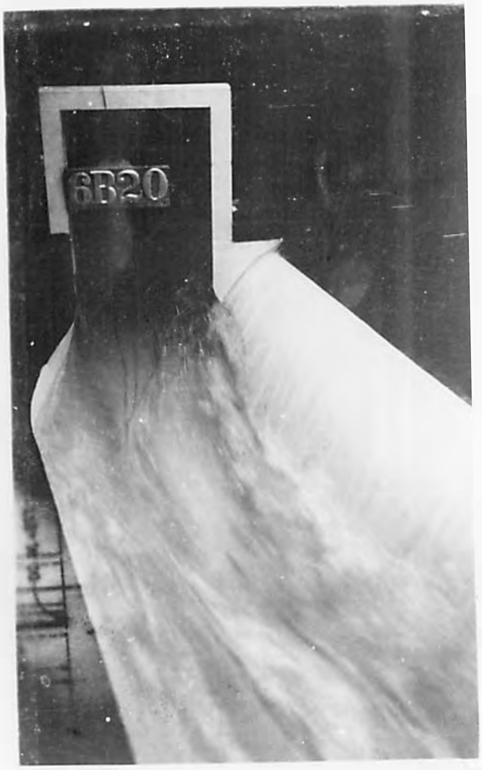


A- LOOKING DOWNSTREAM

B - LOOKING UPSTREAM

DISCHARGE 180.000 SEC FT





C - LOOKING DOWNSTREAM

DISCHARGE 200.000 SEC FT - THE DESIGNED CAPACITY

STONEY GATE SIDE CHANNEL SPILLWAY - MODEL C-3

somewhat smoother, due no doubt to the greater flow beneath the gate and the relatively smaller importance of the flow over the crest, the worst condition being about 120-130,000 second-feet discharge. In all cases the water had a spiral motion and the greater depth on the back side continued through the transition and into the inclined tunnel. With the flow over the gate and crest, the condition at the downstream end of the transition is shown on Plate 21-A. With a 200,000 second-feet flow the water still had some spiral flow but the longitudinal flow from the gate gave it a greater longitudinal component, as shown on View B. With all the flow under the gate (View C) there was no spiral, but with 200,000 second-feet over the crest only, the spiral was very decided (View D).

Flow over the Crest Only and through the Gate Only

Observations were made on the flow in the channel with the gate blocked off and all the water falling over the ogee crest. The discharges were carried up to 200,000 second-feet although this was a much greater discharge than the crest was designed for and necessitated a higher level in the reservoir than would be possible at Hoover Dam. Pictures of the conditions for various discharges are given on Plate 22. The conditions of flow were similar to those found in the free crest side channel (Model C-2), having the high ridge on the rear side of the channel.

Observations were also made on the flow through the gate

A-FLOW OVER CREST AND GATE
POND AT ELEV 1232

B-FLOW OVER CREST AND THRU GATE
DISCHARGE 200,000 SEC. FT.

C-FLOW THRU GATE ONLY
DISCHARGE 200,000 SEC. FT.

D-FLOW OVER CREST ONLY
DISCHARGE 200,000 SEC FT

CONDITION OF FLOW AT END OF TRANSITION
SECTION OF STONEY GATE SIDE CHANNEL SPILLWAY - MODEL C-3

A-DISCHARGE 40,000 SEC. FT.

B-DISCHARGE 100,000 SEC. FT.

C-DISCHARGE 150,000 SEC. FT.

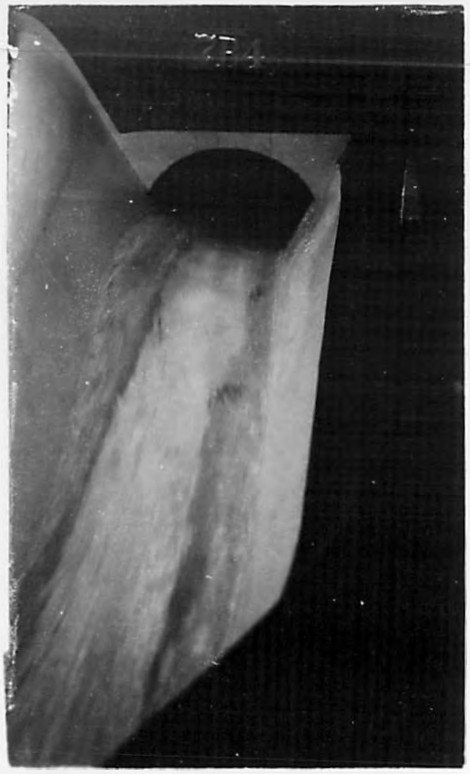
D-DISCHARGE 200,000 SEC. FT.

FLOW OVER CREST ONLY-MODEL C-3

only with the flow over the crest blocked off. The conditions for several discharges are shown on Plate 23. At all flows there was a distinct piling up of the water on the crest side near the entrance to the tunnel, as shown on Views A and C. This was of about the same magnitude for flows of 30,000 second-feet, and therefore represented a relatively greater disturbance for the small flows. There was also some piling up on the rear side of the channel, due no doubt to the curvature of the channel. This was especially noticeable at high discharges; the water rose on a ridge first on the rear side of the channel and then swung across and piled up on the crest side. For low and medium flows the water passed under the Stoney gate with but little disturbance, as shown by View B. At high discharges (View D) there was a tendency to form a wave on the crest side just below the gate, evidently due to the offset between the gate structure wall and the ogee crest at this point.

Dentated Baffles on Channel Floor

A row of dentated baffles was placed on the channel floor to reduce the rotary motion of the water by dissipating the velocity of the water falling over the ogee crest, and thus reducing the height of the ridge on the side of the spillway channel opposite the crest. Those baffles had curved faces which tended to throw the water, striking them back toward the crest side. Two heights were used, the larger height (15 feet) is shown on Plate 24-A. For the lower dis-



A-LOOKING DOWNSTREAM

B-LOOKING UPSTREAM

DISCHARGE 100,000 SEC FT

C-LOOKING DOWNSTREAM

D-LOO [illegible]

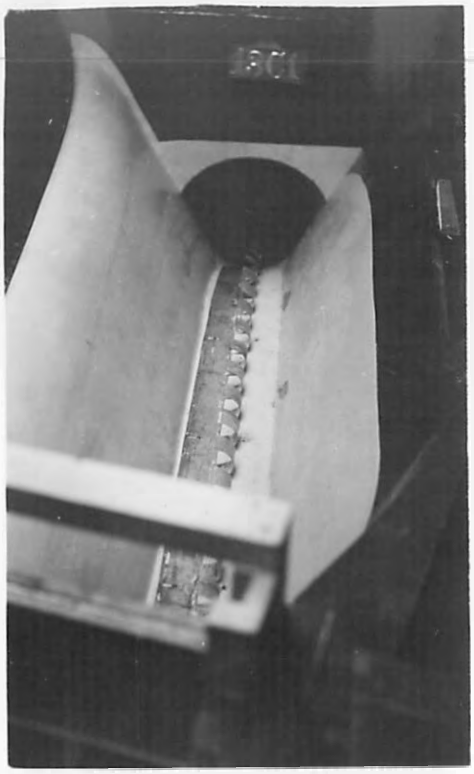
DISCHARGE 155,000 SEC FT

FLOW THRU GATE ONLY-MODEL C-3

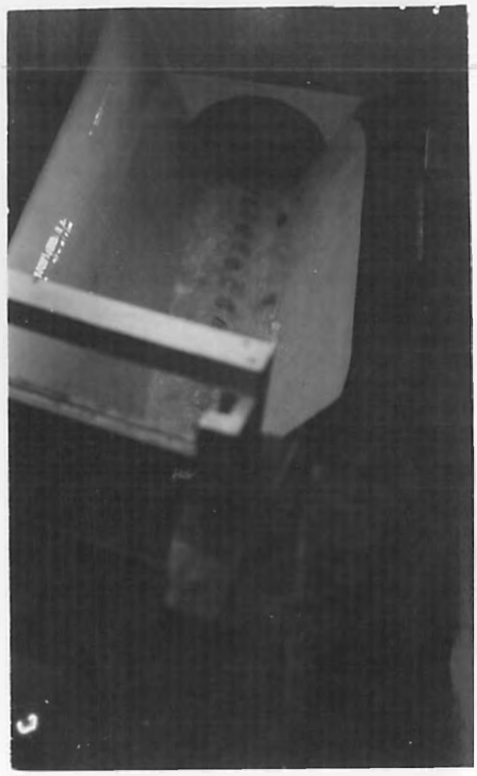
charges the action of these baffles was very satisfactory, as they reduced to a large extent the height of the ridge on the rear side of the spillway (Plates 24-B, C and D and Plate 25-A). A comparison of the action with and without sills for a discharge of 150,000 second-feet is shown on Plates 25-C and D respectively. For higher discharges the effect of the baffles was less, as the water passing over the crest represented a smaller proportion of the total discharge, and the deeper water in the channel dissipated more of the energy of the water coming over the crest before it reached the baffles. A flow of 200,000 second-feet is shown on Plate 25-B. The baffles used in the model were constructed of sheet iron and could not be duplicated in the prototype without a considerable thickening, which would have offered more obstruction to the flow, but as the improvement resulting from the baffles at high flows was slight, no further study was made along this line.

Types of Stoney Gate Piers

In this spillway as originally designed, the structure in which the gate was set presented sharp corners on both sides to the flow of the water entering the gate (Plate 26-A). The corner on the hill side did not cause much disturbance, as it did not project far out from the side of the channel leading toward the gate. On the other side, however, a pronounced swirl was set up, due largely to the water being drawn toward the gate at a sharp angle past the end

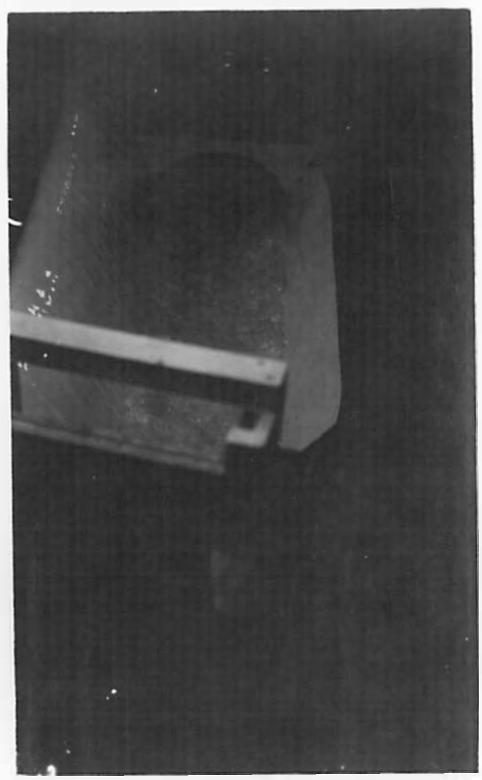


A-15FT. DENTATED BAFFLES



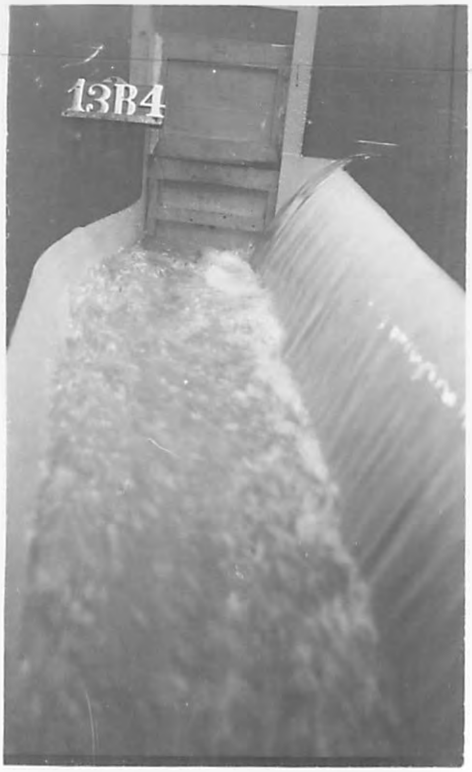
B-FLOW OF 20,000 SEC FT.
WITH BAFFLES

C-FLOW OF 40,000 SEC. FT.
WITH BAFFLES



D-FLOW OF 60,000 SEC FT
WITH BAFFLES

DENTATED BAFFLES ON CHANNEL FLOOR-MODEL C-3



A - FLOW OF 100,000 SEC. FT.
WITH BAFFLES

B - FLOW OF 200,000 SEC. [illegible]
WITH BAFFLES

C - FLOW OF 150,000 SEC. [illegible]
WITH BAFFLES

[illegible] FLOW OF [illegible]
WITHOUT [illegible]

DENTATED BAFFLES ON CHANNEL FLOOR - MODEL C-[illegible]

A - ENTRANCE CONDITIONS FOR STONEY GATE

B - SWIRL AROUND OUTSIDE GATE PIER

C - ROUNDED NOSE ON GATE PIER

D - FLOW WITH ROUNDED NOSE ON PIER

EFFECT OF STONEY GATE PIERS - MODEL C-3

of the structure supporting the gate. To correct this swirl a rounded nose was placed on the outer pier as shown on Plate 26-C, but this did not cause appreciable improvement as the water still flowed from around the end of the gate pier. A curved approach wall was then placed above the outer gate pier, as shown on Plate 27-A. This cut off the flow around the end of the pier and produced smooth entrance conditions as shown in View B. The exact position and dimensions of these walls together with a discussion of their effect on the flow through the gate are discussed in the later section of the report dealing with the hydraulics of the Stoney gate.

Shape of Stoney Gate Approach Channel

Experiments were undertaken to determine how much the cost of the spillway might be reduced by cutting down the excavation in the approach channel to the Stoney gate. Four shapes of approach channel were tested. The dimensions of these are shown in the section on Stoney gate hydraulics. The sides of the channels were formed by temporary sheet iron walls, as shown on Plate 27-C. In the original design there was a space of nearly dead water along the right side of the approach channel. With the channel narrowed, this condition was removed and the flow conditions of the water approaching the gate were not adversely affected. It was found that the size of the channel could be reduced nearly one-third without appreciable detriment to the action of the incoming water. Especially good conditions were

A-NO FLOW

B-DISCHARGE 200,000 SEC FT

CURVED APPROACH WALL ON OUTSIDE OF STONEY GATE PIER

C-TYPE I-NO FLOW

D-TYPE I-DISCHARGE WITH RESERVOIR AT ELEV 1232

EFFECT OF SHAPE OF GATE APPROACH CHANNEL-MODEL C-3 obtained by a combination of narrowing the approach channel on the right side and with the curved wall on the left side, as shown on Plate 28-A. This produced very smooth, well balanced entrance conditions, as shown on Views B and C. The discharge coefficients for these conditions will be discussed later.

Transition from Gate to Channel Section

The shape of the gate approach channel did not greatly influence the flow in the side channel section, as the water upstream from the gate was at greater than the critical depth. It passed through the critical depth at the gate section and downstream from that point was at less than critical depth. In this condition it is very difficult to change its direction without producing a disturbance. As shown (Figure 9), the transition from the gate section to the trapezoidal channel section takes place rather suddenly, giving rise to a rather rapid enlargement on the rear or hill side of the channel, and a change of direction between the gate section and the channel sections. These conditions combined to cause a wave to rise on the rear side of the channel just below the gate as described on page 27. By making the transition longer and the change of direction less abrupt, as shown on Plates 29-A and B, the height of the wave below the transition was considerably reduced, although it was still pronounced, as shown on View C.

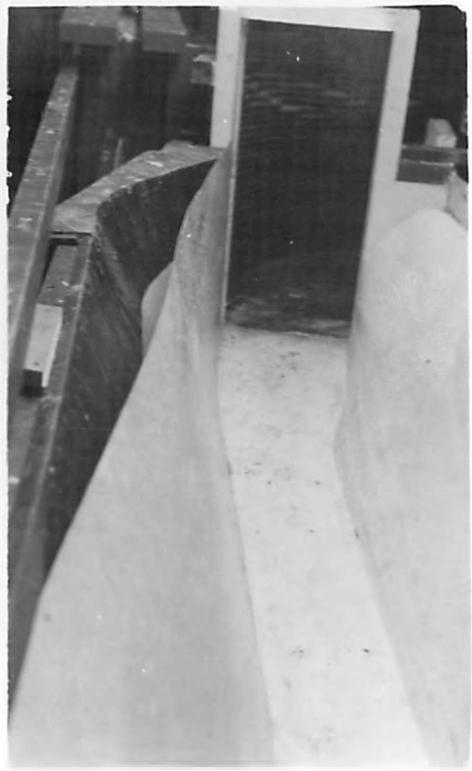
A-TYPE IV NARROWED CHANNEL WITH LEFT APPROACH WALL

B - TYPE IV
DISCHARGE 100,000 SEC FT

C - TYPE IV
DISCHARGE 200,000 SEC FT

EFFECT OF SHAPE OF GATE APPROACH CHANNEL - MODEL C - 3

A-LOOKING DOWNSTREAM



B-LOOKING UPSTREAM

MODEL WITH MORE GRADUAL TRANSITION

C-LOOKING DOWNSTREAM

D-LOOKING UPSTREAM

DISCHARGE WITH RESERVOIR AT ELEVATON 1232

EFFECT OF TRANSITION FROM GATE TO CHANNEL SECTION - MODEL C-3

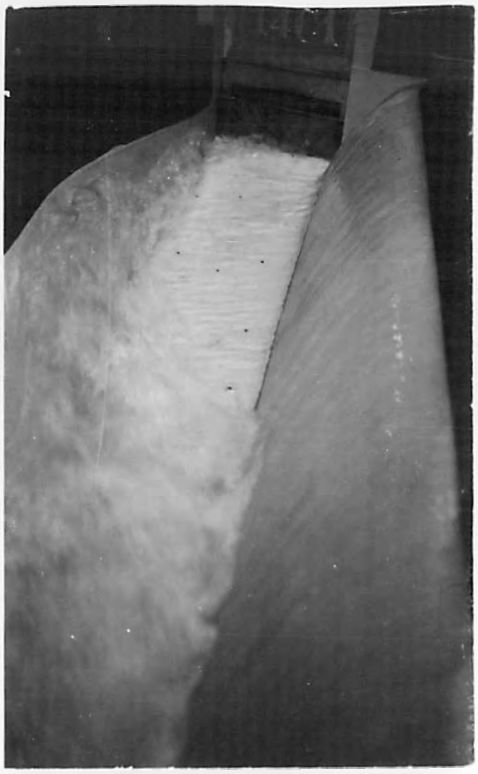
Stoney Gate Type Abandoned

Although several changes indicated that improvement was possible in the Stoney gate spillway as first designed, no major improvements were obtained. It is believed that by making the spillway channel straight and putting all the curvature into the gate approach channel where the flow was below the critical velocity and therefore easily changed in direction, much better flow conditions in the spillway and tunnel could have been obtained. This, however, would have greatly increased the cost. A decrease in the drop between the water surface in the reservoir and that in the channel,brought about by raising the channel floor, would have improved conditions, but would have decreased the head on the gate and required an increase in its already unprecedented size. Moreover, the geological conditions at the site of the Arizona spillway of this type were unfavorable, and as estimates showed that the cost of a spillway with drum gates was very favorable, attempts to further perfect the Stoney gate model were discontinued and further tests on it were made only to aid in the design of the drum gate type. When later experiments developed a satisfactory type of drum gate spillway, the idea of using the Stoney gate type was abandoned.

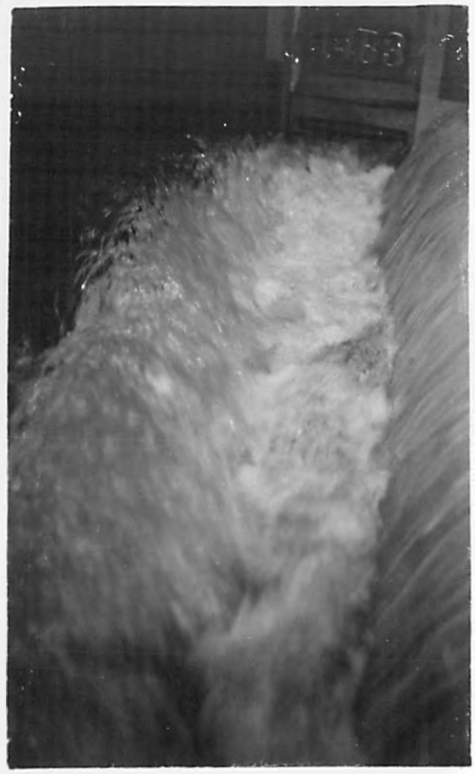
Effect of Channel Slope

The latter part of the testing of the Stoney gate model was carried on at the same time as those on the first drum gate model at the Montrose laboratory. When unsatisfactory conditions developed in the drum gate model, experiments were made on the Stoney gate model to indicate how steep a floor might be satisfactory in the drum gate spillway. In these tests the Stoney gate was blocked off and the model operated as a free crest side channel spillway. The slope of the floor of the Stoney gate model was relatively flat and the conditions of flow when it was operated with flow over the crest only are described on page 26. Experiments were made with bottoms having 15, 30 and 45-foot greater slopes than the original design by placing in the model a false floor of uniform slope with its upstream end at these distances above the floor of the original model.

Some of the results of these experiments are shown on Plate 30. At the lower discharges the flow over the crest swept entirely across the channel floor and formed a high ridge on the rear side of the spillway. At the maximum discharge all the various slopes produced very rough conditions in the spillway, as shown on Views B, C and D. The corresponding condition for the original model is shown on Plate 22-D. In general, the flatter the slope the better the resulting condition, since flatter slopes produced less longitudinal velocity in the spillway and therefore greater depths, which provided a greater mass of water in which the energy of the stream passing over the crest could dissipate itself.



A-BOTTOM AND UPPER END
SWEPT CLEAR



B-BOTTOM WITH 45FT RISE
DISCHARGE 200,000 SEC. FT.

C-BOTTOM WITH 3[illegible]FT RISE
DISCHARGE 200000 SEC FT

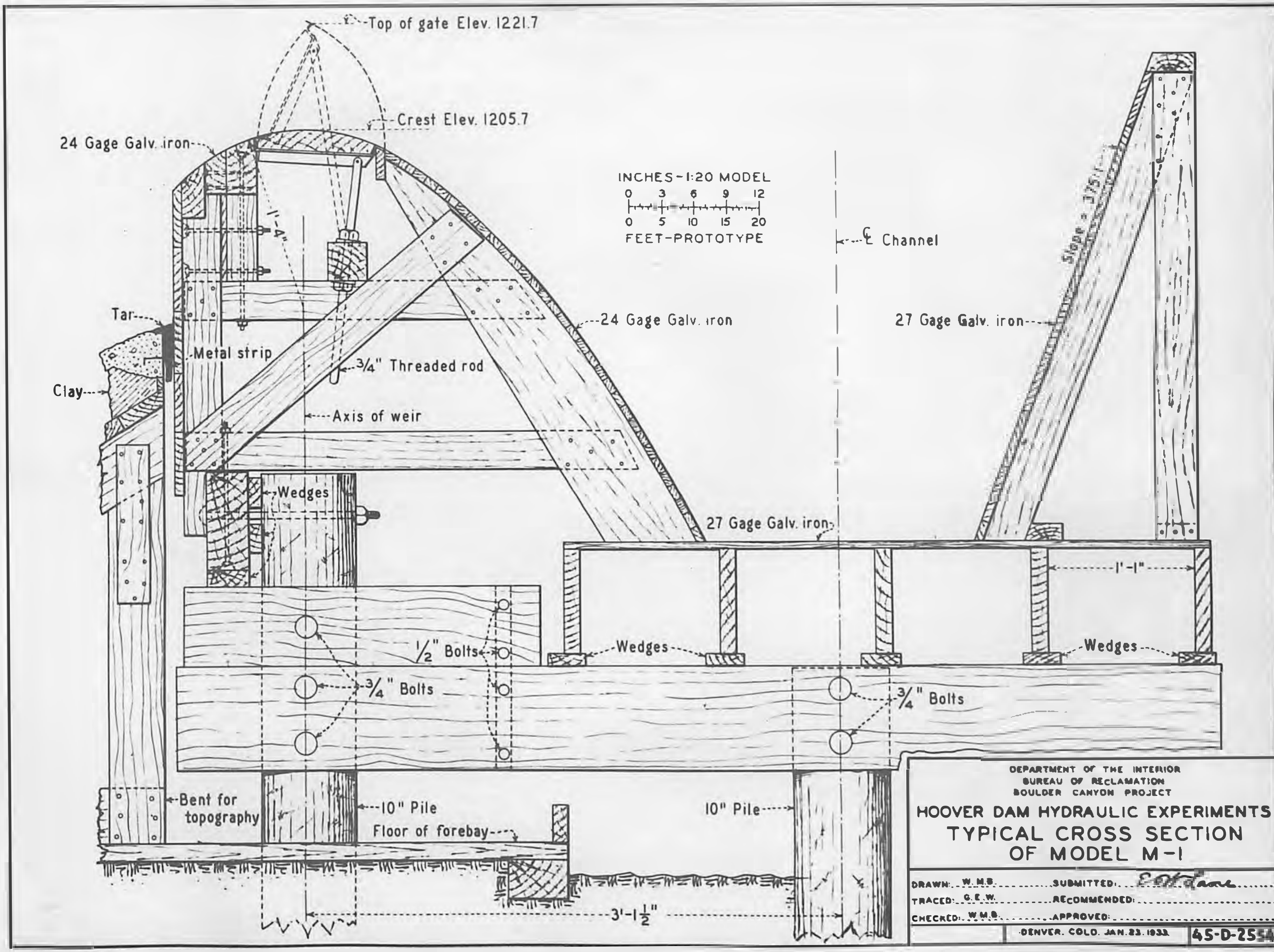
D-BOTTOM WITH 5 [illegible]
DISCHARGE [illegible] SEC FT

EFFECT OF SLOPING FLOORS IN MODEL C-3

THE FIRST DRUM GATE SPILLWAY MODEL M-1

Estimates of cost for spillways of various types showed that a spillway with drum gates could be built at a lower cost than the Stoney gate type and as the model experiments indicated that a spillway of the Stoney gate type would be somewhat unsatisfactory in its action, a model of a drum gate type of spillway was constructed on a 1:20 scale at the Montrose laboratory.

The method of construction of this model is shown on Figure 11. In order to prevent possible settlement of the model, it was supported on piles driven into firm earth or to shale bedrock. A sloping floor was first constructed at the level of the side channel bottom as shown on Plate 31-A. This floor carried the buttresses supporting the rear wall of the spillway channel (Plate 31-B) and one end of the ribs of the downstream face of the ogee crest (Plate 31-C). The other end of these ribs and the upstream face of the ogee crest were supported by a framing resting directly on the piling. The piers were constructed of solid blocks of wood accurately shaped. The gates were not built to represent the entire drum gate, but only the upper face, and were raised and lowered by means of adjustable screw supports inside the ogee crest, as shown on Figure 11. This permitted a much simpler construction than would be required by an exact duplication of the drum gate, and from the standpoint of hydraulic action produced the same result. The completed ogee and channel are shown on Plates 31-D and 34-A.



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

HOOVER DAM HYDRAULIC EXPERIMENTS

TYPICAL CROSS SECTION OF MODEL M-1

DRAWN: W.M.B. SUBMITTED: E.O. Lane
TRACED: G.E.W. RECOMMENDED:
CHECKED: W.M.B. APPROVED:
DENVER, COLO. JAN. 23, 1933 45-D-2554

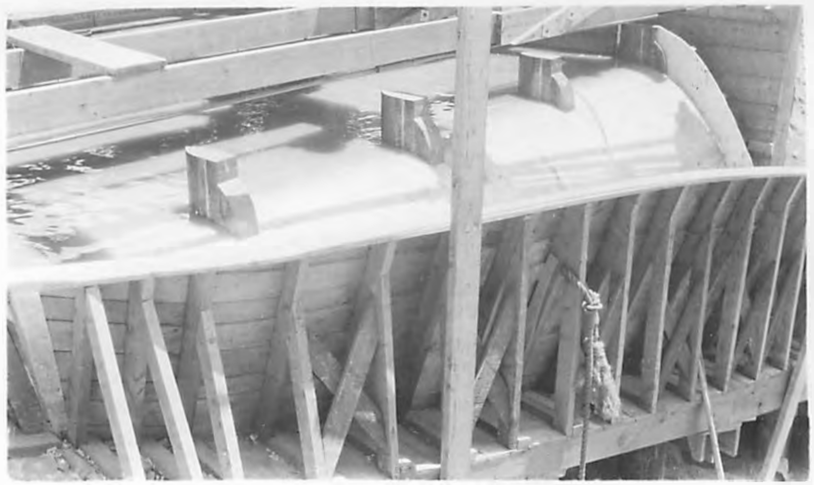
FIG. 11

CONSTRUCTION OF DRUM GATE SPILLWAY – MODEL M-1

A-FLOOR SUPPORTING REAR WALL AND OGEE CREST RIBS

B-CHANNEL PARTIALLY COMPLETE

C-RIBS ON OGEE CREST



D-COMPLETED CHANNEL

PLATE 31

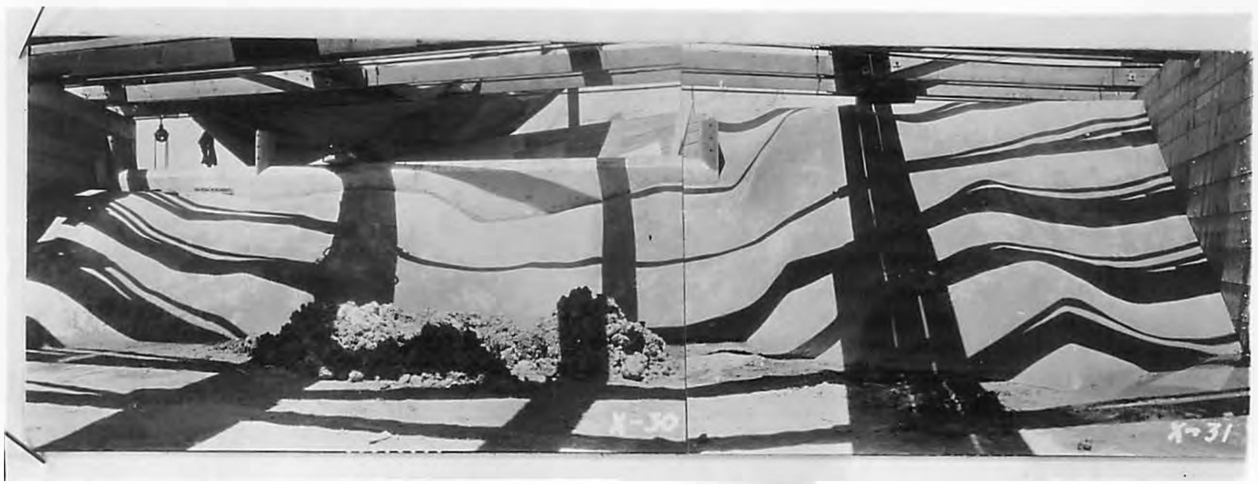
The model of the mountain-side in the vicinity of the spillway was constructed to form the bulkhead across the end of the forebay section of the laboratory flume. It was supported by a sloping floor resting on buttresses carried in turn by the floor of the forebay. In effect, this model topography support formed a framed timber dam across the end of the forebay. Templates cut to represent the topography of the mountain-side were placed on the sloping floor, as shown on Plate 32-A, and the space between the templates was filled with rammed clay (Plate 32-B). The clay was covered with a layer of tar to prevent the alkali which the clay contained from injuring the mortar cover. This was then covered with a 2-inch layer of dense mortar reinforced with chicken-wire, the thickness of the mortar being gaged by means of nails projecting 2 inches above the wooden templates (Plate 32-C). A water seal strip of sheet iron was provided to join the mortar coat to the floor and sides of the forebay and to the ogee crest. A space filled with tar was provided where the mortar joined the sides of the forebay and the ogee crest to permit the model topography to settle independently of them without danger of cracking the mortar. This model topography proved to be very satisfactory and with minor changes was used also for models M-3 and M-5.

The model of the transition from the trapezoidal model section to the circular tunnel section was made by forming splined redwood staves on the inside of accurately cut wooden collars, in a

MODEL OF MOUNTAIN SIDE TOPOGRAPHY - MODEL M-I-3 AND 5

A-SLOPING FLOOR SUPPORTING TOPOGRAPHY WITH TEMPLATES

B-RAMMED CLAY FILLING AND WATER SEAL STRIPS



C-COMPLETED MODEL OF TOPOGRAPHY

PLATE 32

manner similar to the transition of the Stoney gate model previously described. In order to prevent distortion from the forces arising from the bending of the staves, these collars had to be heavy and securely held in position by an outside boxing, reinforced with iron rods. This transition was supported on piles driven to shale.

The contracting circular section of the inclined tunnel was formed by using wedge-shaped staves in a wood-stave pipe. The pipe was carried in a cradle supported at its upper end on the piles carrying the lower end of the transition section, and at the other by the concrete block forming the model of the vertical bend in the tunnel (Plate 33-A). The concrete block was connected to the wood-stave pipe above and below it by means of a steel sleeve. Two windows fitted with removable concrete plugs were cast into the top of this bend in order that the action of the water in it might be observed.

Because of the topography at the laboratory site, the model of the tunnel was built in a deep trench. The horizontal tunnel and horizontal bend was formed of 30-inch wood-stave pipe. The pipe throughout was carried in collars and braced to the sides of the trench to prevent displacement in case of an accident which might fill the trench with water and tend to float the pipe. The horizontal bend had a radius of 41 feet (model dimension) which was much smaller than ordinarily considered feasible in a wood-stave pipe. By closely spacing collars and soaking the staves, the staves were, with considerable

MODEL OF INCLINED TUNNEL, VERTICAL BEND,
HORIZONTAL TUNNEL AND HORIZONTAL BEND – MODEL M-1-3 AND 5

A–INCLINED TUNNEL & VERTICAL BEND

B–CONSTRUCTION OF HORIZONTAL BEND

C–COMPLETED TUNNEL MODEL

PLATE 33

effort, jacked into place, temporarily nailed to the collars, and finally banded into a complete and satisfactory pipe, with the breakage of only a few staves. In this construction all the staves were carried along simultaneously around the bend, in the manner shown on Plate 33-B. For the benefit of anyone who may have occasion to construct a sharp curve with wood-stave pipe it may be said, however, that better results would probably have been secured if the center bottom stave had been carried entirely around the bend and for some distance beyond first, and adjacent staves be placed next throughout this entire length, each successive stave being carried the entire length before another is added. The completed model tunnel is shown on Plate 33-C. Windows were placed in the top of the pipe at intervals in order that the conditions of flow in it might be observed.

Except for a movement of the inclined tunnel and vertical bend a short distance downstream, the model of the entire tunnel remained the same for Models M-1, M-3 and M-5. In order to guard against sliding banks resulting from the slight leakage from the forebay and model, retaining walls were placed in the trench at the vertical bend, and a gravel-covered tile collecting drain was run along the bottom of the trench to carry any leakage or seepage back to the irrigation ditch.

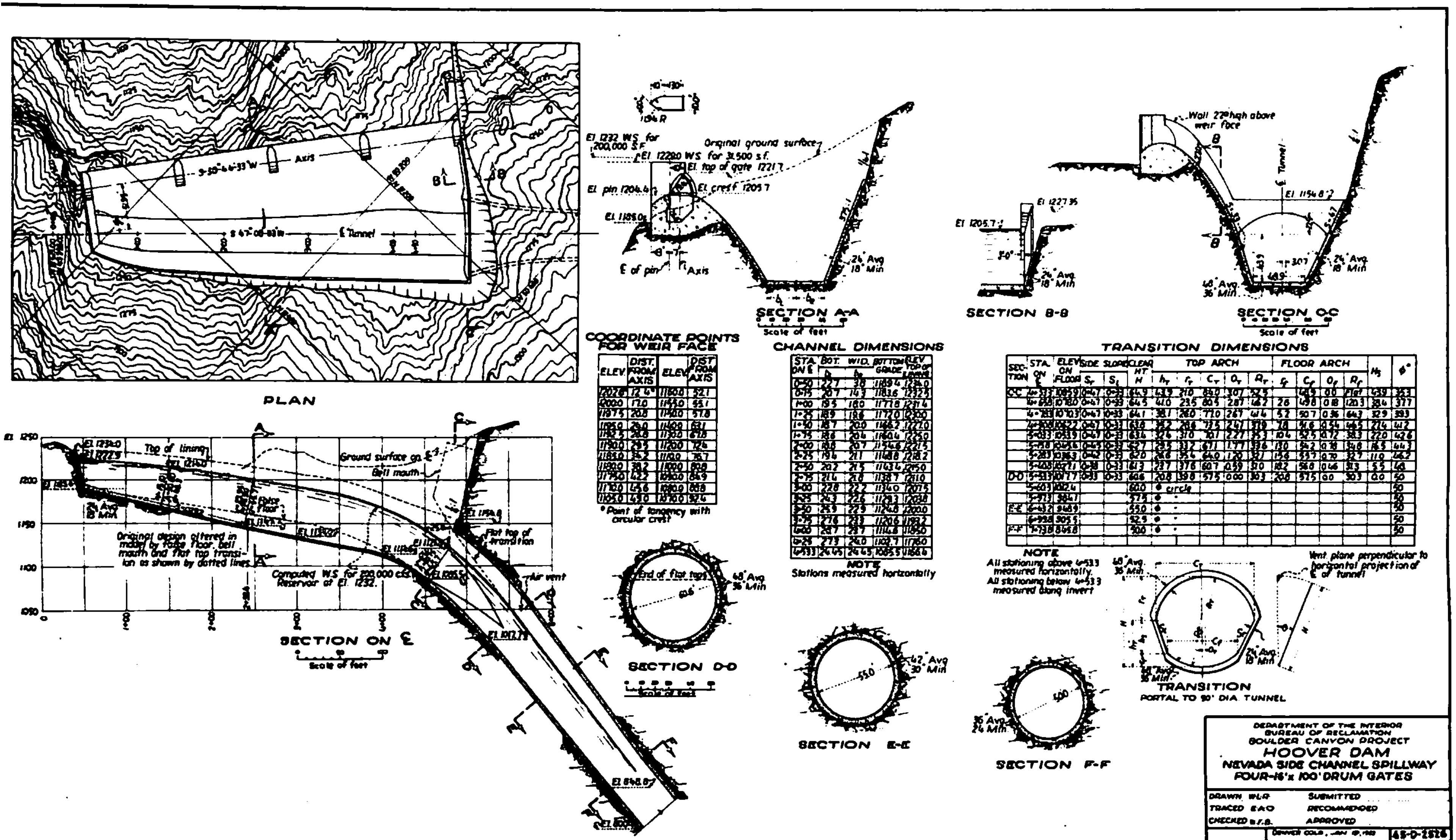
Tests on the M-1 Model

Extensive experiments were carried out on the M-1 model. A large variety of devices was tried to improve the flow conditions in the spillway channel and down the tunnel. These are described in detail in the following paragraphs. To give the details of these setups would unduly expand this report, but in order to preserve a record a copy of a sketch of each setup is filed with the office file copies of this report. Quantitative experiments were made on the depth of flow in the spillway channel and on the discharge coefficients of the spillway crest. The results of these are discussed in detail in the later sections of this report which deal with these phases of spillway design.

Results with Original Design of Model M-1

The prototype for model M-1 is shown on Figure 12. It had four drum gates each 100 feet long with 10-foot piers between. The channel bottom had widths varying from 26.5 to 52.4 feet and sloped rather steeply, having a fall of 104 feet in the 403-foot length to the beginning of the transition section. The rear wall had a slope of 0.375:1. The dimensions were computed according to the theory developed by Hinds, as described on page 19.

Plate 34-A shows the completed model. The action of the model was not favorable. Because of the steep slope, the velocity of flow along the channel was great, and the cross section of the flowing water was relatively small. The depth of flow was therefore less than in the models previously investigated. The average fall of the water passing over the crest between the reservoir level and



COORDINATE POINTS FOR WEIR FACE

ELEV	DIST. FROM AXIS	ELEV	DIST. FROM AXIS
1207.8*	12.4*	1160.0	52.1
1200.0	17.0	1155.0	55.1
1197.5	20.8	1150.0	57.8
1195.0	24.0	1140.0	63.1
1192.5	26.8	1130.0	67.8
1190.0	29.5	1120.0	72.6
1185.0	34.2	1110.0	76.7
1180.0	38.2	1100.0	80.8
1175.0	42.2	1090.0	84.9
1170.0	45.6	1080.0	88.8
1165.0	49.0	1070.0	92.4

*Point of tangency with circular crest

CHANNEL DIMENSIONS

STA. ON ℄	BOT. WID. b_1	BOT. WID. b_2	BOTTOM GRADE	ELEV. TOP OF LINING
0+50	22.7	3.8	1189.4	1234.0
0+75	20.7	14.3	1183.6	1232.5
1+00	19.5	16.0	1177.8	1231.4
1+25	18.9	18.6	1172.0	1230.0
1+50	18.7	20.0	1166.2	1227.0
1+75	18.6	20.4	1160.4	1225.0
2+00	18.8	20.7	1154.6	1221.5
2+25	19.6	21.1	1148.8	1218.2
2+50	20.2	21.5	1143.4	1215.0
2+75	21.4	21.8	1138.7	1211.0
3+00	22.8	22.2	1134.0	1207.5
3+25	24.3	22.6	1129.3	1203.8
3+50	25.9	22.9	1124.6	1200.0
3+75	27.6	23.3	1120.6	1193.2
4+00	28.7	23.7	1116.8	1186.0
4+25	27.3	24.0	1102.7	1176.0
4+533	24.45	24.45	1085.5	1166.4

NOTE
Stations measured horizontally

TRANSITION DIMENSIONS

SECTION	STA. ON ℄	ELEV. ON FLOOR	SIDE SLOPE S_r	SIDE SLOPE S_l	CLEAR HT. H	TOP ARCH h_T	TOP ARCH r_T	TOP ARCH C_T	TOP ARCH O_T	TOP ARCH R_T	FLOOR ARCH r_F	FLOOR ARCH C_F	FLOOR ARCH O_F	FLOOR ARCH R_F	H_S	$\phi°$
C-C	4+533	1085.9	0.47	0.33	64.9	43.9	21.0	86.0	30.7	52.5		48.9	0.0	Flat	43.9	35.3
	4+658	1078.0	0.47	0.33	64.5	41.0	23.5	80.5	28.7	46.2	2.6	49.8	0.18	120.3	38.4	37.7
	4+783	1070.3	0.47	0.33	64.1	38.1	26.0	77.0	26.7	41.4	5.2	50.7	0.36	64.3	32.9	39.3
	4+908	1062.2	0.47	0.33	63.6	35.2	28.6	73.5	24.7	37.9	7.8	51.6	0.54	46.5	27.4	41.2
	5+033	1053.9	0.47	0.33	63.4	32.6	31.0	70.1	22.7	35.3	10.4	52.5	0.72	38.3	22.0	42.6
	5+158	1045.6	0.45	0.33	62.7	29.5	33.2	67.1	1.77	33.6	13.0	54.2	0.18	34.6	16.5	44.3
	5+283	1036.3	0.42	0.33	62.0	26.6	35.4	64.0	1.20	32.7	15.6	55.7	0.70	32.7	11.0	46.2
	5+408	1027.1	0.38	0.33	61.3	23.7	37.6	60.7	0.59	32.0	18.2	56.0	0.46	31.3	5.5	48
D-D	5+533	1017.7	0.33	0.33	60.6	20.8	39.8	57.5	0.00	30.3	20.8	57.5	0.0	30.3	0.0	50
	5+603	1012.4			60.0	●	circle									50
	5+713	984.7			57.5	●	"									50
E-E	6+432	948.9			55.0	●	"									50
	6+928	905.5			52.5	●	"									50
F-F	7+738	849.8			50.0	●	"									50

NOTE
All stationing above 4+533 measured horizontally.
All stationing below 4+533 measured along invert

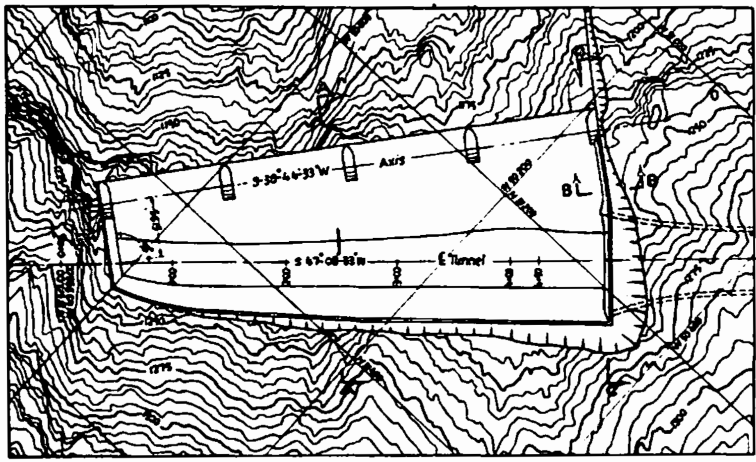
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
NEVADA SIDE CHANNEL SPILLWAY
FOUR-16'x 100' DRUM GATES

DRAWN W.L.R. SUBMITTED
TRACED E.A.O. RECOMMENDED
CHECKED B.F.B. APPROVED
DENVER COLO., JAN 19, 1933 45-D-2526

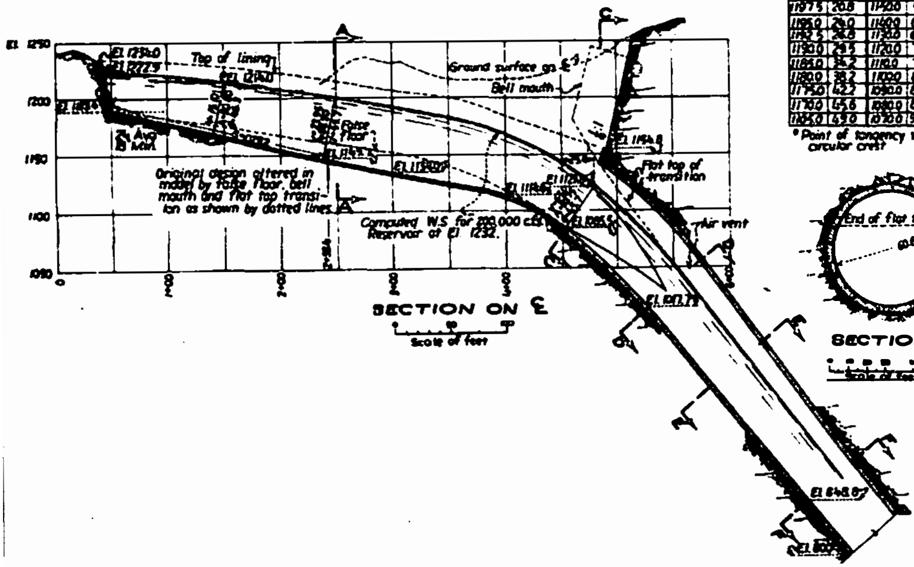
FIG. 12

the channel level was also somewhat greater. This greater fall tended to make the water pile up on the side of the channel opposite the crest to a higher elevation. The smaller depth of flow in the channel also offered less resistance to this tendency, since the greater the depth of water the greater is the tendency to dissipate the energy of the overflowing stream. The combination of greater fall and smaller depth of flow resulted in a high wave on the back side of the channel, which caused considerable impact on the end wall at the portal of the tunnel for discharges of 100,000 second-feet and over. The action of the model is shown on Plates 34 and 35. Plate 34-B shows the conditions for a 35,000-second-foot discharge. This would be approximately the conditions with the largest flood of which there is a record. At the upper end of the channel at discharges up to 150,000 second-feet, the water flowing over the crest swept the floor clear and formed on the back side of the channel a ridge resembling a wave on an ocean beach just as it breaks, shown on Plates 34-B and C and Plates 35-A and B. The conditions for flows of 150,000 and 200,000 second-feet, the latter being the capacity for which the spillway was designed, are shown on Plates 35-C and D.

The relatively large drop of the water between the reservoir and the channel together with the slight depth in the channel caused a decided spiral motion in the flow down the channel, which

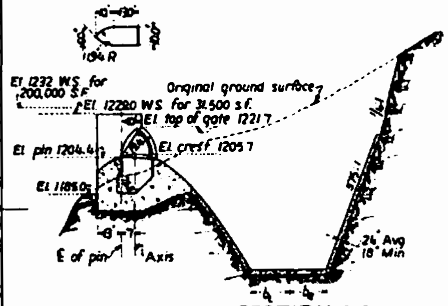


PLAN



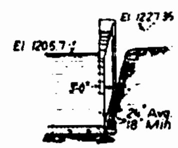
SECTION ON ℄

Scale of feet

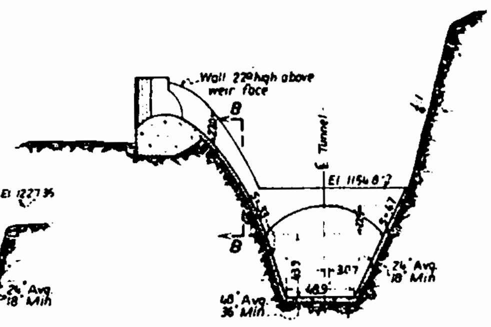


SECTION A-A

Scale of feet



SECTION B-B



SECTION C-C

Scale of feet

COORDINATE POINTS FOR WEIR FACE

ELEV.	DIST. FROM AXIS	ELEV.	DIST. FROM AXIS
1207.8*	12.4*	1160.0	52.1
1200.0	17.0	1155.0	55.1
1197.5	20.8	1150.0	57.8
1195.0	24.0	1140.0	63.1
1192.5	26.8	1130.0	67.8
1190.0	29.5	1120.0	72.6
1185.0	34.2	1110.0	76.7
1180.0	38.2	1100.0	80.8
1175.0	42.2	1090.0	84.9
1170.0	45.6	1080.0	88.8
1165.0	49.0	1070.0	92.4

* Point of tangency with circular crest

CHANNEL DIMENSIONS

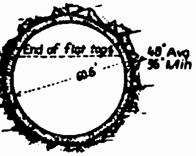
STA. ON ℄	BOT. WID. b_1	BOT. WID. b_2	BOTTOM GRADE	ELEV. TOP OF LINING
0+50	22.7	3.8	1189.4	1234.0
0+75	20.7	14.3	1183.6	1232.5
1+00	19.5	18.0	1177.8	1231.4
1+25	18.9	19.6	1172.0	1230.0
1+50	18.7	20.0	1166.2	1227.0
1+75	18.6	20.4	1160.4	1225.0
2+00	18.8	20.7	1154.6	1221.5
2+25	19.4	21.1	1148.8	1218.2
2+50	20.2	21.5	1143.4	1215.0
2+75	21.4	21.8	1139.7	1211.0
3+00	22.8	22.2	1134.0	1207.5
3+25	24.3	22.6	1128.3	1203.8
3+50	25.9	22.9	1124.8	1200.0
3+75	27.6	23.3	1120.6	1193.2
4+00	28.7	23.7	1114.8	1185.0
4+25	27.3	24.0	1102.7	1178.0
4+33.3	24.45	24.45	1085.5	1166.4

NOTE
Stations measured horizontally

TRANSITION DIMENSIONS

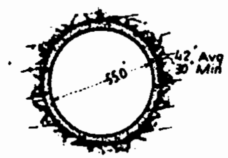
SECTION	STA. ON ℄	ELEV. ON FLOOR	SIDE SLOPE S_T	SIDE SLOPE S_f	CLEAR HT. H	TOP ARCH h_T	TOP ARCH r_T	TOP ARCH C_T	TOP ARCH O_T	TOP ARCH R_T	FLOOR ARCH r_f	FLOOR ARCH C_f	FLOOR ARCH O_f	FLOOR ARCH R_f	H_S	$\phi°$
C-C	4+53.3	1085.9	0.67	0.33	64.9	43.9	21.0	84.0	30.7	52.5		48.9	0.0	Flat	43.9	35.3
	4+69.8	1078.0	0.67	0.33	64.5	41.0	23.5	80.5	28.7	46.2	2.6	49.8	0.18	120.3	38.4	37.7
	4+85.3	1070.3	0.67	0.33	64.1	38.1	26.0	77.0	26.7	41.4	5.2	50.7	0.36	64.3	32.9	39.3
	5+00.8	1062.2	0.67	0.33	63.8	35.2	28.6	73.5	24.7	37.9	7.8	51.6	0.54	46.5	27.4	41.2
	5+03.3	1053.5	0.67	0.33	63.4	32.6	31.0	70.7	22.7	35.3	10.4	52.5	0.72	38.3	22.0	42.6
	5+15.8	1045.6	0.65	0.33	62.7	29.5	33.2	67.1	17.7	33.6	13.0	54.2	0.79	34.0	16.5	44.3
	5+28.3	1036.3	0.62	0.33	62.0	26.6	35.4	64.0	12.0	32.1	15.6	55.7	0.79	32.7	11.0	46.2
	5+40.8	1027.1	0.38	0.33	61.3	23.7	37.6	60.7	0.59	31.0	18.2	56.0	0.46	31.3	5.5	48
D-D	5+53.3	1017.7	0.33	0.33	60.6	20.8	39.8	57.5	0.00	30.3	20.8	57.5	0.0	30.3	0.0	50
	5+60.3	1012.4			60.0	⌀ circle										50
	5+92.3	984.7			57.5	⌀ "										50
E-E	6+43.2	948.9			55.0	⌀ "										50
	6+93.8	905.5			52.5	⌀ "										50
F-F	7+73.8	849.8			50.0	⌀ "										50

NOTE
All stationing above 4+53.3 measured horizontally.
All stationing below 4+53.3 measured along invert

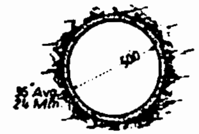


SECTION D-D

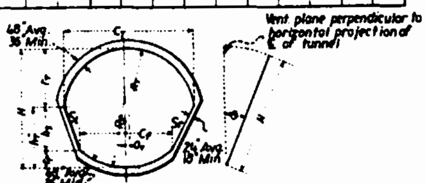
Scale of feet



SECTION E-E



SECTION F-F



TRANSITION
PORTAL TO 50' DIA. TUNNEL

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
NEVADA SIDE CHANNEL SPILLWAY
FOUR-16'x 100' DRUM GATES

DRAWN W.L.R.	SUBMITTED
TRACED E.A.O.	RECOMMENDED
CHECKED B.F.B.	APPROVED
	DENVER COLO., JAN 19, 1932
	45-D-2526

FIG. 12

the channel level was also somewhat greater. This greater fall tended to make the water pile up on the side of the channel opposite the crest to a higher elevation. The smaller depth of flow in the channel also offered less resistance to this tendency, since the greater the depth of water the greater is the tendency to dissipate the energy of the overflowing stream. The combination of greater fall and smaller depth of flow resulted in a high wave on the back side of the channel, which caused considerable impact on the end wall at the portal of the tunnel for discharges of 100,000 second-feet and over. The action of the model is shown on Plates 34 and 35. Plate 34-B shows the conditions for a 35,000-second-foot discharge. This would be approximately the conditions with the largest flood of which there is a record. At the upper end of the channel at discharges up to 150,000 second-feet, the water flowing over the crest swept the floor clear and formed on the back side of the channel a ridge resembling a wave on an ocean beach just as it breaks, shown on Plates 34-B and C and Plates 35-A and B. The conditions for flows of 150,000 and 200,000 second-feet, the latter being the capacity for which the spillway was designed, are shown on Plates 35-C and D.

The relatively large drop of the water between the reservoir and the channel together with the slight depth in the channel caused a decided spiral motion in the flow down the channel, which

A-NO FLOW

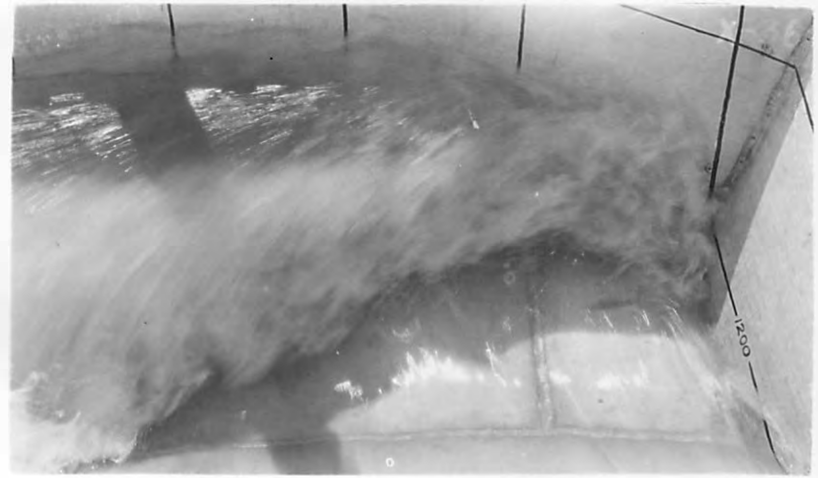
B-DISCHARGE 35000 SEC. FT.

C-DISCHARGE 50,000 SEC. FT.

FLOW IN ORIGINAL DESIGN DRUM GATE SPILLWAY-MODEL M-1

FLOW IN ORIGINAL DESIGN DRUM GATE SPILLWAY - MODEL M-

A- DISCHARGE 100,000 SEC FT



B- WAVE AT UPPER END-DISCHARGE OF 100,000 SEC. FT.

C- DISCHARGE 150,000 SEC. FT

D- DISCHARGE 200000 SEC FT - THE DESIGNED CAPACITY set up considerable commotion in the transition section at the top of the inclined tunnel. Severe splashing also resulted at the vertical bend at the bottom of the inclined tunnel. Water began to splash out of the lower of the two windows in the bend at a discharge of 110,000 and severe splashing occurred at 120,000 second-feet. With a discharge of 200,000 second-feet there was a very disturbed condition of flow at this point and the splashing together with the air brought down by the water exerted considerable pressure on the windows in the top of the bend. The flow through the pipe, as observed through the windows, was very much disturbed and near the end of the tunnel gave rise to severe vibration.

The cause of this vibration or thumping is not certain but seems to be related to the well-known fact that resistance to flow of water in pipes decreases as the depth of flow increases until a depth of about 0.95 of the diameter is reached, beyond which point the resistance rapidly increases. In an inclined pipe into which water is emitted in gradually increasing quantity, when a depth of 0.87 of the diameter is reached, if the water splashes to the top it suddenly fills the whole pipe and in this condition the pipe will not carry as great a discharge as when flowing 0.87 full. As the water flowed down the tunnel its velocity was reduced by friction and the depth of flow increased. Near the downstream end, for discharges of near 200,000 second-feet, the depth approached 0.87 of the diameter of the pipe. When a surge or splash occurred the flow suddenly jumped to the full condition, instantly increasing the resistance to flow and causing a hammer or thump in the pipe. The flow did not continue at the full depth, however, but quickly dropped down to part depth flow. This may have been due to air brought in at the upper end of the tunnel forcing itself out at the lower end. Another possible explanation is that the sudden increase in resistance tended to form temporarily a hydraulic jump, which could not maintain itself because the friction loss to the end of the pipe was not sufficient to create the required back pressure. These changes between full and partially full flow gave rise to a series of thumps or blows of varying intensity at irregular intervals, which were felt only near the downstream end of the pipe. For the original design a severe blow came about once every ten seconds. In the prototype, these disturbances would no doubt set up a perceptible jar in the rock of the canyon wall, which would be undesirable.

Raised Floor in M-1 Model

In order to reduce the height of the wave on the back wall of the channel, the floor of the channel was raised a maximum of 19 feet at station 3+00 and sloped from this point to the original bottom levels at the upper end of the channel and down to the beginning of the transition. The grade of this channel is shown on Figure 12. This decreased the slope of the upper end of the channel, at the same time decreasing the height of the fall of the water over the crest.

The conditions of flow in the channel were improved but there was still considerable splashing at the portal of the tunnel. The effect of the raised floor with a discharge of 100,000 second-feet may be seen by a comparison of Plate 35-A with Plate 40-B. (In the latter view the spillway was provided with a different shaped tunnel portal and transition but these did not affect flows as low as 100,000 second-feet.)

The smoother condition of flow is evident, particularly near the downstream end of the channel and in the improvement of the wave formation at the upper end. The water, however, still rose to an undesirable height on the rear wall, but on account of these improvements, in all the succeeding tests on this model the raised floor was used.

Although the original design was computed to have a cross section just sufficient to carry the 200,000 second-foot discharge without causing sufficient submergence of the ogee crest at the upstream end of the channel to reduce the flow over it, it was found that the floor could be raised and the flow over the crest at the upper end still maintained. This was probably due to the fact that the water level determined in the computations is the mean water level in the channel cross section, while the level which controlled the submergence effect was probably the level on the side of the channel near the weir, which was considerably lower than the mean level.

Coping on the Back Wall

In order to reduce the height to which the water rose against the back wall a curved-faced baffle in the form of a coping was placed along this wall, with the top of the curved face about 20 feet above the floor. The position of this baffle is shown on Plate 36-D. This tended to throw the water out toward the center of the channel and reduced the height of the wave against the wall. It considerably improved the conditions at the entrance to the tunnel and reduced the disturbances throughout the tunnel. The improvement due to this baffle at 100,000 second-feet discharge may be seen by comparing Plate 38-B with Plate 40-D. In the former, the height of the wave against the rear wall was materially less.

Flat Top Contraction in the Transition

The original design contemplated a free surface for the water flowing in the spillway and tunnel. On account of the turbulent conditions of flow, this surface was very rough and produced a great deal of splashing and impact. In order to smooth out the water surface a contraction was placed in the transition section as shown in Figure 12 and on Plates 37-A and 40-A. This consisted of a flat top in the transition gradually contracting the waterway from zero at the upper end of the transition to a distance 20 feet down from the top at the lower end. A vent was built at the lower end to admit air above the water in the inclined tunnel downstream from the contraction. For discharges over about 175,000 second-feet this formed

A-OUTLET OF TUNNEL-ORIGINAL DESIGN MODEL M-I
TUNNEL FLOWING FULL DURING THUMP

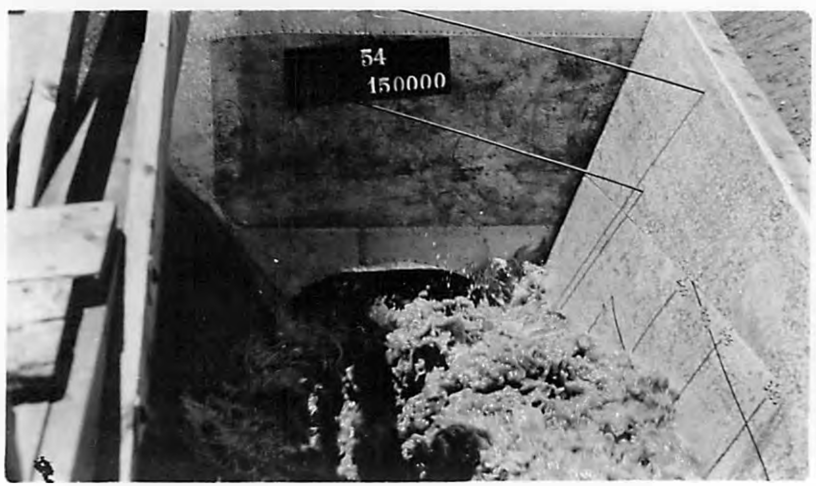
B-OUTLET OF TUNNEL FLOWING PARTLY FULL-MODEL M-I

C-CHANNEL WITH RAISED FLOOR AND COPING BAFFLE
MODEL M-I

FLOW WITH RAISED BOTTOM, COPING BAFFLE AND
20 FT. FLAT TOP IN TRANSITION-DRUM GATE SPILLWAY-MODEL M-1

A-NO DISCHARGE

B-DISCHARGE 150,000 SEC. FT.



C-DISCHARGE 150,000 SEC. FT.

D-DISCHARGE 180,000 SEC. FT.

A-DISCHARGE 180.000 SEC FT

B-DISCHARGE 200.000 SEC FT

C-TUNNEL OUTLET DURING THUMP

FLOW WITH RAISED BOTTOM, COPING BAFFLE AND 15 FT. FLAT TOP IN TRANSITION AND IMPROVED PORTAL SHAPE-DRUM GATE SPILLWAY-MODEL M-1 an orifice with a horizontal top edge at the upper end of the inclined tunnel and caused the water to start down the tunnel with a flat upper surface, thus largely reducing the splashing in the pipe. When this contraction acted as an orifice it raised the surface levels in the side channel, reducing the drop of the water falling over the crest and increasing the depth in the channel, both of which effects tended to improve the flow conditions. Plate 37 shows the conditions of flow in the channel with this contraction acting in conjunction with the raised bottom and the coping on the back wall. Although the contraction considerably raised the water level in the channel at the lower end, it did not back up enough at the upper end to reduce the flow over the spillway crest. The conditions of flow in the pipe were considerably improved, both in regard to the splashing at the vertical bend and the thumping at the end of the pipe. The splashing from the lower window in the vertical bend started at 130,000 second-feet.

After the improved portal mentioned in the following paragraph was installed, a contraction in the transition, which restricted the opening less than that previously described, was also tested. The flat top in this case extended 15 feet down from the top of the transition section as compared with 20 feet in the previous case. The conditions of flow in the channel, vertical bend, and at the end of the tunnel were all somewhat less desirable than with the greater contraction. In all the following experiments, therefore, the greater contraction was used.

Improved Portal Shape

In the original design the face of the portal was at right angles to the tunnel and there was considerable impact on it. To improve this entrance the portal was altered to present the form of bell mouth, as shown on Plate 39-A. Since the tunnel portal as originally built was practically opposite the end of the ogee crest, these bell mouths extended beyond the end of the crest and it was necessary to leave a space on the crest side of them through which the flow from the end of the crest could pass. This would not be a practical form of construction for the prototype, but the set-up served to indicate what results might be secured with a bell mouth entrance and if it were installed in the prototype the necessary adjustments could be made. The larger bell mouth improved entrance conditions considerably and it was used throughout the remainder of the tests. The flow conditions with this portal for various discharges are shown on Plate 39. This set-up was also tested with the coping baffle removed, as shown on Plate 40, but the conditions in the channel, vertical bend, and at the end of the tunnel were all less satisfactory and the coping baffle was therefore reinstalled.

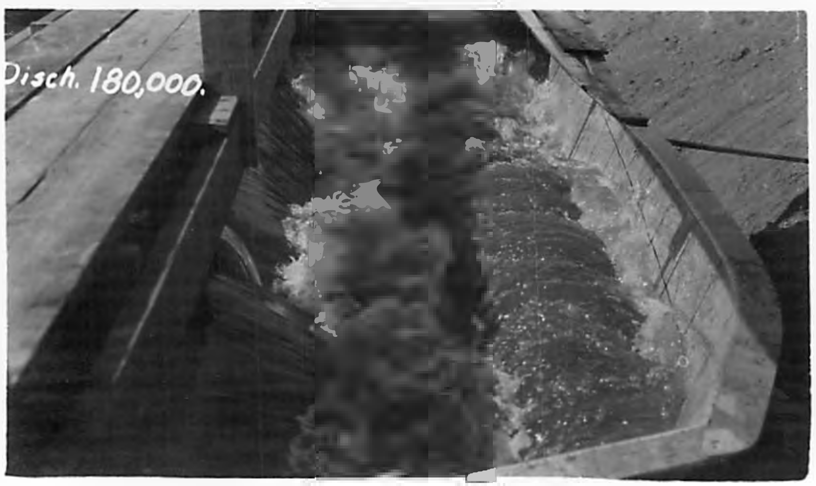
Vane in Inclined Pipe

The water flowing over the ogee crest imparted to the water in the side channel a spiral motion, which persisted for a long distance down the tunnel, and caused at least a portion of the disturbance in the flow. In order to reduce this spiral motion, and hence

FLOW WITH RAISED BOTTOM, COPING BAFFLE,
20 FT. FLAT TOP IN TRANSITION AND IMPROVED PORTAL SHAPE.
DRUM GATE SPILLWAY - MODEL M-1

A - NO DISCHARGE

B - DISCHARGE 100,000 SEC. FT.



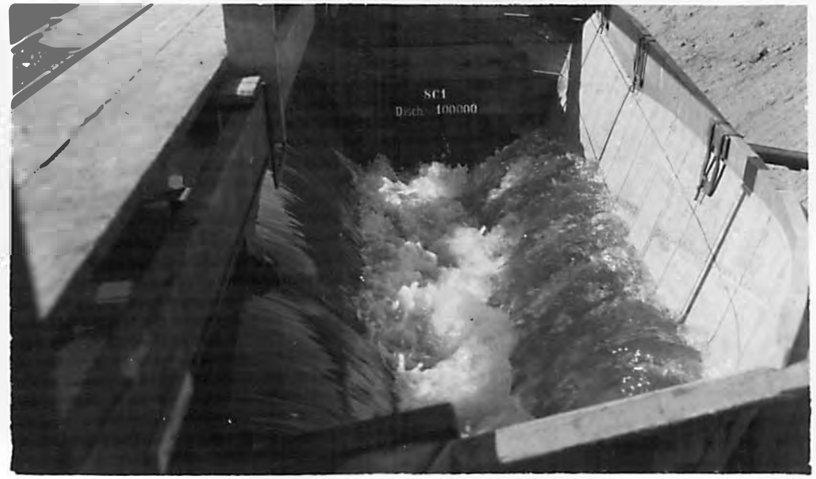
C - DISCHARGE 180,000 SEC. FT.

D - DISCHARGE 200000 SEC. FT.

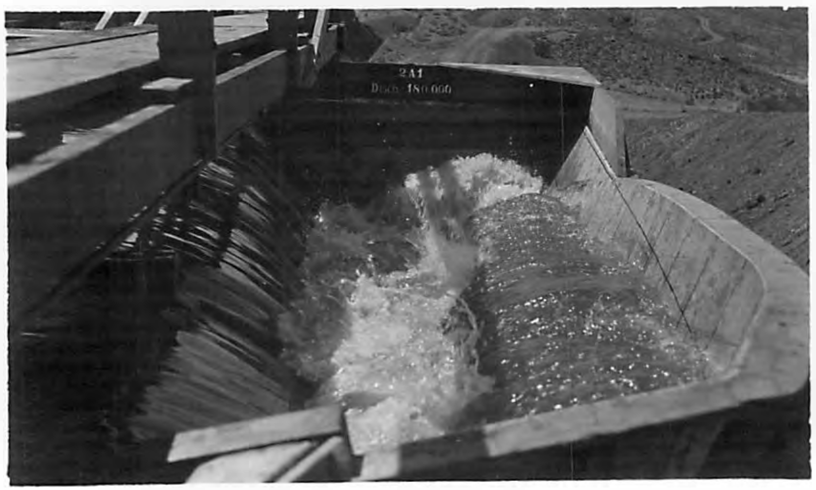
PLATE 39

FLOW WITH RAISED BOTTOM, 20 FT. FLAT TOP IN TRANSITION,
IMPROVED PORTAL SHAPE BUT WITHOUT COPING BAFFLE.
DRUM GATE SPILLWAY - MODEL M-1

A-NO DISCHARGE



B-DISCHARGE 100,000 SEC. FT.



C-DISCHARGE 180,000 SEC. FT.

D-DISCHARGE 200,000 SEC. FT.

PLATE 40

the disturbance, vanes in various positions in the transition and inclined tunnel were experimented upon. The first vane was about 6 feet high and extended down the center of the flat top contraction in the transition. No beneficial effect from this was observed. A vane 6 feet high was then placed along the bottom of the inclined tunnel from the end of the transition to the beginning of the vertical bend. This resulted in a marked improvement in the splashing at the vertical bend. With the flat top contraction in the transition and the coping on the rear wall this vane increased the discharge at which splashing began at the lower window from 130,000 second feet to 162,000 second feet. Severe splashing did not occur until the discharge reached 190,000 second-feet.

As the splashing out at the window always occurred on the left side, it was at first thought that by inclining the vane toward the right side of pipe more water could be forced to that side and the splashing relieved. The direction of the vane was therefore changed so that it extended from the middle of the bottom of the pipe at the upper end to 5 feet to the right of the middle at the lower end. This change, however, increased the splashing, reducing the discharge at which splashing began from 162,000 second-feet to 155,000 second-feet and the discharge for severe splashing from 190,000 second-feet to 170,000 second-feet. The vane was then moved so that the lower end was 5 feet to the left of the center of the bottom. In this position the splashing from the lower window began at about the same discharge as with the vane in the center, but no severe splashing occurred for discharges up to 200,000 second-feet.

The explanation of this unexpected action seems to be that the water approached the vertical bend with a greater depth on the right side. In passing around the vertical bend it was acted upon by a centripetal force, and since the mass of water on the right side was greater than on the left, the force was greater on the right side. This greater force caused a greater pressure in the water at the bottom of the right side than at the bottom of the left side and the water moved from the position of greatest pressure toward that of less pressure; that is, from the right side to the left side. The greater mass at the beginning of the bend was on the right side and in passing around the bend moved across the bottom toward the left side, up the left side and across the top of the bend toward the right, having moved with a spiral clockwise motion and across the face of the window, which is in the upper side of the bend, in a direction inclined toward the right. The vane with its end toward the left reduced this motion because it forced more water in the inclined tunnel toward the left side of the pipe and thus made the centrifugal force on the two sides of the pipe more nearly equal. Moving the vane toward the right side at the lower end increased the mass on the right side, making the two sides more unequal, and, therefore, increasing the cross motion which caused the splashing.

The best conditions with the M-1 model were obtained with the raised floor, coping baffle, 20 feet flat top in the transition, improved portal form and vane in the pipe as previously described. The conditions of flow in the channel were as shown on Plate 39. The condition of flow at the outlet of the tunnel was similar to that shown on Plate 36-B except at the rare intervals when shocks occurred.

Cylindrically-Faced Baffles

In order to break up the wave which piled up on the side wall of the spillway, and to more equally distribute the flowing water in the cross section of the channel, various combinations of baffles with cylindrical faces were investigated. The first set-up of these baffles is shown in Plate 41-A. It consisted of sections of baffles 10 feet long with 10-foot spaces between, set on the center line of the channel. The cylindrical face of the baffle had a radius of 6 feet, giving it a height of 12 feet, and the baffle had an overall height of 14 feet. These baffles improved the flow in the channel and tunnel somewhat, but not as much as the baffle on the rear wall. The splashing began at the lower window in the vertical bend with a discharge of 130,000 second-feet. The conditions of flow are shown on Plates 41-B, C and D. The baffles considerably obstructed the flow in the channel, however, and submerged the weir at the upper end to sufficient extent to obstruct the flow over it and require a higher headwater level than contemplated in the design to produce the 200,000 second-foot discharge.

A - NO DISCHARGE

B - DISCHARGE 13,000 SEC FT

C - DISCHARGE 180,000 SEC FT

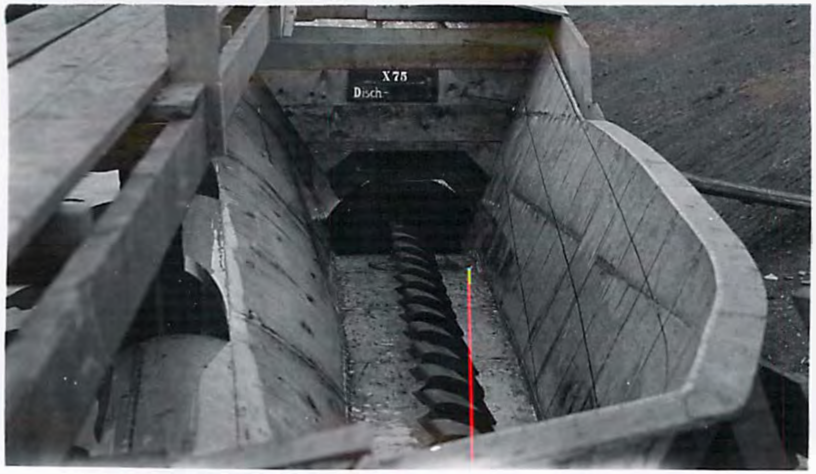
D - DISCHARGE 200,000 SEC FT.

CYLINDRICALLY FACED DENTATED BAFFLES ON
CENTER LINE OF CHANNEL-DRUM GATE SPILLWAY - MODEL M-1

The baffles located on the center line of the channel, with their faces parallel to it, were not exactly at right angles to the direction of flow, and a considerable portion of the water was deflected downstream and impinged on the next baffle below, as shown in Plate 41-B. To remedy this condition the baffles were placed with their faces inclined upstream 15 degrees with the center line of the channel, the row of baffles still extending down the channel center line as shown in Plate 42-A. The flow conditions (Plates 42-B and C) with this set-up were a slight improvement over those with the previous one, both in the channel and in the tunnel, but the baffles still so obstructed the flow in the channel that a water level above elevation 1232 was necessary in the forebay to produce a discharge of 200,000 second-feet. The location of the baffles was then changed from the center line to a line halfway between the center line and the bottom of the crest, the individual baffles being inclined 15 degrees with the center line of the channel as before. This set-up was slightly better than the preceding one (Plate 42-D) but the obstruction caused by the baffles was still too great. As the two upstream baffles seemed to obstruct the flow without bettering conditions, they were removed, but the result was not a material improvement.

The cylindrically-faced baffles were next placed in a continuous line down the channel, beginning $6\frac{1}{2}$ feet to the right of the center line at station 1+00 and 15 feet to the left of the center line at station 3+50. The submergence effect was not as great as for the preceding test (Plate 42) but was still too great, and severe thump-

INCLINED, CYLINDRICALLY FACED DENTATED BAFFLES, DRUM GATE SPILLWAY - MODEL M-I



A- NO DISCHARGE
BAFFLES ON CENTER LINE OF CHANNEL

B- DISCHARGE 13000 SECOND FEET
BAFFLES ON CENTER LINE OF CHANNEL

C- DISCHARGE 200000 SECOND FEET
BAFFLES ON CENTER LINE OF CHANNEL

D- DISCHARGE 200,000 SECOND FEET
BAFFLES MIDWAY BETWEEN CENTER LINE AND CREST

PLATE 42

ing occurred at the downstream end of the pipe.

In order to reduce the submergence, smaller-sized baffles were used in this test (Plate 43-A). These had a cylinder radius of $4\frac{1}{2}$ feet and an overall height of 11 feet. They extended in a line from $8\frac{1}{3}$ feet to the right of the center line at station 0+65 to 20 feet to the left of the center line at station 3+75. With this baffle the submergence did not obstruct the flow over the crest (Plates 43-C and D). The results of this set-up seemed to be somewhat of an improvement over the previous ones and the baffle was extended downstream to station 4+50. The extension, however, resulted in bad splashing at the vertical bend. The splashing began at 132,000 second-feet, and was severe at 150,000 second-feet. The shocks at the lower end of the tunnel were not large and occurred on an average of once every 5 seconds.

The continuous baffle of the previous run was made up of blocks 10 feet long. A test was made with every other block removed. This set-up was similar to the first ones run with the cylindrically-faced baffles but the baffles in this case were smaller. The conditions of flow in the channel were fairly good and excessive submergence at the upper end did not occur. The splashing from the lower window began at 155,000 second-feet and became severe at 178,000 second-feet. The conditions at the end of the tunnel were very desirable, shocks occurring at average intervals of 40 seconds.

CYLINDRICALLY FACED INCLINED BAFFLE
DRUM GATE SPILLWAY - MODEL M-1

A-SET UP-NO DISCHARGE

B-DISCHARGE 90.000 SEC FT

C-DISCHARGE 200000 SEC FT

D-DISCHARGE 200.000 SEC. FT.

PLATE 43

The conditions of flow with small cylindrical-faced baffle in channel are shown on Plates 43 and 44.

Cylindrically-faced baffles were also placed as shown in Plate 45-A. The water flowing over the crest, upon striking the face of these baffles, was deflected downstream thus tending to increase the velocity in the channel and reduce the cross section necessary and consequently the cost of the spillway. At small discharges the action of the baffle in deflecting the flow from the crest downstream could be observed. This action probably continued for higher flows also, but the obstruction offered by the baffles offset the advantage gained and at the 200,000 second-foot discharge (Plate 45-B) the conditions of flow were not appreciably improved over the conditions for no baffles.

Tests were also made with the cylindrically-faced baffles placed as shown on Plate 45-C. With baffles in this position it was believed that the water would be forced toward the ogee side, making the flow on the two sides of the channel at more nearly the same height. Plate 45-D shows that at a 100,000 second-foot discharge this result was accomplished. The obstruction of the baffles and the effect of the upstream deflection of the water so impeded the flow in the channel that at a 200,000 second-foot discharge submergence at the upper end of the channel became excessive and a water level of more than El. 1232 was required to produce a 200,000 second-foot discharge.

CYLINDRICALLY FACED INCLINED BAFFLE,
DRUM GATE SPILLWAY - MODEL M-1

A- DISCHARGE 100.000 SEC. FT.

B-DISCHARGE 100,000 SEC.FT.

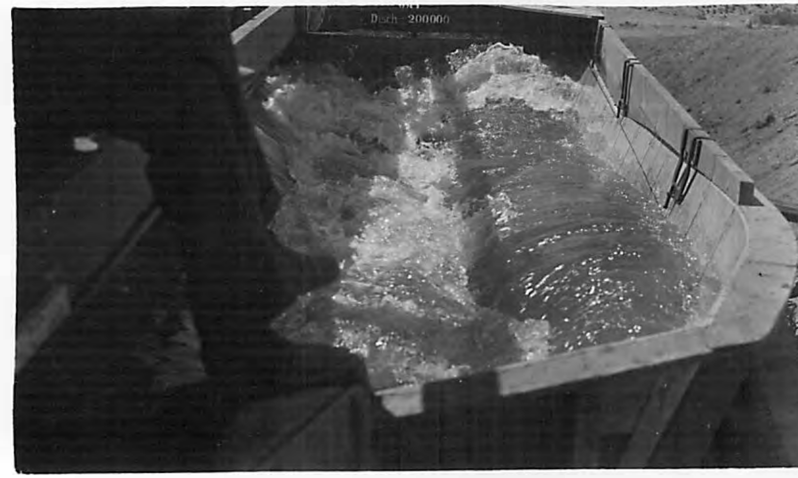
C- DISCHARGE 150.000 SEC. FT

D-DISCHARGE 200.000 SEC FT

PLATE 44

CYLINDRICALLY FACED BAFFLES INCLINED TO DIRECTION OF FLOW.
DRUM GATE SPILLWAY – MODEL M-1

A-NO DISCHARGE – BAFFLES INCLINED DOWNSTREAM



B-DISCHARGE 200,000 SECOND FEET
BAFFLES INCLINED DOWNSTREAM

C-NO DISCHARGE – BAFFLES INCLINED UPSTREAM

D-DISCHARGE 100,000 SECOND FEET
BAFFLES INCLINED UPSTREAM

PLATE 45

Deflector Vanes on the Ogee Crest

Deflector vanes on the ogee crest were also tried to direct the overfalling water downstream and permit the use of a smaller channel. The first set-up is shown in Plate 46-A. The water falling over the weir was not deflected directly downstream but somewhat out to the side with a considerable downstream component, as shown in Plate 46-B. At the larger discharges these vanes obstructed the flow and caused excessive submergence at the upper end of the crest. The conditions in the channel for 200,000 second-foot discharge are shown on Plate 46-C. Considerable vibration also resulted at the downstream end of the tunnel, the average period for severe shocks being 4 seconds. The upstream four vanes were removed, as they appeared to obstruct the flow (Plate 46-D). This slightly improved conditions in the channel but excessive submergence was still present and the frequency of severe shocks at the tunnel exit was increased to a 3-second average. The downstream four vanes were raised to a position where they would not so greatly obstruct the flow (Plate 47). This eliminated the vibration at the end of the tunnel, but produced an unstable condition in the channel. At times the <u>channel</u> at the lower end was full to El. 1225 and at others it dropped to a level 20 feet lower. No reason for this was apparent.

VANES ON OGEE CREST – DRUM GATE SPILLWAY MODEL M-1

B – DISCHARGE 160,000 SEC. FT.

A – DISCHARGE 100,000 SEC.FT.

C – DISCHARGE 200000 SEC.FT.

UPSTREAM FOUR VANES REMOVED – DOWNSTREAM FOUR RAISED

VANES ON OGEE CREST - DRUM GATE SPILLWAY MODEL M-I

A-FULL SET OF VANES ON OGEE CREST

B-DISCHARGE 30,000 SECOND FEET WITH FULL SET OF VANES

C-DISCHARGE 200000 SEC FT. WITH FULL SET OF VANES

D-DISCHARGE 200,000 SECOND FEET WITH UPSTREAM FOUR VANES REMOVED

Trough on Crest Side of Floor

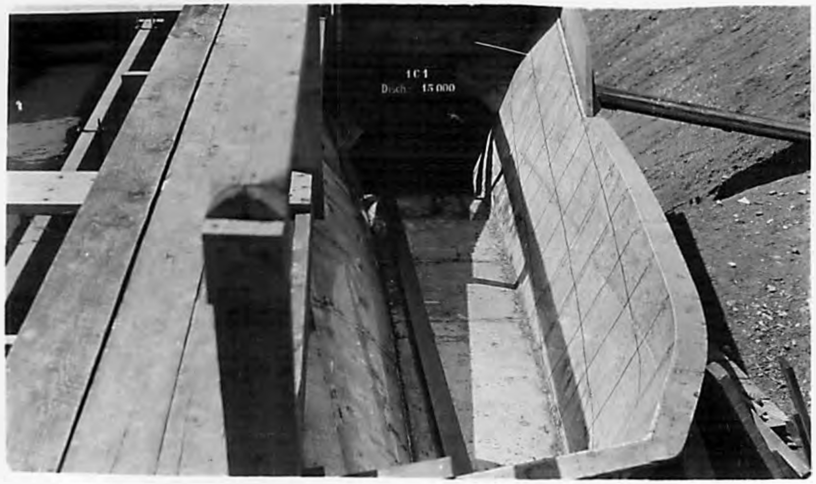
This model (Plate 48-A) was made by removing the false floor and building a raised section on the wall side. For flows up to 100,000 second-feet the raised section caused a very disturbed condition in the channel, with a stream of water rising up in a fountain effect along the center line of the channel. The condition for a 100,000 second-foot discharge is shown in Plate 48-B. Disturbed conditions still existed at 150,000 second-feet (Plate 48-C). At 200,000 second-feet the conditions in the channel were reasonably good (Plate 48-D). The severe shocks at the end of the tunnel occurred at intervals of about 15 seconds.

A dentated sill 6 feet high was placed at the left edge of the raised portion of the channel floor. This tended to hold the water to the right side of the channel and at low flows caused the trough at the upper end to be swept out and produce very trubulent conditions. A weir was also tried across the lower portion of the channel bottom at the downstream end, but this did not noticeably improve the flow.

Plain Sill and Dentated Sill on Channel Floor

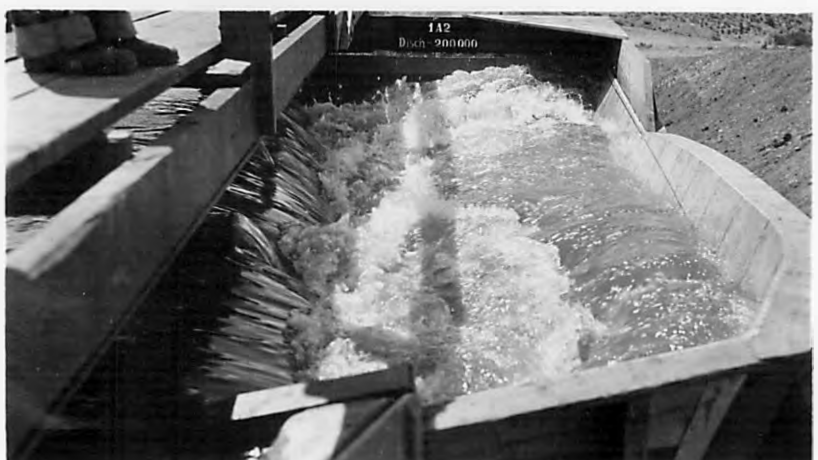
Experiments were made with a plain sill with vertical face 6 feet high (Plate 49-A). At high flows this produced very good conditions in the channel and at the end of the tunnel, small shocks

PLAIN AND DENTATED SILLS ON CHANNEL FLOOR,
DRUM GATE SPILLWAY-MODEL M-1



A-PLAIN SILL ON CHANNEL FLOOR

B-PLAIN SILL-DISCHARGE 15,000 SEC.FT.



C-PLAIN SILL-DISCHARGE 200,000 SEC.FT.

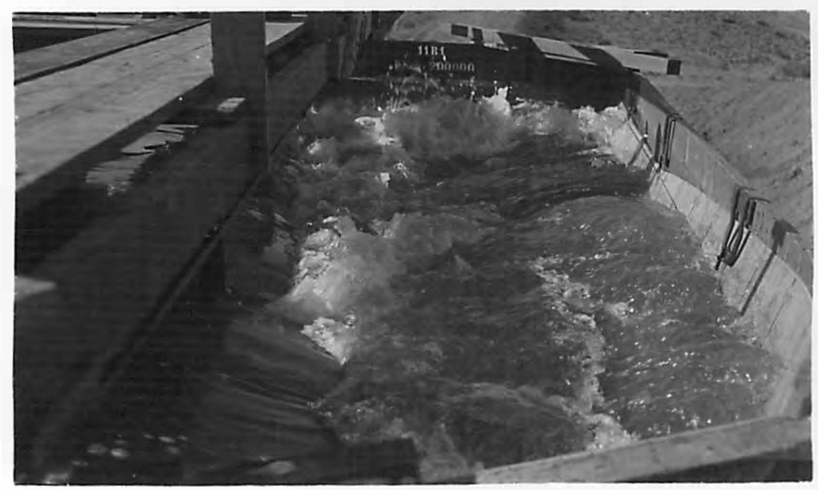
D-DENTATED SILL-DISCHARGE 200000 SEC.FT.

TROUGH ON CREST SIDE OF CHANNEL FLOOR.
DRUM GATE SPILLWAY-MODEL M-1

A-NO DISCHARGE

B-DISCHARGE 100.000 SEC FT

C-DISCHARGE 150.000 SEC. FT



D-DISCHARGE 200000 SEC FT.

PLATE 48

occurring only about once in 15 seconds. At low flows, however, the conditions in the channel were undesirable. At a discharge of 15,000 second-feet the stream was thrown vertically into the air in a fountain formation (Plate 49-B). At a somewhat higher flow a pressure area was formed upstream from the baffle which forced the stream upward so that it passed over the baffle without impinging on it, but instead rose high above the bottom and dashed against the rear wall.

The plain sill was replaced by a dentated sill 6 feet high extending from 23 feet to left of the center line at station 3+00 to a position 14 feet to left of the center line at station 0+50. This improved the conditions of flow in the channel at low discharges. At the discharges the conditions at the end of the pipe, however, were worse, as severe shocks occurred at intervals averaging 1.3 seconds.

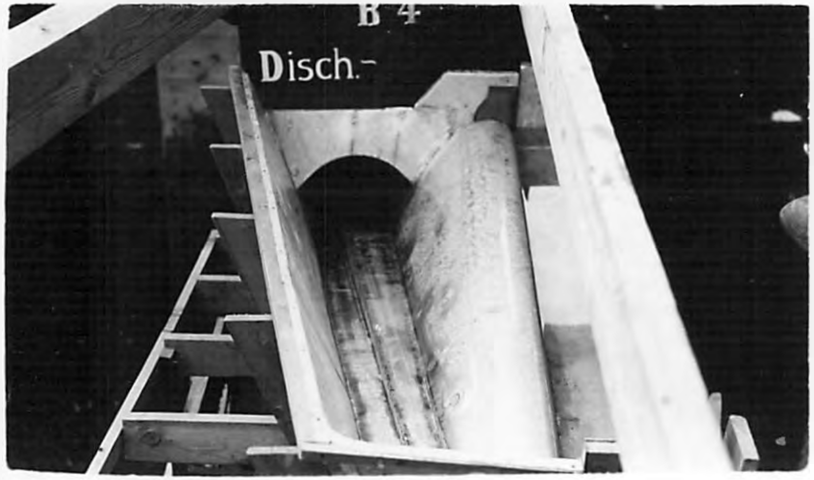
DRUM GATE TYPE SPILLWAY - 1:100 MODEL M-2

A small model of the drum gate type of spillway was constructed by Mr. W. H. Price on a 1:100 scale to test out the possibility of using a trough in the floor of the channel along the bottom of the rear wall. It was expected that this would reduce the height of the wave along the rear wall. The channel of this model was approximately the same design as that of the M-1 model, but the transition of the former was slightly larger in proportion and of less abrupt curvature. The plan of the M-2 model was reversed, the crest being on the right-hand side of the channel, but this did not affect its action. In order to simplify construction it was built without gates or the intermediate piers. Plate 50-A shows the spillwas with no flow in it. The trough on the wall side of the floor proved ineffective in reducing the height of the wave. The stream from the weir seemed to jump off the edge of the trough and flow in an inclined direction downward to the bottom of the trough, thence across the bottom and up the rear wall, with practically as much energy as with no trough. The condition of flow in this model for a 200,000 second-foot discharge is shown in Plate 50-B.

The trough was then moved to the ogee side with better results. The action in this case was very similar to that for the M-1 model. The conditions of flow for 100,000 and 200,000 second-feet are shown on Plates 50-C and D, respectively. A comparison of Plate 50-C with Plate 48-B shows a very close similarity for 100,000 second-foot flows on the M-2 and M-1 models, respectively. The picture of the two models for a 200,000 second-foot flow are not comparable, as the flat top in the transition of the M-1 was not built in the M-2 model. At the 100,000 second-foot discharge this top did not come into action.

This small model was also tested with a flat floor similar to the raised floor in the M-1 model. The conditions for discharges

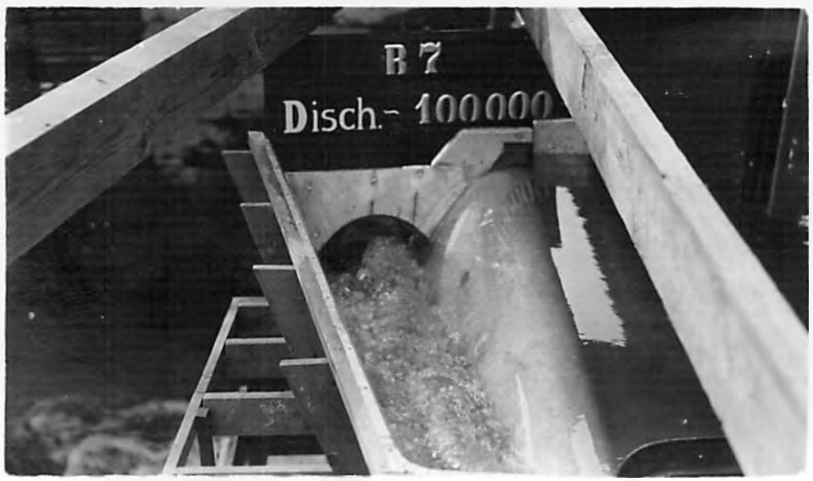
SIDE CHANNEL SPILLWAY-MODEL M-2 - SCALE 1:100
TROUGH ON OGEE SIDE OF FLOOR.



A- NO DISCHARGE

B-DISCHARGE 200,000 SEC. FT

TROUGH IN FLOOR ALONG REAR WALL



C- DISCHARGE 100,000 SEC. FT

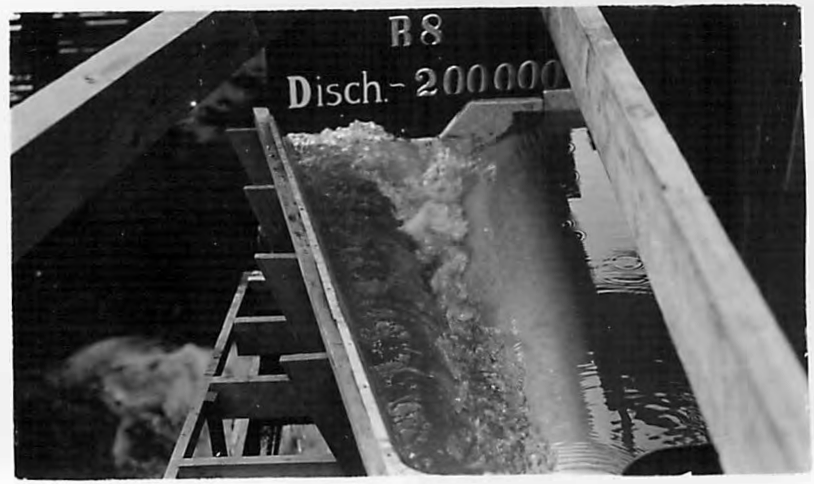
D- DISCHARGE 200,000 SEC. FT

of 100,000 and 200,000 second-feet are shown on Plates 51-A and B. The results on this model were similar to those on the 1:20 model, as may be seen by a comparison of Plate 51-A with Plate 40-B. The improved portal and flat top in the transition were not in action in the latter view.

It was believed that considerable improvement in the flow conditions at the lower end of the spillway could be made by altering the transition between the channel and the inclined tunnel pipe. The top of the transition was raised at the upper end and made to incline more steeply downward. The results, however, were unsatisfactory; at a 200,000 second-foot discharge the water from the ridge against the rear wall impinged on the top of the transition and formed an undesirable wave, as shown in Plate 52-B. The conditions for flows of 100,000, 150,000 and 180,000 are shown on Plates 51-C and D and Plate 52-A, respectively.

A few quantitative observations were made on this model to determine the cross section of flow in the channel. The results of these are discussed in the section of this report dealing with this feature of the spillway design.

A - DISCHARGE 100,000 SEC.FT



B - DISCHARGE 200,000 SEC.FT.

SIDE CHANNEL SPILLWAY WITH RAISED BOTTOM

C - DISCHARGE 100,000 SEC. FT.

D - DISCHARGE 150,000 SEC. FT.

SIDE CHANNEL WITH RAISED BOTTOM AND ALTERED TRANSITION
SIDE CHANNEL SPILLWAY - MODEL M-2 - SCALE 1:100

PLATE 51

A- DISCHARGE 180,000 SEC.FT.

C-SLIGHTLY SPIRAL FLOW AT
END OF TRANSITION

DISCHARGE 200,000 SEC FT.

SIDE CHANNEL WITH RAISED BOTTOM AND ALTERED TRANSITION
SIDE CHANNEL SPILLWAY - MODEL M-2 - SCALE 1:100