G J Hornsby

UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: HYDRAULIC MODEL EXPERIMENTS FOR THE DESIGN OF THE WHEELER DAM.

By JAMES W. BALL, JUNIOR ENGINEER

Under direction of

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and

JACOB E. WARNOCK, ASSOCIATE ENGINEER

TECHNICAL MEMORANDUM NO. 407

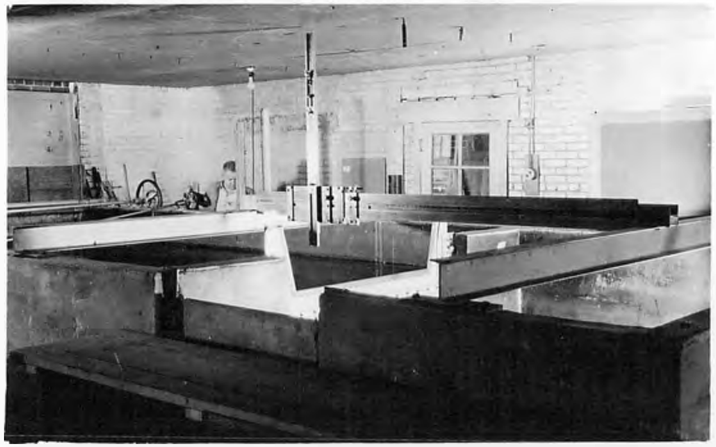
Fort Collins, Colorado

October 30, 1934.

PRICE ($13.00)

A. THE BUILDING.

B. THE SUPPLY RESERVOIR



C. MEASURING WEIR AND CHANNEL.

D. INTERIOR VIEW

THE HYDRAULIC LABORATORY
OF THE COLORADO AGRICULTURAL EXPERIMENT STATION.

ACKNOWLEDGEMENTS

The studies described in this report were made in the laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado by the hydraulic research division of the United States Bureau of Reclamation with Jacob E. Warnock, Associate Engineer, as resident engineer. During the greater portion of the period of tests he was assisted by Grover J. Hornsby, Assistant Engineer. The tests on the model of the Wheeler Dam were conducted under the immediate supervision of James W. Ball, Junior Engineer, and this report was prepared by him. He was assisted in the tests of the model and preparation of the report by Rob Roy Buirgy, Junior Engineer, and Aubrey N. Smith, Junior Engineer. The model was constructed by William J. Colson, William O. Parker, Emerson D. Helbig, Arthur J. Warfield, Lloyd W. Watson, Floyd L. Kelly and John H. Wood.

The tests were conducted under the general supervision of E. W. Lane, Research Engineer, and under the direction of J. L. Savage, Chief Designing Engineer. All engineering work of the Bureau of Reclamation is under R. F. Walter, Chief Engineer, and all activities of the Bureau are under the direction of Dr. Elwood Mead, Commissioner.

It is desired to acknowledge the cooperation of the Colorado State Board of Agriculture (governing board of the Colorado Agricultural College), and the staff of the U. S. Bureau

of Agricultural Engineering in permitting the use of the hydraulic laboratory of the Colorado Agricultural Experiment Station in conducting the experiments on the model of the Wheeler Dam.

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I. SYNOPSIS.

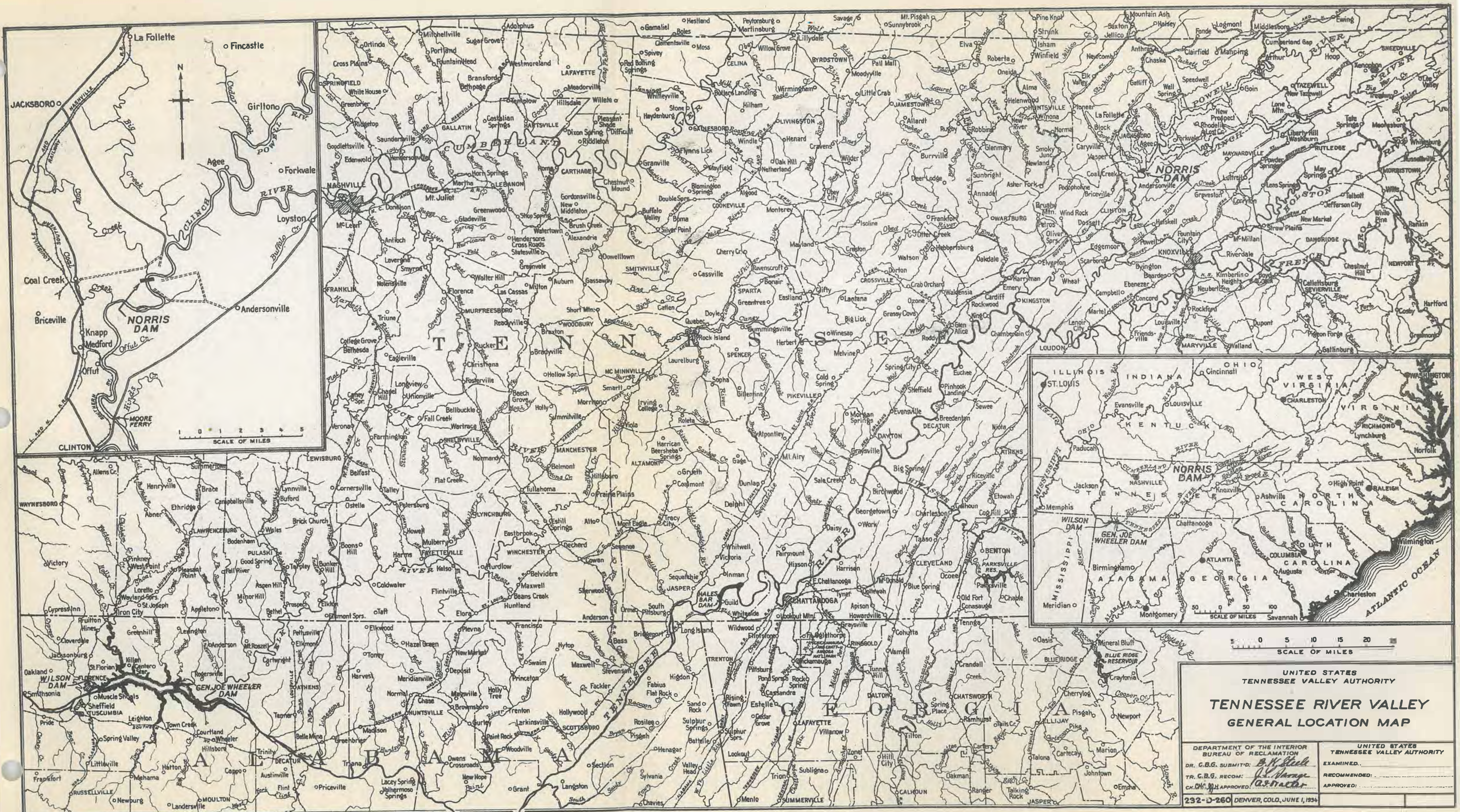
The determination of the best form of protection against scour at the toe of the Wheeler Dam being built by the Tennessee Valley Authority in the Tennessee River approximately 20 miles east of Florence, Alabama (Fig. 1), has been the primary subject of extensive hydraulic model experiments conducted by the hydraulic research division of the U. S. Bureau of Reclamation in the hydraulic laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado.

From these experiments, a satisfactory design has been developed such that the immense potentially destructive forces contained in the spillway discharge will be rendered harmless.

Besides the experiments on the design of the stilling pool, studies were made on the design of the downstream nose of the piers to improve the hydraulic conditions on the apron.

Studies were made to determine a gate operating program that would enable the operating engineer to control a flood without needlessly endangering the structure, and extensive calibrations were made to provide data for the construction of a spillway discharge diagram.

FIGURE 1



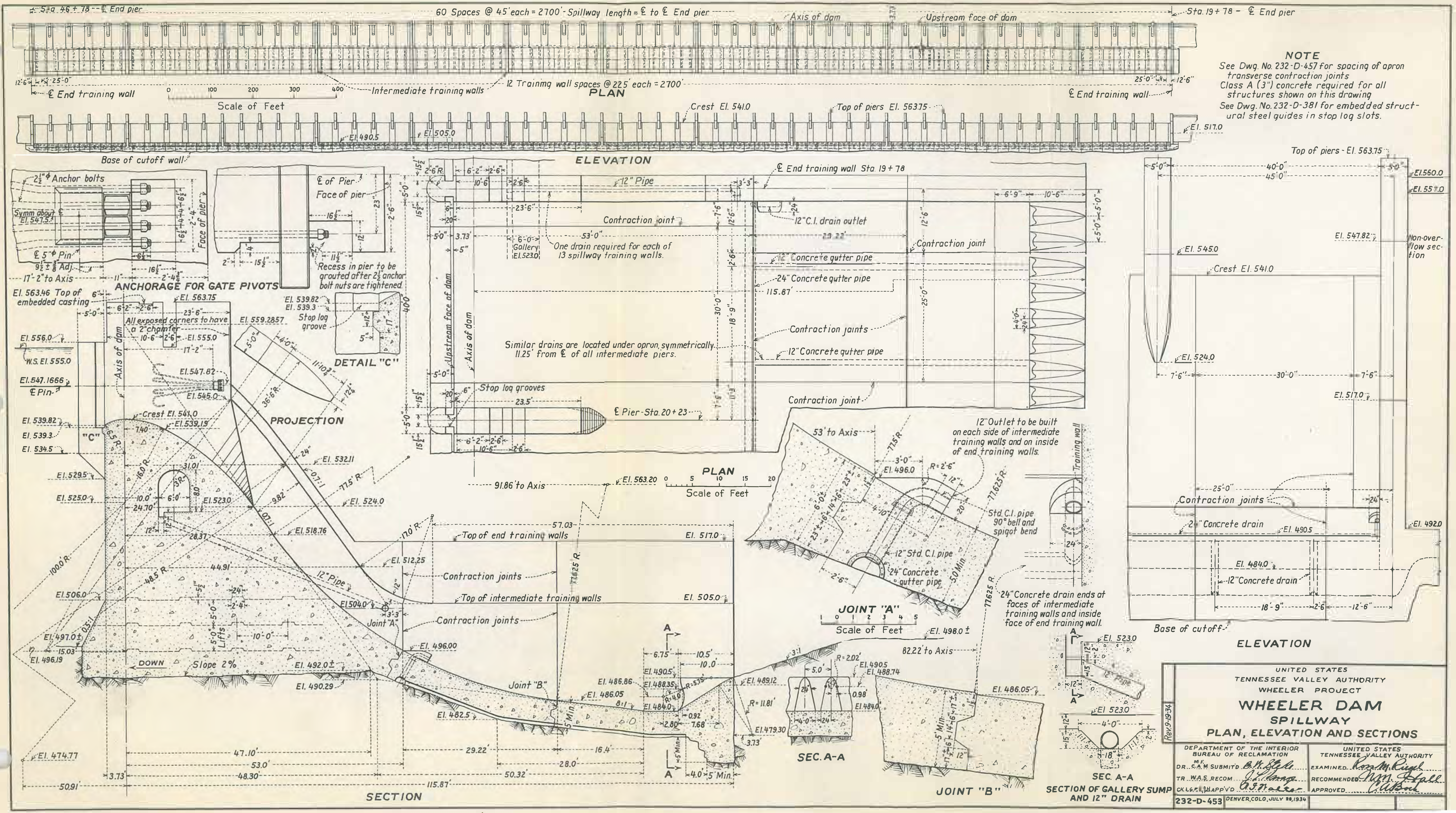
UNITED STATES
TENNESSEE VALLEY AUTHORITY
TENNESSEE RIVER VALLEY
GENERAL LOCATION MAP
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
UNITED STATES
TENNESSEE VALLEY AUTHORITY
EXAMINED
RECOMMENDED
APPROVED
232-D-260 DENVER, COLO., JUNE 1, 1934
SCALE OF MILES
NORRIS DAM
WILSON DAM
GEN. JOE WHEELER DAM
HALES BAR DAM
TENNESSEE
GEORGIA
ALABAMA
ATLANTIC OCEAN

II. INTRODUCTION.

The Wheeler Dam (formerly Dam No. 3 of Muscle Shoals), which is located in the Tennessee River about 20 miles upstream from Florence, Alabama (Figure 1), is being constructed by the Tennessee Valley Authority for flood control, power production and improvement of navigation on the Tennessee River.

The dam is of the gravity type, and, when completed, will be about 50 feet high, 6,350 feet long, and will have as an integral part of the structure a 2,700-foot spillway capable of discharging a maximum flood of 650,000 second-feet. The reservoir will be maintained at Elevation 555 by sixty Tainter gates, each forty feet long and fourteen feet high. A surcharge of three feet is provided for large floods giving a reservoir elevation of 558 for the maximum discharge. The piers which separate the gates are five feet in thickness and will serve as anchorages for the gate hinges. The structure will be provided with a trash gate near each end for removing trash which accumulates behind the non-overflow sections and will also be equipped with a navigation lock. The reservoir, when full, will be about 85 miles long with a maximum depth of 57 feet (Reservoir Elevation 555) and a capacity of approximately 700,000 acre-feet. Figure 2 shows a section and plan of the structure.

FIGURE 2



PLAN
60 Spaces @ 45' each = 2700' - Spillway length = ℄ to ℄ End pier
Sta. 19+78 - ℄ End pier
Axis of dam
Upstream face of dam
℄ End training wall
Intermediate training walls
12 Training wall spaces @ 225' each = 2700'
Scale of Feet
NOTE
See Dwg. No. 232-D-457 for spacing of apron transverse contraction joints
Class A (3") concrete required for all structures shown on this drawing
See Dwg. No. 232-D-381 for embedded structural steel guides in stop log slots.
ELEVATION
Crest El. 541.0
Top of piers El. 563.75
El. 490.5
El. 505.0
El. 517.0
Base of cutoff wall
ANCHORAGE FOR GATE PIVOTS
2½" ϕ Anchor bolts
Symm about ℄ El. 547.5
℄ 5" ϕ Pin
Face of pier
℄ of Pier
Recess in pier to be grouted after 2½" anchor bolt nuts are tightened
DETAIL "C"
Stop log groove
El. 539.82
El. 539.3
PROJECTION
All exposed corners to have a 2" chamfer
El. 563.46 Top of embedded casting
El. 563.75
El. 559.2857
El. 555.0
El. 547.82
El. 545.0
W.S. El. 555.0
El. 547.1666 ℄ Pin
Crest El. 541.0
El. 539.15
El. 532.11
El. 524.0
El. 523.0
El. 518.76
El. 512.25
El. 504.0
El. 496.00
El. 492.0±
El. 490.29
El. 497.0±
El. 496.19
El. 474.77
Axis of dam
Top of end training walls
Top of intermediate training walls
Contraction joints
12" Pipe
Joint "A"
Joint "B"
Slope 2%
DOWN
Lifts
SECTION
PLAN
℄ End training wall Sta. 19+78
℄ Pier - Sta. 20+23
Stop log grooves
Gallery El. 523.0
One drain required for each of 13 spillway training walls.
Contraction joint
12" C.I. drain outlet
12" Concrete gutter pipe
24" Concrete gutter pipe
Contraction joints
Similar drains are located under apron symmetrically 11.25' from ℄ of all intermediate piers.
Upstream face of dam
91.86' to Axis
El. 563.20
Scale of Feet
JOINT "A"
53' to Axis
12" Outlet to be built on each side of intermediate training walls and on inside of end training walls.
Std. C.I. pipe 90° bell and spigot bend
12" Std. C.I. pipe
24" Concrete gutter pipe
24" Concrete drain ends at faces of intermediate training walls and inside face of end training wall.
Training wall
El. 498.0±
82.22' to Axis
SEC. A-A
JOINT "B"
SEC. A-A
SECTION OF GALLERY SUMP AND 12" DRAIN
ELEVATION
Top of piers - El. 563.75
El. 560.0
El. 547.82
Non-overflow section
El. 545.0
Crest El. 541.0
El. 524.0
El. 517.0
Contraction joints
24" Concrete drain
El. 490.5
El. 484.0
12" Concrete drain
El. 492.0
Base of cutoff
UNITED STATES
TENNESSEE VALLEY AUTHORITY
WHEELER PROJECT
WHEELER DAM
SPILLWAY
PLAN, ELEVATION AND SECTIONS
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
UNITED STATES TENNESSEE VALLEY AUTHORITY
SUBMIT'D
EXAMINED
RECOM.
RECOMMENDED
APP'V'D
APPROVED
Rev. 9-1934
232-D-453
DENVER, COLO., JULY 28, 1934

The design of this dam, for which frequent reference was made to House Document No. 328, 71st Congress, 2nd Session, "Tennessee River and Tributaries, North Carolina, Tennessee, Alabama and Kentucky", was developed in the Denver office of the Bureau of Reclamation. The following is pertinent data obtained from the above mentioned document:

Maximum recorded flood at Florence, Alabama, March, 19, 1897 - 465,000 second-feet.

Average recorded flow for 25 years - 53,000 second-feet

Estimated 500,~~000~~ year flood - 494,000 second-feet.

Minimum tailwater (Headwater Wilson Dam) Elev. 505.1.

Because of the characteristics of the Wheeler Dam site, the problem of protection against scour at the toe was of major importance and it was primarily for the purpose of determining the most effective and economical form that model tests were made. Other problems encountered and solved by model studies were the design of the downstream nose of the piers to give better hydraulic conditions on the apron, and the design of the intermediate training walls to resist the differential pressures that might occur when the gates were operated.

Studies were made on the crest to determine a coefficient curve, to obtain a satisfactory program for operating the gates, and to obtain a discharge diagram to be used by the operating engineer in determining the discharge through the Tainter gates.

Studies were also made to determine the existence of an undesirable partial vacuum on any portion of the diffuser sill.

The determination of the best form of protection against scour at the toe of a dam is an important factor in the design of the structure, because of the vast amount of energy contained in the water flowing over the spillway and the destructive action to the foundation of the dam which is possible if this energy is not properly controlled. In the case of the Wheeler Dam, the maximum fall is about 70 feet (from maximum water surface at Elev. 558 to water surface immediately upstream from the hydraulic jump) but the quantity is so large that should a flood of 650,000 second-feet occur, the energy of the overfalling water would be 5,170,000 horsepower. The proper design of the apron is, therefore, vital to the safety of the structure.

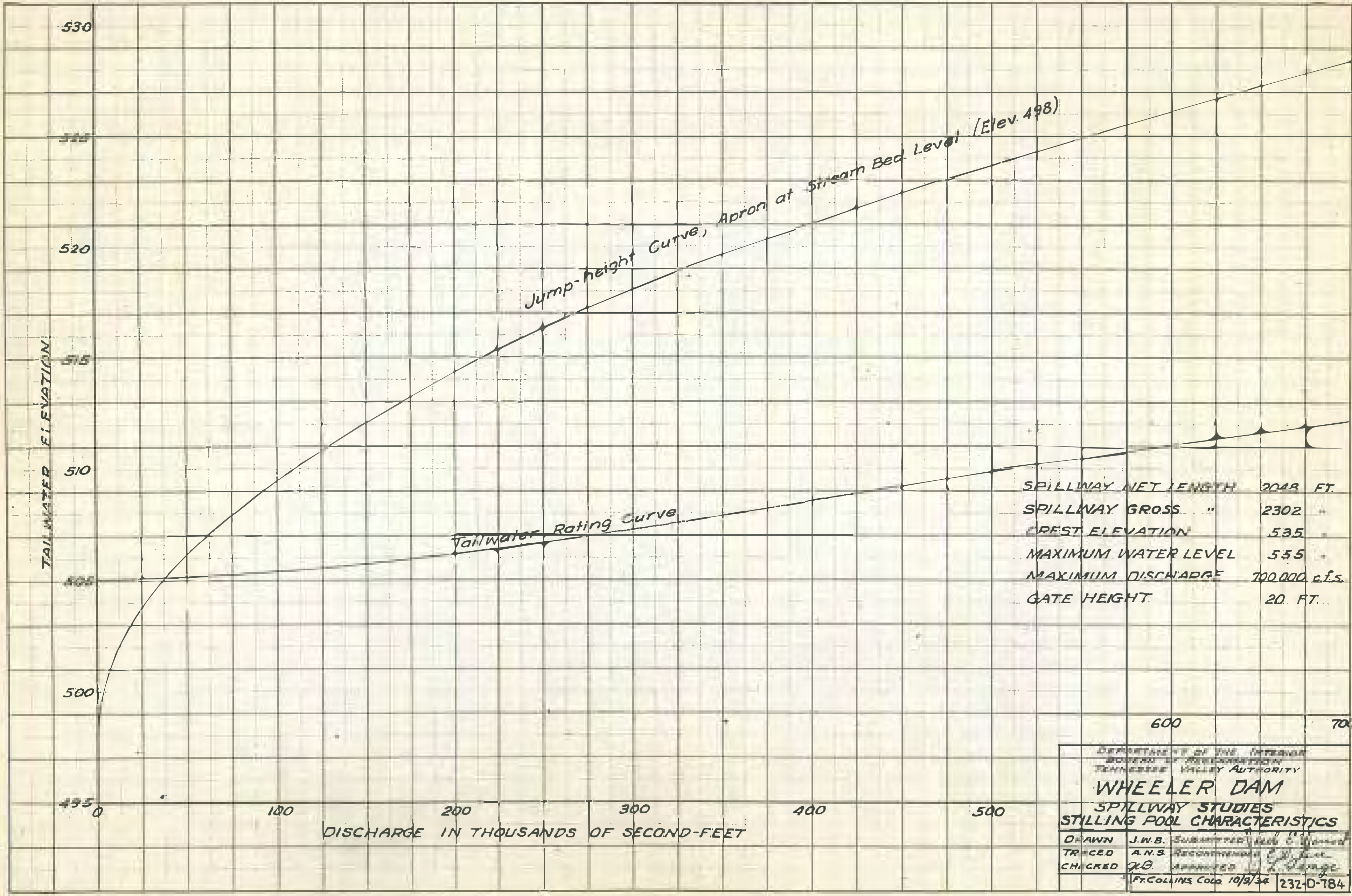
Until recently, the principle of protection against scour below dams has not been very completely understood. A determination for each individual case by means of model testing has rapidly grown into use, but the principles involved were not in all cases well considered and the results of these tests were, therefore, not as reliable as otherwise might have been the case. As a result of these studies by the Bureau of

Reclamation in connection with the Madden, Cle Elum and other dams, the principles governing the design of scour protection have been worked out and the types of protection applicable to conditions at various dam sites have been classified. Model studies are still necessary to determine which of the several forms that are applicable to the conditions at a given dam site is the best, but the classification of the principles narrows the field which is necessary to study and insures that unsatisfactory conditions are not overlooked.

The principles governing scour protection are outlined in U. S. Bureau of Reclamation Technical Memorandum #323, entitled "Protection against Scour below Overfall Dams" (Appendix No. I). Briefly stated, the type of scour protection depends upon the relation, throughout the entire range of discharge, of the elevation of the tailwater below the dam to the elevation necessary to cause a hydraulic jump to form on a level apron at the elevation of the stream bed, or, in other words, to the relative position of the tailwater rating curve and a jump-forming height curve, for an apron at stream bed level. This is true whether or not the hydraulic jump is used for the scour protection. The type applicable to the various relations of the elevation of the tailwater rating curve are given on Page 2, Appendix I.

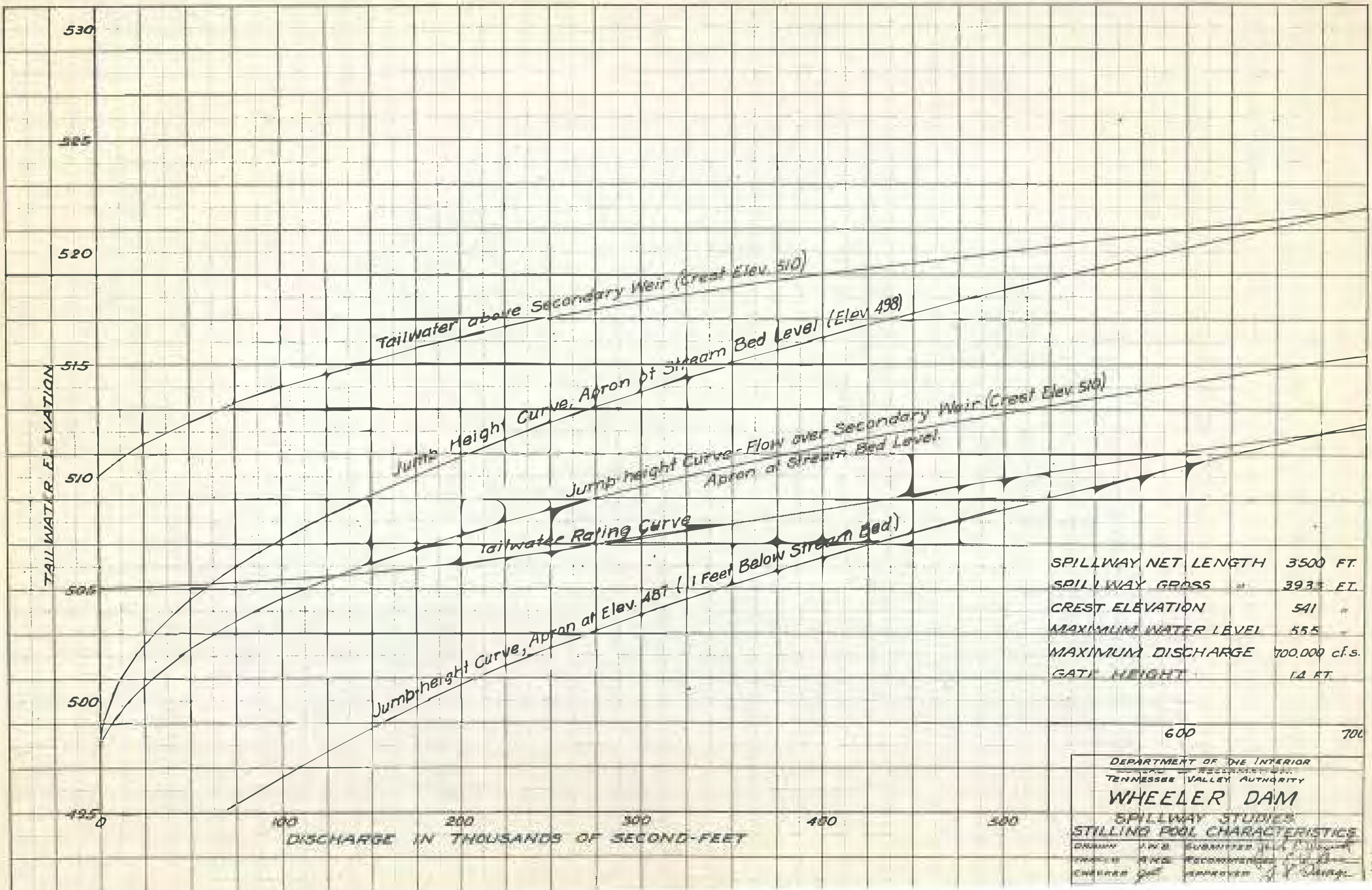
The characteristics of the Wheeler Dam site from the standpoint of scour protection are shown in Figures 3 and 4. The conditions at this site are especially severe because the slope of the river bed is steep, the river is wide and the tailwater depth is, therefore, shallow. Further rigid restrictions are inflicted by the small range of fluctuation of tailwater depth due to the regulation of the reservoir of Wilson Dam to a constant elevation of 505. The original design of the Wheeler Dam, with the crest at Elev. 535, provided for a discharge of 700,000 second-feet with a maximum head of 20 feet. The jump-height curve for an apron at stream bed level (Figure 3) falls considerably above the tailwater curve, and this dam, therefore, is in Case I, Appendix I. For this condition, the possible solutions are: (1) shaping the bucket to throw the overfalling stream far from the base of the dam, (2) raising the tailwater level by a secondary dam, (3) deepening the pool below the dam, and (4) various forms of baffles.

A better agreement of jump-height and tailwater curve was obtained by using 14-foot gates. As a result, the original design was altered by raising the crest elevation six feet to Elev. 541 and providing for a discharge of 700,000 second-feet with a maximum head of 14 feet. With this layout a length of spillway of 3,500 feet was necessary. Estimates showed that,



530
525
520
515
510
505
500
495
0
100
200
300
400
500
600
700
TAILWATER ELEVATION
DISCHARGE IN THOUSANDS OF SECOND-FEET
Jump-height Curve, Apron at Stream Bed Level (Elev. 498)
Tailwater Rating Curve
SPILLWAY NET LENGTH 2048 FT.
SPILLWAY GROSS " 2302 "
CREST ELEVATION 535 "
MAXIMUM WATER LEVEL 555 "
MAXIMUM DISCHARGE 700,000 c.f.s.
GATE HEIGHT 20 FT.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
STILLING POOL CHARACTERISTICS
DRAWN J.W.B. SUBMITTED
TRACED A.N.S. RECOMMENDED
CHECKED APPROVED
FT. COLLINS COLO. 10/9/34
232-D-784

FIG. 3



TAILWATER ELEVATION
530
525
520
515
510
505
500
495
0
100
200
300
400
500
600
700
DISCHARGE IN THOUSANDS OF SECOND-FEET
Tailwater above Secondary Weir (Crest Elev. 510)
Jump Height Curve, Apron at Stream Bed Level (Elev. 498)
Jump height Curve - Flow over Secondary Weir (Crest Elev. 510)
Apron at Stream Bed Level.
Tailwater Rating Curve
Jump height Curve, Apron at Elev. 487 (11 Feet Below Stream Bed)
SPILLWAY NET LENGTH 3500 FT.
SPILLWAY GROSS " 3933 FT.
CREST ELEVATION 541 "
MAXIMUM WATER LEVEL 555 "
MAXIMUM DISCHARGE 700,000 c.f.s.
GATE HEIGHT 14 FT.
DEPARTMENT OF THE INTERIOR
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
STILLING POOL CHARACTERISTICS
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED

FIG.

although the lower gates required a longer spillway section, and, therefore, a greater length of protection, the decrease in cost of protection per foot for the lower gates almost entirely offset the increased length and resulted in practically the same total cost on account of less severe scour conditions resulting and the less depth of pool required.

The principal dimensions required for certain forms of protection may be predicted from Figure 4. To produce a sufficient depth to maintain the jump on an apron at stream bed level for all discharges up to the maximum would have required a secondary weir with a crest at Elev. 510. Below this secondary weir, there would be another problem of protection, since the jump-height curve for the flow over the secondary weir is also higher than the natural tailwater rating curve. Figure 4 shows that a depressed pool formed by excavation to place the apron at Elev. 487 would produce a jump for all discharges up to 700,000 second-feet.

As the tailwater rating curve is important in determining the action of the stilling pool, it was constructed as accurately as possible. Up to a discharge of 318,000 second-feet, the elevation of the tailwater was based upon observation of actual floods. The value of roughness of the river channel was computed from the observed discharge and the channel size as determined by surveys. The stages which would be reached for discharges of 500,000 and 700,000 second-feet were computed using these roughness values. It is believed that this method gives results which are sufficiently accurate for the purpose.

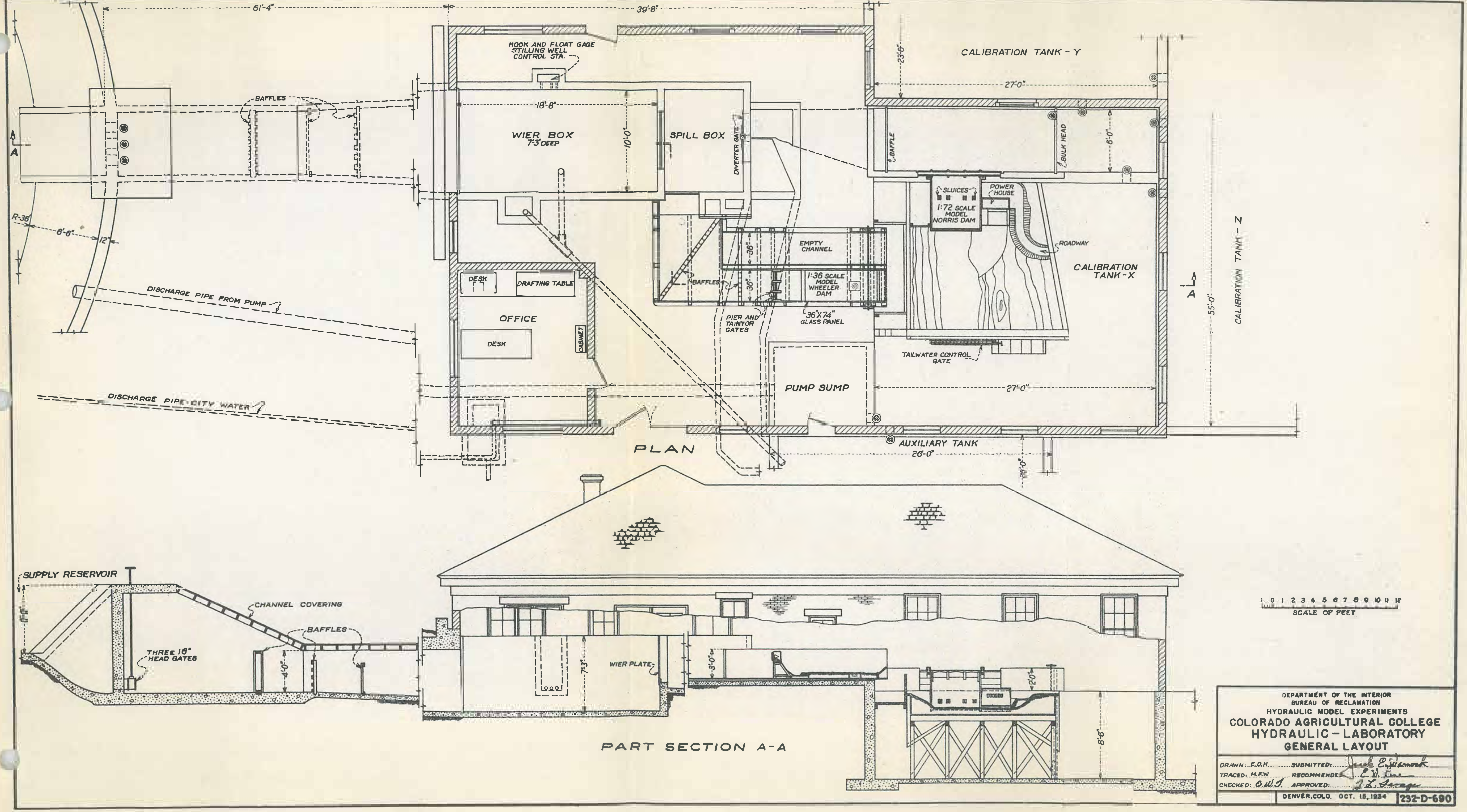
III. APPARATUS.

a. Hydraulic Laboratory.

The experiments for the design of the Wheeler Dam were conducted in the hydraulic laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado, a general layout of which, together with the location of the model, is shown on Figure 5.

The flow of water for the model was obtained from the 200,000-gallon reservoir above the laboratory and was discharged by hand-operated gates into a diverging flume and thence into a concrete weir channel 19½ feet long, 10 feet wide and 7 feet 3 inches deep. In the side of this channel, 13 feet upstream from the weir was a by-pass gate and a 4-inch by-pass valve. Small adjustments of the discharge quantity through the weir were made by varying the flow through these by-passes. The head on the weir was observed by a hook gage and a Cornell type float gage. The gages were located in a stilling well, 9 inches by 24 inches, connected to the weir channel by a pipe. The weir settings and weir are shown in the frontispiece. The model discharge in the case of the Wheeler model was measured by two

FIGURE 5



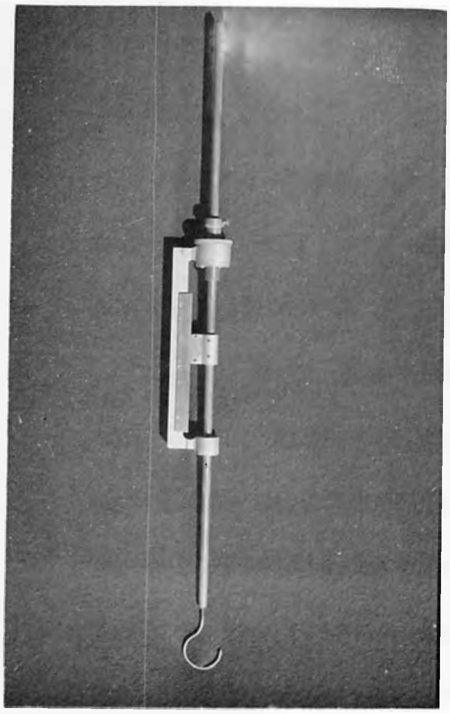
61'-4"
39'-8"
HOOK AND FLOAT GAGE STILLING WELL CONTROL STA.
CALIBRATION TANK - Y
23'-6"
27'-0"
BAFFLES
18'-8"
WIER BOX
7'-3" DEEP
10'-0"
SPILL BOX
DIVERTER GATE
BAFFLE
BULK HEAD
6'-0"
A
R-36'
6'-6"
12"
SLUICES
POWER HOUSE
1:72 SCALE MODEL NORRIS DAM
EMPTY CHANNEL
ROADWAY
CALIBRATION TANK-X
CALIBRATION TANK - Z
55'-0"
DESK
DRAFTING TABLE
BAFFLES
1:36 SCALE MODEL WHEELER DAM
DISCHARGE PIPE FROM PUMP
OFFICE
PIER AND TAINTOR GATES
36"X74" GLASS PANEL
DESK
CABINET
TAILWATER CONTROL GATE
DISCHARGE PIPE-CITY WATER
PUMP SUMP
27'-0"
AUXILIARY TANK
26'-0"
PLAN
SUPPLY RESERVOIR
CHANNEL COVERING
BAFFLES
THREE 16" HEAD GATES
4'-0"
7'-3"
WIER PLATE
3'-0"
2'-0"
8'-6"
SCALE OF FEET
PART SECTION A-A
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HYDRAULIC MODEL EXPERIMENTS
COLORADO AGRICULTURAL COLLEGE
HYDRAULIC – LABORATORY
GENERAL LAYOUT
DRAWN: E.D.H.
TRACED: M.F.W.
CHECKED: O.W.T.
SUBMITTED:
RECOMMENDED:
APPROVED:
DENVER, COLO. OCT. 15, 1934
232-D-690

different types of weirs depending upon the quantity of water to be measured and the type of experiment in progress. For all visual tests, the two-foot Cipolletti weir was sufficiently accurate, but when experiments required very small quantities, as in calibration work, the 90-degree V-notch weir was used.

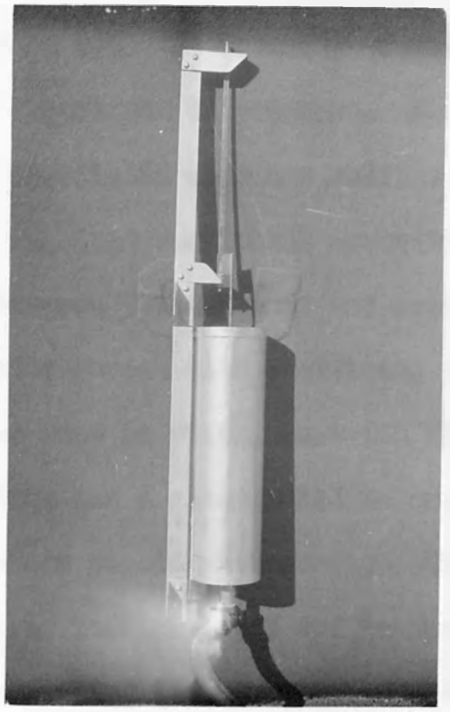
From the weir, the water passed into a stilling pool and thence into the flume, three feet in width, in which the model was constructed. This flume had a glass panel in one side to facilitate visual tests and permit photography. The head on the crest was regulated by Tainter gates and measured by a float gage connected to an opening in the center of the three-foot flume, three feet upstream from the model. The tailwater was regulated to the proper level by a spill gate at the discharge end of the three-foot flume. The elevation of the tailwater was measured by a float gage connected to an opening in the center of the flume at a position where it was not affected by the jump or the drawdown over the tailwater gate. Profiles of the water surface and erosion were measured by a point gage. Plate I shows the gages and type of recording apparatus for setting the Tainter gates.

B. The Model.

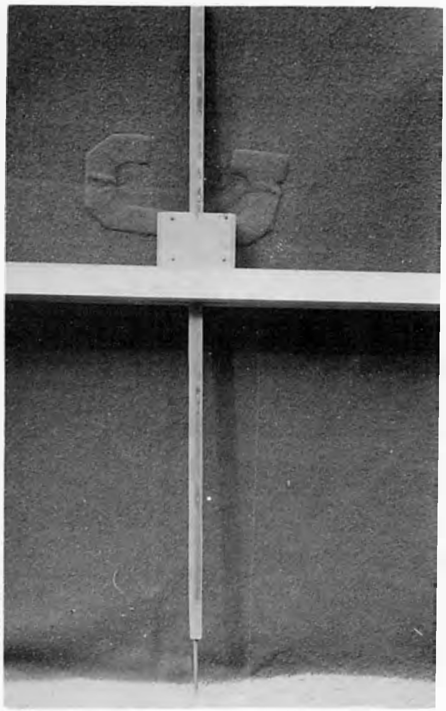
Due to the excessive length of the dam and the



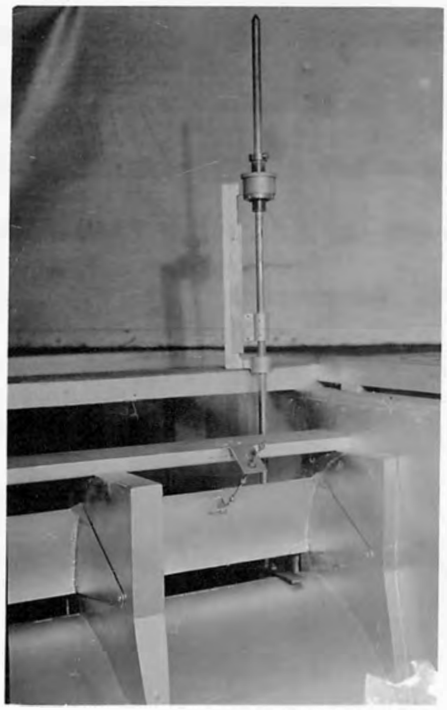
A. HOOK GAGE.



B. FLOAT GAGE.



C. POINT GAGE.



D. BAR GAGE.

INSTRUMENTS FOR MEASURING QUANTITIES ON MODEL.

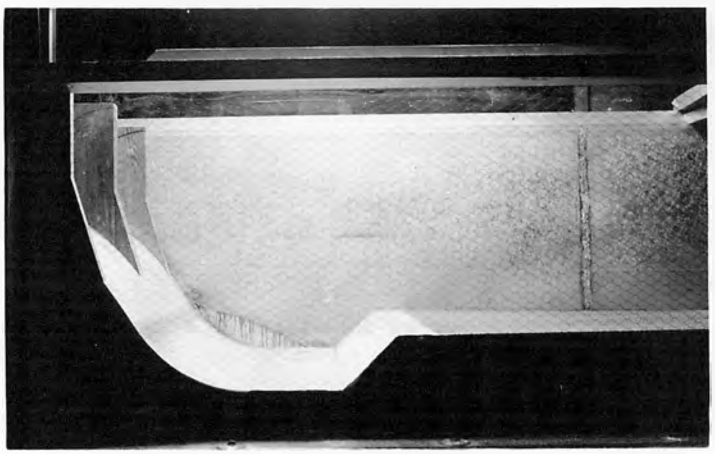
quantity of water required, a complete model on a workable scale could not be built and tested in the laboratory. It was not deemed necessary, however, because of the flatness of the river bed between banks at the dam site and the nature of the tests required. Therefore, a model (Plate 2) on a scale ratio of 1:36, three feet long, representing 108 feet of the prototype spillway was used for all tests performed by the research staff of the U. S. Bureau of Reclamation.

These studies were based on the laws of hydraulic similitude. The relation of the dimensions and quantities of the model and prototype are shown in the following table, using the symbol N to express the scale ratio, i.e. N equals 36.

RATIO OF PROTOTYPE TO MODEL.

Quantity	Formula	Value
Linear Dimensions	$N^{1.0}$	36.0
Head	$N^{1.0}$	36.0
Velocity	$N^{0.5}$	6.0
Area	N^{2}	1296.0
Discharge	$N^{5/2}$	7776.0
Time	$N^{0.5}$	6.0

A. LOOKING UPSTREAM.



B. SIDE VIEW.

MODEL READY FOR OPERATION.

	Prototype	Full Model	Fraction of Model
Length, ft.	6,336	176	3
Gross Spillway, ft.	2,700	75	3
Net Spillway, ft.	2,400	66.667	2.653
Height, ft.	50	1.389	1.389
Gate Length, ft.	40	1.111	1.111
Number of Gates	60	60	2.390
Fall, ft.	74	2.056	2.056
Velocity, f.p.s.	69	11.5	11.5
Design Discharge, Second-feet.	650,000	83.59	3.327

To expedite tests on the design of the stilling pool, so as not to delay construction, the first model built with a wooden understructure covered with sheet metal was used for Tests 1 to 66, inclusive.

The section was bolted to the floor of the three-foot flume, which had been previously used for similar models. The model was made water-tight by soldering the ends to the flume lining. The aprons and buckets were made of galvanized iron to facilitate fastening the base of the model to the floor. Piers of redwood were fastened to the crest by screws through the covering into the understructure of the model. The gates were made of heavy galvanized iron rolled to the proper radius and placed in an arc-shaped groove in the piers.

Although the design of the dam was changed from time to time during the period of preliminary testing, the same model, with only the most critical dimensions changed, was used throughout. The most critical dimension in this case was the difference in elevation of the crest and apron.

When the general form of scour protection for the toe of the Wheeler Dam had been determined, the design had been so materially changed that it was necessary to replace the model before making investigations concerning other parts of the structure.

Because of previous experience with the distortion of wooden models by swelling from contact with water, the least amount of wood possible was used in the construction of the second model, which was used in Test 67, et seq. Redwood was used in most cases, because, when properly treated it was least affected by moisture.

The base of the model was a framework of five angle iron bents mounted on channel iron and installed in the same manner as the first model. The upstream and downstream faces, of galvanized iron, were fastened to the bents with screws. The redwood crest, treated with hot linseed oil, was fastened with screws to the tops of the bents. Smooth and water-tight joints between the crest and the faces were made by rabbeting the crest and fastening the edges of the galvanized iron with screws into the rabbet. Piers were also made of redwood and were attached to the crest by wood screws. Galvanized iron Tainter gates were rolled to the proper radius and supplied with hinges to simulate the operation on the prototype. The aprons tested on the first model were used and were installed in the same manner as previously described.

When the model was completely installed and ready for operation, it was carefully finished with emery and garnet paper and painted with extra bright aluminum paint to prevent rusting of exposed metal surface, swelling of wood surfaces, and to improve the photographic qualities.

IV. PROCEDURE.

The design of the Wheeler spillway from a hydraulic standpoint involved one major problem with a number of subdivisions. The major problem was to determine the best form of stilling pool for dissipating the vast amount of energy contained in the overfalling water without causing damage to the dam or surroundings. The following is an outline of the problems investigated by means of the model:

A. Type of Scour Portection.

(1) Four General Types.

a. Shaping the bucket to throw the overfalling stream away from the base of the dam.

b. Raising the tailwater level by a secondary dam.

c. Deepening the pool below the dam.

d. Various forms of baffles.

B. Intermediate Training Wall.

(1) Differential Pressures.

(2) Usefulness.

C. Pier Studies.

(1) Design of Downstream Nose.

D. Crest Studies.

(1) Gate Operating Program.

(2) Discharge Coefficients.

a. First Model.

b. Final design.

(3) Discharge Diagrams.

a. Adjacent gates discharging.

b. Adjacent gates not discharging.

The four types of protection for Case I conditions were all tested and in some cases combined in order to improve the conditions in the stilling pool. Many major forms were also tested with a number of different shapes.

For the study pertaining to scour protection, gravel was used to simulate the river bed downstream from the apron. In previous model studies in the laboratory, some consideration has been given this subject. Conclusions were that the size of material should depend upon the scale ratio of the model and on the characteristics of the river bed, i.e. a larger material would be used for a model with a scale ratio of 1:20 than one with a ratio of 1:100. Also larger material should be used when the river bed was solid rock, the size depending upon the degree of hardness. No fixed ratio has been determined, but the selection of the size and characteristics of the material is left to the judgment of the testing engineer with the above only as a guide. However, it was kept in mind that if satisfactory conditions could be obtained with material representing a river bed, composed of loose material such as boulders and gravel, a certain factor of safety was taken into account, especially in the case of the Wheeler Dam site where the river bed is presumably of solid rock with a laminated horizontal strata fairly well bonded together. With a scale ratio of 1:36 for the model, a gravel containing some fair sized stones was used. Even with the size used, it is believed to give more severe conditions than will be encountered at the dam site.

The size of the gravel was obtained by a screen analysis of a representative sample and is shown by the following tabulation:

> 100% passed a 2-inch mesh; 98.7% a $1\frac{1}{2}$-inch; 91.9% a 1-inch; 81.8% a $\frac{3}{4}$-inch; 70% a $\frac{1}{2}$-inch; 51% a 3/8-inch; 37% a No. 4; 28.2% a No. 10; 15.1% a No. 20; 7.4% a No. 30; 3.6% a No. 40; 1.2% a No. 50; 0.55% a

No. 80; 0.21% a No. 100; and 0.08% a No. 200.

Each test was given a number, which was chronological in nature, as was each run of a test. Pictures were given a letter also of chronological nature. That is, for photographs the number was such that it could be associated with its proper test and run. A picture with the number 1-A-2 is interpreted as Test or Set-up 1, Picture A of Run 2. In each picture the number was placed where it could not interfere with the photography.

During the period of testing several radical changes in design were made by the design office, and it is mainly to clarify this report that the following table, which outlines the design conditions, was prepared.

Design Conditions.

Cond. No.	Max. Disch. Sec-ft.	Crest Elev.	Max. W.S. Elev.	Controlled W.S. Elev.	Design Head Feet	Length Net Spill	Test No.
1	700,000	535	555	555	20	2050	1 & 2
2	700,000	541	555	555	14	3500	3-26
3	700,000	541	557	555	16	2865	27 & 28
4	733,000	541	557	555	14	3000	29-32
5	650,000	541	?	555	?	2880	33 & 34
6	650,000	541	558	555	17	2400	35-81

Numerous minor changes were made on the model during the period of testing and although each is given in detail on ensuing figures, the record is not complete. To clarify the text the following outline was prepared.

Test No. 1 - Design Condition No. 1.

Set-up: Level Depressed Apron at Elevation 484.

Various pool lengths with end of pool paved on a 1:1 slope to Elevation 498 (Figure 6, Test 1).

The purpose of this test was to determine the length of pool necessary to form a satisfactory jump on the apron.

The results show a minimum length of pool as well as a minimum depth is required to form a satisfactory jump. Although this design was satisfactory from the standpoint of hydraulics and erosion, it was not recommended because of the undesirably deep excavation necessary at the toe of the dam and the excessive quantity of excavated material.

Test No. 2 - Design Condition No. 1.

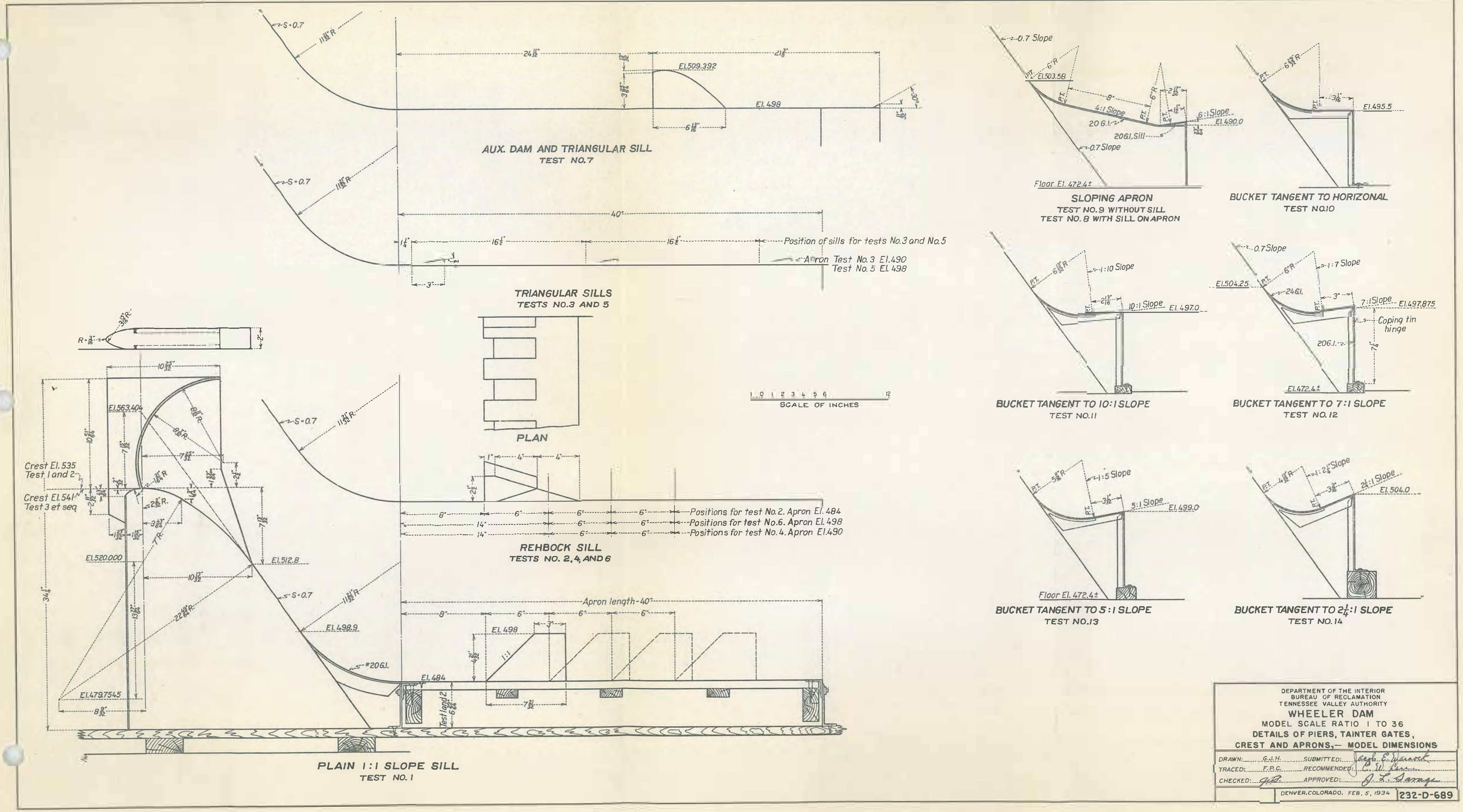
Set-up: Level Depressed Apron at Elevation 484 with the Rehbock Dentated Sill (Figure 6, Test 2).

The purpose of this test was to determine the most satisfactory position of the sill. The design condition was changed before the test was finished.

Test No. 3 - Design Condition No. 2.

Set-up: Level Depressed Apron at Elevation 490. Three triangular sills suggested by Mr. Savage (Fig. 6, Test 3).

FIGURE 6


AUX. DAM AND TRIANGULAR SILL
TEST NO.7
TRIANGULAR SILLS
TESTS NO.3 AND 5
Position of sills for tests No.3 and No.5
Apron Test No.3 El.490
Test No.5 El.498
PLAN
REHBOCK SILL
TESTS NO. 2, 4, AND 6
Positions for test No.2. Apron El.484
Positions for test No.6. Apron El.498
Positions for test No.4. Apron El.490
Apron length-40"
PLAIN 1:1 SLOPE SILL
TEST NO.1
Crest El.535 Test 1 and 2
Crest El.541 Test 3 et seq
SCALE OF INCHES
SLOPING APRON
TEST NO.9 WITHOUT SILL
TEST NO.8 WITH SILL ON APRON
Floor El. 472.4±
BUCKET TANGENT TO HORIZONAL
TEST NO.10
BUCKET TANGENT TO 10:1 SLOPE
TEST NO.11
BUCKET TANGENT TO 7:1 SLOPE
TEST NO.12
Coping tin hinge
BUCKET TANGENT TO 5:1 SLOPE
TEST NO.13
BUCKET TANGENT TO 2¼:1 SLOPE
TEST NO.14
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
MODEL SCALE RATIO 1 TO 36
DETAILS OF PIERS, TAINTER GATES,
CREST AND APRONS,— MODEL DIMENSIONS
DRAWN: G.J.H. SUBMITTED: Jacob E. Warnock
TRACED: F.P.C. RECOMMENDED: C. W. Lee
CHECKED: J.B. APPROVED: J. L. Savage
DENVER, COLORADO. FEB. 5, 1934
232-D-689

The purpose of this test was to determine the effect of the sills on the jump. Although the sills aided in the retardation of the water, their action was not sufficient to allow a reduction in pool depth. It would have been necessary to lower the apron to produce satisfactory conditions.

Test No. 4 - Design Condition No. 2.

Set-up: Level Depressed Apron at Elev. 490 with Rehbock Dentated Sill at Various Positions (Fig. 6, Test 4).

The purpose of this test was to determine the position of the sill to give satisfactory conditions and to determine if the pool depth was sufficient.

The test proved that the sill necessitated considerably less excavation, but that the sill depended largely upon direct impact for its action. It was indicated also that a slightly deeper pool would improve the conditions at the higher discharges.

Test No. 5 - Design Condition No. 2.

Set-up: Level Apron at Elev. 498 with three triangular sills suggested by Mr. Savage (Fig. 6, Test 5).

Same purpose as in Test 3.

Same results as in Test 3.

Test No. 6 - Design Condition No. 2.

Set-up: Level Apron at Elev. 498 with Rehbock Dentated Sill at various positions (Fig. 6, Test 6).

The purpose of this test was to determine if the pool depth was sufficient to obtain satisfactory action with sills at various positions.

The test showed that the apron was not sufficiently low.

Test No. 7 - Design Condition No. 2.

Set-up: Level Apron at Elev. 498 with Secondary Dam and Triangular Sill (Figure 6, Test 7).

The purpose of this test was to determine the tail-water elevation required below the secondary dam and to determine the action of the secondary dam.

The test showed that the secondary apron would have to be lowered slightly or a third dam built to form a pool below the secondary dam.

This set-up was not recommended because of the initial cost and because it would offer difficulty in construction.

Test No. 8 - Design Condition No. 2.

Set-up: Sloping Apron at Elev. 490 with an 18-foot radius bucket; 4:1 slope and a triangular sill (Fig. 6, Test 8).

The purpose of this test was to determine the proper elevation of the apron to give satisfactory conditions.

The test showed that the apron produced good conditions if lowered slightly.

When the apron was lowered, however, it required too deep an excavation at the toe of the dam.

Test No. 9 - Design Condition No. 2.

Set-up: Same as in Test 8 except triangular sill removed (Figure 6, Test 9).

The purpose of this test was to determine the necessity of a sill at the end of the apron.

The test proved that a sill reduced considerably the length of apron required for satisfactory jump conditions.

Test No. 10 - Design Condition No. 2.

Set-up: Deflecting Bucket with 20.7-foot radius and a 9-foot horizontal lip at Elev. 495.5 (Fig. 6, Test 10).

The purpose of this test was to determine the elevation of the bucket required to give best results with normal tailwater conditions.

The test showed unstable conditions and excessive erosion near the dam.

<u>Test No. 11 - Design Condition No. 2.</u>

Set-up: Deflecting Bucket with 19.2-foot radius and lip on 10:1 slope to Elev. 497 (Figure 6, Test 11).

Same results as in Test 10.

<u>Tost No. 12 - Design Condition No. 2.</u>

Set-up: Same as Test 11, except radius of bucket 18 feet with lip on 7:1 slope to Elev. 497.875 (Fig. 6, Test 12).

Same purpose as in Test 10.

Same results as shown by Test 11, except erosion is deeper and farther downstream.

<u>Test No. 13 - Design Condition No. 2.</u>

Set-up: Same as Test 11, except radius of bucket 17 feet with lip on 5:1 slope to Elev. 499 (Fig. 6, Test 13).

Same purpose as in Test 10.

Same results as in Test 12 except erosion is farther downstream.

<u>Test No. 14 - Design Condition No. 2.</u>

Set-up: Same as Test 13, except radius of bucket 13.2 feet with lip on $2\frac{1}{4}$:1 slope to Elev. 504 (Fig. 6, Test 14).

Same purpose as in Test 10.

Same results as in Test 13 except erosion is farther downstream.

Test No. 15 - Design Condition No. 2.

Set-up: Same as Test 13, except trajectory-shaped apron extended downstream from bucket lip (Fig. 7, Test 15).

The purpose of this test was to stabilize conditions in the bucket.

The test showed that hydraulic conditions were stabilized, but that the jet clung to the apron and deep erosion occurred at end of apron. A satisfactory condition was obtained by lowering the apron, but again deep excavation was required near the toe of the dam.

Test No. 16 - Design Condition No. 2.

Set-up: Same as Test 15, except triangular sill at end of apron (Figure 7, Test 16).

The purpose of this test was to reduce erosion at end of apron from that in Test 15.

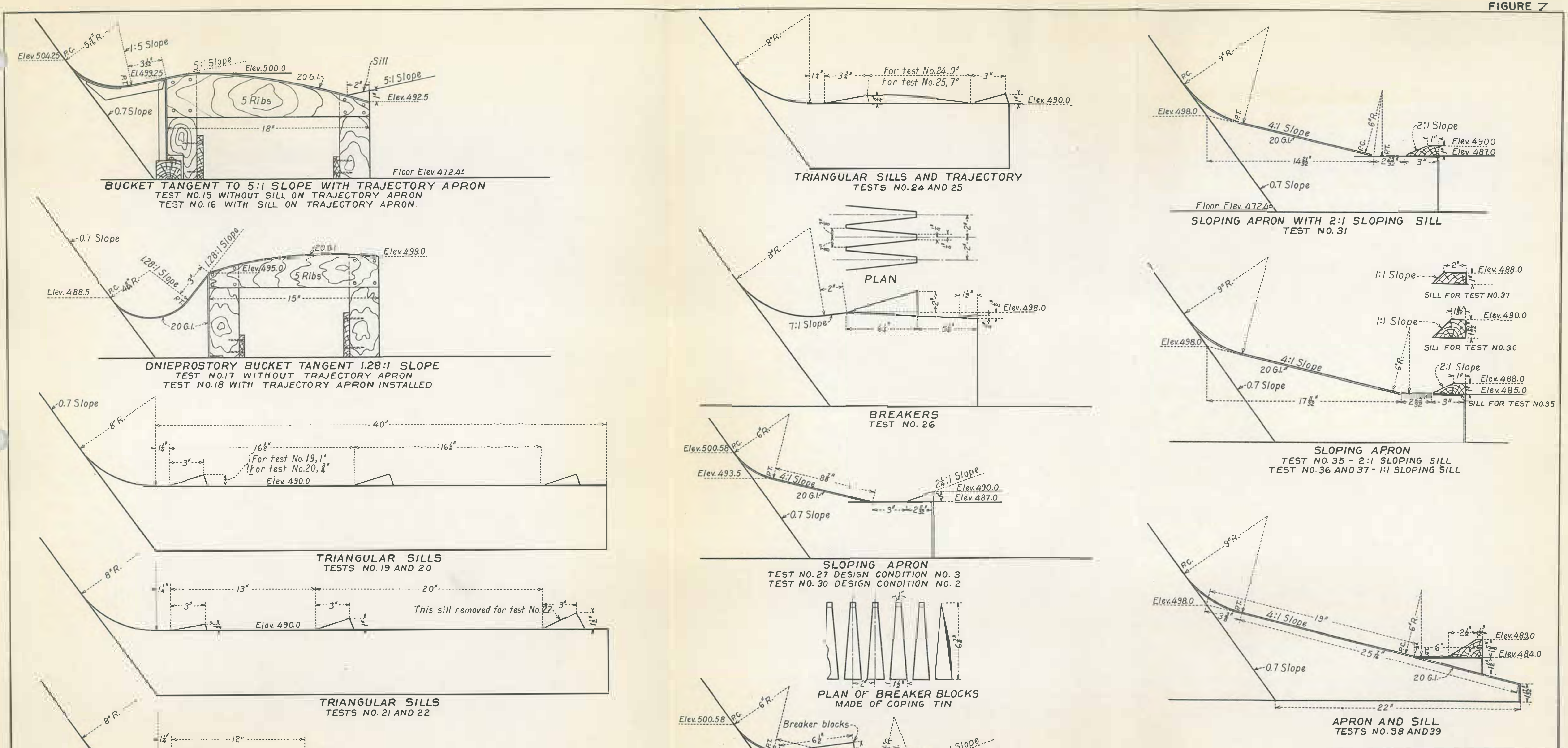
The test showed that erosion was slightly reduced.

Test No. 17 - Design Condition No. 2.

Set-up: Dnieprostroy type bucket with 12.5-foot radius and a 9-foot lip on a 1.28:1 slope to Elev. 495 (Figure 7, Test 17).

The purpose of this test was to determine whether the set-up would be satisfactory, and, if so, at what elevation it would give satisfactory conditions with normal tailwater.

FIGURE 7



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

WHEELER DAM
MODEL SCALE RATIO 1 TO 36
DETAILS OF BUCKETS, APRONS, AND SILLS
MODEL DIMENSIONS

DRAWN: R.R.B. SUBMITTED: Jacob E. Warnock
TRACED: M.C.G. RECOMMENDED: E. W. Lane
CHECKED: J.W.B. APPROVED: J. L. Savage

DENVER, COLO., FEB. 26, 1934 232-D-688

The test showed minimum erosion and satisfactory conditions, however, deep excavation at the toe of the dam was still required.

Test No. 18 - Design Condition No. 2.

Set-up: Same as Test 17, except for curved apron extended downstream from bucket lip (Figure 7, Test 18).

The purpose of this test was to reduce the roughness of the tailwater surface.

The test showed that bad conditions were intensified.

Test No. 19 - Design Condition No. 2.

Set-up: Level depressed apron at Elev. 490 with three 3-foot triangular sills on apron (Figure 7, Test 19).

Same purpose as in Test 3.

Same results as in Test 3.

Test No. 20 - Design Condition No. 2.

Set-up: Same as in Test 19 except $2\frac{1}{4}$-foot sills (Fig. 7, Test 20).

Same purpose as in Test 3.

Same results as in Test 3.

Test No. 21 - Design Condition No. 2.

Set-up: Same as Test 19, except sills $1\frac{1}{2}$, 3 and $4\frac{1}{2}$ feet high (Figure 7, Test 21).

Same purpose as in Test 3.

Same results as in Test 3.

Test No. 22 - Design Condition No. 2.

Set-up: Same as Test 21, except $4\frac{1}{2}$-foot sill removed (Figure 7, Test 22).

Same purpose as in Test 3.

Same results as in Test 3.

Test No. 23 - Design Condition No. 2.

Set-up: Same as Test 22, except sills $2\frac{1}{4}$ and 3 feet high and closer spacing (Figure 7, Test 23).

Same purpose as in Test 3.

Same results as in Test 3.

Test No. 24 - Design Condition No. 2.

Set-up: Same as Test 23, except trajectory shape between sills (Figure 7, Test 24).

The purpose of this test was to make the second sill more active.

The test showed no difference in the action of the sills.

Test No. 25 - Design Condition No. 2.

Set-up: Same as Test 24, except trajectory shape shortened (Figure 7, Test 25).

Same purpose as in Test 24.

Same results as in Test 24.

Test No. 26 - Design Condition No. 2.

Set-up: Deflecting type bucket with horizontal apron at Elev. 496.6 and breakers (initial type diffuser sill suggested by Mr. Hornsby) (Fig. 7, Test 26).

The purpose of this test was to study the action of this type sill and to determine its feasibility.

The test showed that this sill offered some reduction in excavation and aided in the formation of the jump, but not enough to balance the cost of the sill. This sill had the advantage in that it did not depend upon direct impact for its action.

Test No. 27 - Design Condition No. 3.

Set-up: An 18-foot radius bucket with 4:1 sloping apron to Elev. 487 with a 3-foot triangular sill at end (Figure 7, Test 27).

The purpose of this test was to study the action of the sloping apron in relation to the formation of the hydraulic jump.

The test showed that, while conditions were not as good as desired, there was a possibility of obtaining an improvement by lowering the apron.

Test No. 28 - Design Condition No. 3.

Set-up: An 18-foot radius bucket with 5:1 sloping apron to Elev. 487.5; breaker blocks and 3-foot sill (Figure 7, Test 28).

The purpose of this test was to improve conditions without lowering the apron.

The test showed insufficient improvement.

Test No. 29 - Design Condition No. 4.

Set-up: Same as Test 28, (Figure 7, Test 29).

Same purpose as in Test 28.

Same results as in Test 28.

Test No. 30 - Design Condition No. 4.

Set-up: Same as Test 27.

Same purpose as in Test 27.

Same results as in Test 27.

Test No. 31 - Design Condition No. 4.

Set-up: A 27-foot radius bucket with 4:1 sloping apron to Elev. 487 and 3-foot trapezoidal sill. Rock on 3:1 slope from top of sill to Elev. 498 (Fig. 7, Test 31).

The purpose of this test was to further develop the possibilities of the sloping apron.

The test showed some improvement, but indications were that if the apron were lowered, better conditions would result.

Test No. 32 - Design Condition No. 1.

Set-up: Same as Test 31.

The purpose of this test was to determine a coefficient curve for the first model (wooden understructure).

Test No. 33 - Design Condition No. 5.

(Obsolete due to information from the design office being incomplete).

Test No. 34 - Design Condition No. 5.

(Obsolete due to information from design office being incomplete. Superseded by Test 35).

Test No. 35 - Design Condition No. 6.

Set-up: Apron with 4:1 slope tangent at upper end of 27-foot radius and at lower end to 18-foot radius. Horizontal apron at Elev. 485.0. Sill with 2:1 slope to Elev. 488.0 and top width of 3 feet. Rock on 3:1 slope to Elevation 498 (Fig. 7, Test 35).

Same purpose as in Test 31.

The test showed that the depth of pool was insufficient at maximum discharge.

Test No. 36 - Design Condition No. 6.

Set-up: Same as Test 35 except sill with 1:1 slope to Elev. 490 and top width of 4 feet (Figure 7, Test 36).

Same purpose as in Test 31.

The test showed less erosion, but worse hydraulic conditions.

Test No. 37 - Design Condition No. 6.

Set-up: Same as Test 35, except sill with 1:1 slope to Elev. 488.0 and top width of 6 feet (Fig. 7, Test 37).

Same purpose as in Test 31.

The test showed results practically the same as in Test 35.

Test No. 38 - Design Condition No. 6.

Set-up: Same as Test 36 except with horizontal apron at Elev. 484.0. Sill with $1\frac{1}{2}$:1 slope to Elev. 489 and top width of $1\frac{1}{2}$ feet (Figure 7, Test 38).

Same purpose as in Test 31.

The test showed satisfactory conditions at all flows below 650,000 second-feet. With a discharge of 650,000 second-feet, the tailwater was barely deep enough to cause the hydraulic jump to form. This design was tentatively recommended as a general type of scour protection subject to further development.

Test No. 39 - Design Condition No. 6.

Set-up: Same as Test 38.

The purpose of this test was to determine the tail-water elevation at which the hydraulic jump failed to form.

The results are plotted on Figure 26.

Test No. 40 - Design Condition No. 1.

Set-up: Crest designed for 20-foot head with Tainter gate raised 9 feet.

The purpose of this test was to obtain discharge data for Tainter gates.

Test No. 41 - Design Condition No. 1.

Set-up: Same as Test 40, except gate raised $1\frac{1}{2}$ feet.

Same purpose as in Test 40.

Test No. 42 - Design Condition No. 1.

Set-up: Same as Test 40, except gate raised 3 feet.

Same purpose as in Test 40.

Test No. 43 - Design Condition No. 1.

Set-up: Same as Test 40, except gate raised $4\frac{1}{2}$ feet.

Same purpose as in Test 40.

Tests 44, 45, 46 and 47.

These test numbers were earmarked for additional calibration tests similar to Tests 40 to 43 inclusive. Before they could be made, however, the first model was removed. Similar tests were made on the final model.

Test No. 48 - Design Condition No. 6.

Set-up: A 43.5-foot radius bucket with 8:1 sloping apron. Horizontal apron at Elev. 484; Sill with $1\frac{1}{2}$:1 slope to Elev. 489 and top width of $1\frac{1}{2}$ feet. Rock on 3:1 slope to Elev. 498 (Figure 8, Test 48).

The purpose of this test was to make additional study of the use of a sloping apron to obtain satisfactory foundation conditions combined with satisfactory hydraulic conditions, particularly with a flow of 650,000 second-feet.

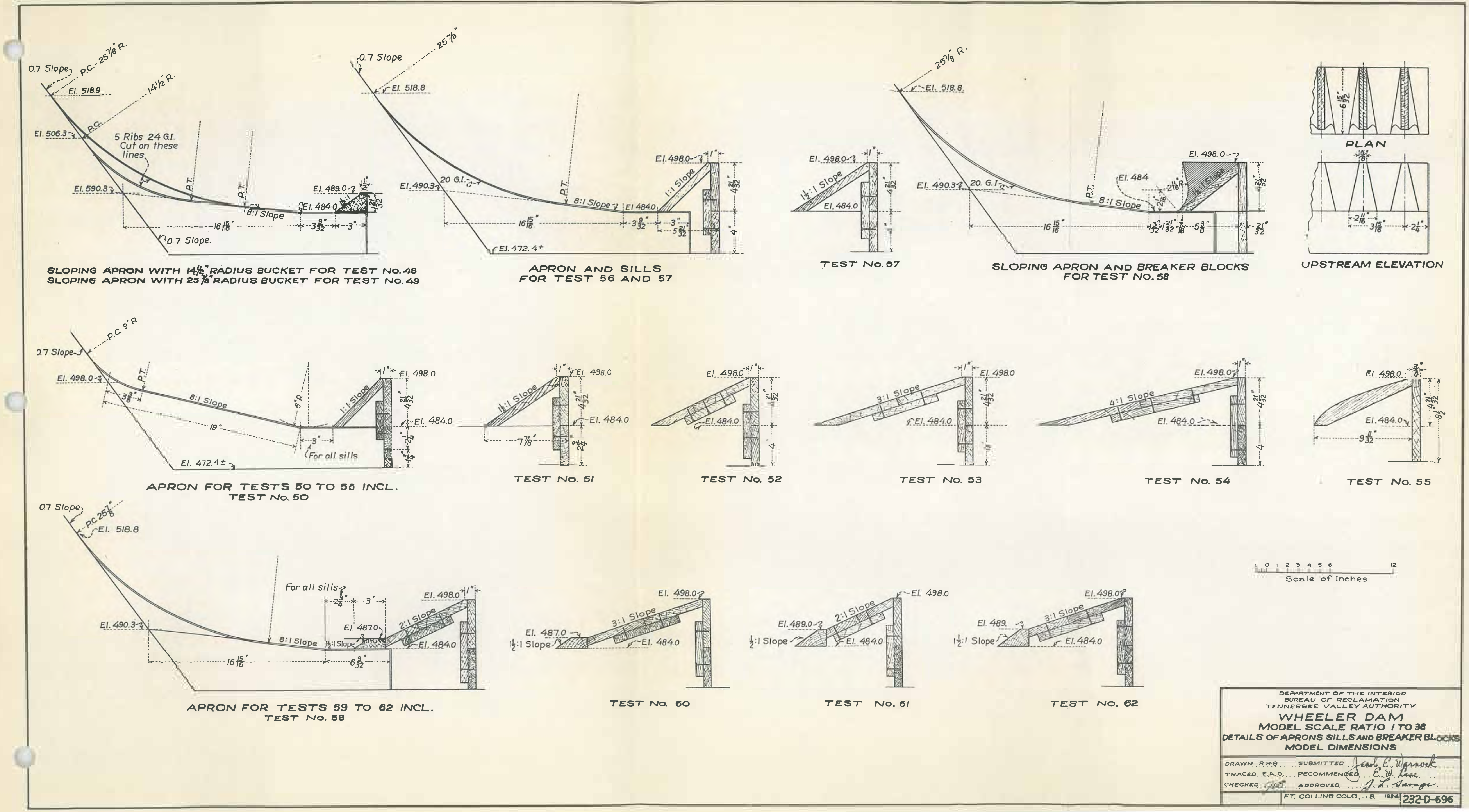
The test showed that conditions were somewhat improved but the excavation required at the toe of the dam was too deep to meet the requirements of the design office.

Test No. 49 - Design Conditions No. 6.

Set-up: Same as Test 48 except 77.6-foot radius bucket (Figure 8, Test 49).

Same purpose as in Test 48.

While the hydraulic conditions in Test 49 were not as satisfactory as in Test 48, they were superior to those in Test 38, and the requirements of the design office as to foundation conditions were satisfied. The 77.6-foot radius bucket with 8:1 sloping apron and horizontal apron at Elev. 484.0 was adopted as the final design with the sill lay-out still subject to change.



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

WHEELER DAM
MODEL SCALE RATIO 1 TO 36
DETAILS OF APRONS SILLS AND BREAKER BLOCKS
MODEL DIMENSIONS

DRAWN R.R.S. SUBMITTED Jacob E. Warnock
TRACED E.A.O. RECOMMENDED E. W. Lane
CHECKED APPROVED J. L. Savage
FT. COLLINS COLO., FEB. 1934 232-D-696

Test No. 50 - Design Condition No. 6.

Set-up: Apron with 4:1 slope tangent at upper end to 27-foot radius and at lower end to 18-foot radius. Horizontal apron at Elev. 484; pavement on 1:1 slope from end of apron to Elev. 498 (Fig. 8, Test 50).

The purpose of this test was to determine the feasibility of paving the rock excavation at the downstream end of the apron instead of constructing a sloping sill.

Hydraulic conditions were rough but the jump formed on the apron at all discharges.

Test No. 51 - Design Condition No. 6.

Set-up: Same as Test 50, except pavement slope was $1\frac{1}{2}$:1 (Figure 8, Test 51).

Same purpose as in Test 50.

The jump did not form on the apron at medium flows. Hydraulic conditions were smoother than with the 1:1 slope when the jump was formed on the apron.

Test No. 52. - Design Condition No. 6.

Set-up: Same as Test 50, except pavement slope was 2:1 (Figure 8, Test 52).

Same purpose as in Test 50.

Same results as in Test 51.

Test No. 53 - Design Condition No. 6.

Set-up: Same as Test 50, except slope of pavement was 3:1 (Figure 8, Test 53).

Same purpose as in Test 50.

Same results as in Test 51.

Test No. 54 - Design Condition No. 6.

Set-up: Same as Test 50, except slope of pavement was 4:1 (Figure 8, Test 54).

Same purpose as in Test 50.

Same results as in Test 51.

Test No. 55 - Design Condition No. 6.

Set-up: Same as Test 50, except pavement was parabolic in shape (Figure 8, Test 55).

Same purpose as in Test 50.

Same results as in Test 51.

Test No. 56 - Design Condition No. 6.

Set-up: Same apron as in Test 49, except 1:1 slope pavement from end of apron to Elev. 498 (Fig. 8, Test 56).

Same purpose as in Test 50.

Same results as in Test 50.

Test No. 57 - Design Condition No. 6.

Set-up: Same as Test 56, except $1\frac{1}{2}$:1 slope pavement (Figure 8, Test 57).

Same purpose as in Test 50.

Same results as in Test 51.

Further tests on sloped pavement were not made because the above did not differ from tests on the 4:1 sloping apron.

Test No. 58 - Design Condition No. 6.

Set-up: Same as Test 49, except 14-foot diffuser sill (formerly called breaker blocks) (Fig. 8, Test 58).

The purpose of this test was to make further development of a sill layout as outlined in Test 49.

The test showed practically no erosion and the sill gave entirely satisfactory results, but objections were made on the grounds that the sill structure was too large and consequently too expensive.

Test No. 59 - Design Condition No. 6.

Set-up: Same as in Test 49, except sill with $1\frac{1}{2}$:1 slope to Elev. 487.0 and top width of 4.5 feet; pavement on 2:1 slope from top of sill to Elev. 498 (Fig. 8, Test 59).

The purpose of this test was to study the action of the pavement in conjunction with the $1\frac{1}{2}$:1 sloping sill.

This pavement did not allow a natural retrogression of the river bed to produce stable hydraulic conditions, and as a result, the jump would not form on the apron for medium flows.

Test No. 60 - Design Condition No. 6.

Set-up: Same as Test 59, except slope of pavement was 3:1 (Figure 8, Test 60).

Same purpose as in Test 59.

Same results as in Test 59.

Test 61 - Design Condition No. 6.

Set-up: Same as Test 59, except sill with $1\frac{1}{2}$:1 slope to Elevation 489 (Figure 8, Test 61).

Same purpose as in Test 59.

Same results as in Test 59.

Test 62 - Design Condition No. 6.

Set-up: Same as in Test 61, except slope of pavement was 3:1 (Figure 8, Test 62).

Same purpose as in Test 59.

Same results as in Test 59.

Test 63 - Design Condition No. 6.

Set-up: Same as in test 49, except with a 5-foot diffuser-type sill with curved upstream face (Figure 9, Test 63).

Same purpose as in Test 58.

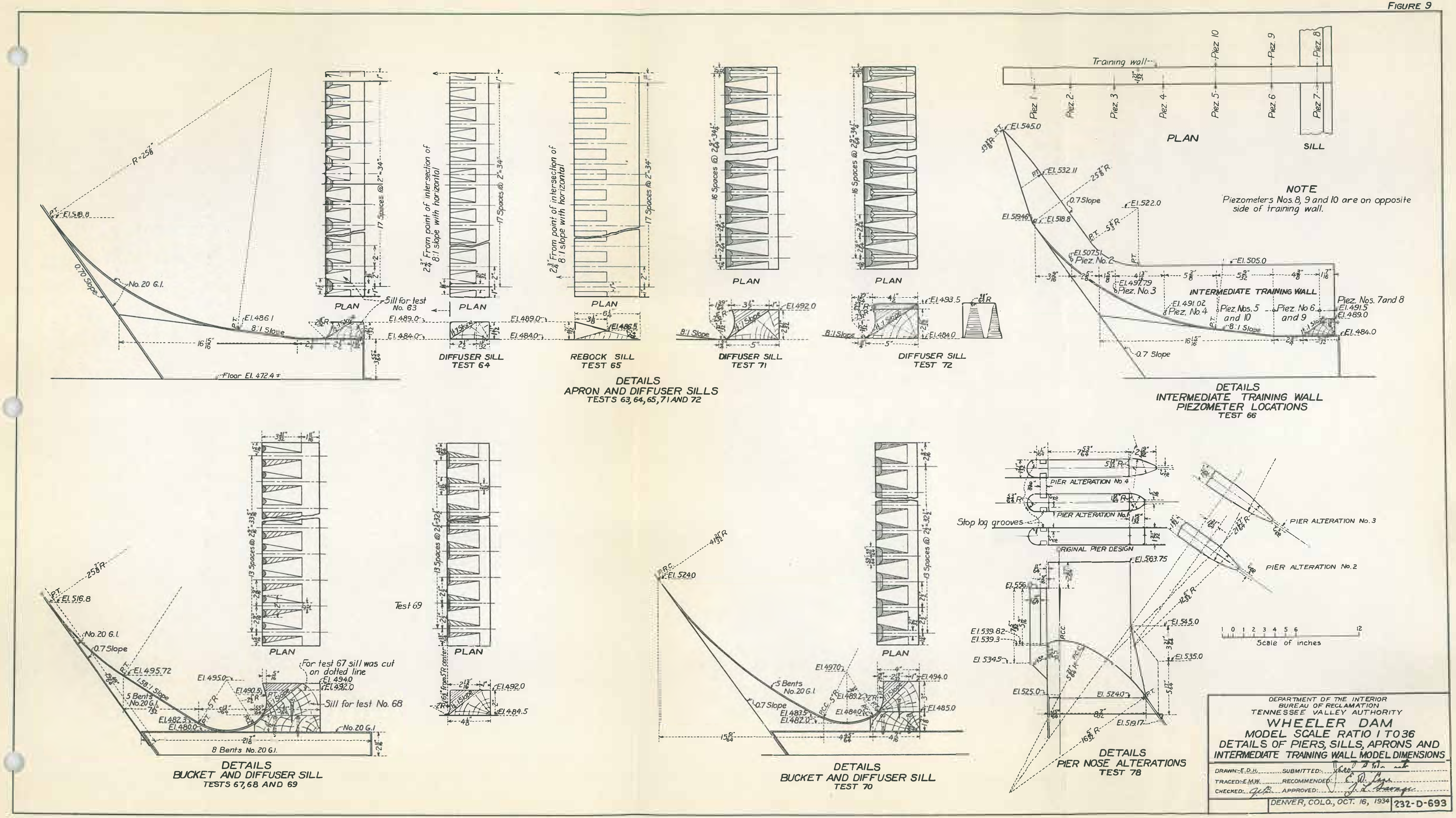
Considerable improvement over the $1\frac{1}{2}$:1 sloping sill was noted. However, additional improvement seemed possible.

Test 64 - Design Condition No. 6.

Set-up: Same as in Test 49, except with 5-foot diffuser-type sill with vertical upstream face (Figure 9, Test 64).

Same purpose as in Test 58.

Action was similar to Test 63, except that the sill was subjected to more direct impact which was objectionable.


PLAN
Sill for test No. 63
DIFFUSER SILL TEST 64
REBOCK SILL TEST 65
DIFFUSER SILL TEST 71
DIFFUSER SILL TEST 72
DETAILS
APRON AND DIFFUSER SILLS
TESTS 63, 64, 65, 71 AND 72
Training wall
SILL
NOTE
Piezometers Nos. 8, 9 and 10 are on opposite side of training wall.
INTERMEDIATE TRAINING WALL
DETAILS
INTERMEDIATE TRAINING WALL
PIEZOMETER LOCATIONS
TEST 66
For test 67 sill was cut on dotted line
Sill for test No. 68
Test 69
DETAILS
BUCKET AND DIFFUSER SILL
TESTS 67, 68 AND 69
DETAILS
BUCKET AND DIFFUSER SILL
TEST 70
PIER ALTERATION No. 4
PIER ALTERATION No. 1
ORIGINAL PIER DESIGN
Stop log grooves
PIER ALTERATION No. 3
PIER ALTERATION No. 2
Scale of inches
DETAILS
PIER NOSE ALTERATIONS
TEST 78
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
MODEL SCALE RATIO 1 TO 36
DETAILS OF PIERS, SILLS, APRONS AND
INTERMEDIATE TRAINING WALL MODEL DIMENSIONS
DRAWN: E.D.H. SUBMITTED:
TRACED: E.M.W. RECOMMENDED:
CHECKED: APPROVED:
DENVER, COLO., OCT. 16, 1934
232-D-693

Test 65 - Design Condition No. 6.

Set-up: Same as Test 49, except 5-foot Rehbock sill at end of apron and 3:1 slope from Elev. 484 to Elev. 498 (Figure 9, Test 65).

Same purpose as in Test 58.

This sill depends upon direct impact for its action and requires more excavation for the same height of sill than either the diffuser or the $1\frac{1}{2}$:1 sloping sill.

Test 66 - Design Condition No. 6.

Set-up: A 77.6-foot radius bucket with 8:1 sloping apron. Horizontal apron at Elev. 484. Sill with $1\frac{1}{2}$:1 slope to Elev. 489.0 and top width of 1.5 feet. Rock on 3:1 slope to Elev. 498.0. Intermediate training wall, downstream from every fifth pier, with top at Elevation 505.0 (Figure 9, Test 66).

The purpose of this test was to obtain design data in the training wall and study its action under extreme gate operation conditions.

The walls facilitated the formation of the jump on the apron for extreme operating conditions of adjacent sets of gates and showed no undesirable erosive conditions, except for maximum discharge.

Test 67 - Design Condition No. 6.

(First Test on Second Model)

Set-up: A 1.58:1 sloping apron tangent at the upper end to a 77.6-foot radius and at the lower end to a 15-foot radius with bottom at Elev. 480.0 and P.C.C. at Elev. 482.0. A 7.5-foot radius to Elev. 484.5. A 7.5-foot diffuser type sill with curved upstream face. Rock on 3:1 slope from top of sill at Elev. 492 to Elev. 498.0 (Figure 9, Test 67).

The purpose of this test was to study the action of the diffuser sill in conjunction with the curved bucket placed at a low elevation.

The test showed more satisfactory conditions than any previous set-up, but was not adopted because the depth of excavation required was not satisfactory to the design department.

Test 68 - Design Condition No. 6.

Set-up: Same as in Test 67, except with $9\frac{1}{2}$-foot diffuser sill (Figure 9, Test 68).

Same purpose as in Test 67.

Same results as in Test 67.

Test 69 - Design Condition No. 6.

Set-up: Same as Test 67, except with vertical upstream face of the diffuser sill and slightly closer spacing of diffusion chambers.

Same purpose as in Test 67.

Same results as in Test 67.

Test 70 - Design Condition No. 6.

Set-up: Deflecting type bucket with 125-foot radius from the face of dam to the 15-foot radius bucket with bottom at Elev. 482, lip at Elev. 484 and diffuser sill downstream from bucket lip (Figure 9, Test 70).

The purpose of this test was to determine if the excavation at the toe of the dam could be reduced without affecting the hydraulic conditions of this particular type.

The test showed that hydraulic conditions were not as desirable as with the bucket at a lower elevation.

Test 71 - Design Condition No. 6.

Set-up: Same as in Test 63, except with an 8-foot diffuser sill with spacing of diffusion chambers to conform to expansion joint spacing (Figure 9, Test 71).

The purpose of this test was to obtain a satisfactory diffuser sill with a fixed number of diffusion chambers between contraction joints.

The test showed this sill action to be very satisfactory with only slight erosion occurring at maximum discharge. Paint tests were made and indications were that a low pressure area existed on top of the chamber walls.

Test 72 - Design Condition No. 6.

Set-up: Same as Test 71, except top of diffusion chamber walls were capped (Figure 9, Test 72).

The purpose of this test was to eliminate the low pressure area on top of the walls.

The test showed the elimination of this undesirable low pressure area. Hydraulic conditions were not affected by the caps. Indications pointed to similar results by the use of a smaller sill.

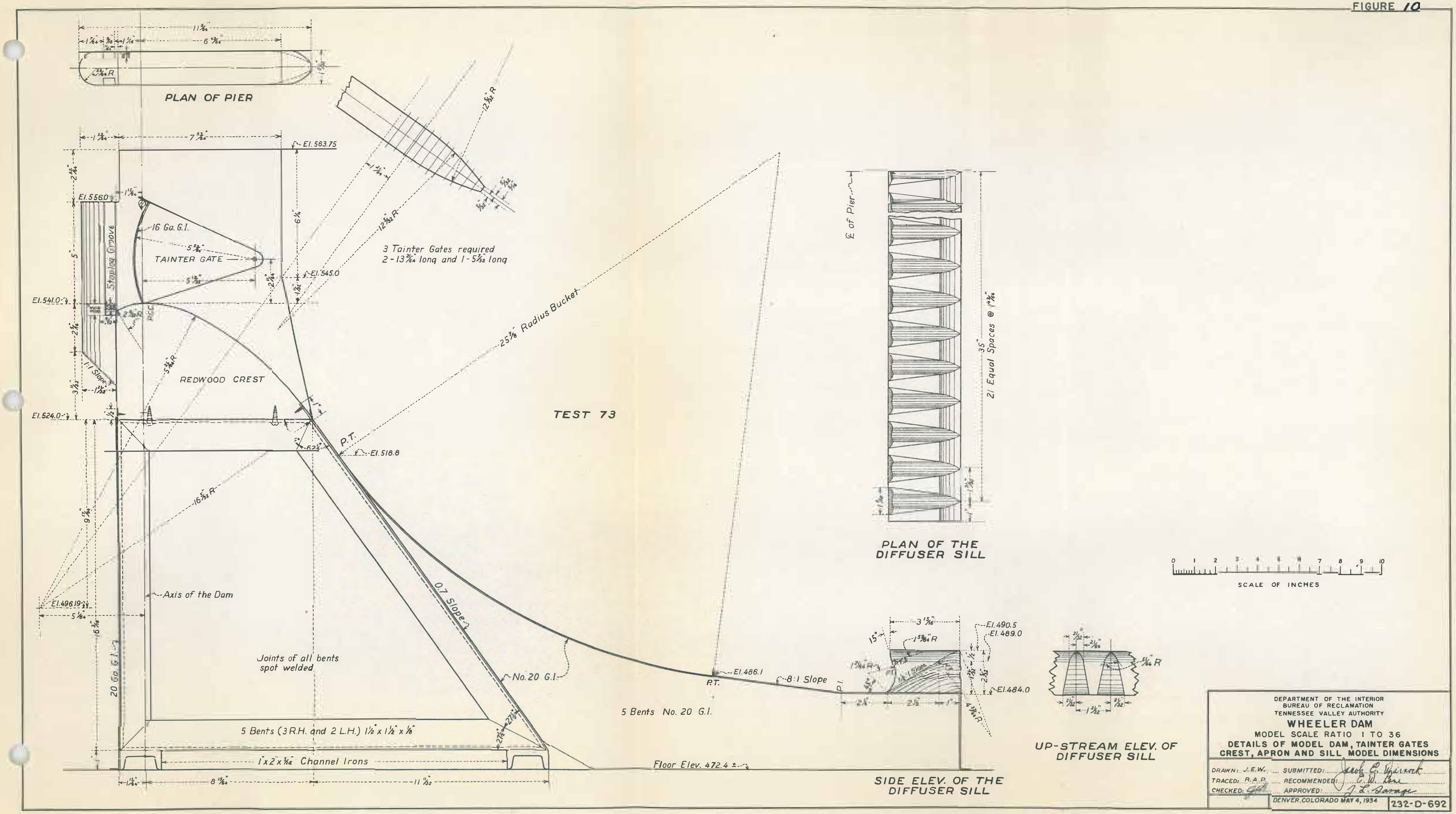
Test 73 - Design Condition No. 6.

Set-up: Same as in Test 63, except 6.5-foot diffuser sill with curved upstream face and spacing to conform to contraction joint spacing (Figure 10, Test 73).

The purpose of this test was to obtain a more economical sill that would give results comparable to those in Test 72.

The test showed that conditions were not quite as good as with the higher sill, but were superior to any other type tested. As this sill gave adequate protection for all flows, including maximum discharge of 650,000 second-feet, it was adopted for use on the Wheeler Dam.

FIGURE 10



PLAN OF PIER
3 Tainter Gates required
2 - 13 7/64" long and 1 - 5 3/32" long
El. 563.75
El. 556.0
16 Ga. G.I.
TAINTER GATE
Stoplog Groove
El. 545.0
El. 541.0
REDWOOD CREST
1-1 Slope
25 7/8" Radius Bucket
El. 524.0
TEST 73
P.T.
El. 518.8
Axis of the Dam
El. 496.19
0.7 Slope
20 Ga. G.I.
Joints of all bents spot welded
No. 20 G.I.
El. 486.1
P.T.
8:1 Slope
5 Bents No. 20 G.I.
5 Bents (3 R.H. and 2 L.H.) 1 1/2" x 1 1/2" x 1/8"
1" x 2" x 5/16" Channel Irons
Floor Elev. 472.4 ±
℄ of Pier
21 Equal Spaces @ 1 43/64"
PLAN OF THE DIFFUSER SILL
SCALE OF INCHES
El. 490.5
El. 489.0
El. 484.0
SIDE ELEV. OF THE DIFFUSER SILL
UP-STREAM ELEV. OF DIFFUSER SILL
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
MODEL SCALE RATIO 1 TO 36
DETAILS OF MODEL DAM, TAINTER GATES
CREST, APRON AND SILL MODEL DIMENSIONS
DRAWN: J.E.W.
TRACED: R.A.P.
CHECKED:
SUBMITTED: Jacob E. Warnock
RECOMMENDED: E. W. Lane
APPROVED: J. L. Savage
DENVER, COLORADO MAY 4, 1934
232-D-692

Test 74 - Design Condition No. 6.

Set-up: Same as in Test 73. Piezometers in diffusion chamber (Figure 10, Test 73).

The purpose of this test was to obtain the pressures within the diffusion chamber of the sill.

The test showed very slight negative pressures at the entrance with a discharge of 650,000 second-feet.

Test 75 - Design Condition No. 6.

Set-up: Same as in Test 73 (Figure 10, Test 73).

The purpose of this test was to obtain discharge coefficients for the final design.

This test was superseded because leaks were found in the flume upstream from the model.

Test 76 - Design Condition No. 6.

Set-up: Same as in Test 73 (Figure 10, Test 73).

The purpose of this test was to obtain extensive calibration data on the Tainter gates with adjacent gates discharging.

A discharge diagram was prepared and coefficients computed from the observed data.

Test 77 - Design Condition No. 6.

Set-up: Same as in Test 73 (Figure 10, Test 73).

The purpose of this test was to obtain the coefficient of discharge for one gate only discharging with free flow conditions.

A coefficient of discharge curve was plotted from the observed data.

Test 78 - Design Condition No. 6.

Set-up: Same as in Test 73, except alterations were made on piers (Figure 9, Test 78).

The purpose of this test was to determine the shape of the downstream nose of the piers to give best conditions on the apron.

Several shapes were tried and as a result a satisfactory solution was obtained (Alteration No. 2, Figure 9, Test 78).

Test 79 - Design Condition No. 6.

Set-up: Same as in Test 73.

The purpose of this test was to obtain discharge data for one Tainter gate only discharging.

Same results as in Test 76.

Test 80 - Design Condition No. 6.

Set-up: Same as in Test 73. Piezometers in the diffuser sill.

The purpose of this test was to obtain pressures on the upstream face of the diffusion chamber walls.

Test 81 - Design Condition No. 6.

Set-up: Final Design.

The purpose of this test was to obtain a gate operating program.

A satisfactory program was prepared from the observed data.

V. RESULTS AND CONCLUSIONS.

The experiments on the model of the spillway for the Wheeler Dam extended over a period of ten months. During this same period, active experimental work was being conducted on a model of the ultimate development of the Grand Coulee Dam where a problem of similar, but more severe, nature was being studied. It was there that the first development was made of the diffuser type sill which was finally adopted for use on the apron of the Wheeler Dam.

Since the outline given previously is chronological in nature, the tests have been rearranged as to types to simplify the compilation of results and conclusions. An outline is as follows:

A. Stilling Pool

1. Level Depressed Apron

a. Level Depressed Apron - Test 1 (Figure 6).

b. Rehbock Sill - Tests 2, 4 and 6 (Figure 6).

c. Triangular Sills - Tests 3 and 5 (Figure 6); 19, 20, 21, 22, 23, 24, and 25 (Figure 7).

d. Breaker Blocks - Test 26, Figure 7.

e. Secondary Weir - Test 7, Figure (6).

2. Deflecting Bucket

a. Deflecting Bucket (High elevation) - Test 10, 11, 12, 13 and 14 (Figure 6) and 15 (Figure 7).

b. Deflecting Bucket (high elevation) with trajectory apron - Test 16 (Figure 7).

c. Deflecting Bucket (low elevation) Tests 17 and 18 (Figure 7).

d. Deflecting Bucket with Diffuser Sill - Tests 67, 68, 69 and 70 (Figure 9).

3. Sloping Apron

a. Triangular Sill - Tests 8 and 9 (Figure 6) and Tests 27 and 30 (Figure 7).

b. Sloping Sills

1. Apron with 4:1 Slope - Tests 8 and 9 (Figure 6) and Tests 27, 30, 31, 35, 36, 37 and 38 (Figure 7).

2. Apron with 8:1 Slope - Tests 48 and 49 (Figure 8).

c. Paved Slopes

1. Apron with 4:1 Slope - Tests 50-54 incl., (Figure 8).

2. Apron with 8:1 Slope - Tests 56 and 57 (Figure 8).

d. Combination Sill and Paved Slope

1. Three-foot Sill - Tests 59 and 60 (Figure 8).

2. Five-foot Sill - Tests 61 and 62 (Figure 8).

e. Breakers - Tests 28 and 29 (Figure 7).

f. Diffuser Sills - Test 58 (Figure 8), 63, 64, 65, 71, 72 (Figure 9), and 73 (Figure 10).

B. Intermediate Training Wall.

C. Pier Studies.

D. Gate Operating Program.

1. With Dividing Wall.

2. Without Dividing Wall.

E. Tainter Gate Discharge Data.

1. Discharge Coefficient.

2. Discharge Diagram.

A. Stilling Pool.

1. Level Depressed Apron.

The simplest and perhaps the most perfect solution of the dissipation of energy in water flowing at a high velocity is the ideal formation of a hydraulic jump. The energy is dissipated in internal impact to such an extent that little energy remains to cause erosion. In the case of overfall dams, there is a distinct advantage in designing the dam so that the jump will occur, thus reducing to a minimum the protection required to prevent erosion, especially since the action can be predicted within reasonably close limits.

The formula for the hydraulic jump in a rectangular horizontal channel is

$$D_2 = \frac{-D_1}{2} + \sqrt{\frac{2 V_1^2 D_1}{g} + \frac{D_1^2}{4}}$$

where D_1 = the depth upstream from the jump,

D_2 = the depth downstream from the jump,

V_1 = the velocity corresponding to D_1

V_2 = the velocity corresponding to D_2.

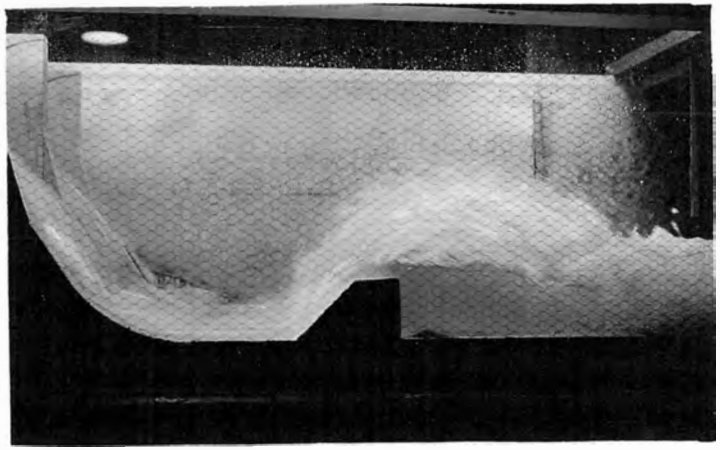
As outlined in Appendix I, "The ideal condition would be to have a tailwater at such a height above the river bed for each discharge that it would form a perfect jump for the depth and velocity which would occur in the overfalling stream at the toe of the dam for that discharge. The height of the tailwater, however, is controlled by the conditions in the stream channel downstream from the dam and this ideal condition is never attained."

As further explained, the relations between the tailwater rating curve and a curve of the depth of water necessary to form a hydraulic jump may be divided into four classes. The case of the Wheeler Dam, where the jump-height curve for an apron at stream bed level (Figure 3) falls considerably above the tailwater curve, is Case I, Appendix I.

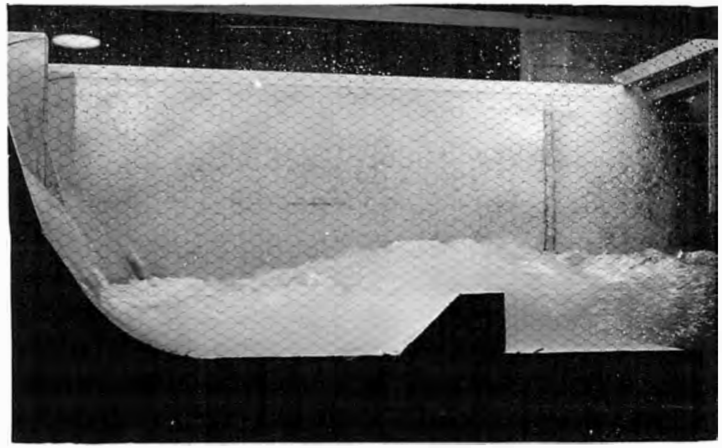
A study of the conditions at the Wheeler Dam site resulted in the initial use of a depressed apron on the model at Elev. 484 with the end of the pool paved on a 1:1 slope upward to the river bed (Figure 6, Test 1). The above elevation was computed to give a satisfactory jump on the apron for the maximum discharge with the first design condition.

Studies with different pool lengths proved that a minimum length, as well as depth, is required for the satisfactory formation of the hydraulic jump (Plate 3). As the length of pool is increased, the effectiveness of the stilling action is improved (Plates 3 and 4). This form of pool would be very satisfactory from the hydraulic standpoint, but would have the disadvantages of a high initial cost and a deep excavation at the toe of the dam,

In an attempt to both shorten the necessary length of apron and decrease the depth of excavation, various types of baffles were investigated on the original depressed apron placed at different elevations. From a study of the action of a perfect hydraulic jump, it was concluded that any form of baffle or sill used on a level apron must be so placed as to decrease the velocity, V_2, rather than V_1, for if any obstruction is placed in the path of the high velocity water it will destroy only a small amount of the energy by impact but at the same time it will deflect the jet from its natural course and break up the perfect formation of the jump. The effect of any sill should be to increase the downstream depth, D_2, hence decreasing V_2 and moving the formation of the jump upstream. Another effect of the sill should be to properly distribute the flow of water beyond the jump and prevent the jets of water, flowing at relatively low velocity, from impinging on the river bed thereby causing erosion.

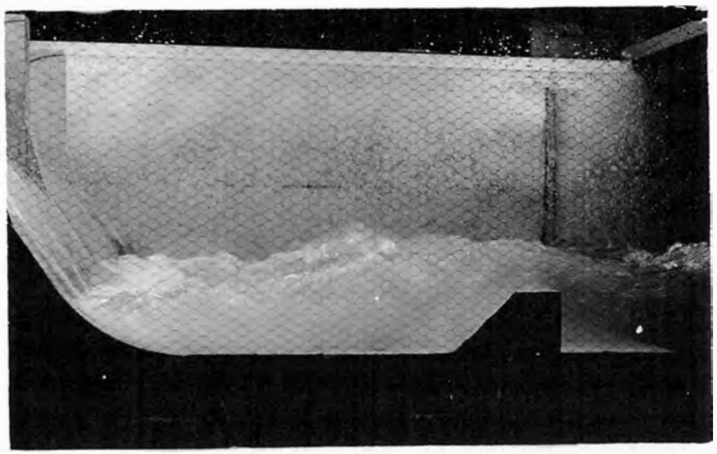


A. POOL LENGTH 24 FEET.

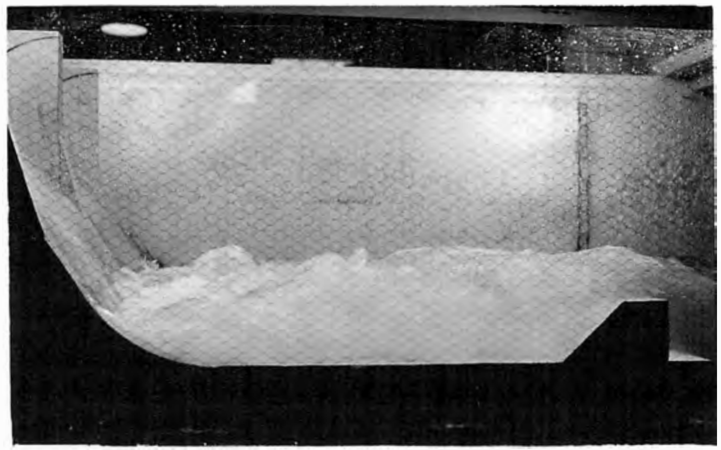


B. POOL LENGTH 42 FEET.

DISCHARGE 300,000 SECOND-FEET.



C. POOL LENGTH 60 FEET.

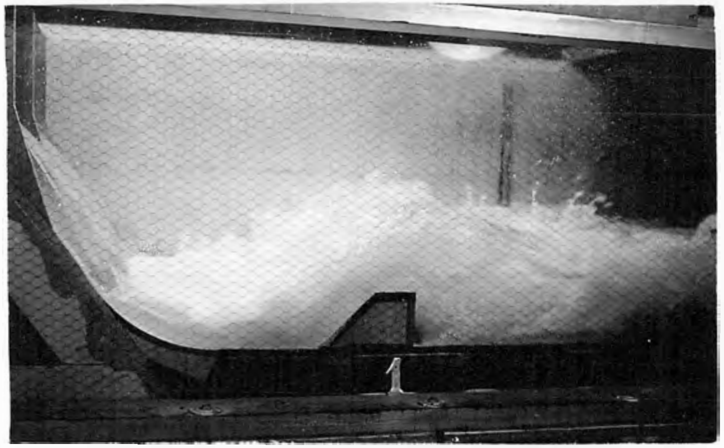


D. POOL LENGTH 78 FEET.

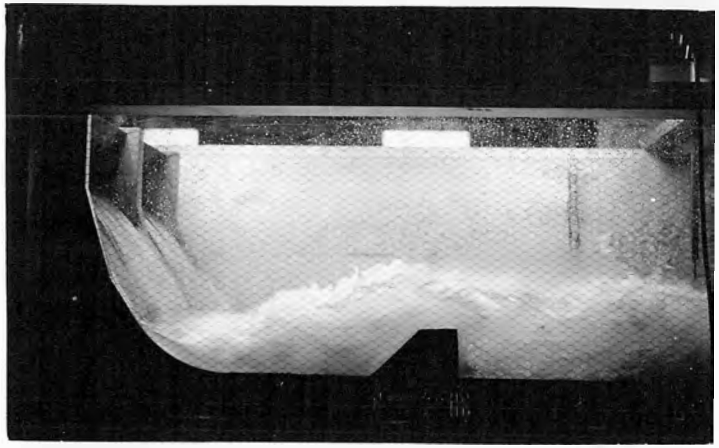
DISCHARGE 300,000 SECOND-FEET.

ACTION OF LEVEL APRON AT ELEVATION 484.

PLATE 3

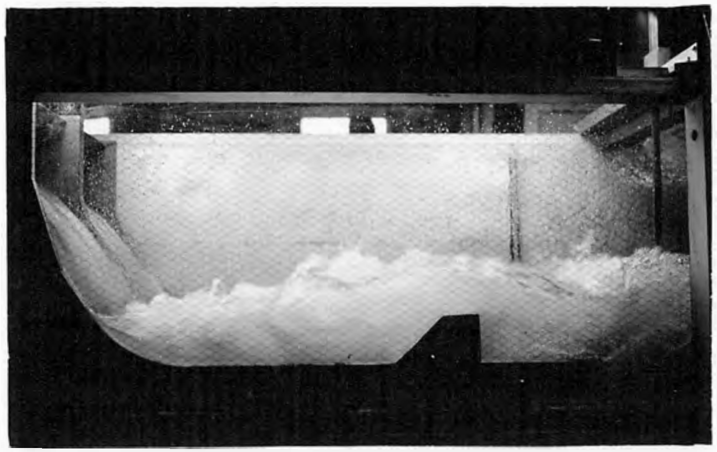


A. POOL LENGTH 24 FEET.

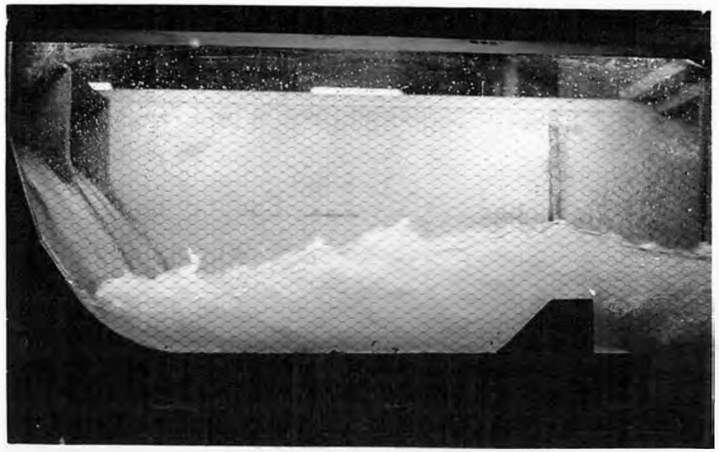


B. POOL LENGTH 42 FEET.

DISCHARGE 700,000 SECOND-FEET.



C. POOL LENGTH 60 FEET.



D. POOL LENGTH 78 FEET.

DISCHARGE 700,000 SECOND-FEET.

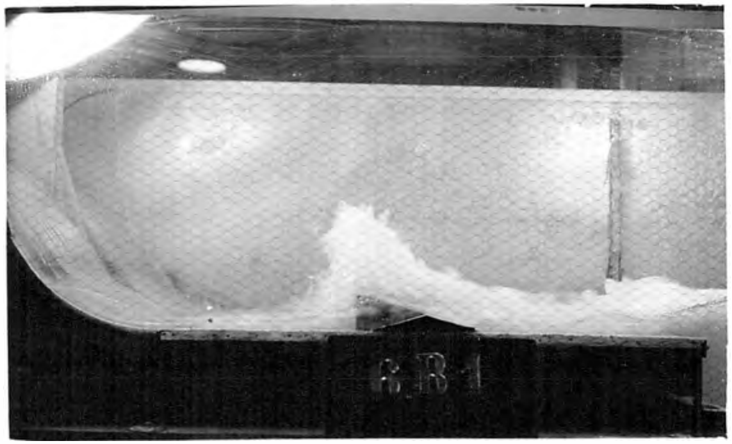
ACTION OF LEVEL APRON AT ELEVATION 484.

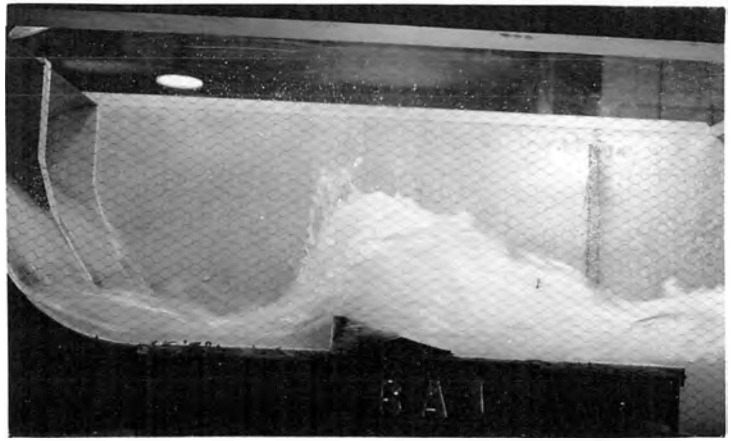
PLATE 4

Since the hydraulic jump must form in more or less perfect condition upstream from a sill to make it satisfactory, with the same length of apron, the sill will make possible the use of a slightly less pool depth and the additional cost of a sill can be balanced against the cost of a deeper pool. Rehbock method

The first of these was the Rehbock dentated sill (Tests 2, 4 and 6, Figure 6), patented by Dr. Theodor Rehbock, of the University of Karlsruhe, Germany. With this sill at various positions on the apron at stream bed level, a jump would not form upstream from the sill and the dentates were subject to excessive impact (Plate 5). The distance of the sill from the dam had little effect on its action. Better results were obtained by placing the sills on a depressed apron at Elevation 490 (Plate 6).

Sets of smaller triangular sills of different sizes and shapes were tried (Tests 3 and 5, Figure 6 and Tests 19 and 25, inclusive, Figure 7) but were not satisfactory, because the depth of pool required was practically the same as for the depressed apron alone. At stream bed level, the water swept across them at a high velocity to the end of the apron (Plate 7 A & B). A somewhat improved action is obtained by lowering the apron (Plate 7 C & D).

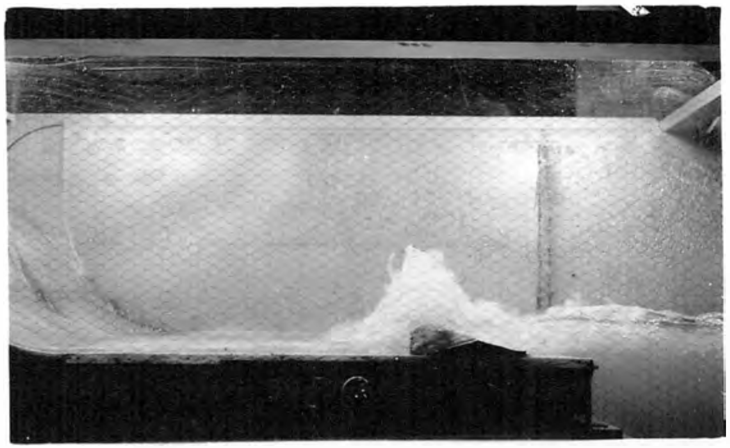


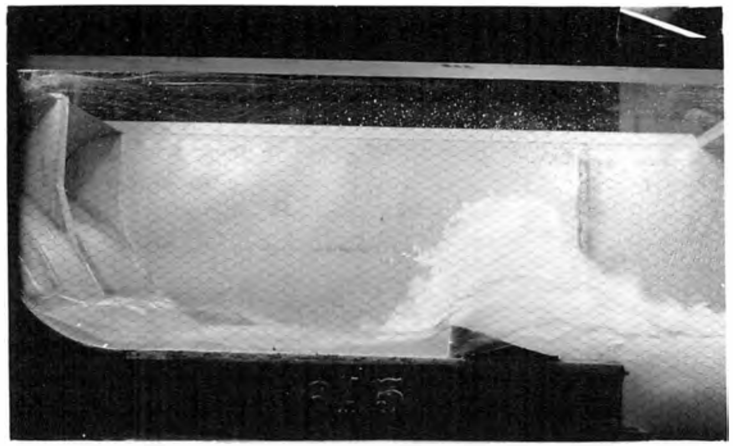


A. DISCHARGE 350,000 SECOND-FEET.

B. DISCHARGE 700,000 SECOND-FEET.

SILL 42 FEET FROM TOE OF DAM.





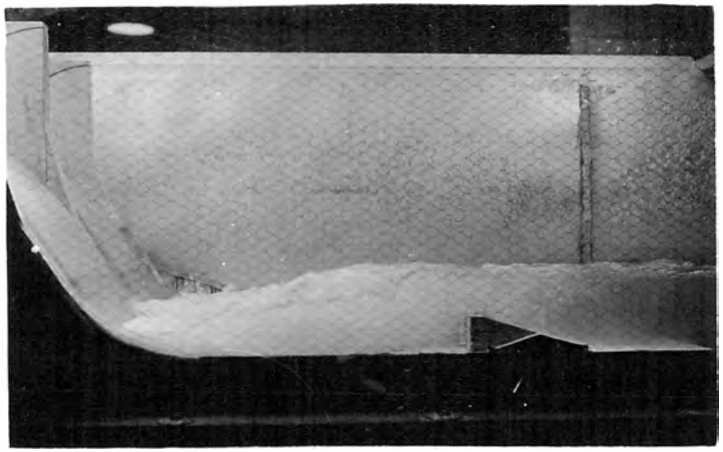
C. DISCHARGE 350,000 SECOND-FEET.

D. DISCHARGE 700,000 SECOND-FEET.

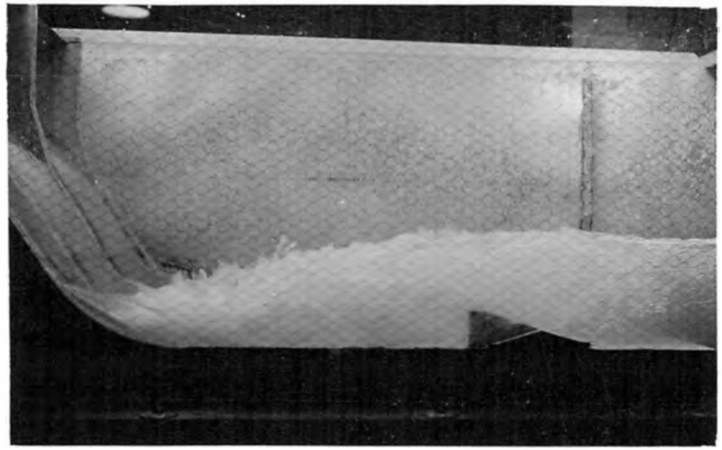
SILL 78 FEET FROM TOE OF DAM.

ACTION OF REHBOCK SILL ON LEVEL APRON AT ELEVATION 498.

PLATE 5

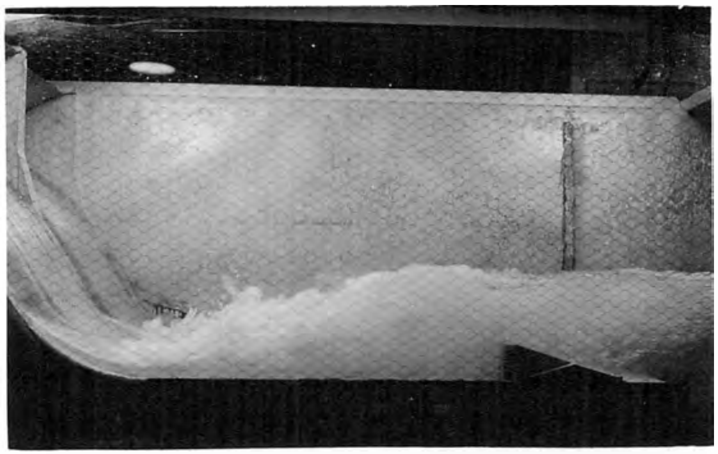


A. DISCHARGE 350,000 SECOND-FEET.

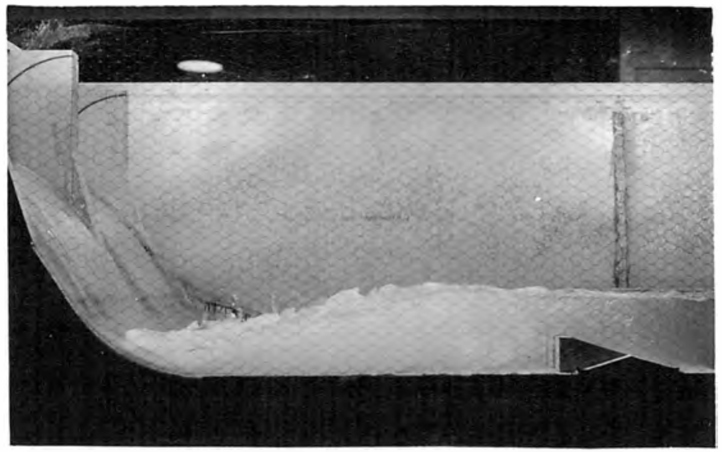


B. DISCHARGE 700,000 SECOND-FEET.

SILL 60 FEET FROM TOE OF DAM.



C. DISCHARGE 350,000 SECOND-FEET.

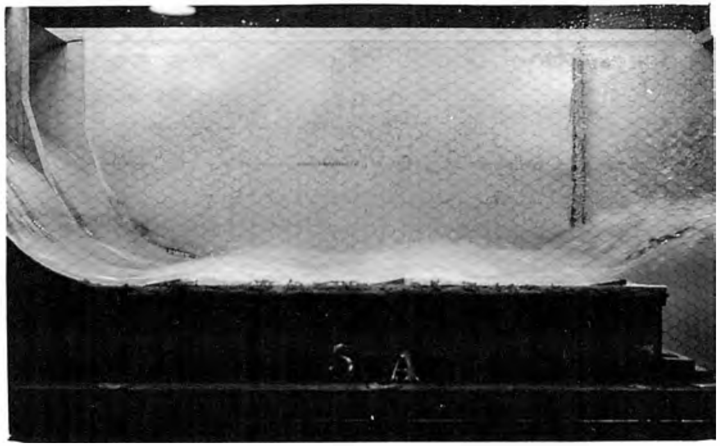


D. DISCHARGE 700,000 SECOND-FEET.

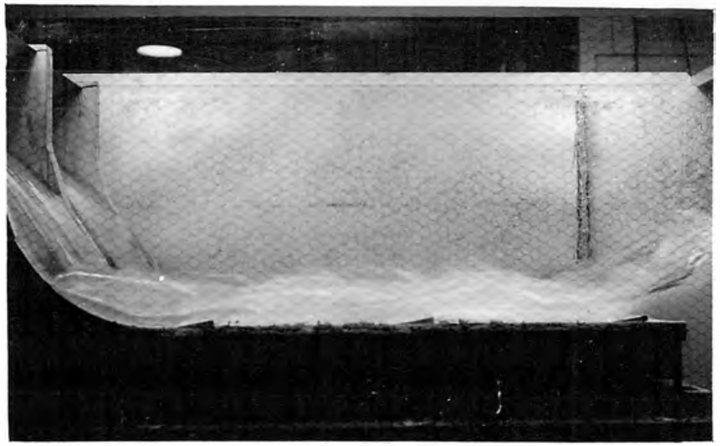
SILL 78 FEET FROM TOE OF DAM.

ACTION OF REHBOCK SILL ON LEVEL APRON AT ELEVATION 490.

PLATE 6

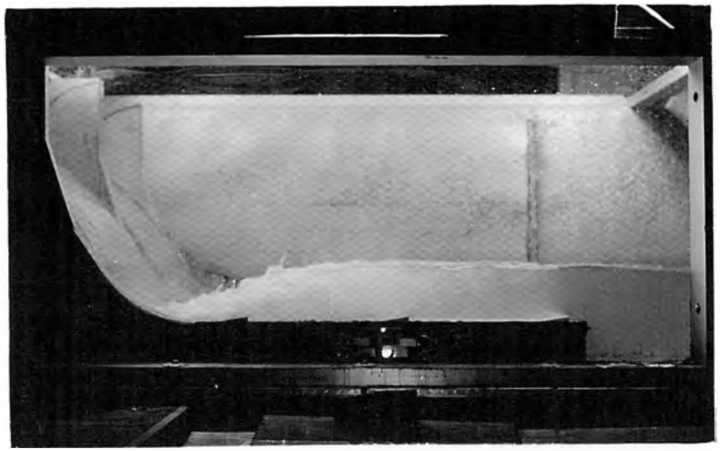


A. DISCHARGE 350,000 SECOND-FEET.

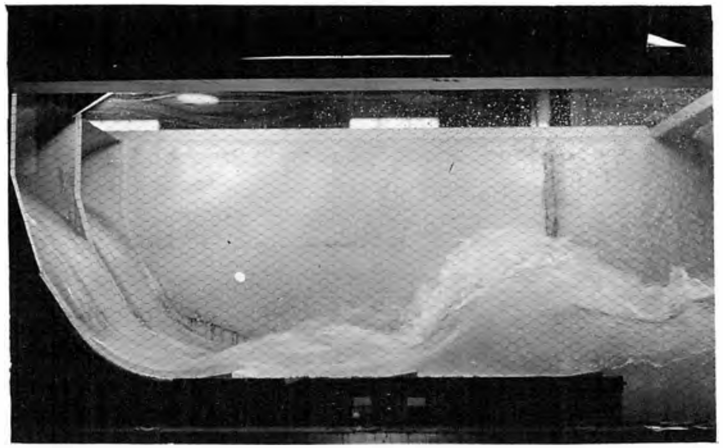


B. DISCHARGE 700,000 SECOND-FEET.

APRON AT ELEVATION 498.



C. DISCHARGE 350,000 SECOND-FEET.



D. DISCHARGE 700,000 SECOND-FEET.

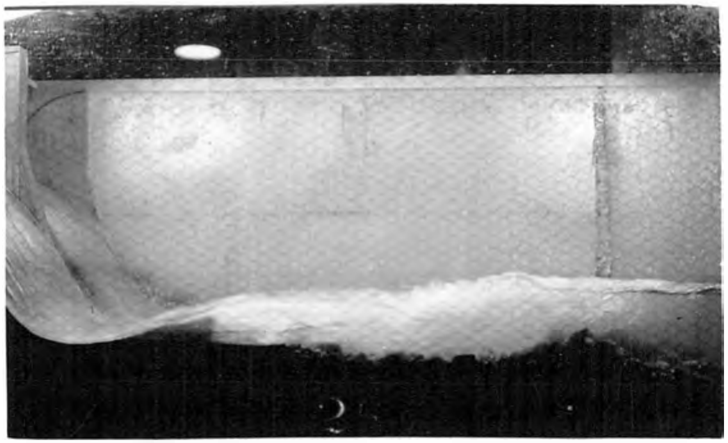
APRON AT ELEVATION 490.

ACTION OF TRIANGULAR SILLS ON LEVEL APRON.

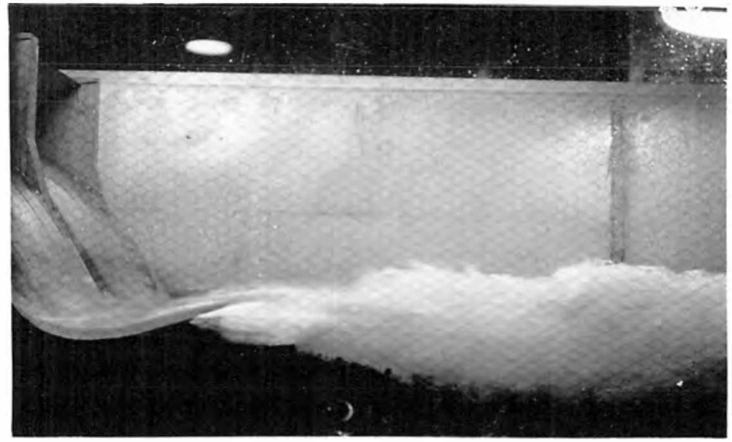
PLATE 7

Tests were also made on a breaker type sill (Forerunner of the diffuser type sill finally adopted) suggested by Mr. G. J. Hornsby (Test 26, Figure 7). This sill had the advantage over previous forms of sills in that there was no direct impact of water on it. The action of this sill was not sufficiently effective to permit the elimination of the depressed apron, but it would make possible some reduction in the length of apron. Plate 8 shows the action at stream bed level and depressed five feet.

The use of a secondary weir or dam (Test 7, Fig. 6) to raise the tailwater to a height sufficient to form a jump on an apron at stream bed level showed a satisfactory action below the dam (Plate 9 A & B), but confirmed the prediction that the depth of tailwater below the secondary dam would be insufficient to form a jump. As the velocities below this weir could reach 35 feet per second, some form of protection would be necessary. A depression of the secondary apron to about three feet below stream bed level, or some form of baffle would probably accomplish this result (Plate 9 C and D). The secondary weir form of protection is undesirable from a construction standpoint as it would necessitate a more extensive cofferdam and prevent the use of a compact construction plant. It also would interfere with the diversion during construction.

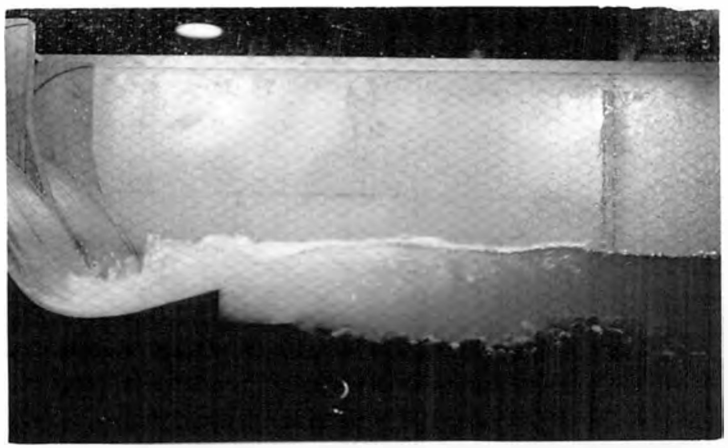


A. DISCHARGE 350,000 SECOND-FEET.

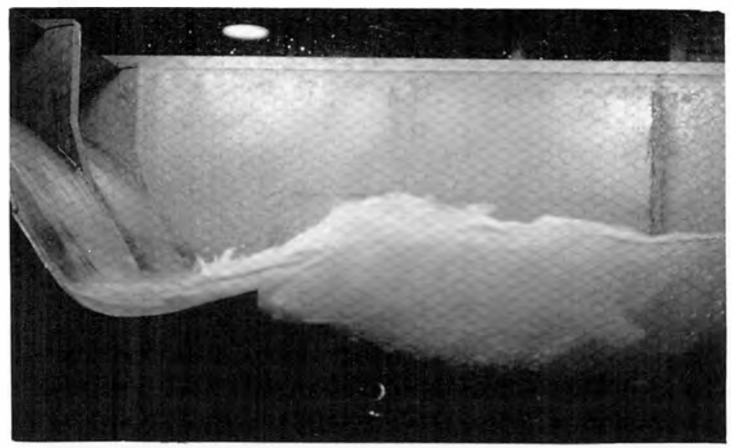


B. DISCHARGE 700,000 SECOND-FEET.

APRON AT ELEVATION 498.



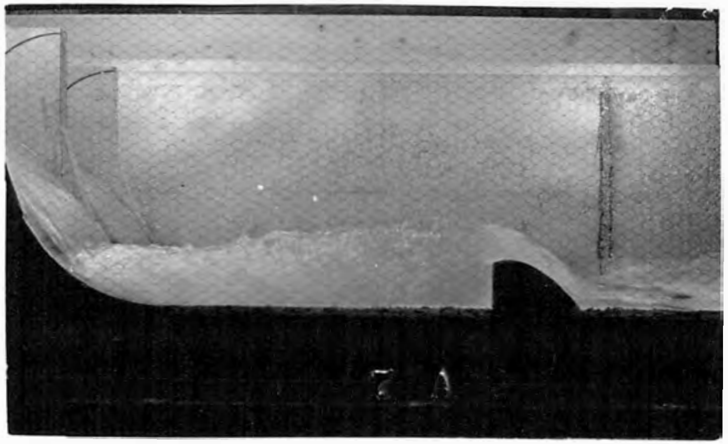
C. DISCHARGE 350,000 SECOND-FEET.



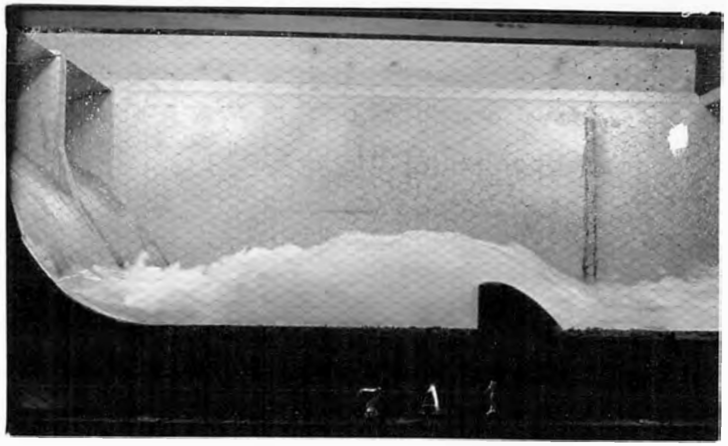
D. DISCHARGE 700,000 SECOND-FEET.

APRON AT ELEVATION 493.

ACTION OF DIFFUSER SILL, INITIAL TYPE, ON LEVEL APRON.

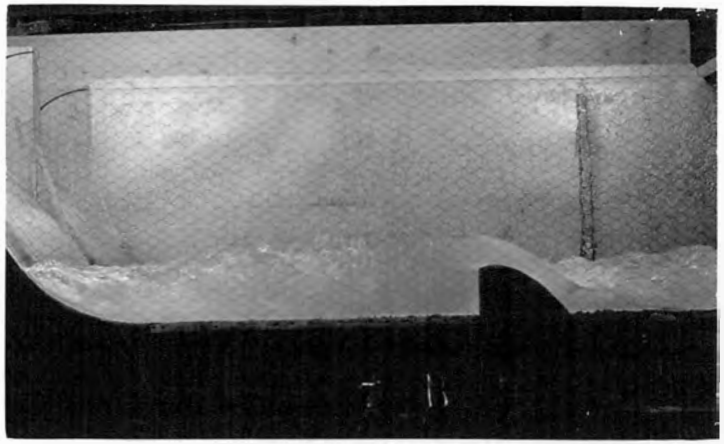


A. DISCHARGE 350,000 SECOND-FEET.

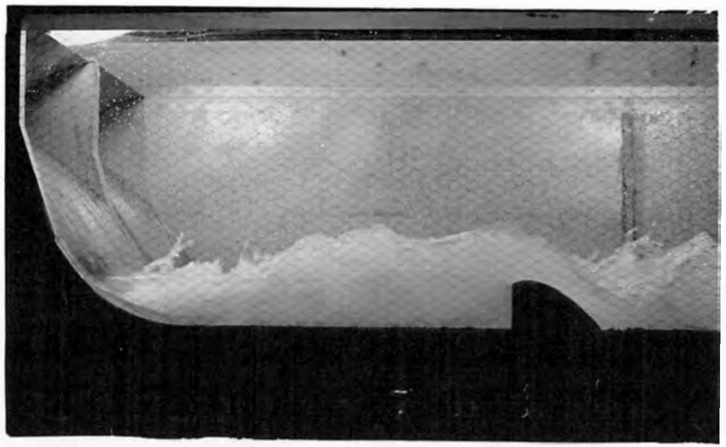


B. DISCHARGE 700,000 SECOND-FEET.

APRON AT ELEVATION 498.



C. DISCHARGE 350,000 SECOND-FEET.



D. DISCHARGE 700,000 SECOND-FEET.

APRON DEPRESSED.

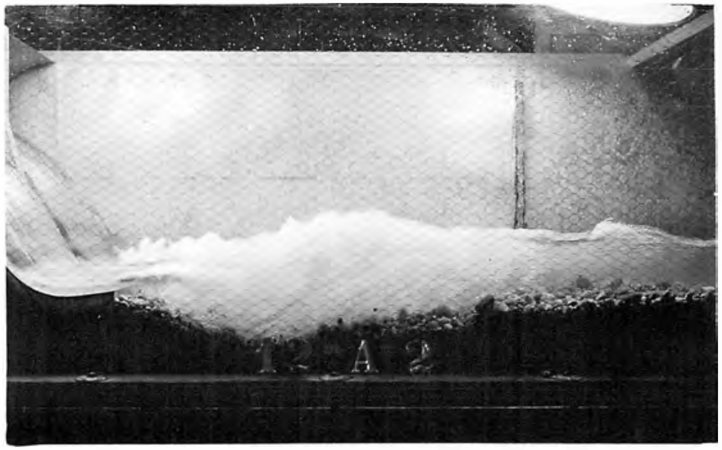
ACTION OF SECONDARY WEIR.

2. Deflecting Bucket.

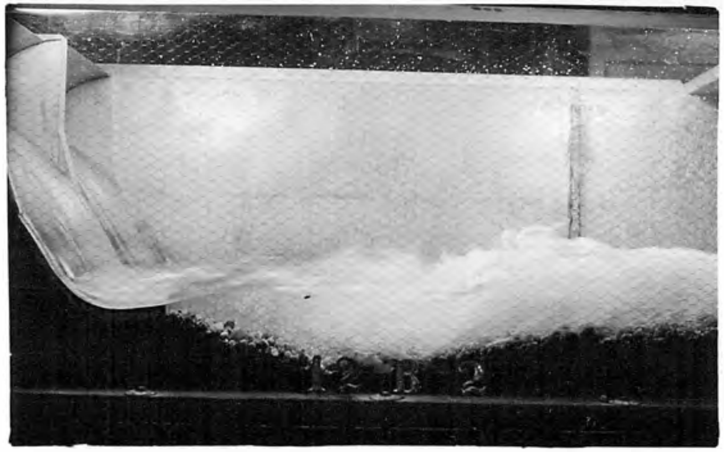
Another possible solution of the dissipation of energy at the toe of the Wheeler Dam would be the use of a short curving apron called a "deflecting bucket" with the low point placed near the minimum tailwater elevation so that the stream of high velocity water at the toe of the dam is thrown upward and outward striking the stream bed some distance away from the structure where the energy is dissipated by impact on the river bottom and adjacent water.

For these tests a flexible adjustable bucket was used on the model (Tests 10 to 14 inclusive, Figure 6 and Tests 15 and 16, Figure 7). Buckets with lips at Elev. 495.5, 497.0, 497.9, 499.0 and 504.0 were tested (Plates 10 and 11). The upward slope of the bucket increased with the lip elevation and moved the point of scour downstream, producing a deeper scour and unstable flow conditions with the stream diving under part of the time (Plate 10 A and B). If any of these types were used, the scour below the dam would probably not endanger it, but a piling up of rock would occur.

To eliminate the unstable condition of the jet, a trajectory shaped apron was extended downstream from the lip of the deflecting bucket (Tests 15 and 16, Figure 7). While the condition was stabilized, the stream clung to the apron

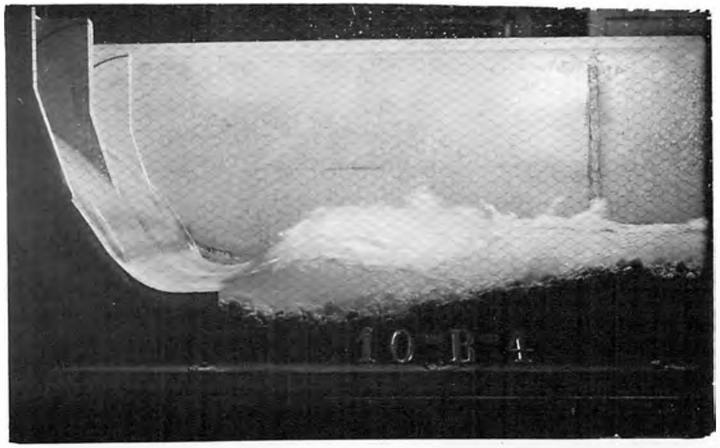


A. JET DIVING UNDER.

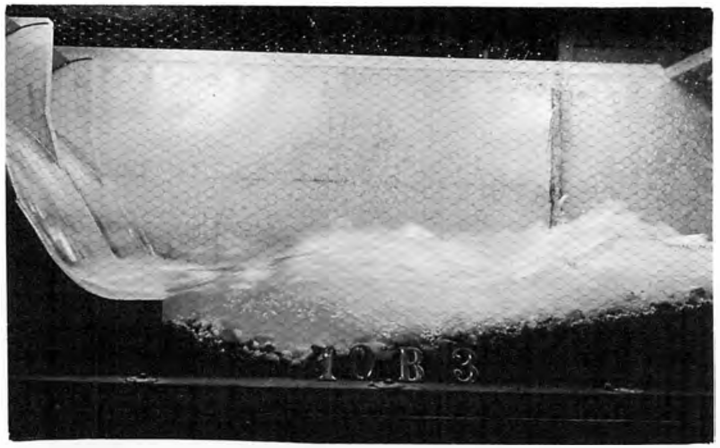


B. JET SHOOTING ALONG TAILWATER SURFACE.

BUCKET LIP AT ELEVATION 497.8 - DISCHARGE 700,000 SECOND-FEET.



C. DISCHARGE 350,000 SECOND-FEET.

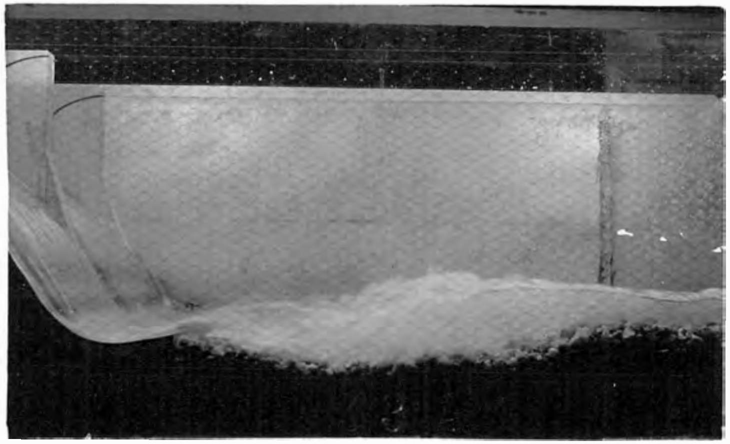


D. DISCHARGE 700,000 SECOND-FEET.

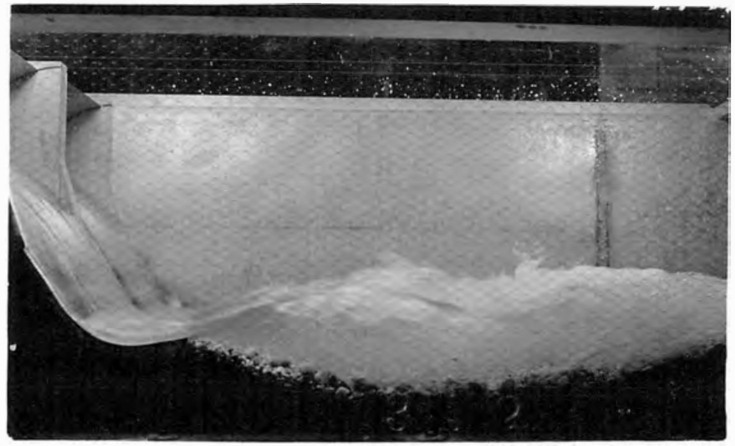
BUCKET LIP AT ELEVATION 495.5.

ACTION OF DEFLECTING TYPE BUCKET.

PLATE 10

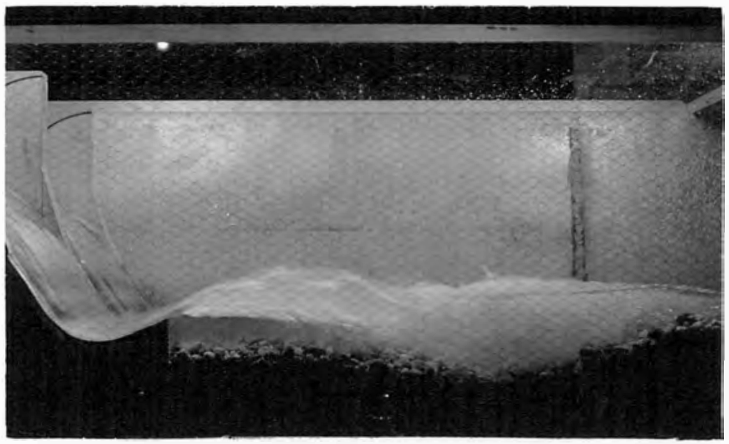


A. DISCHARGE 350,000 SECOND-FEET.

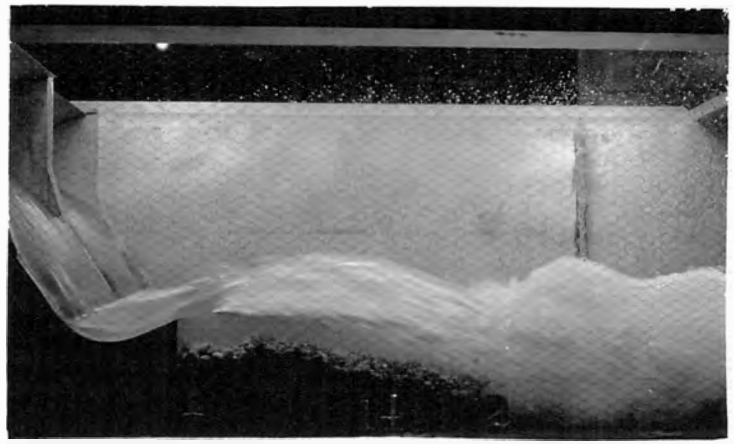


B. DISCHARGE 700,000 SECOND-FEET.

BUCKET LIP AT ELEVATION 499.0.



C. DISCHARGE 350,000 SECOND-FEET.



D. DISCHARGE 700,000 SECOND-FEET.

BUCKET LIP AT ELEVATION 504.0.

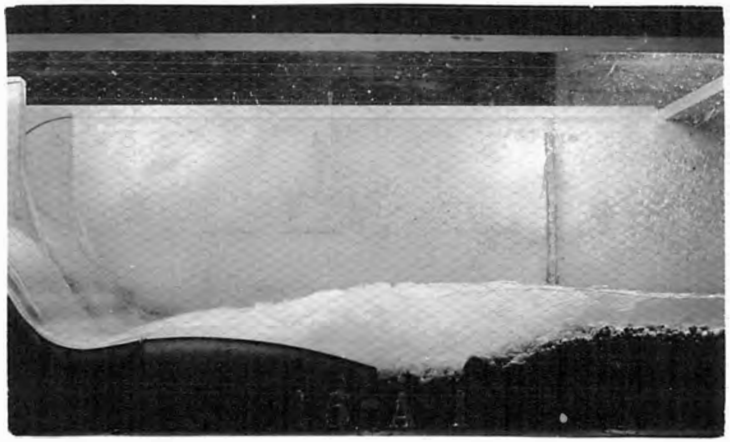
ACTION OF DEFLECTING TYPE BUCKET.

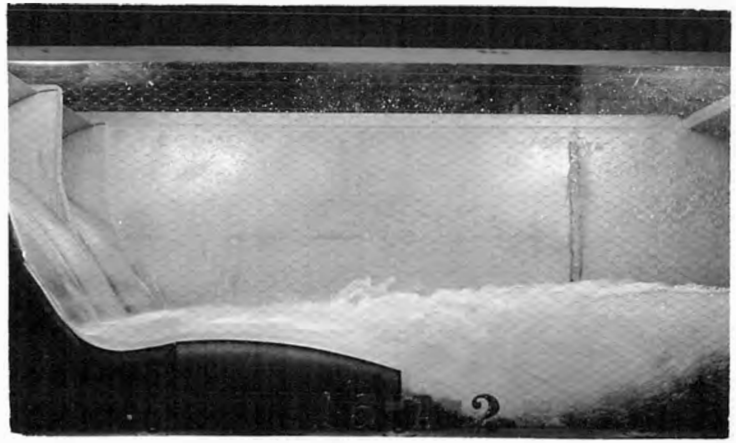
and eroded a deep hole at the downstream end (Plate 12 A and B). It was found that by lowering the apron about eight feet the hydraulic jump had more of a tendency to form and very little erosion resulted (Plete 12). Such a design would require a deep excavation at the toe of the dam which would be undesirable from the standpoint of both cost and design.

This same type of short curved apron, when placed at a lower elevation produced an entirely different action by allowing the higher tailwater to act against the high velocity sheet tending to cause it to become nearly vertical and part of it falling backward into the bucket along the face of the dam. This serves to form a highly turbulent roller which dissipates a large amount of energy in itself and acts as a brake on the approaching high velocity sheet. The energy is thoroughly dissipated without excessive erosion below the apron.

This particular phenomenon was studied by raising the tailwater in Tests 10 to 16, inclusive, to an abnormally high elevation thus simulating the placing of the bucket at a low elevation (Plate 13 A and B).

This action could only be accomplished at the Wheeler Dam by placing the bucket at a sufficiently low elevation, which, again would result in a deep excavation at the toe of the dam with its relatively high cost, both of which would be objectionable.

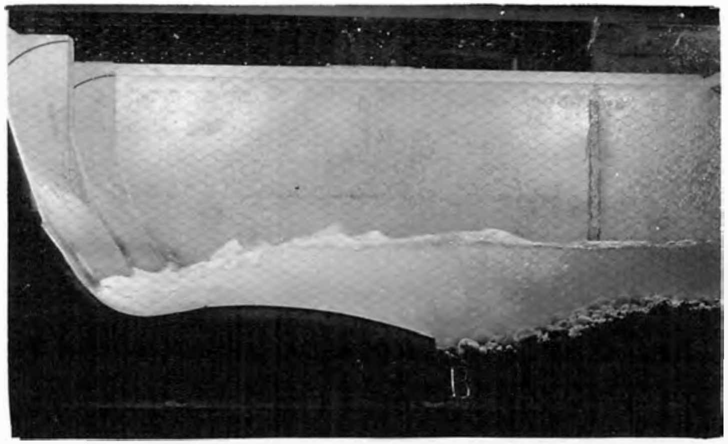


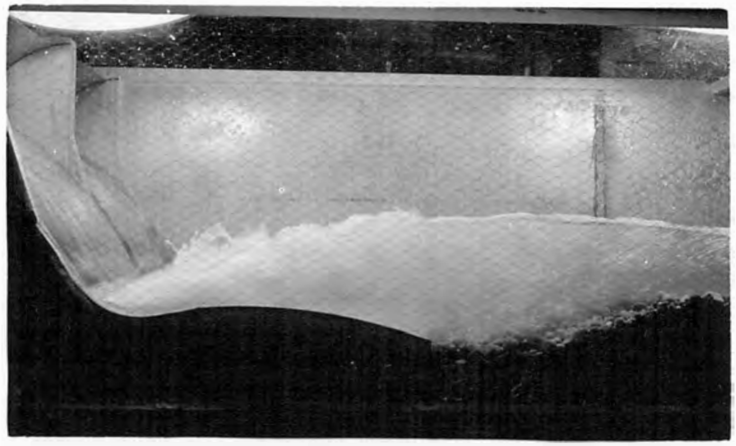


A. DISCHARGE 350,000 SECOND-FEET.

B. DISCHARGE 700,000 SECOND-FEET.

HIGH POINT OF APRON AT ELEVATION 500.



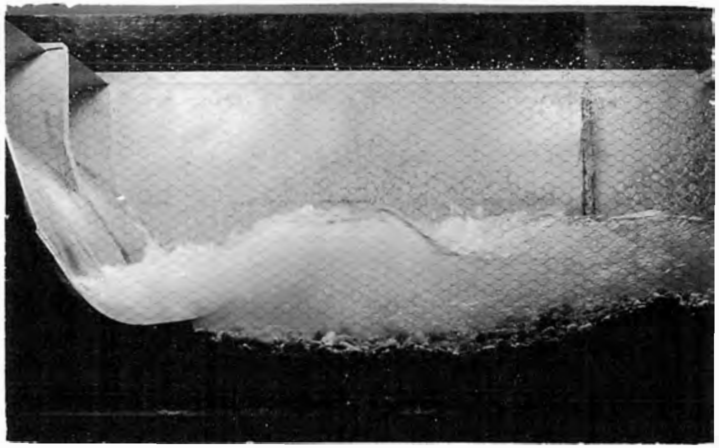


C. DISCHARGE 350,000 SECOND-FEET.

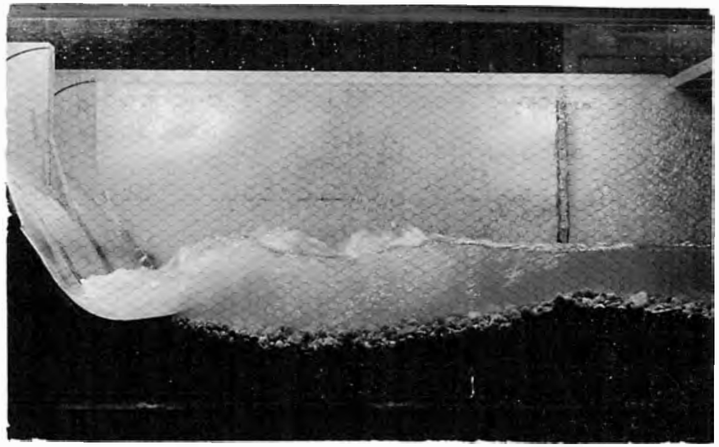
D. DISCHARGE 700,000 SECOND-FEET.

HIGH POINT OF APRON AT ELEVATION 492.

ACTION OF DEFLECTING TYPE BUCKET AND TRAJECTORY SHAPED APRON.

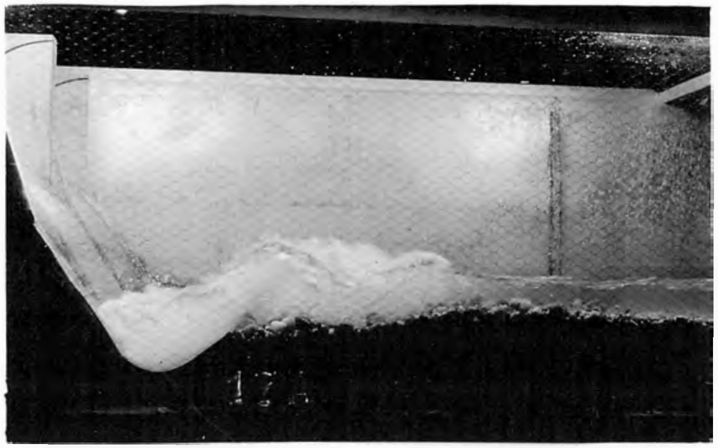


A. DISCHARGE 350,000 SECOND-FEET.

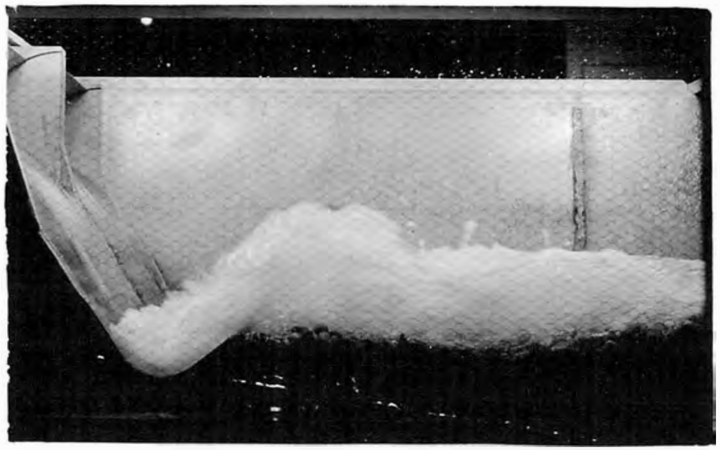


B. DISCHARGE 700,000 SECOND-FEET.

DEFLECTING BUCKET LOWERED.



C. DISCHARGE 350,000 SECOND-FEET.



D. DISCHARGE 700,000 SECOND-FEET.

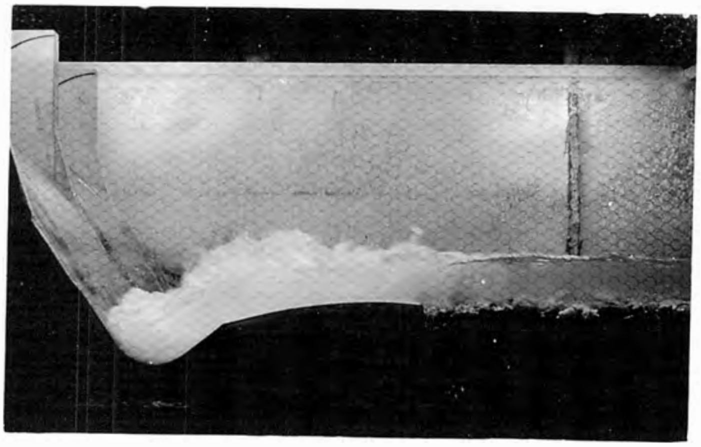
DEFLECTING BUCKET WITH STEEP LIP.

DEFLECTING TYPE BUCKET PLACED AT LOW ELEVATION.

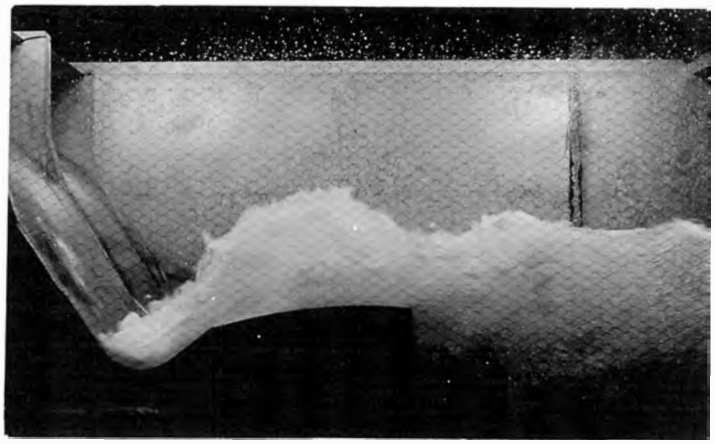
To completely study this action, the bucket on the model was lowered and altered (Test 17, Figure 7) with a steeper slope of the bucket lip which produced a better action and reduced the scour in the river bed (Plate 13 C and D). This form of protection is not expensive where there is a normally deep tailwater sufficient to allow the construction of the bucket without excessive excavation.

While the erosion in Test 17 was very slight, there was considerable roughness of the tailwater surface. It was for the purpose of smoothing the water near the bucket that a curved apron was extended downstream from the bucket lip (Test 18, Figure 7). This gave unstable conditions (Plate 14 B and C) and increased erosion.

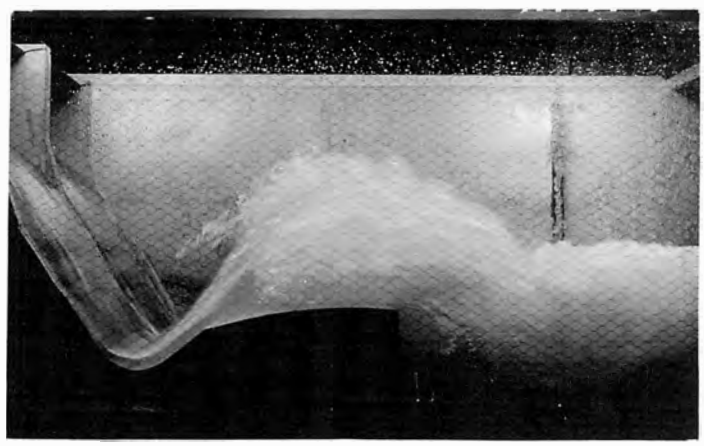
During the lapse of time between Test 26, which was the breaker blocks (forerunner of the diffuser sill) and Test 58, the diffuser sill had been developed to such an extent for the Grand Coulee Dam that a similar solution was sought for the dissipation of energy at the toe of the Wheeler Dam. The apron of the Wheeler model was altered to use a diffuser sill in conjunction with the deflecting bucket placed at a low elevation (Tests 67 to 69, inclusive, Figure 9). The tests on this set-up gave very satisfactory flow conditions with very little erosion (Plate 15). At all discharges a back roll, which was influenced by the curved upstream



A. DISCHARGE 350,000 SECOND-FEET.

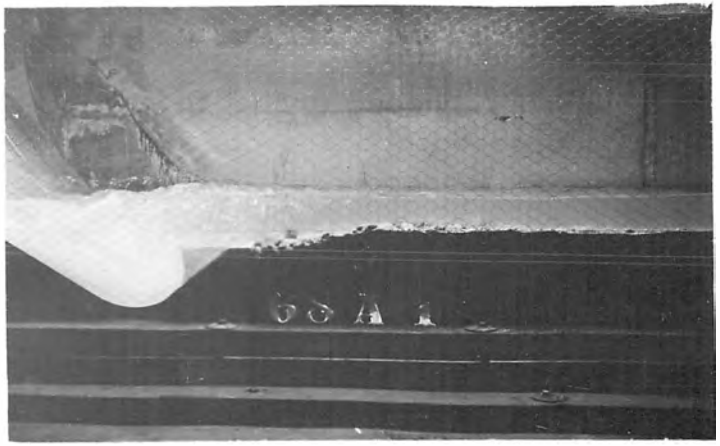


B. DISCHARGE 700,000 SECOND-FEET.
TAILWATER ELEVATION 509.

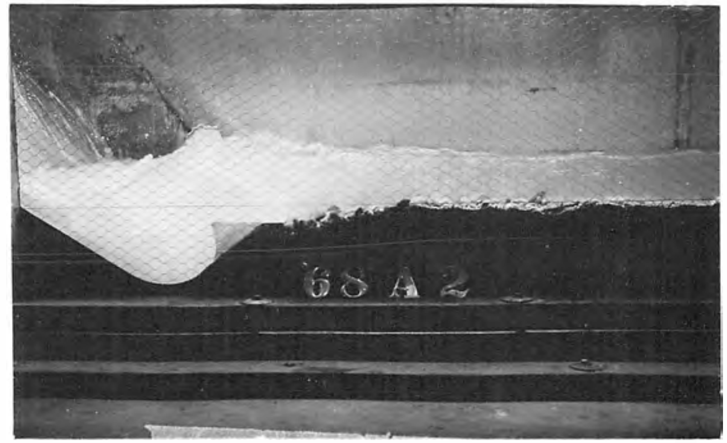


C. DISCHARGE 700,000 SECOND-FEET.
TAILWATER ELEVATION 509.

DEFLECTING TYPE BUCKET PLACED AT LOW ELEVATION.
ACTION WITH CURVED APRON.



A. DISCHARGE 150,000 SECOND-FEET.



B. DISCHARGE 350,000 SECOND-FEET.

BOTTOM OF BUCKET AT ELEV. 480.

C. DISCHARGE 500,000 SECOND-FEET.

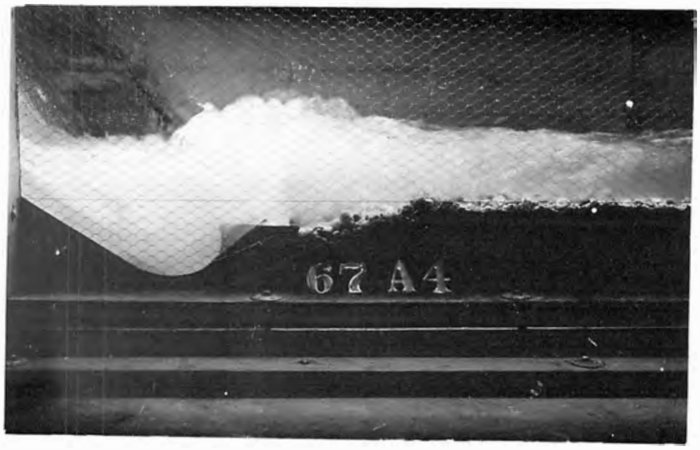
D. DISCHARGE 650,000 SECOND-FEET.

BOTTOM OF BUCKET AT ELEV. 460.

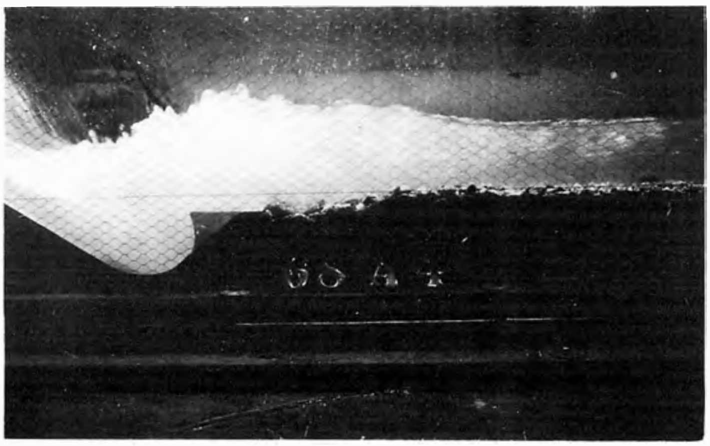
DEFLECTING TYPE BUCKET PLACED AT LOW ELEVATION.
ACTION WITH DIFFUSER SILL.

face of the diffusion chamber walls, was formed in the bucket and impinged on the inflowing stream. This facilitated the dissipation of energy as did the action in the diffusion chamber. The area of exit of the chamber was greater than that of the entrance, thus causing a decrease in the velocity of the water flowing through the chamber. The combined action of roller and the sill distributes the velocities in the tailwater below such that the higher occur along the upper surface and the lower along the river bed.

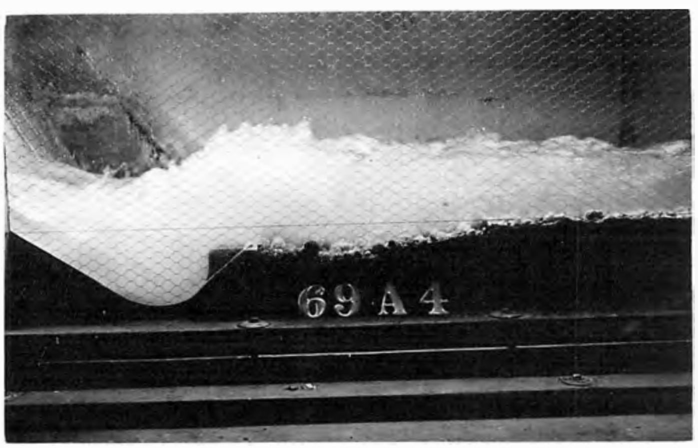
The first sill tested (Test 68, Figure 9), with top at Elev. 494 and large diffusion chambers, was $9\frac{1}{2}$ feet high. A more economical set-up was sought by removing two feet from the top of the first sill (Test 67, Figure 9). There was no apparent difference (Plate 16 A and B) and a sill $7\frac{1}{2}$ feet high, top at Elev. 492, with smaller diffusion chambers (Test 69, Figure 9) was constructed. This also proved sufficient (Plate 16 C) and an attempt was made to decrease the undesirable excavation at the toe of the dam by altering and raising the bucket to Elev. 482 (Test 70, Figure 9). The action proved satisfactory for small quantities over the spillway, but proved undesirable from both the hydraulic and erosion standpoint with the maximum discharge (Plate 17).



A. 7½-FOOT SILL - LARGE CHAMBERS.

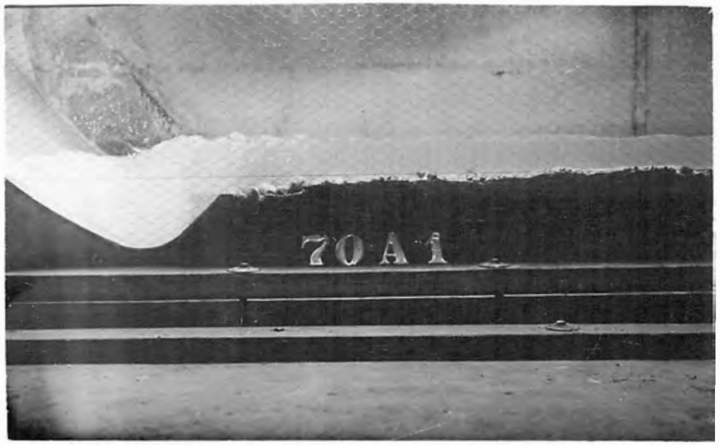


B. 9½-FOOT SILL - LARGE CHAMBERS.

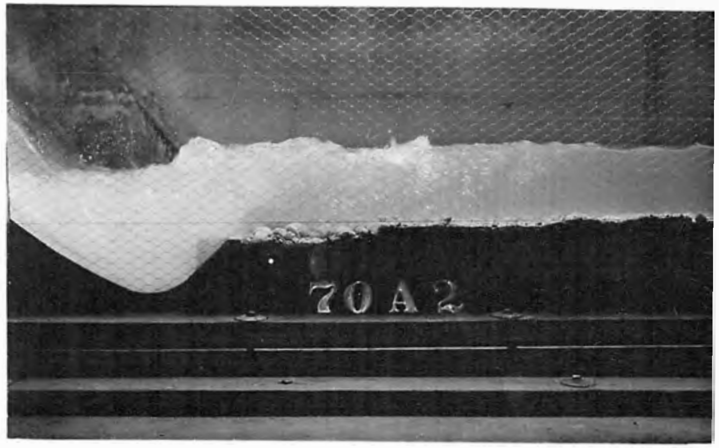


C. 7½-FOOT SILL - SMALLER CHAMBERS.

DEFLECTING TYPE BUCKET PLACED AT ELEVATION 480.
ACTION WITH DIFFUSER SILL - DISCHARGE 650,000 SECOND-FEET.

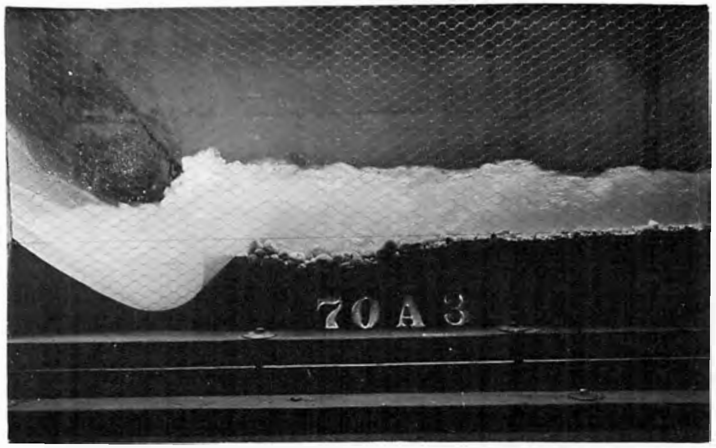


A. DISCHARGE 150,000 SECOND-FEET.

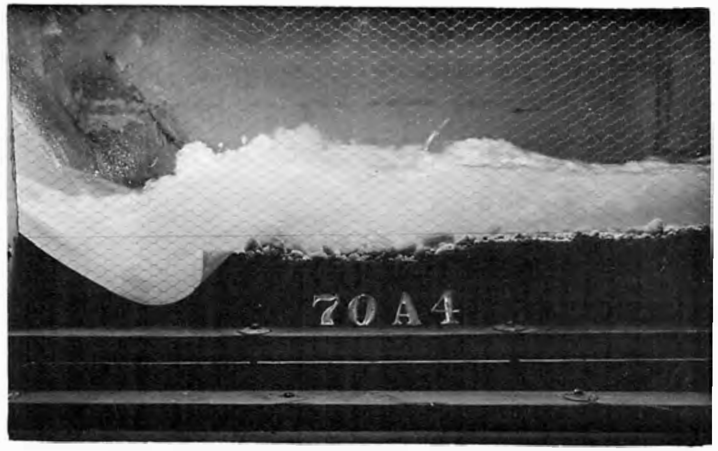


B. DISCHARGE 350,000 SECOND-FEET.

BOTTOM OF BUCKET AT ELEV. 482.



C. DISCHARGE 500,000 SECOND-FEET.



D. DISCHARGE 650,000 SECOND-FEET.

BOTTOM OF BUCKET AT ELEV. 482.

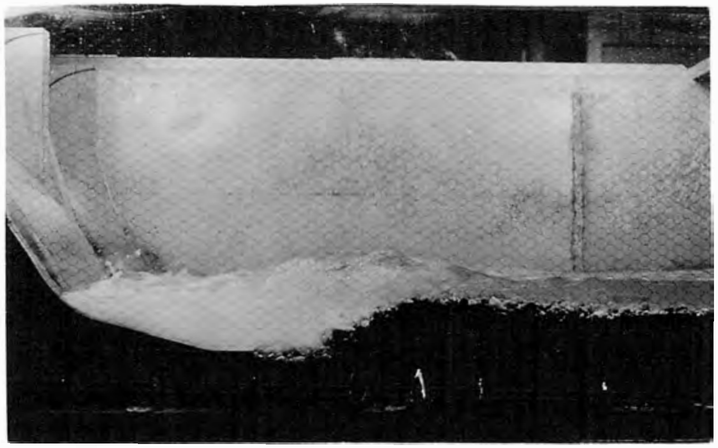
DEFLECTING TYPE BUCKET PLACED AT LOW ELEVATION.
ACTION WITH DIFFUSER SILL.

3. Sloping Apron.

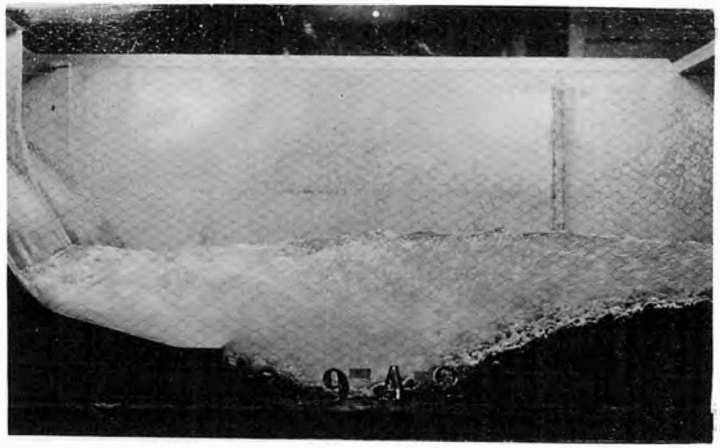
A satisfactory solution would result from the use of either the level apron or the deflecting bucket, but in each case a deep excavation was required near the toe of the dam. As this produced foundation conditions which were not satisfactory to the design office, exhaustive model studies were made to determine the possibilities of the sloping apron, which, if properly constructed would require some excavation downstream from the toe of the dam.

The sloping apron, built above the river bed, would not have been a solution as the jump would not form on it. With the apron depressed to obtain the proper relation to the tailwater level, the action was the same as described in Appendix I. Many tests were made to determine the properties of a sloping apron that would satisfy all conditions.

The first apron investigated (Tests 8 & 9, Figure 6) had a 4:1 slope with the lowest point at Elev. 490, first with a horizontal section at the end and second, with a 6:1 slope upward. Improvement was indicated (Plate 18). The apron was lowered 3 feet with the horizontal section lengthened and a trinagular sill placed at the end (Tests 27 & 30, Figure 7). Conditions were slightly improved, but the

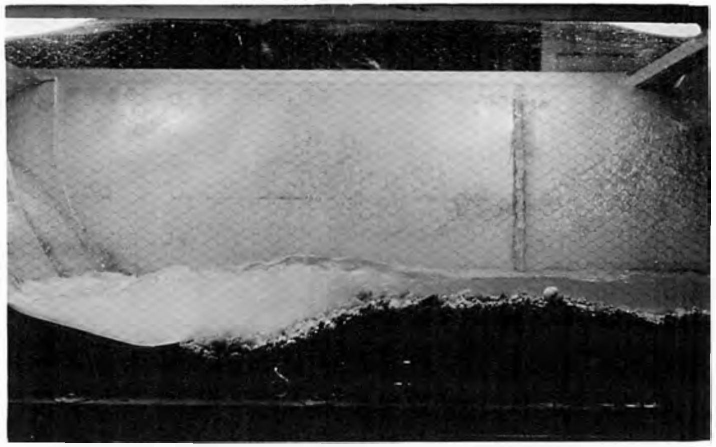


A. DISCHARGE 350,000 SECOND-FEET.

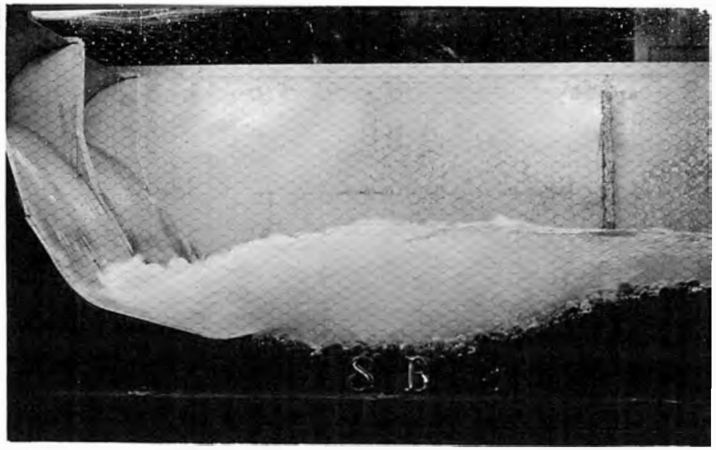


B. DISCHARGE 700,000 SECOND-FEET.

HORIZONTAL SECTION AT END.



C. DISCHARGE 350,000 SECOND-FEET.



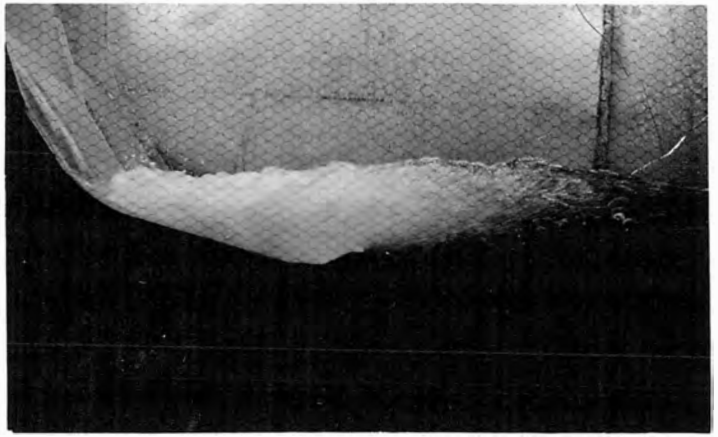
D. DISCHARGE 700,000 SECOND-FEET.

SLOPE OF 6:1 AT END.

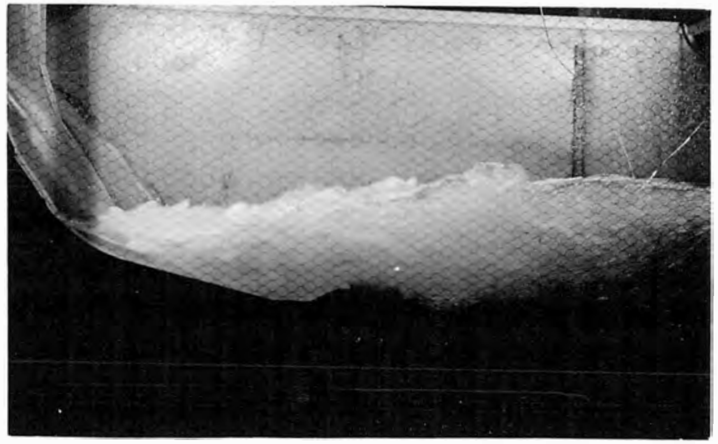
ACTION OF SLOPING APRON - PRELIMINARY DESIGN.

excavation at the toe of the dam was still too deep. As a result, the upstream end of the apron was fixed by the design office such that the elevation of the point of intersection of the 0.7 slope on the downstream face of the dam and the 4:1 slope of the apron should not fall below Elev. 498. In order to obtain satisfactory conditions, therefore, the downstream end of the apron was lowered. This was done by lowering the horizontal section along the 4:1 slope at the same time lengthening the apron. Tests were first made with the horizontal part of the apron at Elev. 487 and a trapezoidal shaped sloping sill placed at the end (Test 31, Figure 7). For a flood of 350,000 second-feet, for which the average frequency is once in ten years, there would be practically no erosion with the river bed excavated on a 3:1 upward slope downstream from the end of the apron (Plate 19). Should a flood occur which would require the full capacity of the spillway with a surcharge of two feet, giving a flow of 733,000* second-feet, some erosion would take place (Plate 19, B). Although the tailwater proved to be materially lower than that computed, the scour would not be serious (Plate 19, C). However, the tailwater is more likely to be higher than computed, especially if portions of the cofferdam and the excavated rock are left in the river bottom, producing an

* Design Condition No. 4, Page 23.

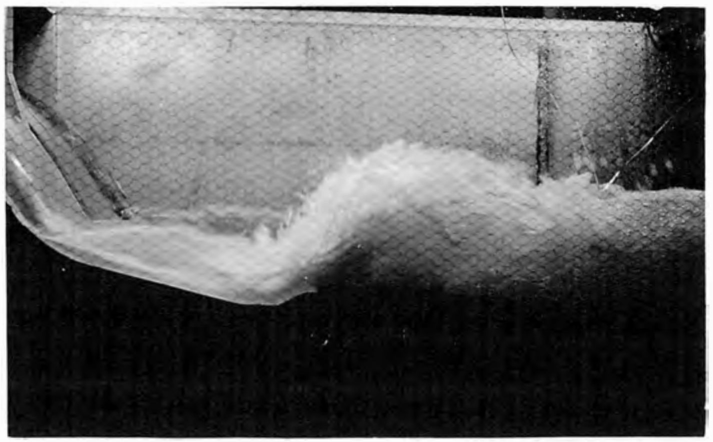


A. DISCHARGE 350,000 SECOND-FEET.

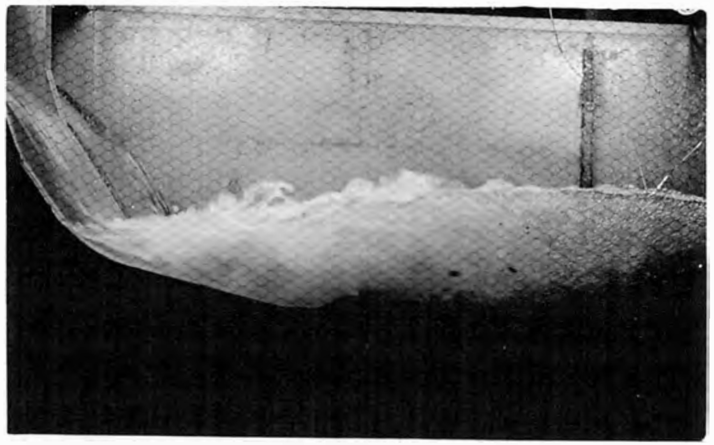


B. DISCHARGE 733,000 SECOND-FEET.

NORMAL TAILWATER.



SHALLOW TAILWATER.



D. DEEP TAILWATER.

DISCHARGE 733,000 SECOND-FEET.

ACTION OF SLOPING APRON AT ELEVATION 487.

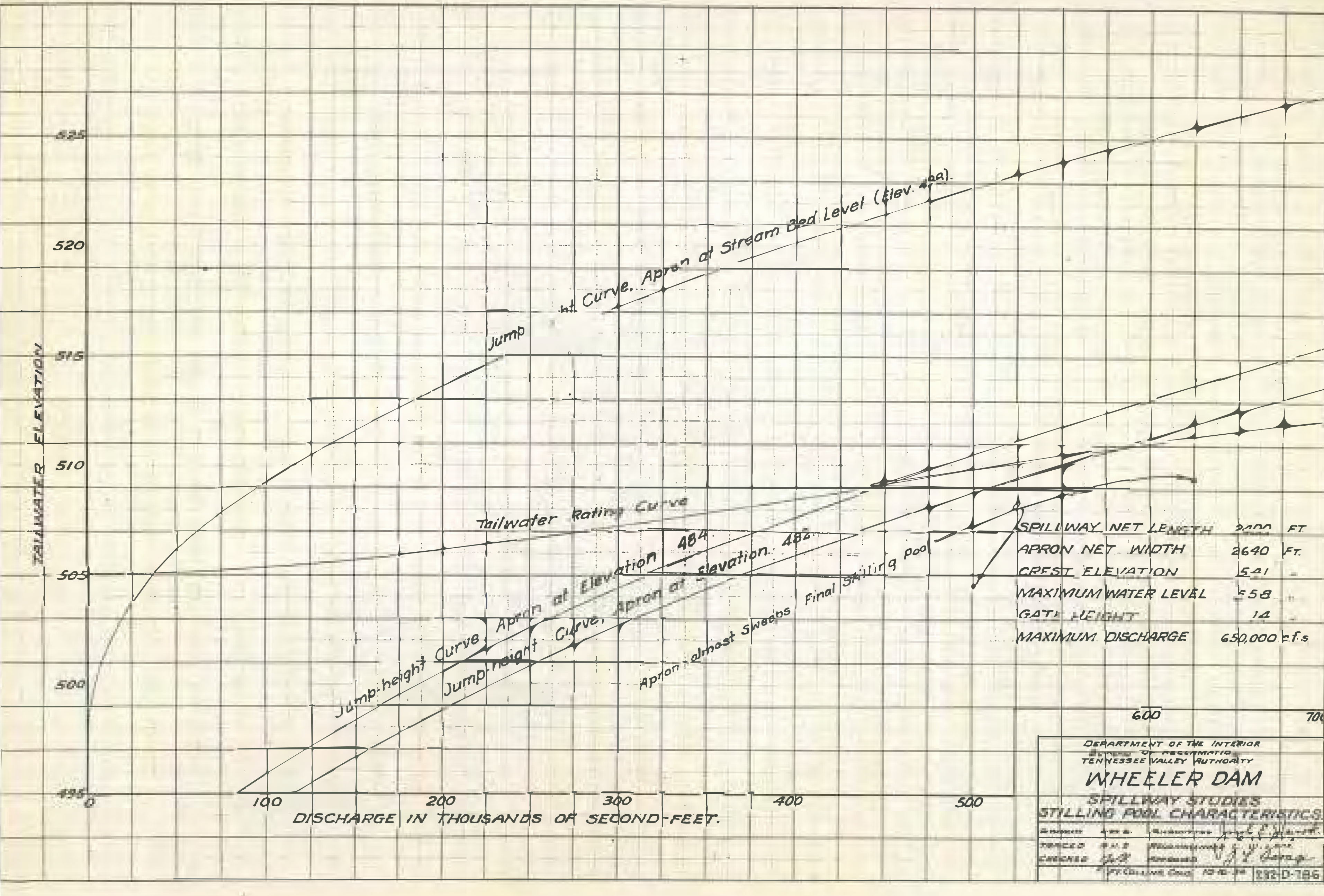
PLATE 19

obstructing effect. Also the spillway occupies only part of the width of the river and some drop is necessary for the flow over it to spread the full width. If the higher tailwater exists, the conditions would be much better than indicated by the model tests (Plate 19, D). The action of this form of protection would, therefore, be satisfactory from the hydraulic standpoint and since its cost would be below any other form giving satisfactory results, this general type was recommended for use on the spillway of the Wheeler Dam.

All the tests leading to the recommended apron were not thoroughly investigated. It was necessary to expedite designs and only enough testing was done on each type to investigate its feasibility and obtain comparable data.

A change in crest design, which provided for 650,000 second-feet through 60 gates, each 40 feet long, made it necessary to further investigate the sloping apron.

The relation between the tailwater rating curve and the jump-height curve (Figure 11) for aprons at various elevations, shows that a level apron at Elev. 484 would not be adequate for flows greater than 450,000 second-feet and that one at Elev. 482 would be much better. The apron would be below the stream bed and the end of the pool would have the same effect as a secondary dam, 14 feet high with the crest

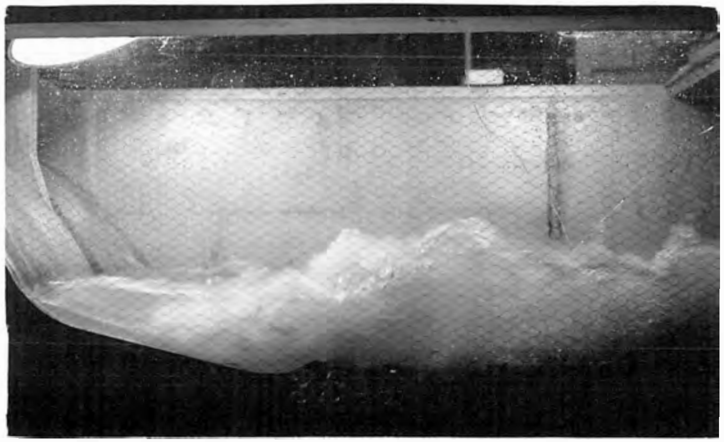


TAILWATER ELEVATION
525
520
515
510
505
500
495
0
100
200
300
400
500
600
700
DISCHARGE IN THOUSANDS OF SECOND-FEET.
Jump-height Curve, Apron at Stream Bed Level (Elev. 488).
Tailwater Rating Curve
Jump-height Curve, Apron at Elevation 484
Jump-height Curve, Apron at Elevation 482
Apron almost Sweeps Final Stilling Pool
SPILLWAY NET LENGTH 2400 FT.
APRON NET WIDTH 2640 FT.
CREST ELEVATION 541
MAXIMUM WATER LEVEL 558
GATE HEIGHT 14
MAXIMUM DISCHARGE 650,000 c.f.s.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
STILLING POOL CHARACTERISTICS
TRACED
CHECKED
FT. COLLINS, COLO.
292-D-786

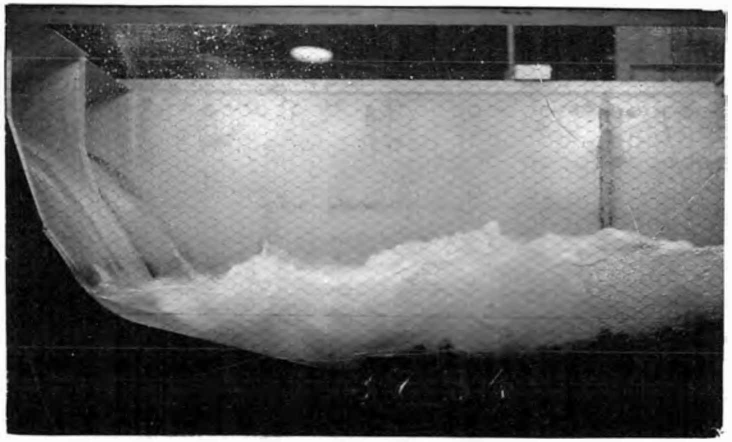
FIG. 11

at Elev. 498, thus causing the jump to form more easily for higher flows. A study of Figure 11 and a consideration of the depressed apron resulted in the horizontal part of the apron being lowered to Elev. 485 (Test 35, Figure 7). This did not give satisfactory results (Plate 20, A) and different sills were tried (Tests 36 & 37, Figure 7) to improve conditions. Very little difference was noted (Plate 20). The flat part of the apron was lowered to Elev. 484 (Test 38, Figure 7) and a 5-foot trapezoidal shaped sill with a $1\frac{1}{2}$:1 slope on the upstream face was placed at the end. Considerable improvement was noted (Plate 20, D) but no factor of safety was provided for the maximum discharge (Plate 21, A) and it was deemed necessary to provide more pool length. One method would have been to lower and lengthen the downstream end of the apron; another, to lengthen the horizontal portion of the apron by moving the sill farther downstream. Either would have accomplished the desired results, but would have naturally increased the cost of protection and a more economical method was sought.

The pool was lengthened by flattening the sloping part of the apron and using a larger radius from the face of the dam to the slope. Two set-ups were made on an 8:1 sloping apron and the results compared with those obtained

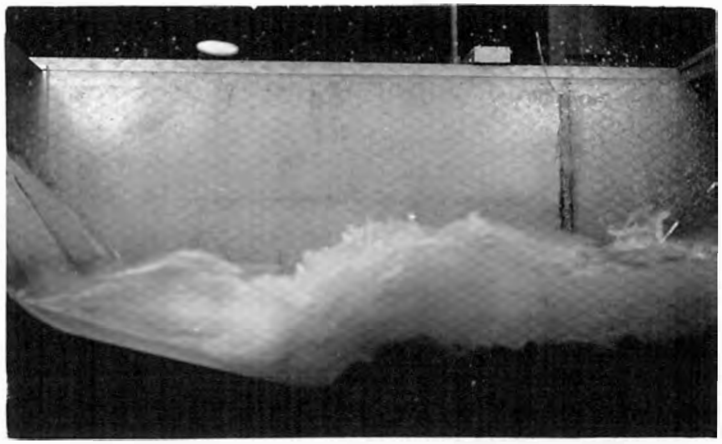


A. 3-FOOT SILL WITH 2:1 SLOPE.

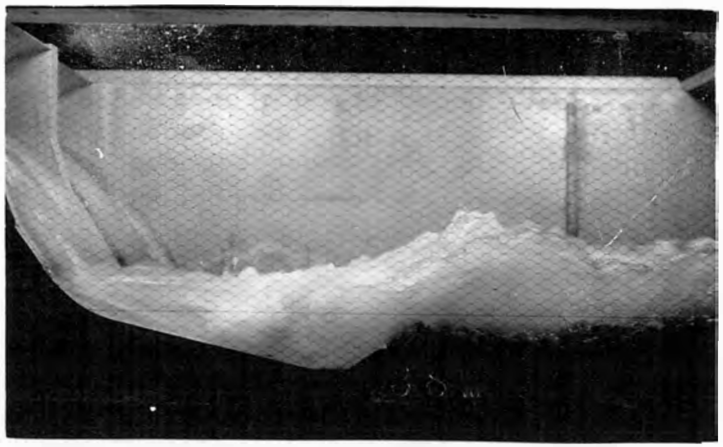


B. 3-FOOT SILL WITH 1:1 SLOPE.

DISCHARGE 650,000 SECOND-FEET.



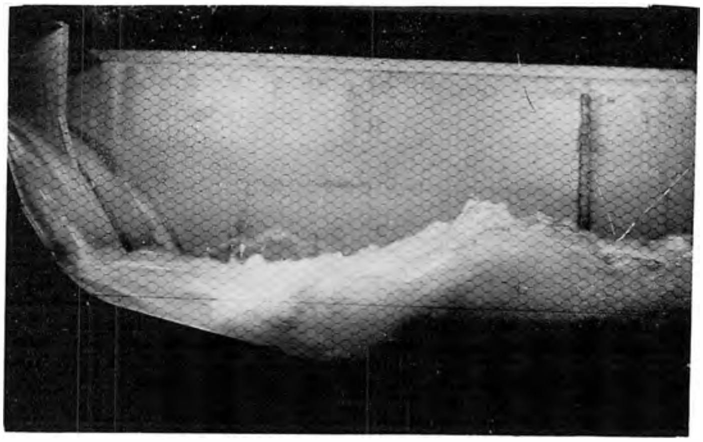
C. 5-FOOT SILL WITH 1:1 SLOPE.



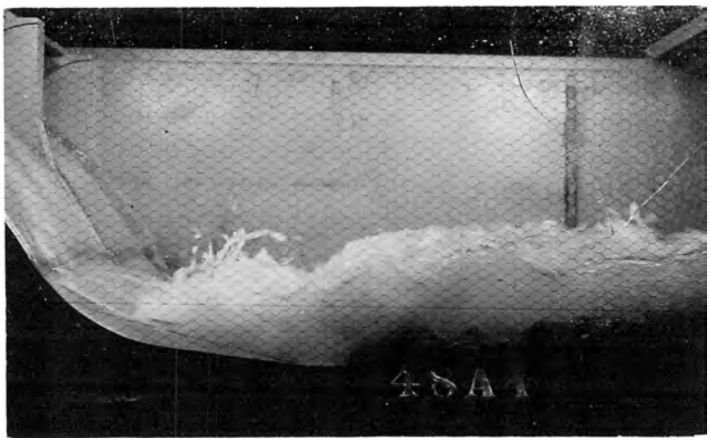
D. 5-FOOT SILL WITH 1½:1 SLOPE.

DISCHARGE 650,000 SECOND-FEET.

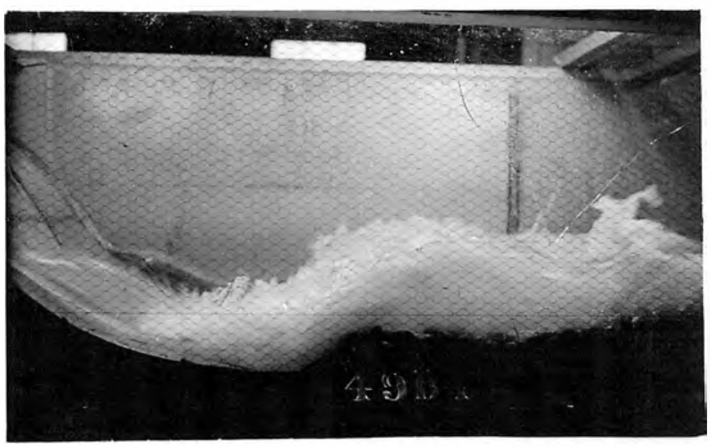
ACTION OF TRAPEZOIDAL SILLS AT END OF APRON.



A. 27-FOOT RADIUS BUCKET - 4:1 SLOPING APRON.



B. 43½-FOOT RADIUS BUCKET - 8:1 SLOPING APRON.



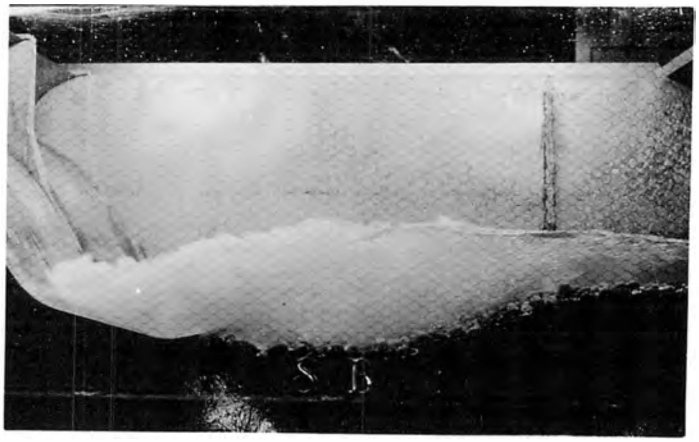
C. 77.6-FOOT RADIUS BUCKET - 8:1 SLOPING APRON.

ACTION OF SLOPING APRONS - DISCHARGE 650,000 SECOND-FEET.

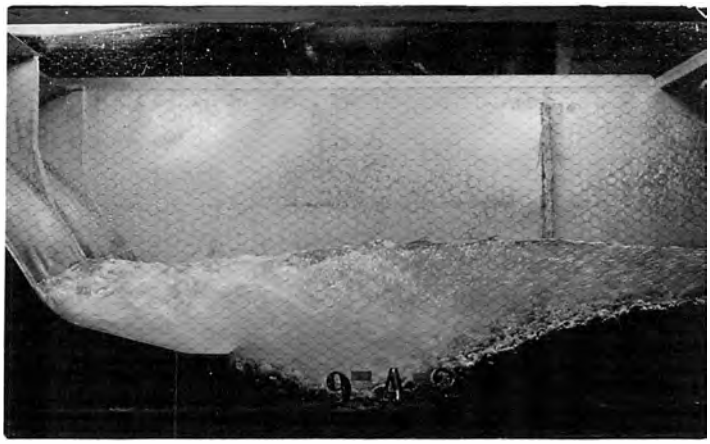
on the apron with the 27-foot radius bucket and 4:1 slope. The first 8:1 sloping apron had a 43.5-foot radius bucket (Test 48, Figure 8) and the second had a 77.6-foot radius (Test 49, Figure 8). A comparison of the results (Figure 12 and Plate 21) shows that more water was retained on the apron with the 43.5-foot radius which resulted in slightly less erosion. However, this particular set-up would not satisfy the foundation conditions stipulated by the design office and a compromise resulted between minimum excavation and best hydraulic conditions. The 8:1 sloping apron with a 77.6-foot radius bucket with the horizontal part at Elev. 484 was adopted with the type of sill at the end of the apron subject to change.

The necessity of a sill on an apron, for discharges other than those for which the hydraulic jump would form within the length of the apron, was revealed by the initial tests on the sloping apron (Plate 22). A sill allows a considerable shortening of an apron which reduces the cost of excavation in any case. The cost of material in the sill is balanced by the saving in the length of apron.

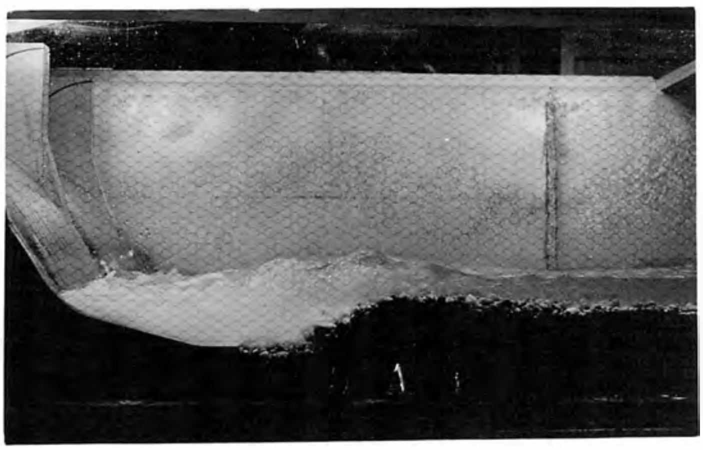
Numerous sills, some of which were used in the preliminary tests, were investigated to improve the hydraulic conditions in the stilling pool and give the necessary factor of safety for the maximum flood. The first, a triangular



A. DISCHARGE 700,000 SECOND-FEET.

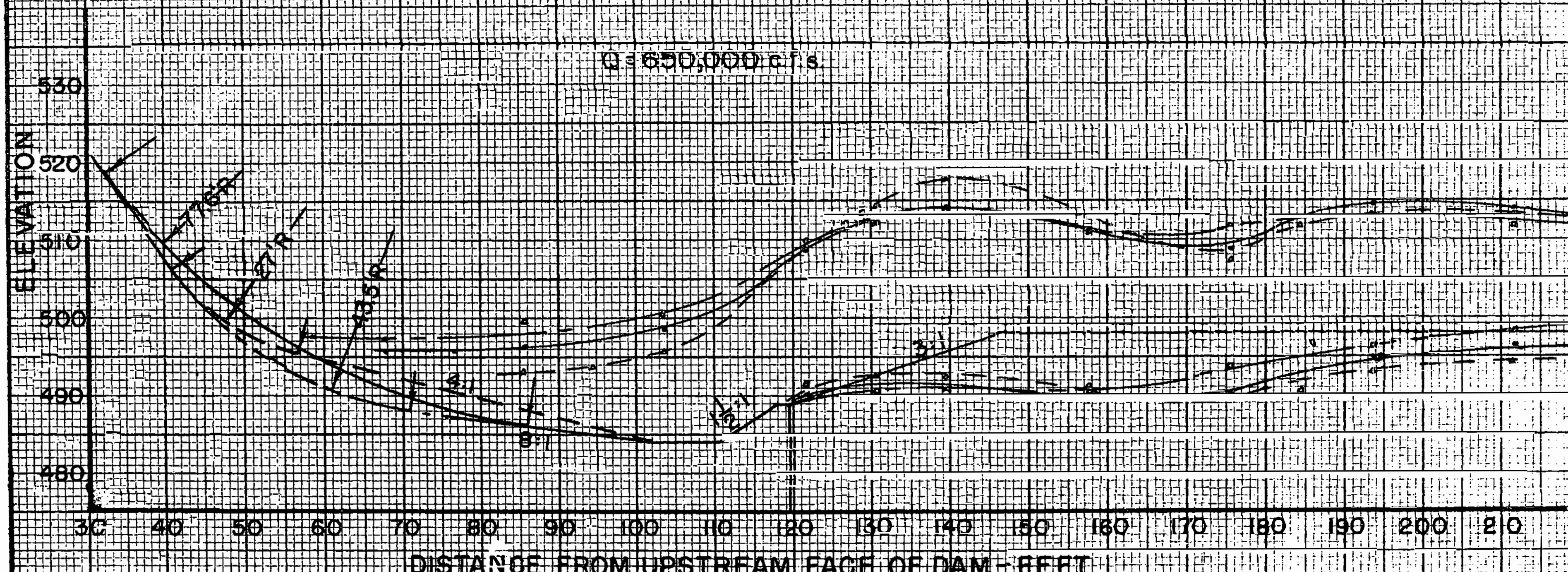


B. DISCHARGE 700,000 SECOND-FEET.



C. DISCHARGE 350,000 SECOND-FEET.

ACTION SHOWING NECESSITY OF SILL AT END OF APRON.



EXPLANATION

Profile of water surface and river bed, normal tailwater

Test No. 38, Run 4C. A 4:1 sloping apron with a 27 ft. radius bucket

Test No. 48, Run 4C. An 8:1 sloping apron with a 43.5 ft. radius bucket

Test No. 49, Run 4C. An 8:1 sloping apron with a 77.6 ft. radius bucket

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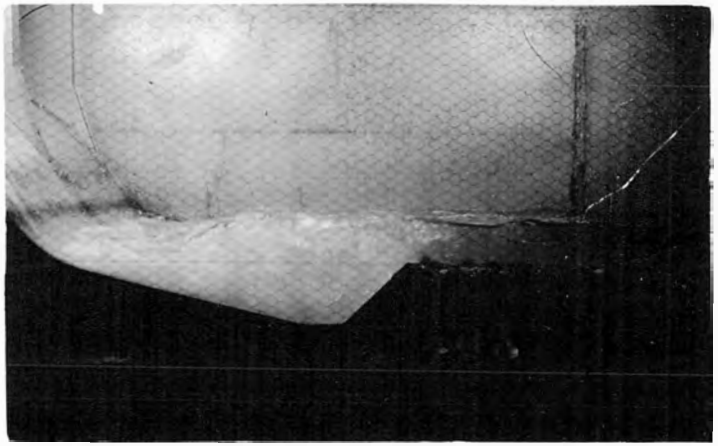
FIG. 12

sloping sill (Tests 27 & 30, Figure 7), was unsatisfactory because of the sharp corner on the downstream top edge. This condition was much improved by extending the sill a short distance horizontally downstream thus forming a flat top (Test 31, Figure 7).

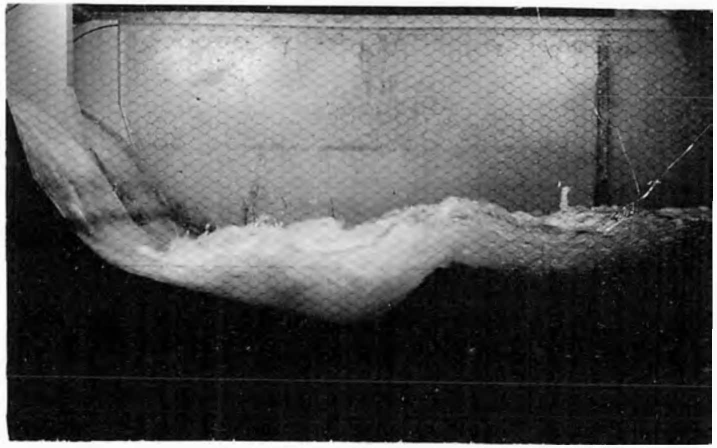
Sills 3 feet high (Tests 35 & 37, Figure 7) with different slopes on the upstream face were tested (Plate 20, A and B). These were found inadequate from the standpoint of erosion and the height of the sill was increased to five feet (Tests 36 & 38, Figure 7). The 5-foot sills reduced erosion but apparently produced less desirable hydraulic conditions (Plate 20, C and D). A close study of Plate 20 shows that, in the case of the smaller sill, the river bed was eroded to such an extent that the effective length of the pool was increased and thus allowed a better formation of the jump. As in the case of the smaller sill, tests were made on the 5-foot sill with different slopes on the upstream face. Visual tests proved that the $1\frac{1}{2}$:1 sloping sill gave better surface conditions than the 1:1 slope (Plate 20, C and D) and reduced the erosion from that with the 2:1 slope. It can be noted also (Plate 20) that in all cases the jump is very imperfect at the maximum discharge. This would indicate the necessity of lengthening the apron. Because of the remote possibility of the occurrence of the maximum flood and the slight erosion caused by it, the cost of lengthening the apron was not justified. It was felt that any improvement must be made by the use of an auxiliary structure at the end of the apron, such as a sill or baffle.

Mr. Charles H. Paul suggested excavating on a slope at the end of the apron and paving to the river bed at Elev. 498. Several slopes were tried with the 4:1 sloping apron (Tests 50 to 54 inclusive, Figure 8), but none were satisfactory from the hydraulic standpoint and in all cases construction would have been more expensive than the 5-foot sloping (trapezoidal shape) sill.

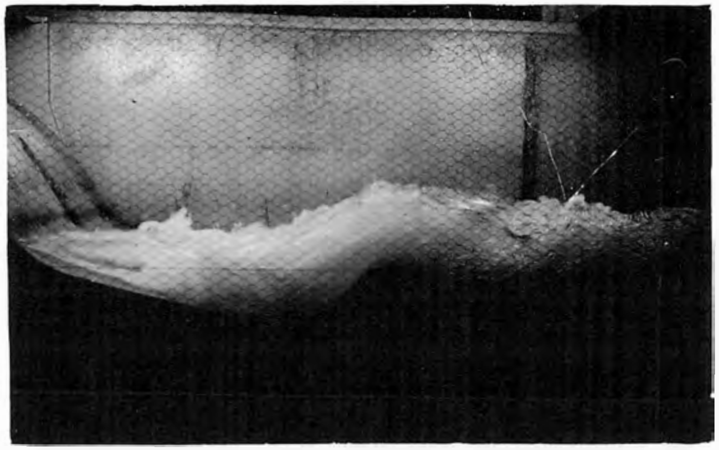
With the 1:1 slope and maximum discharge, a very imperfect jump was formed near the end of the apron (Plate 23, D), but it was the only slope that kept the jump on the apron for all discharges. The action was very poor with a flow of 350,000 second-feet (Plate 23), and, although it seemed more satisfactory in some respects than the 8:1 sloping apron with a 5-foot sloping sill, its action was precarious since a change in slope from 1:1 to $1\frac{1}{2}$:1 prevented the jump from forming on the apron at maximum discharges (Figure 13). There was more water in the pool with the 1:1 paved slope than with the 5-foot sill, giving a more effective energy dissipation with less scour (Figure 14).



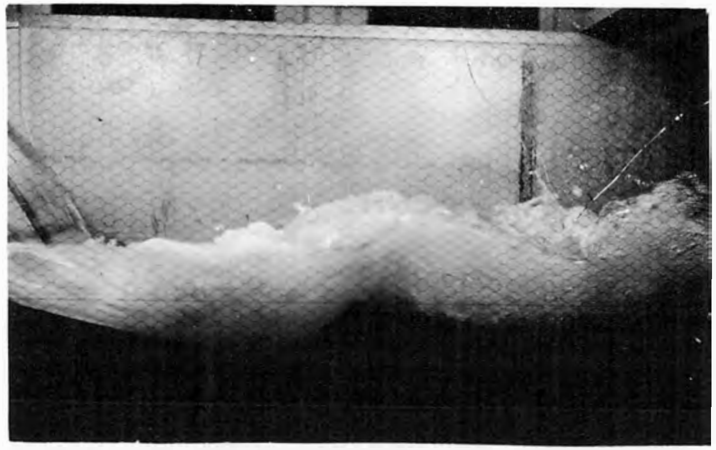
A. DISCHARGE 150,000 SECOND-FEET.



B. DISCHARGE 350,000 SECOND-FEET.



C. DISCHARGE 500,000 SECOND-FEET.



D. DISCHARGE 650,000 SECOND-FEET.

4:1 SLOPING APRON - NO SILL.
ACTION WITH PAVEMENT ON 1:1 SLOPE FROM END OF APRON TO ELEV. 498.

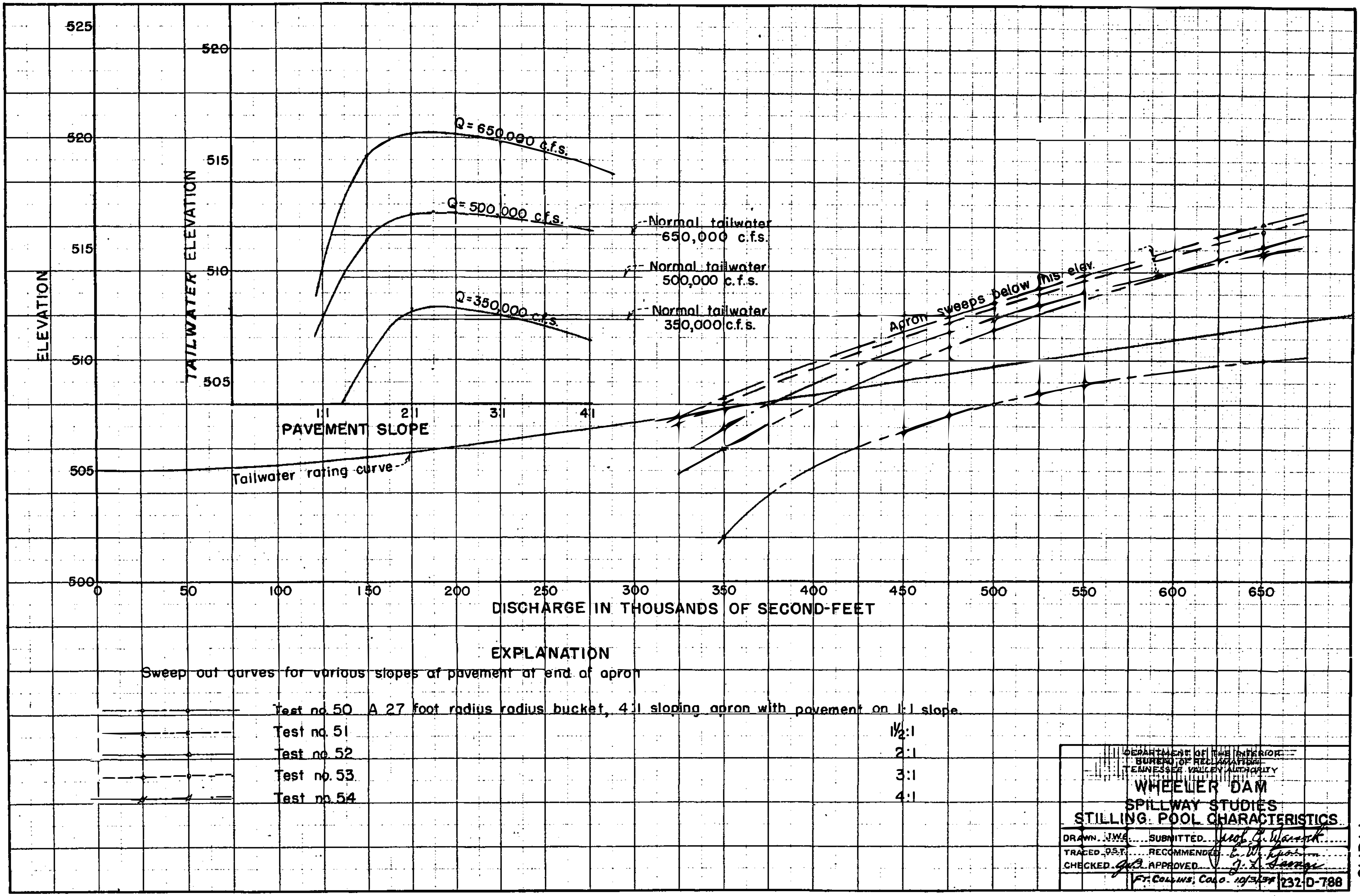

ELEVATION
TAILWATER ELEVATION
Q=650,000 c.f.s.
Q=500,000 c.f.s.
Q=350,000 c.f.s.
Normal tailwater 650,000 c.f.s.
Normal tailwater 500,000 c.f.s.
Normal tailwater 350,000 c.f.s.
PAVEMENT SLOPE
1:1
2:1
3:1
4:1
Apron sweeps below this elev.
Tailwater rating curve
525
520
515
510
505
500
0
50
100
150
200
250
300
350
400
450
500
550
600
650
DISCHARGE IN THOUSANDS OF SECOND-FEET
EXPLANATION
Sweep out curves for various slopes of pavement at end of apron
Test no. 50 A 27 foot radius radius bucket, 4:1 sloping apron with pavement on 1:1 slope.
Test no. 51 1½:1
Test no. 52 2:1
Test no. 53 3:1
Test no. 54 4:1
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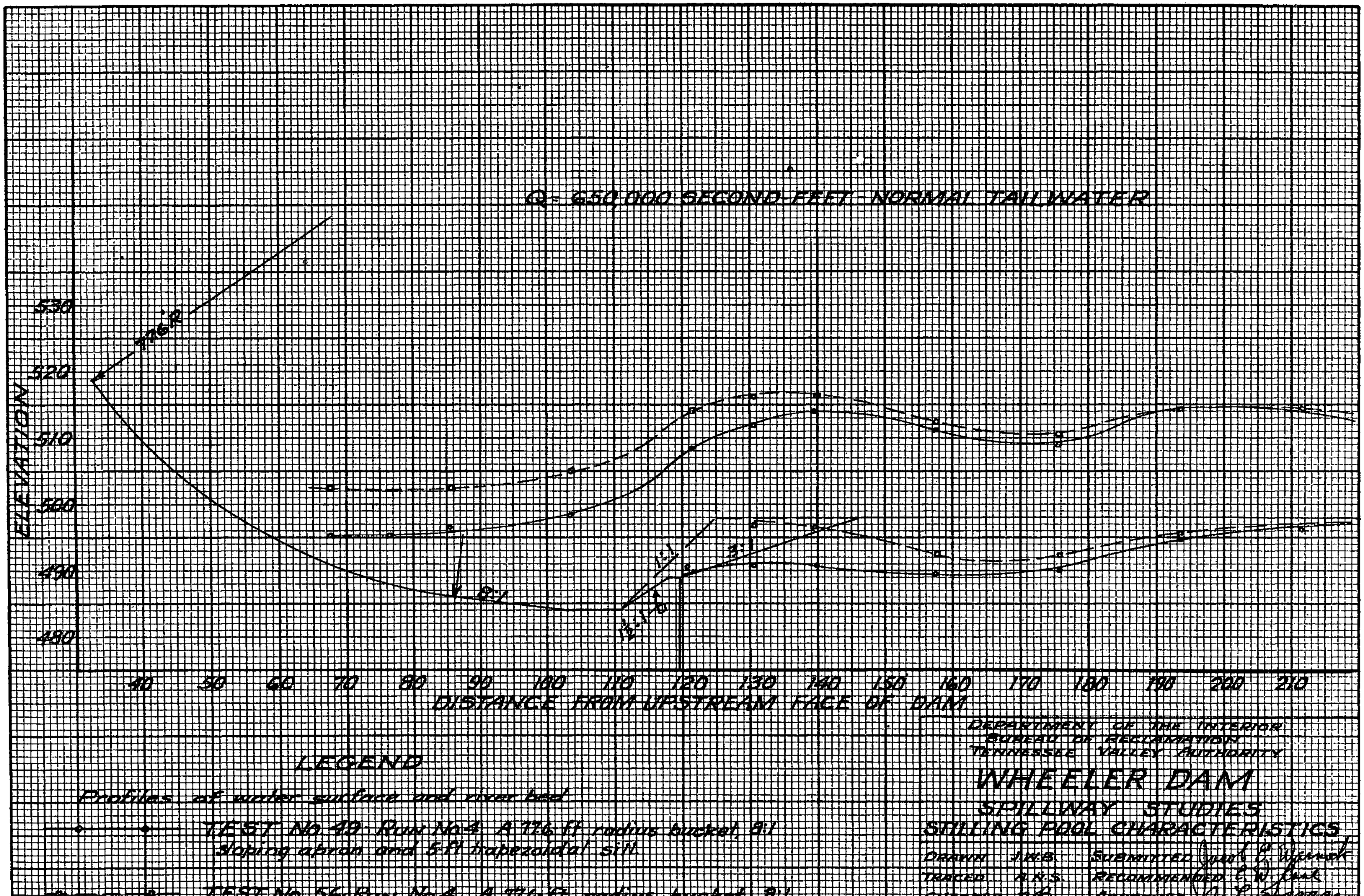
FIG. 13



Q = 650,000 SECOND-FEET - NORMAL TAILWATER
ELEVATION
530
520
510
500
490
480
77.6 R
8:1
1:1
3:1
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
DISTANCE FROM UPSTREAM FACE OF DAM
LEGEND
Profiles of water surface and river bed
TEST No. 49 Run No. 4 A 77.6 ft. radius bucket, 8:1 sloping apron and 5 ft. trapezoidal sill.
TEST No. 56 Run No. 4 A 77.6 ft. radius bucket, 8:1 sloping apron and 1:1 slope pavement to river bed
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FIG. 14

The results of these tests proved that, with the paved slopes, the apron had to be lengthened or lowered in order to obtain a satisfactory jump for all discharges.

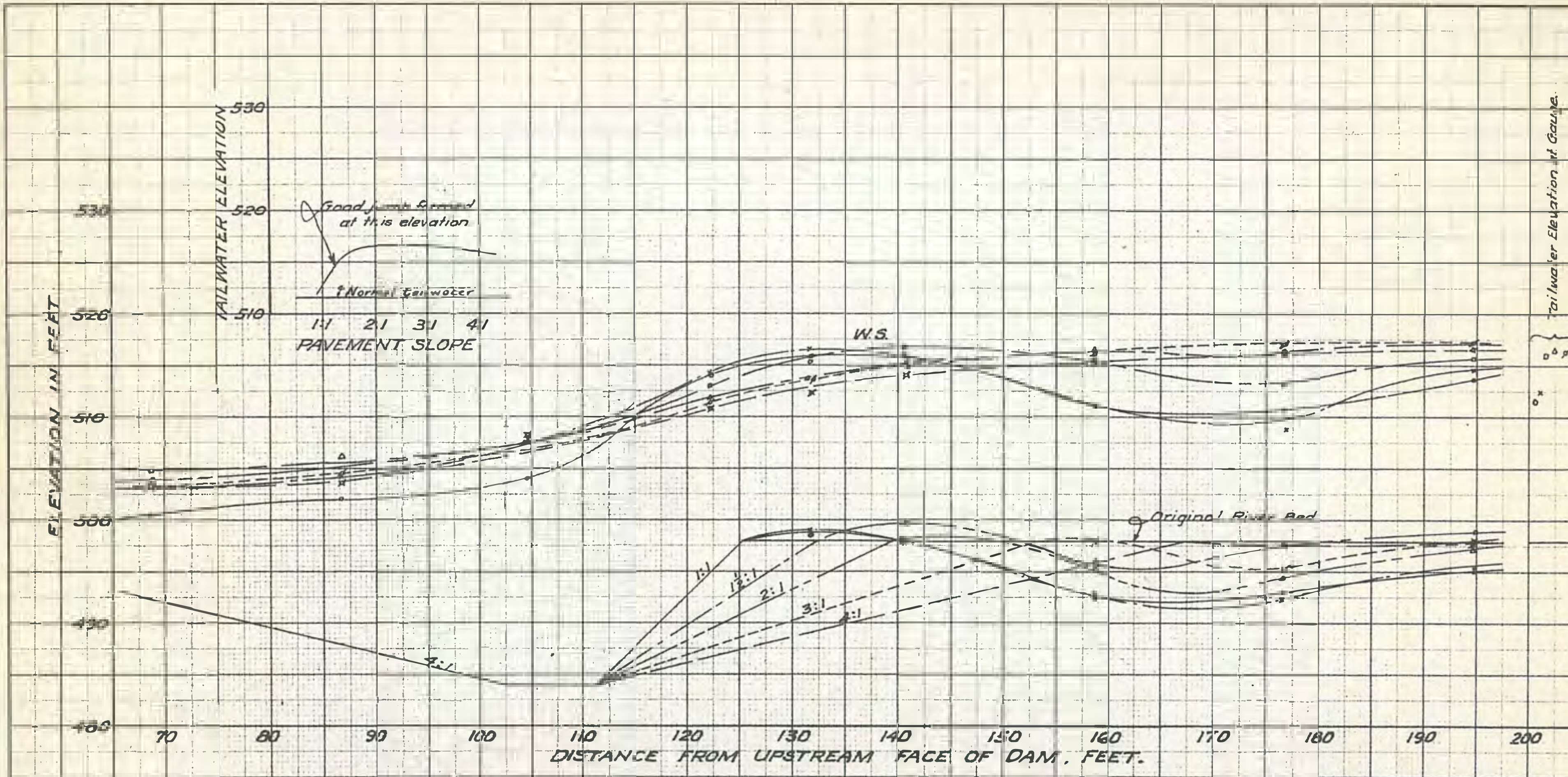
An analysis of the recorded data shows that, holding the apron length constant, for each slope there is a definite amount the apron must be lowered to obtain satisfactory results (Figure 15). This amount is approximate because of the method used in obtaining it, but is sufficiently accurate for the purpose. Instead of actually lowering the apron, the tailwater was raised, thus slightly reducing the velocity. The effectiveness of the jump was determined by observation and some variation might have occurred without being detected.

A comparison of these different slopes (Figure 15) shows that considerably more depth of tailwater was necessary to cause a satisfactory jump to form on the apron with the $1\frac{1}{2}$:1 slope than the 1:1; that the maximum depth was required for a 2:1 slope; that as the slope becames flatter less tailwater was required and the hydraulic conditions became less violent with less erosion resulting (Figure 15).

A parabolic shaped pavement was also tested (Test 55, Figure 8), but no apparent improvement was noted.

The 8:1 sloping apron (Tests 56 and 57, Figure 8)

FIG. 15



LEGEND

Profile of water surface and river bed with tailwater elevation to give good jump on apron and a discharge of 650,000 second-feet.

- TEST NO. 50, RUN No. 4 A 27-ft. radius bucket, 4:1 sloping apron and 1:1 pavement. (See Note)
- TEST NO. 50, RUN No. 4A " " " " " " " " " 1:1 "
- TEST NO. 51, RUN No. 4 " " " " " " " " " 1½:1 "
- TEST NO. 52, RUN No. 4 " " " " " " " " " 2:1 "
- TEST NO. 53, RUN No. 4 " " " " " " " " " 3:1 "
- TEST NO. 54, RUN No. 4 " " " " " " " " " 4:1 "

Tailwater elevation normal for Test 50, Run 4 only

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WHEELER DAM

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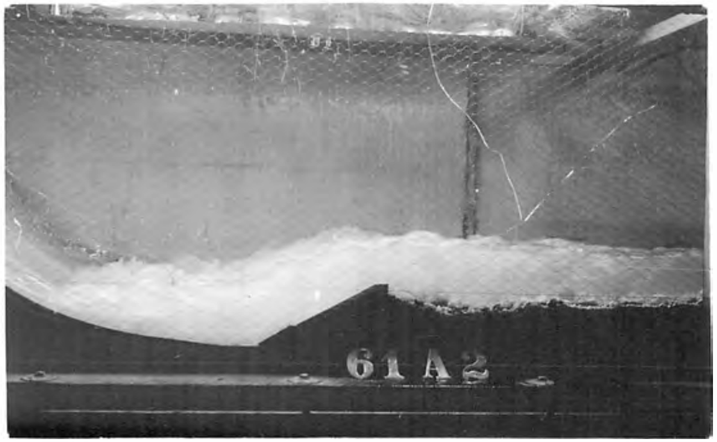
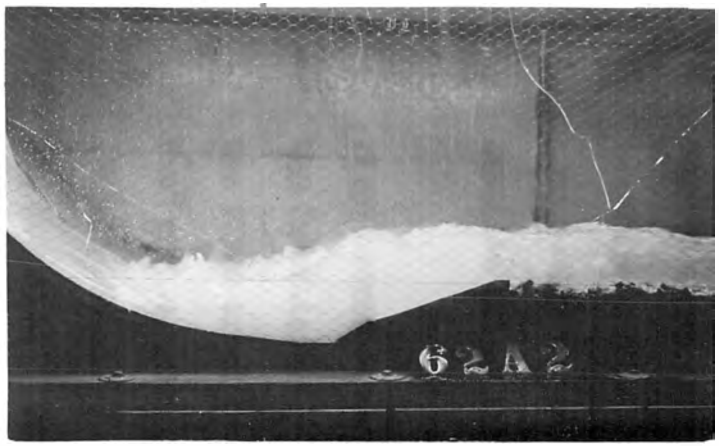
was tested to determine its effect upon the action of the sloped pavement. Results on the 1:1 and the $1\frac{1}{2}$:1 slope were practically the same as for the 4:1 sloping apron and the complete set of tests was not made.

The advisability of excavating and paving on a slope upward from the top of a sloping sill to the river bed was investigated. A 3-foot sill with a 2:1 sloped pavement was first tried (Test 59, Figure 8) but the jump failed to form on the apron at flows as low as 430,000 second-feet (Figure 16) and the pavement slope was changed to 3:1 (Test 60, Figure 8). The conditions were somewhat improved with a very imperfect jump forming at a discharge of about 485,000 second-feet (Figure 16). The 3-foot sill was replaced by a 5-foot sill and both slopes (Tests 61 and 62, Figure 8) were tested. The conditions with the 5-foot sill were less desirable (Figure 16).

With the lower sill and the 3:1 slope, more excavation was required and the pool size was increased (Figure 16), thus allowing the jump to form on the apron for larger discharges than any of the other set-ups. A difference in hydraulic conditions at medium flows was apparent (Plate 24) and became more pronounced as the flow increased. With a discharge of 650,000 second-feet and the tailwater depth to give a satisfactory jump on the apron, the same difference was noted (Plate 25). The high sill and the shorter pool

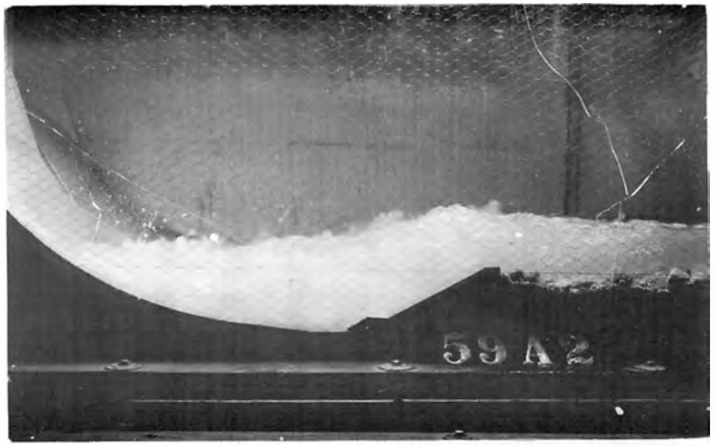


A. PAVEMENT ON 2:1 SLOPE.

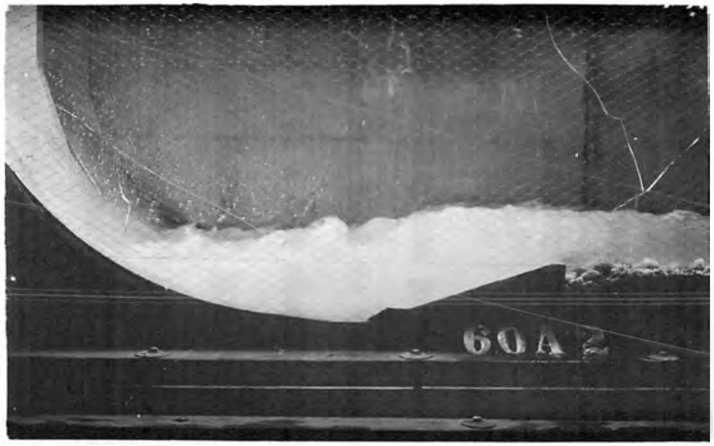


B. PAVEMENT ON 3:1 SLOPE.

5-FOOT SILL - DISCHARGE 350,000 SECOND-FEET - NORMAL TAILWATER.



C. PAVEMENT ON 2:1 SLOPE.

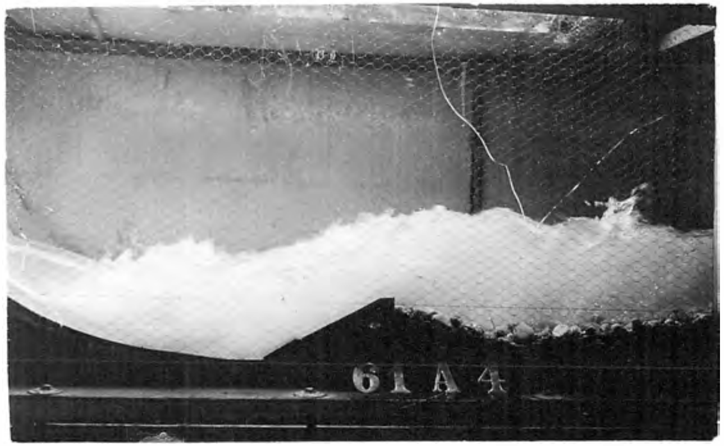


D. PAVEMENT ON 3:1 SLOPE.

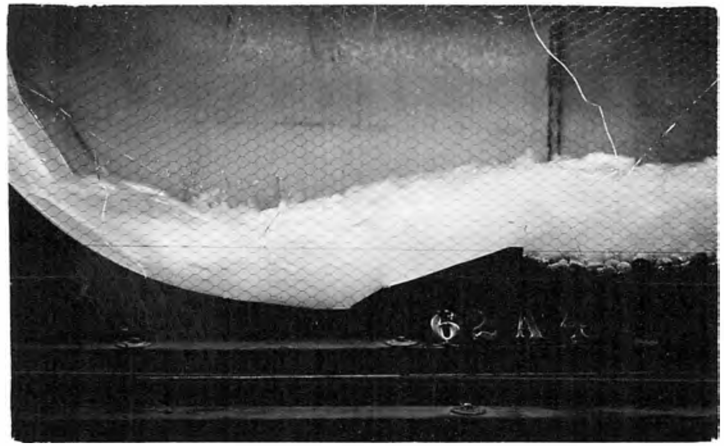
3-FOOT SILL - DISCHARGE 350,000 SECOND-FEET - NORMAL TAILWATER.

ACTION ON APRON WITH SLOPING PAVEMENT FROM TOP OF SILL TO RIVER BED.

PLATE 24

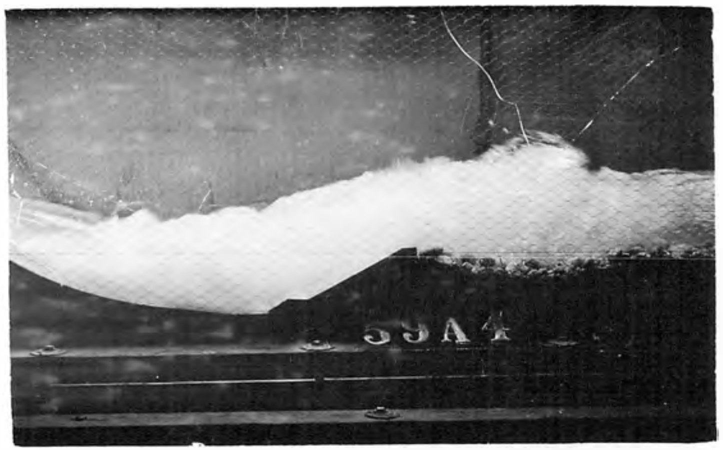


A. PAVEMENT ON 2:1 SLOPE.

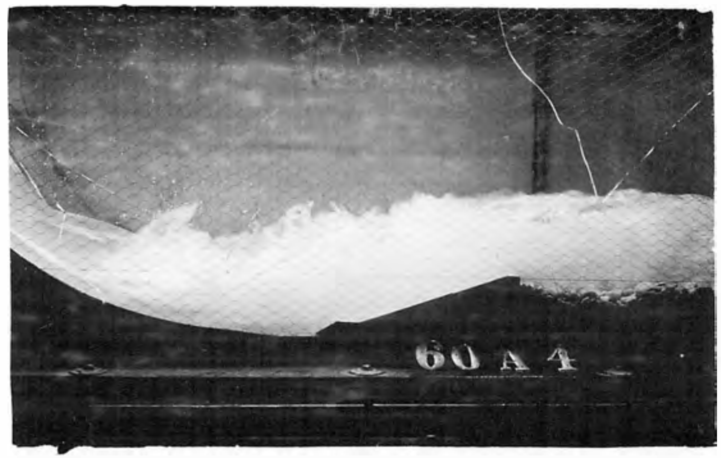


B. PAVEMENT ON 3:1 SLOPE.

5-FOOT SILL - DISCHARGE 650,000 SECOND-FEET - HIGH TAILWATER.



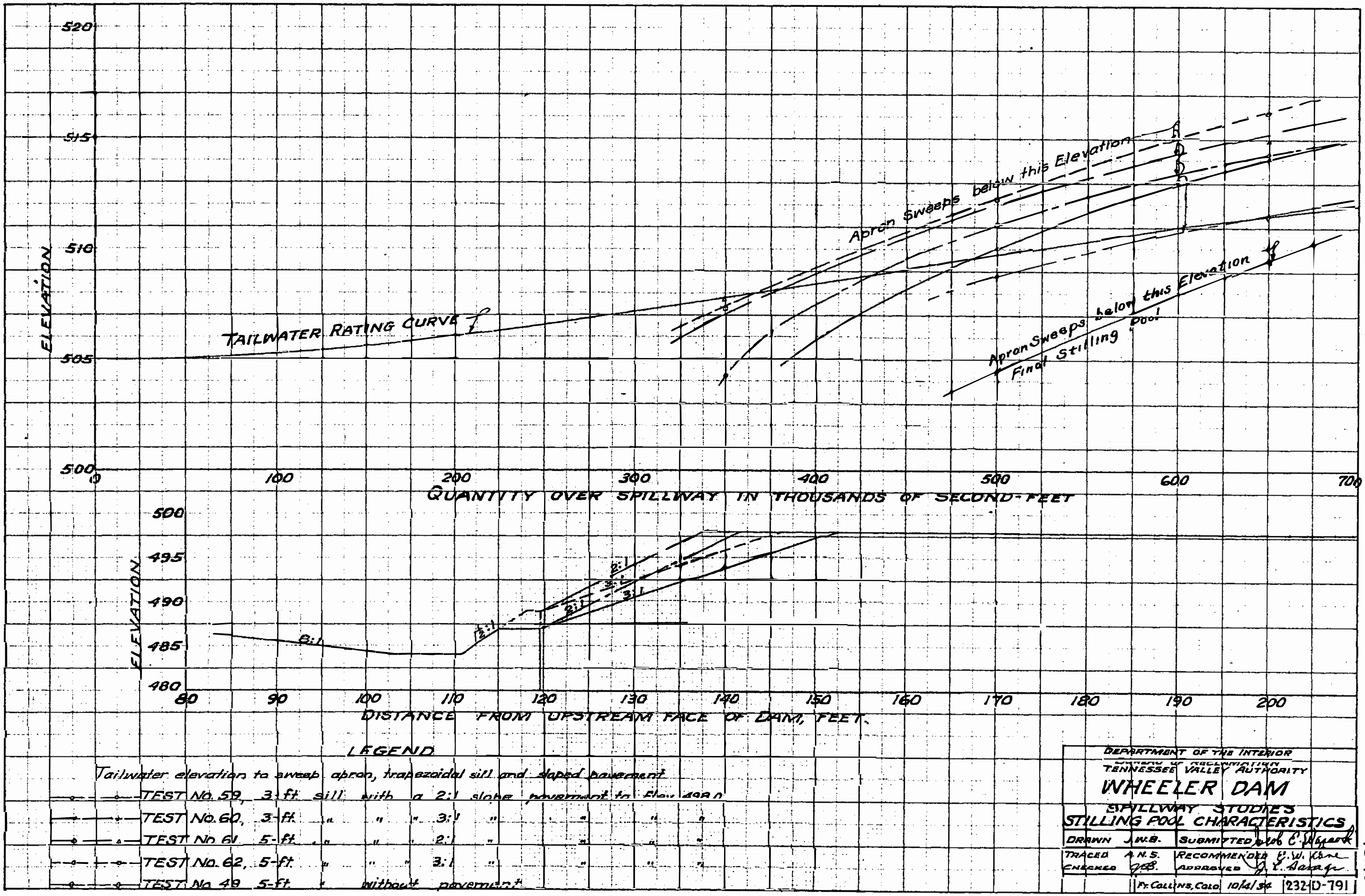
C. PAVEMENT ON 2:1 SLOPE.



D. PAVEMENT ON 3:1 SLOPE.

3-FOOT SILL - DISCHARGE 650,000 SECOND-FEET - HIGH TAILWATER.

ACTION ON APRON WITH SLOPING PAVEMENT FROM TOP OF SILL TO RIVER BED.



520
515
510
505
500
ELEVATION
0
100
200
300
400
500
600
700
QUANTITY OVER SPILLWAY IN THOUSANDS OF SECOND-FEET
TAILWATER RATING CURVE
Apron Sweeps below this Elevation
Apron Sweeps below this Elevation
Final Stilling Pool
500
495
490
485
480
ELEVATION
80
90
100
110
120
130
140
150
160
170
180
190
200
DISTANCE FROM UPSTREAM FACE OF DAM, FEET.
2:1
3:1
2:1
3:1
8:1
LEGEND
Tailwater elevation to sweep apron, trapezoidal sill and sloped pavement
TEST No. 59, 3-ft. sill with a 2:1 slope pavement to Elev. 498.0
TEST No. 60, 3-ft. " " " 3:1 " " " "
TEST No. 61 5-ft. " " " 2:1 " " " "
TEST No. 62, 5-ft. " " " 3:1 " " " "
TEST No. 49 5-ft. " without pavement
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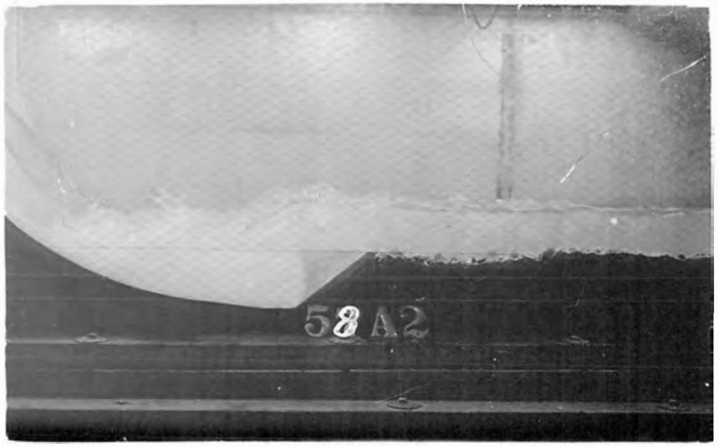
FIG. 10

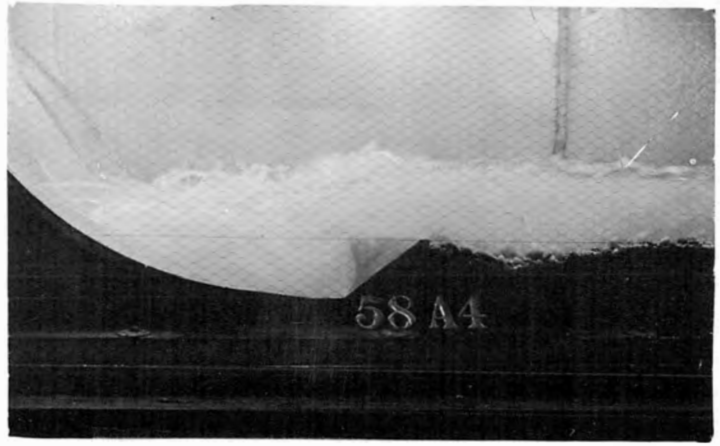
gave rough hydraulic conditions which resulted in more erosion (Plate 25) and it was thought that this type of protection was not as satisfactory as the sills alone because they did not allow a natural retrogression of the river bed necessary to form a good jump.

A natural retrogression of the river bed was proven essential to the formation of a satisfactory jump on the apron. The diffuser type sill had proven so effective on the Grand Coulee model that tests were made to obtain the properties of such a sill for the Wheeler apron.

The diffuser sill was made up of numerous diverging chambers in which the area of exit was greater than the area of entrance thus simulating the action of a draft tube and causing a decrease in the velocity of the water as it passed through the chambers. Part of the stream of water is turned upward and backward by the chamber walls forming a roll on the apron (Plate 26). This roll impinges upon and tends to reduce the velocity of the inflowing sheet. Also, the impingement of the jets issuing from the chamber exits reduces the velocity, aiding in the dissipation of the contained energy. Probably the most useful feature of this type of sill was the distribution of velocities in the tailwater downstream. The higher velocities were near the surface and the lower along



A. DISCHARGE 350,000 SECOND-FEET.



B. DISCHARGE 650,000 SECOND-FEET.

LARGE DIFFUSER SILL.

C. DISCHARGE 350,000 SECOND-FEET.

D. DISCHARGE 650,000 SECOND-FEET.

SMALL DIFFUSER SILL WITH CURVED UPSTREAM FACE.

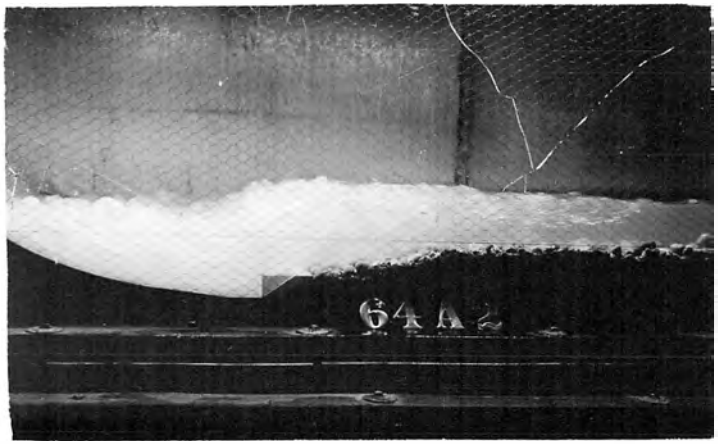
ACTION OF DIFFUSER SILLS.

PLATE 26

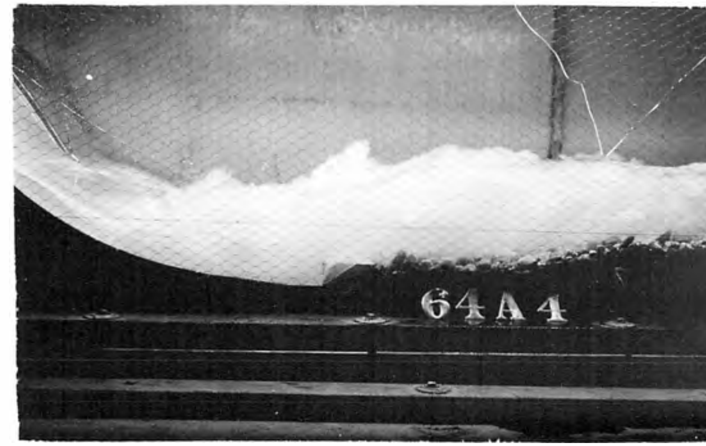
the river bed which caused a formation of currents such that loose material was deposited immediately below the sill with only slight erosion at a considerable distance downstream. The above characteristics apply to all diffuser sills with their effectiveness depending upon the Correctness efficiency of design. for a given condition

Different sizes and shapes of diffuser sills (formerly called breaker blocks) were tried. The first, a 14-foot sill with curved upstream face and top at Elev. 498 (Test 58, Figure 8), gave good hydraulic action (Plate 26, A and B) and slight erosion (Figure 17). The second, a 5-foot sill with curved upstream face and top at Elev. 489 (Test 63, Figure 9; Plate 26, C and D), caused considerable increase in erosion (Figure 17). Some improvement over the 5-foot sloping sill was noted in the second case (Figure 18). The upstream face of the sill was made vertical (Test 64, Figure 9) in an attempt to improve the conditions. More impact resulted and slight improvement was noted (Plate 27, A and B, and Figure 19) but the increased impact was undesirable and further studies were made.

A comparison between the sill described above and the 5-foot Rehbock dentated sill was made (Test 65, Figure 9). Conditions were improved (Figure 19) but since this sill

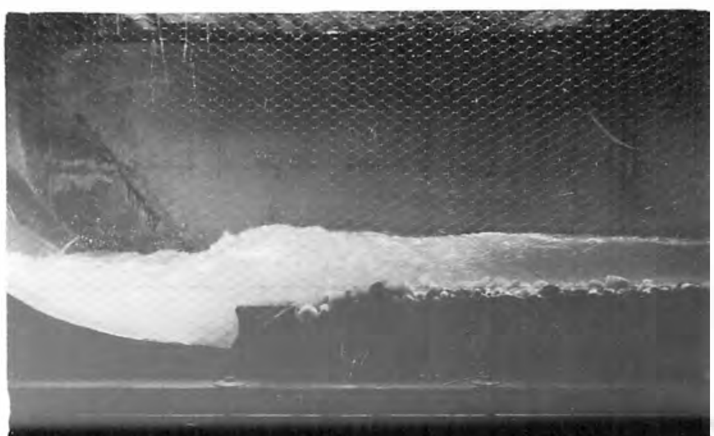


A. DISCHARGE 350,000 SECOND-FEET.

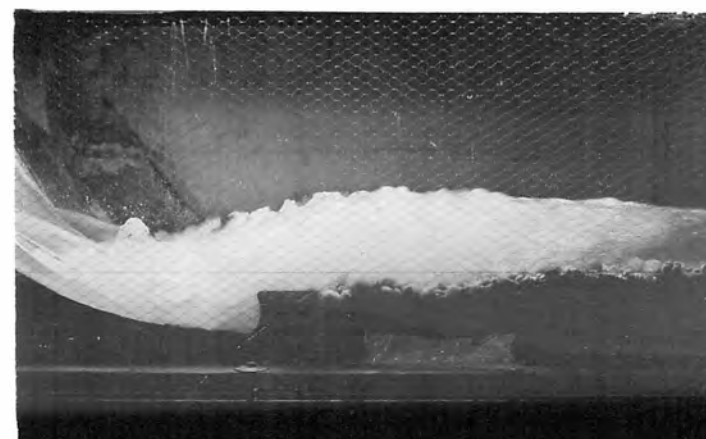


B. DISCHARGE 650,000 SECOND-FEET.

SMALL DIFFUSER SILL WITH VERTICAL UPSTREAM FACE.



C. DISCHARGE 350,000 SECOND-FEET.



D. DISCHARGE 650,000 SECOND-FEET.

DIFFUSER SILL OF FINAL DESIGN TYPE.

ACTION OF DIFFUSER SILLS.

PLATE 27

A. UPSTREAM SIDE VIEW.

B. TOP VIEW.

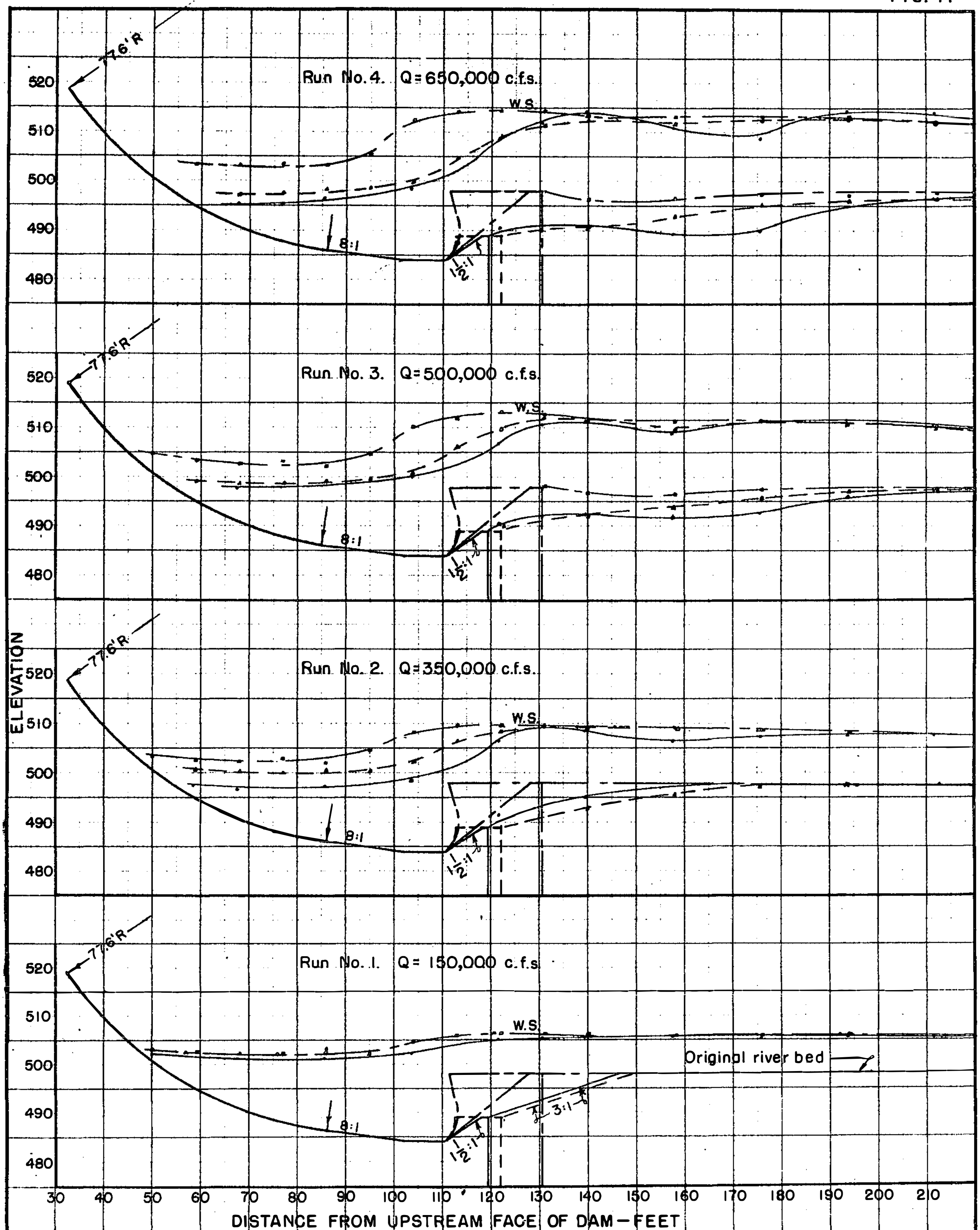
DIFFUSER SILL WITH FLAT TOP.

C. UPSTREAM VIEW.

D. DOWNSTREAM SIDE VIEW.

CAPPED DIFFUSER SILL.

FLOW LINES AS SHOWN BY PAINT TESTS.



EXPLANATION

Profile of water surface and river bed, normal tailwater.

———— Test No. 49. A 77.6' radius bucket with 8:1 sloping apron at elevation 484. Trapezoidal sill at end of apron.

— — — Test No. 63. A 77.6' radius bucket with 8:1 sloping apron at elevation 484. Modified diffuser sill with curved upstream face.

—·—·— Test No. 58. A 77.6' radius bucket with 8:1 sloping apron at elevation 484. Diffuser sill to elevation 498 at end of apron.

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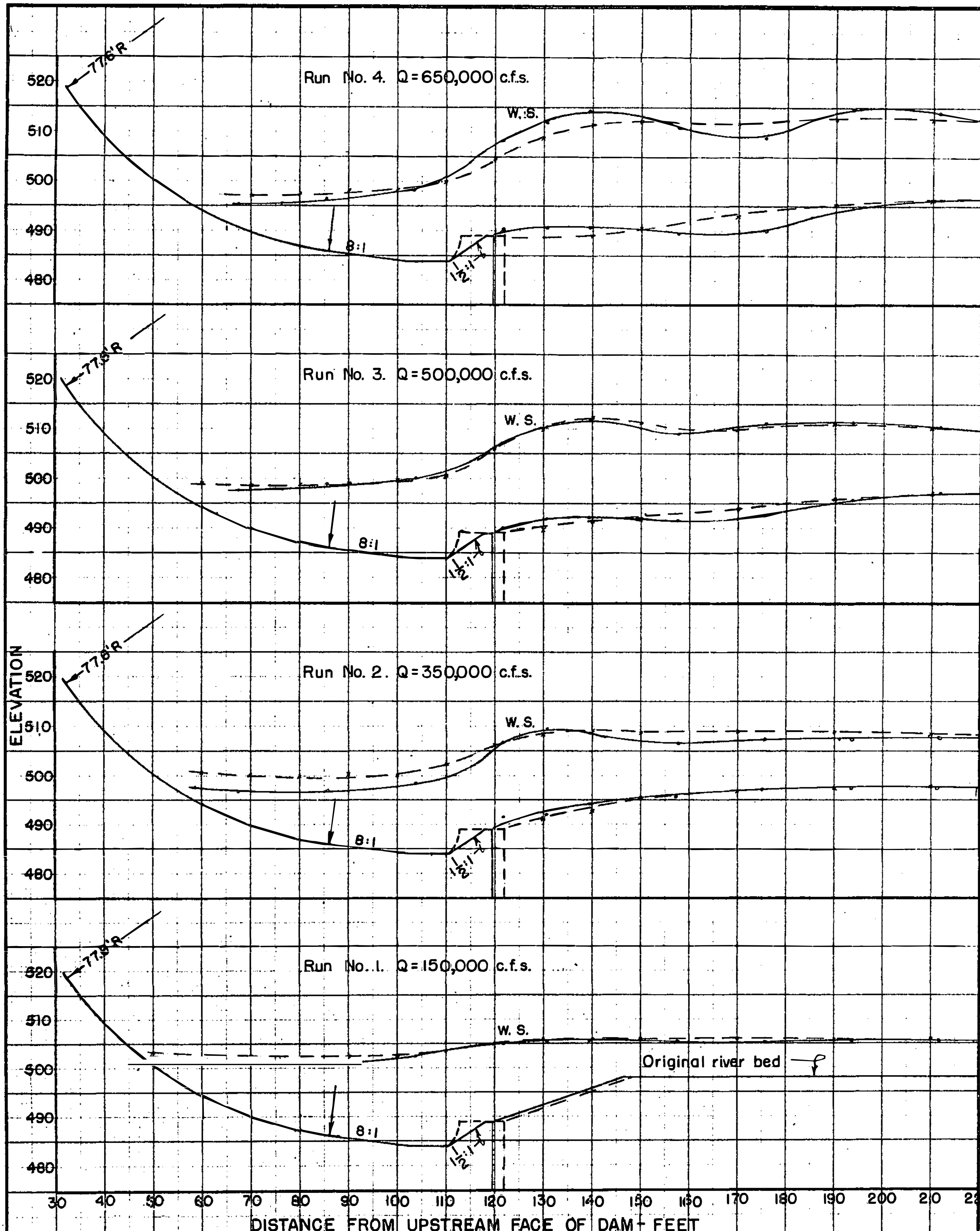

EXPLANATION

Profiles of water surface and river bed, normal tailwater.

———— Test No. 49. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Trapezoidal sill at end of apron.

— — — Test No. 63. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Modified diffuser sill with curved upstream face, at end of apron.

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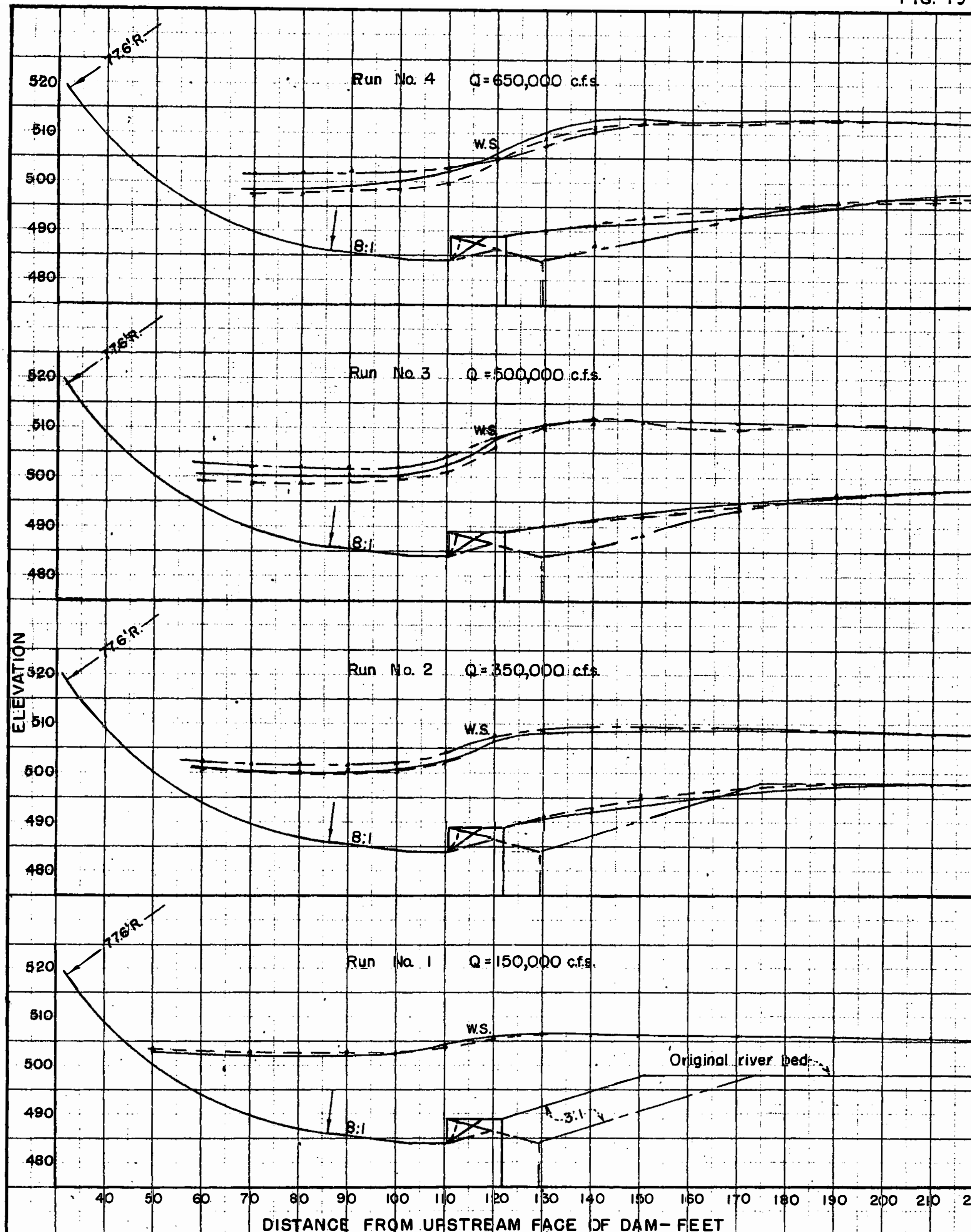
FIG. 19



EXPLANATION

Profile of water surface and river bed, normal tailwater

Test No. 64. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Modified diffuser sill at end of apron.

Test No. 63. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Modified diffuser sill with curved upstream face, at end of apron.

Test No. 65. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Rehback dentated sill at end of apron.

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STILLING POOL CHARACTERISTICS

DRAWN J.W.B. SUBMITTED Joseph E. Warnock
TRACED D.W.S. RECOMMENDED E. W. Lane
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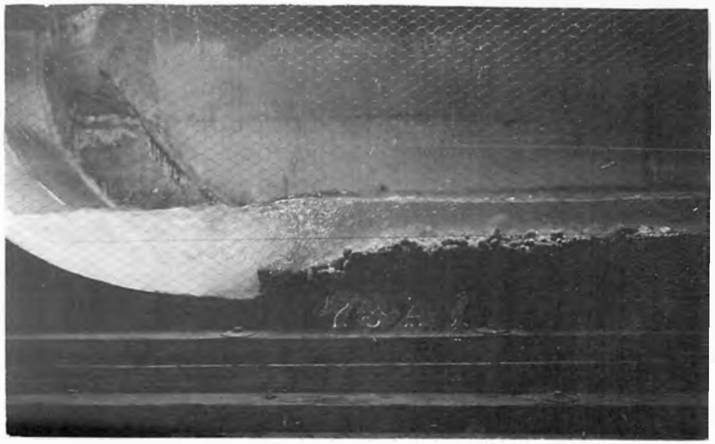
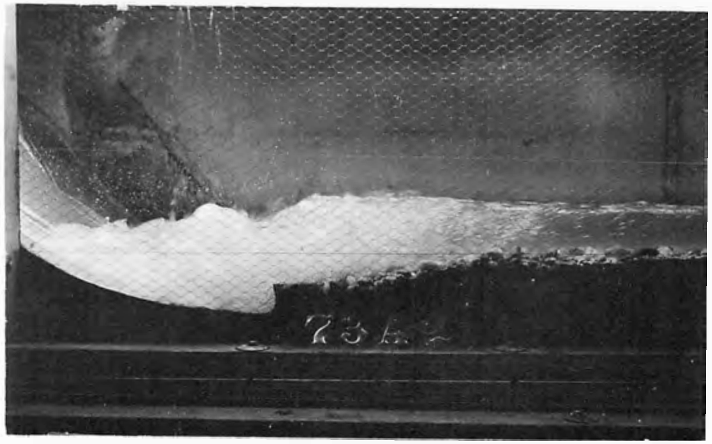
Ft Collins, Colo. 7/26/34 232-D-794

depends upon direct impact for its action and requires excessive excavation, it was considered undesirable.

No consideration up to this time had been given to obtaining an agreement between the spacing of the diffusion chambers and the spacing of the contraction joints in the dam. With a contraction joint spacing of 45 feet, a sill 8 feet high with top at Elev. 492 (Test 71, Figure 9) was constructed to provide seven chambers between the contraction joints. This sill gave very good hydraulic and erosion conditions, but paint tests indicated the presence of a low pressure area on top of the walls (Plate 28, A and B) and caps were added (Test 72, Figure 9) to eliminate it. The caps produced no apparent effect on the hydraulic conditions and paint tests indicated that no vacuum was present (Plate 28, C and D). Although the sill was very effective, it was thought to be more than adequate and a smaller sill with nine chambers between contraction joints (Test 73, Figure 10) was tried. A comparison (Plate 27 C and D and Plate 29) shows that while the action is not as desirable as the higher sill, it is very effective and the difference in erosion is slight. Profiles of water surface and river bed (Figure 20) show these conditions. There was also a noted improvement over the 5-foot sloping sill (Figure 21) and the diffuser sill was

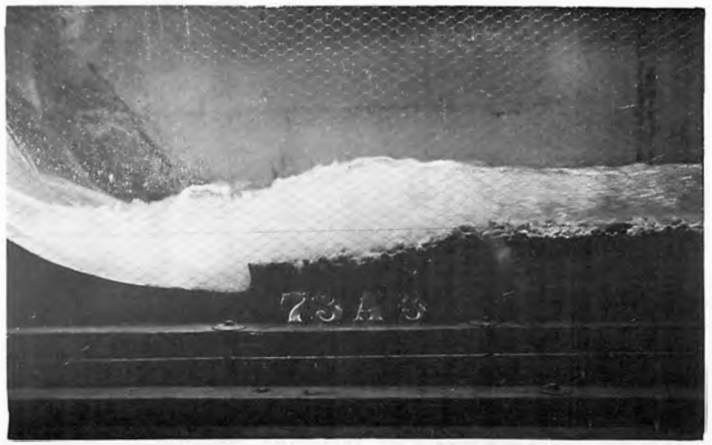


A. DISCHARGE 150,000 SECOND-FEET.

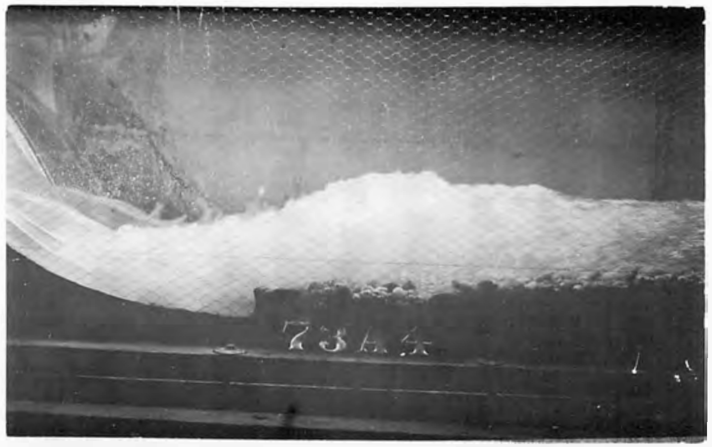


B. DISCHARGE 350,000 SECOND-FEET.

DIFFUSER SILL - FINAL DESIGN.



C. DISCHARGE 500,000 SECOND-FEET.

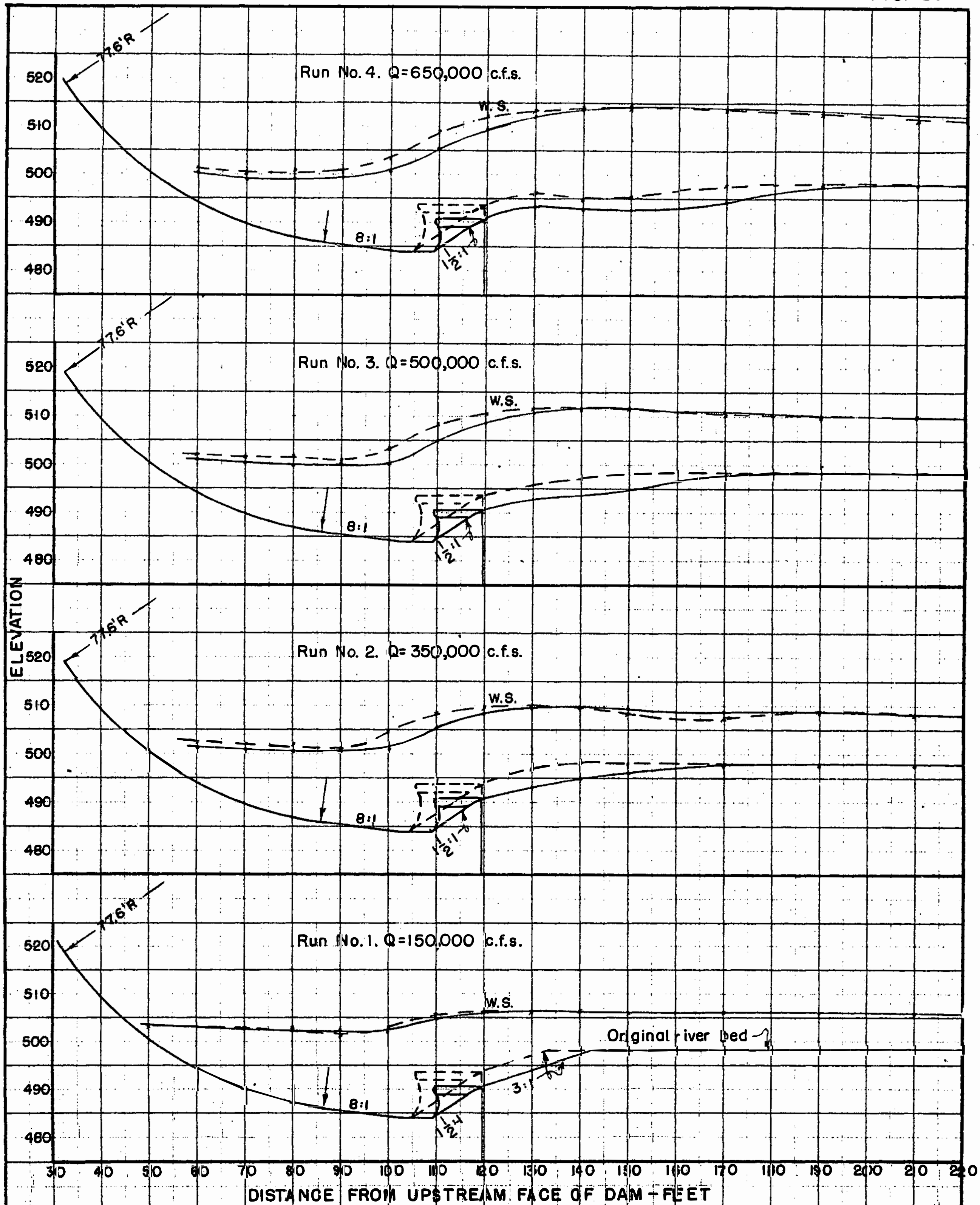


D. DISCHARGE 650,000 SECOND-FEET.

DIFFUSER SILL - FINAL DESIGN.

ACTION OF ADOPTED DIFFUSER SILL AND SLOPING APRON.

Fig. 20



EXPLANATION

Profile of water surface and river bed, normal tailwater.

Test No. 73. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Diffuser sill of final design at end of apron, top of elevation 490.5.

Test No. 72. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Diffuser sill at end of apron, top at elevation 493.5.

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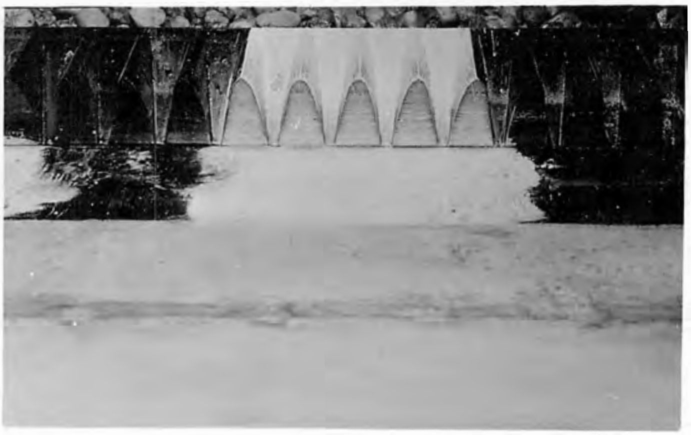
232-D-795

recommended for use on the Wheeler apron.

After its tentative adoption, tests were made on the sill to eliminate any undesirable features that might be present. Paint tests were made and results comparable to those for the high sill were obtained (Plate 30). Profiles of water surface and river bed were taken which show (Figure 22) a comparatively smooth tailwater surface and slight erosion.

The tailwater elevation for which the jump failed to form on the apron for various flows was obtained (Figure 23). It was found that for discharges up to about 500,000 second-feet, the jump could not be made to sweep off the apron. At higher flows it would not fail to form except with abnormally low tailwater. The jump almost fails to form on the apron for the maximum discharge with the tailwater at Elev. 509.5 or 2.1 feet below normal. A greater factor of safety was also obtained by using the diffuser sill.

Piezometers were installed (Figure 24) to verify the existence of negative pressures within the sill chambers and to determine the impact on the chamber wall. These openings were connected to glass tubes mounted on a graduated board outside the flume to which the center of each opening was referred. With steady flow conditions, readings to the



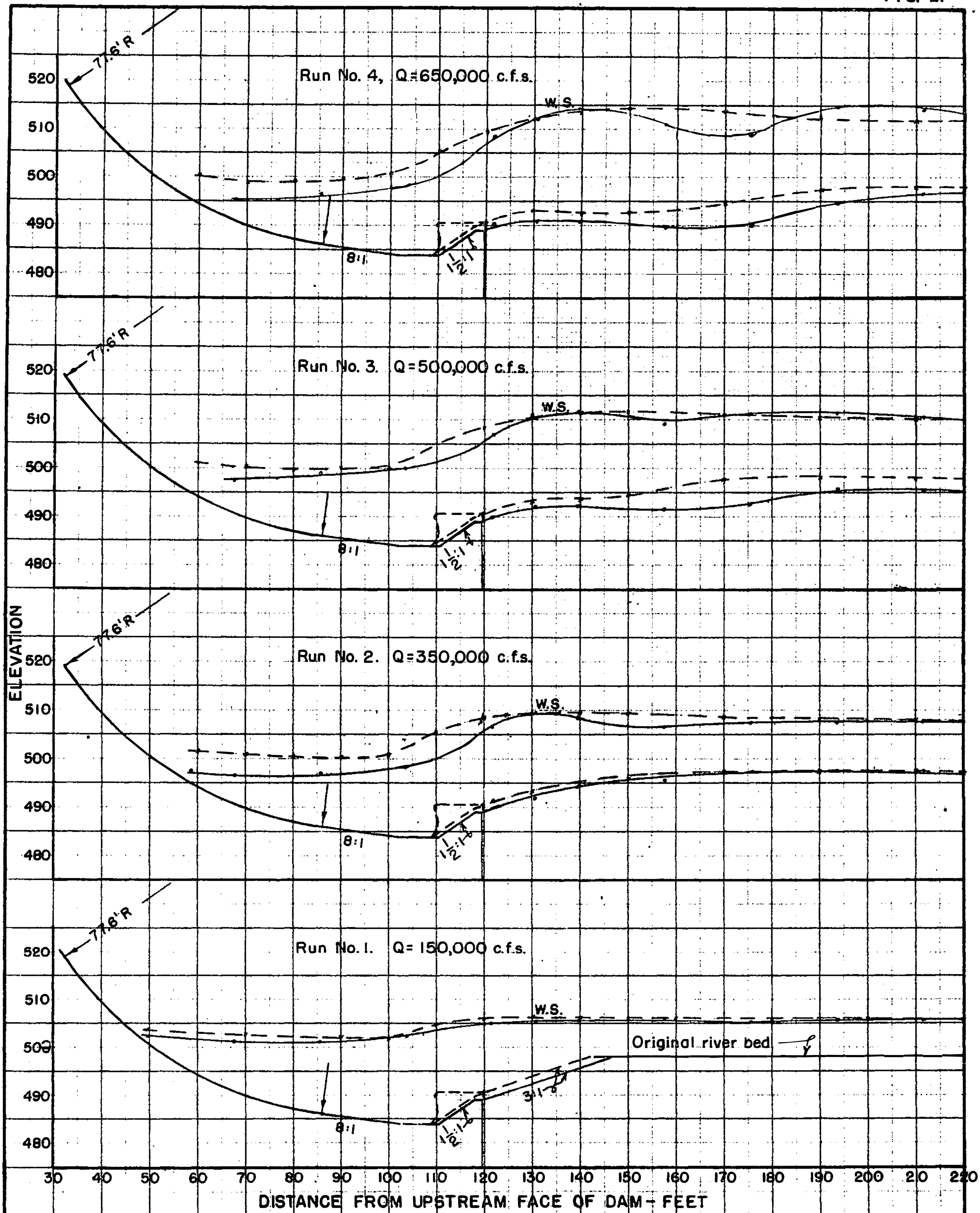
A. UPSTREAM VIEW.

B. DOWNSTREAM SIDE VIEW.

C. TOP VIEW.

FINAL DESIGN DIFFUSER SILL - FLOW LINES AS SHOWN BY PAINT TESTS.

FIG. 21



EXPLANATION

Profile of water surface and river bed, normal tailwater.

——— Test No. 49. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Trapezoidal sill at end of apron.

— — - — Test No. 73. A 77.6 ft. radius bucket with 8:1 sloping apron at elevation 484. Diffuser sill of final design at end of apron.

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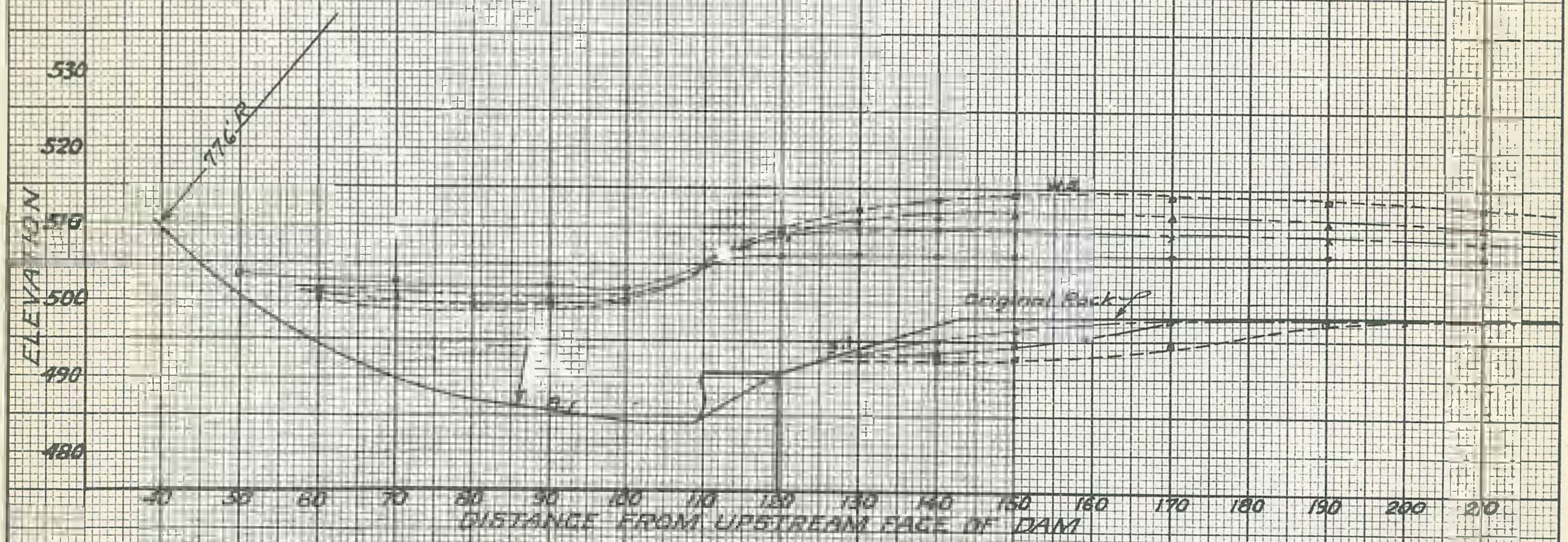

LEGEND

Profiles of water surface and river bed - Normal Tailwater
Final Design of Stilling Pool

TEST No. 73

Run No. 1 - Discharge 150 000 Second-feet
Run No. 2 - " 350 000
Run No. 3 - " 500 000
Run No. 4 - " 650 000

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WHEELER DAM
SPILLWAY STUDIES
STILLING POOL CHARACTERISTICS.

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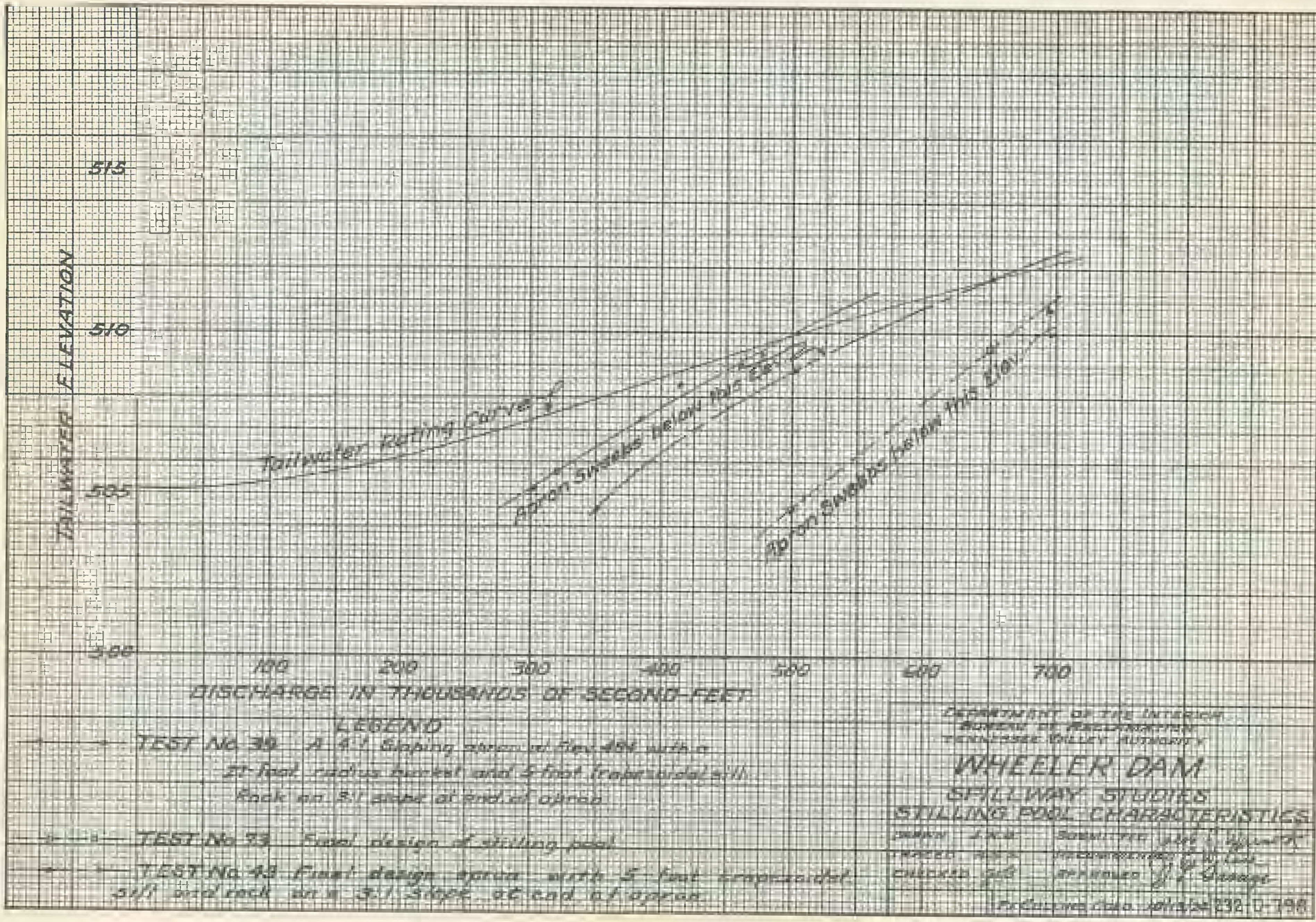
CHECKED	[illegible]	APPROVED	J. L. Savage
	FT. COLLINS, COLO.	10/12/34	237-D-197

FIG. 22



TAILWATER ELEVATION
515
510
505
500
Tailwater Rating Curve
Apron Sweeps below this Elev.
Apron Sweeps below this Elev.
100
200
300
400
500
600
700
DISCHARGE IN THOUSANDS OF SECOND-FEET
LEGEND
TEST No. 39 A 4:1 Sloping apron at Elev. 484 with a 25-foot radius bucket and 5-foot trapezoidal sill. Rock on 3:1 slope at end of apron.
TEST No. 73 Final design of stilling pool.
TEST No. 48 Final design apron with 5-foot trapezoidal sill and rock on a 3:1 slope at end of apron.
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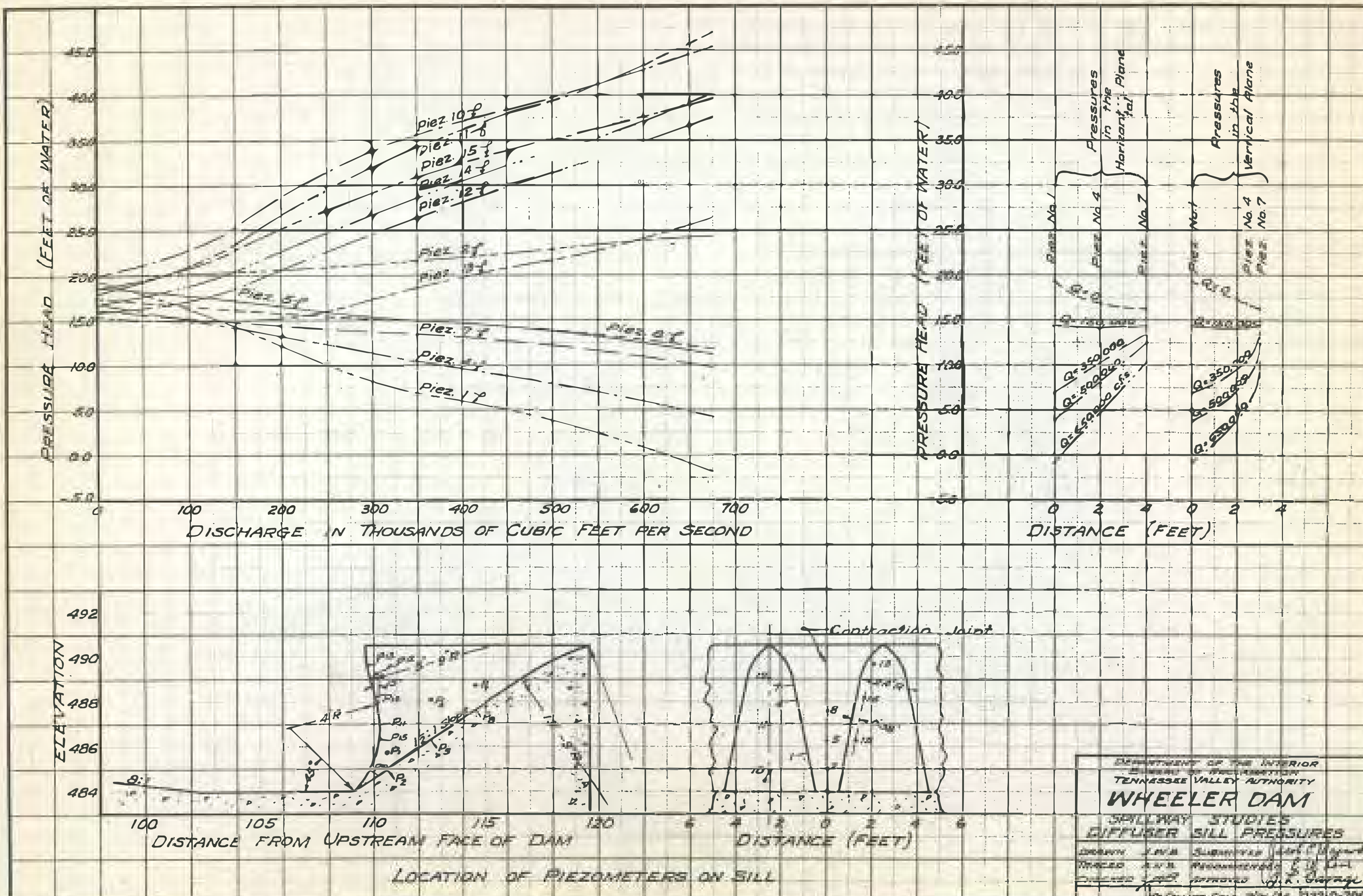
FIG. 23



PRESSURE HEAD (FEET OF WATER)
Piez. 10
Piez. 11
Piez. 15
Piez. 14
Piez. 12
Piez. 8
Piez. 13
Piez. 5
Piez. 9
Piez. 6
Piez. 4
Piez. 1
DISCHARGE IN THOUSANDS OF CUBIC FEET PER SECOND
Pressures in the Horizontal Plane
Pressures in the Vertical Plane
Piez. No. 1
Piez. No. 4
Piez. No. 7
Q=0
Q=350,000
Q=500,000
Q=650,000 c.f.s.
DISTANCE (FEET)
ELEVATION
Contraction Joint
4' R
8:1
1.5:1 Slope
DISTANCE FROM UPSTREAM FACE OF DAM
DISTANCE (FEET)
LOCATION OF PIEZOMETERS ON SILL
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WHEELER DAM
SPILLWAY STUDIES
DIFFUSER SILL PRESSURES
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
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FIG. 24

thousandth of a foot were made, which, when referred to the piezometer opening, gave the pressure at that point on the model in feet of water. These results were converted to prototype pressure by the equation

$$H_p = H_m N^{1.0} \quad \text{where } N = 36.$$

The prototype pressures (Table 1 are plotted on Figure 24. Some of the piezometers were located in pairs, at the same relative position on each wall of the chamber, and the average reading was used in plotting the results.

With a flow of 650,000 second-feet, a very slight negative pressure was found to occur on the walls at the entrance end of the chamber. Results showed that with the smaller discharges, a pressure existed at all piezometers and that as the flow increased the pressures increased or decreased (Figure 24) depending upon the action of the water at the piezometer opening. The draft tube action was substantiated by the gradual increase in pressure from the entrance to the exit of the chamber. Some impact by the inflowing stream was recorded on the upstream end of the chamber walls.

As a result of the extensive studies on the stilling pool of the Wheeler model, the sloping apron (Test 73, Figure 10) with a 77.6-foot radius bucket and 8:1 downward slope to Elev. 484 and 6.5-foot diffuser sill at the end was adopted as the final design.

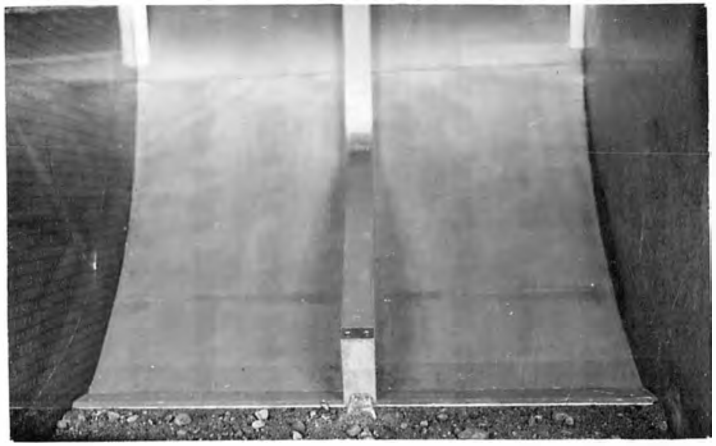
TABLE NO. 1

SUMMARY OF DATA AND TRANSFERENCE OF RESULTS - WHEELER DAM
Diffuser Sill Pressures

Piezometer	Pressures in Feet of Water								Piezometer
	Run No. 1 Q=150,000		Run No. 2 Q=350,000		Run No. 3 Q=500,000		Run No. 4 Q=650,000		
Number	Model	Proto	Model	Proto	Model	Proto	Model	Proto	Elevation
1	0.3913	14.09	0.1847	6.65	0.1365	4.91	0.0443	1.59	485.72
2	.5656	20.36	.6008	21.63	.6366	22.92	.6696	24.11	485.01
3	.4066	14.64	.1958	7.05	.0844	3.04	-.0664	- 2.39	485.67
4	.3959	14.25	.2889	10.40	.2299	8.28	.1429	5.14	488.11
5	.4738	17.06	.4154	14.95	.3828	13.78	.3248	11.69	486.28
6	.3936	14.17	.2856	10.28	.2166	7.80	.1266	4.56	488.16
7	.4120	14.83	.3795	13.64	.3500	12.60	.3050	9.98	488.72
8	.4484	16.14	.4150	14.94	.3864	13.91	.3374	12.15	487.59
9	.4116	14.82	.3700	13.32	.3360	12.10	.2910	10.48	488.72
10	.7045	25.36	.9905	35.66	1.1031	39.71	1.2355	44.48	484.87
11	.6477	23.32	.9497	34.19	1.0923	39.32	1.2707	45.75	487.03
12	.5133	18.48	.7369	26.53	.8727	31.42	1.0187	36.37	489.04
13	.4149	14.94	.5203	18.73	.6007	21.63	.7077	25.48	489.65
14	.5703	20.53	.7953	28.63	.9343	33.63	1.0883	39.18	488.16
15	.6367	22.92	.8581	30.89	.9721	35.00	1.0831	38.99	486.28

B. Intermediate Training Walls.

The length of the Wheeler Dam is such that it would not be logical to operate the sixty gates individually, and as a result they were divided into sets of five gates each. In the operation of these sets, cross-currents would be encountered in the stilling pool when the gate opening of adjacent sets was different. It was for the purpose of facilitating the action in the pool that the intermediate training walls were included in the design of the dam. These walls were located downstream from every fifth pier and extended at Elev. 505.0 to the end of the apron (Test 66, Figure 9). The necessity of the walls was questioned and tests were made to determine their feasibility. The results from the model used which represented only 108 feet of the prototype spillway indicated that this question would be satisfactorily answered only by tests on a model representing a length of spillway containing at least three sets of gates. The tests were made with the assumption that all gates except one set at the end of the spillway were discharging. The reason for such an assumption was that it gave the worst conceivable cross-currents caused by the high headwater and low tailwater. Photographs of the action of the wall (Plate 31) and of the position of the



A. INTERMEDIATE TRAINING WALL.

B. DISCHARGE 350,000 SECOND-FEET.

C. DISCHARGE 500,000 SECOND-FEET.

D. DISCHARGE 650,000 SECOND-FEET.

ACTION OF INTERMEDIATE TRAINING WALL.

gravel after each run (Plate 32) indicated that at all discharges, water flowed from the static side over the wall and into the jump on the other side. This action had very little effect at the lower flows, but proved undesirable for the larger floods, in that the jet eroded just downstream from the end of the wall (Plate 32) and a whirl on the apron below the closed set piled a considerable amount of sand on the apron. Tests were not made with the final design but it is reasonable to believe that the action would be improved with the use of the diffuser sill. In fact one of the reasons leading to the design of the final diffuser sill was an effort to eliminate the possible undesirable cross-currents in the stilling pool caused by an unbalanced operation of the spillway gates. The possibility of an extreme unbalanced condition occuring with the larger discharges is very remote, thus only the action at lower flows need be seriously considered. This action would be immediately eliminated as soon as the closed setswere allowed to discharge. The results would indicate that the walls stabilize the unbalanced condition of high headwater and low tailwater, thus aiding in simplifying the gate operating program.

To design the wall to withstand the differential pressures existing, when the gates on one side of the wall were discharging and those on the other side were

A. DISCHARGE 150,000 SECOND-FEET.

B. DISCHARGE 350,000 SECOND-FEET.

C. DISCHARGE 500,000 SECOND-FEET.

D. DISCHARGE 650,000 SECOND-FEET.

ACTION OF INTERMEDIATE TRAINING WALL.
POSITION OF GRAVEL AFTER RUN.

closed, it was necessary to obtain the pressures which were not obtainable except by model studies. Piezometers installed at intervals along each side of the wall (Figure 25) were connected to glass tubes mounted on a graduated board outside the flume to which the center of each opening was referred. With steady flow conditions, readings were made which, when referred to the piezometer opening, gave the pressure at that point on the model in feet of water. These results were converted to prototype pressure by the equation

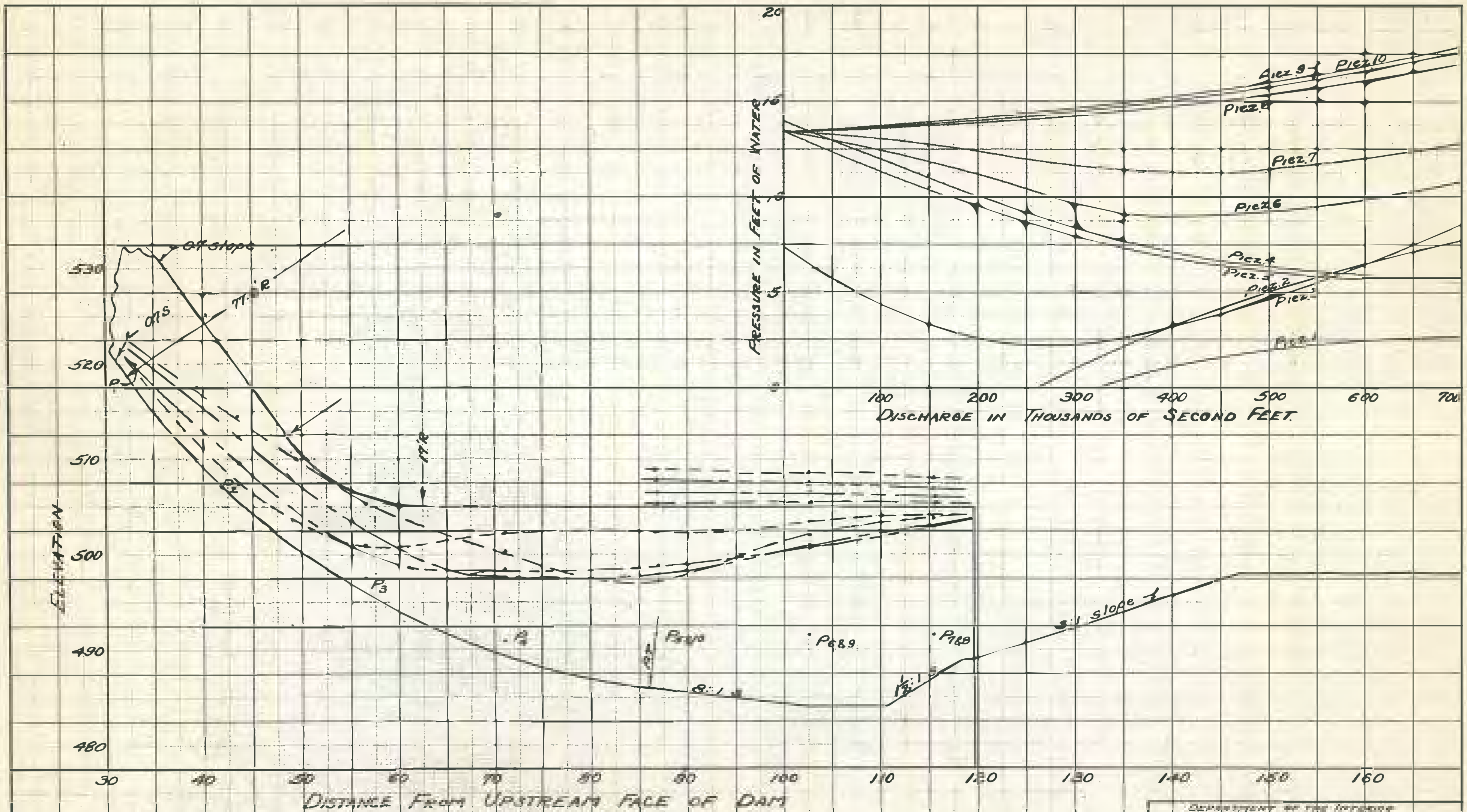
$$H_p = H_m N^{1.0} \quad \text{where } N = 36.$$

The prototype pressures (Table 2) are plotted on Figure 25.

The pressure exerted on the static side was due to the tailwater elevation while that on the kinetic side depends upon the formation of the jump. It is obvious (Figure 25), that as the discharge increases the jump moves toward the end of the apron causing a greater differential pressure. The safety of the wall would be increased in the final design as the diffuser sill holds more water in the pool which increases the pressure on the kinetic side and decreases the differential pressure.

C. Piers

During the entire set of tests on the action of the stilling pool an undesirable condition of flow occurred downstream from the spillway piers.



LEGEND

• Pressure on static side of wall (Gate Closed) Piez 8, 9 & 10.
• Pressure on jump " " " (Gate Open) " 1, 2, 3, 4, 5, 6, & 7

– – – – – –	TEST No 66, Run No 1,	Discharge of 150,000 c.f.s.	Reservoir Elev. 555	Tailwater Elev.	505.7
—+—–—+—	" " " " 2,	" " 350,000 "	" "	"	507.7
—+—–—+—	" " " " 3,	" " 500,000 "	" 555.4 "	"	509.7
—— — ——	" " " " 4,	" " 640,000 "	" 555.1 "	"	511.7

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WHEELER DAM
SPILLWAY STUDIES
INTERMEDIATE TRAINING WALL

DRAWN J.W.B. SUBMITTED
TRACED J.W.B. RECOMMENDED
CHECKED ... APPROVED J.L. Savage
... 232-D-821

FIGURE 25

TABLE NO. 2.

SUMMARY OF DATA AND TRANSFERENCE OF RESULTS - WHEELER DAM.

Intermediate Training Wall Pressures.

Piezometer Number	Pressures in Feet of Water: Run No. 1 Q=150,000		Run No. 2 Q=350,000		Run No. 3 Q=500,000		Run No. 4 Q=650,000		Piezometer Elevation
	Model	Proto.	Model	Proto.	Model	Proto.	Model	Proto.	
1	0.0016	0.06	0.0114	0.41	0.0538	1.94	0.0720	2.59	519.46
2	.0012	0.04	.0620	2.23	.1472	5.30	.2004	7.21	507.51
3	.0924	3.33	.0682	2.46	.1328	4.78	.2090	7.52	497.79
4	.3104	11.17	.2086	7.51	.1864	6.71	.2566	9.24	491.02
5	.2946	10.61	.2038	7.34	.1752	6.31	.1556	5.60	491.67
6	.3268	11.76	.2512	9.04	.2580	9.29	.2866	10.32	491.60
7	.3520	12.67	.3164	11.39	.3152	11.35	.3448	12.41	491.56
8	.3832	13.80	.4032	14.51	.4238	15.26	.4602	16.57	491.60
9	.3812	13.72	.4360	14.67	.4408	15.87	.4740	17.06	491.63
10	.3800	13.68	.4138	14.90	.4492	16.17	.4798	17.27	491.70

The original design on the piers had a flat face on the downstream side (Figure 9, Test 78). The streams from the adjacent gates came abruptly together behind the piers and formed a fin shaped jet near the P.C. of the bucket (Plate 33, A and B). This condition was intensified by the tendency of the streams to flatten as they flowed down the face of the dam and by the centrifugal action in the bucket. At the larger discharges, the fin-shaped stream continued across the apron above the sill and slightly eroded the river bed. The stream interfered with the formation of the jump on the apron and produced an intermittent boiling effect downstream from the sill (Plate 33 A). Visual tests indicated that the elimination of this fin would prevent these undesirable conditions. Several alterations were made and a satisfactory design was developed. The alterations were made on a single pier as this expedited testing and at the same time gave a comparison with the original design. In nearly all photographs the original and altered conditions are shown. In each case the pier to the left is the altered one.

The first alteration (Test 78, Figure 9) was the use of a 5.87-foot radius in the horizontal plane on each side of the sloping part of the pier nose. Considerable

A. TOP VIEW.

B. DOWNSTREAM VIEW.

ORIGINAL DESIGN - DISCHARGE 500,000 SECOND-FEET.

C. TOP VIEW.

D. DOWNSTREAM VIEW.

ALTERATION NO. 1 - DISCHARGE 500,000 SECOND-FEET.

ACTION AROUND DOWNSTREAM NOSE OF PIER.

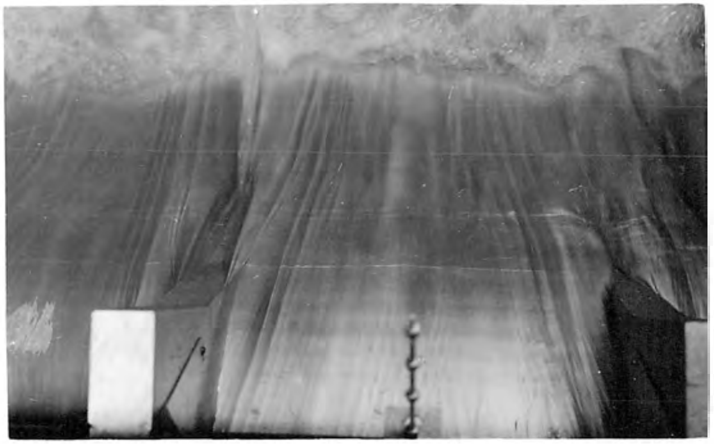
improvement was noted (Plate 33, C and D) and the size of the fin was materially reduced.

A second alteration (Test 78, Figure 9) was the use of a 36.5-foot radius in a plane perpendicular to a line through the downstream face of the pier at Elev. 545 and perpendicular to the surface of the spillway crest. The fin was noticeable only at high flows and all undesirable conditions were eliminated (Plate 34, A and B). The fin of water was lowered to such an extent that it was practically eliminated by the sill and no erosion occurred.

Alteration No. 3 (Test 78, Figure 9) with a 64.3-foot radius in the same plane as Alteration No. 2 produced no apparent improvement (Plate 34, C and D). An undesirable feature of this design was the undercutting of the pier which reduced the concrete under the gate hinge.

A fourth shape (Test 78, Figure 9), designed to prevent the undercutting of the pier, did not allow the streams to meet far enough upstream from the bucket and the results were not as desirable as in Alterations 2 and 3 (Plate 35). The added concrete also increased the cost of each pier.

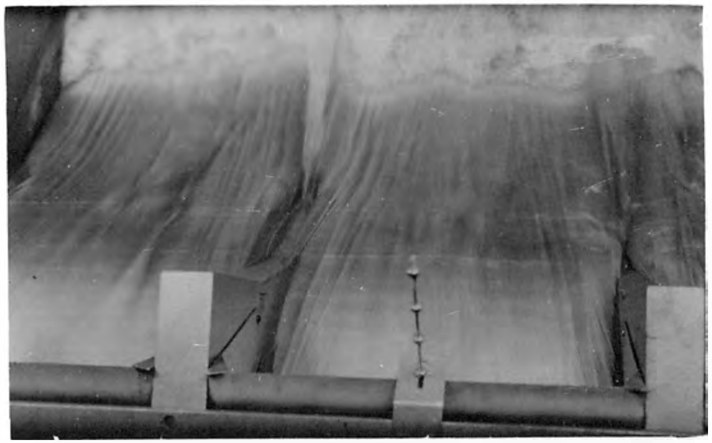
The improved hydraulic conditions and reduction in the amount of concrete resulted in the adoption of Alteration No. 2.



A. TOP VIEW

B. DOWNSTREAM VIEW.

ALTERATION NO. 2 - ADOPTED DESIGN - DISCHARGE 500,000 SECOND-FEET.

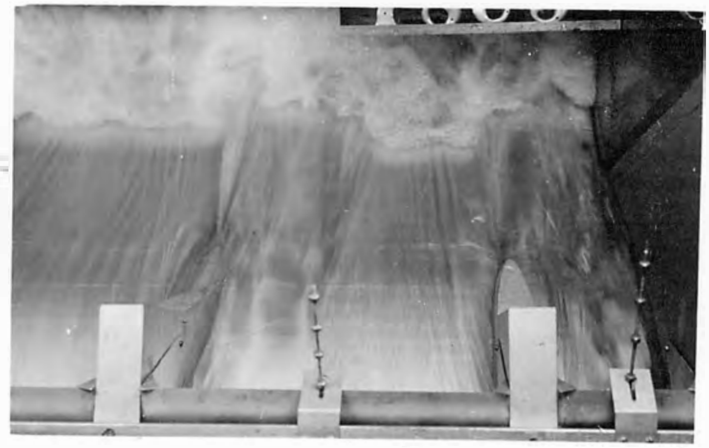


C. TOP VIEW.

D. DOWNSTREAM VIEW.

ALTERATION NO. 3 - DISCHARGE 500,000 SECOND-FEET.

ACTION AROUND DOWNSTREAM NOSE OF PIER.



A. TOP VIEW

B. DOWNSTREAM VIEW.

C. SIDE VIEW.

ACTION AROUND DOWNSTREAM NOSE OF PIER.
ALTERATION NO. 4 - DISCHARGE 500,000 SECOND-FEET.

D. Gate Operating Program for Wheeler Dam.

Assuming a flow condition such that only the power house is operating, in which case the flood gates would all be in a closed position, the reservoir would be at Elev. 555 and the tailwater surface would be at Elev. 505.1. Further, assuming that the gates are divided into sets of five by intermediate training walls, with the approach of an oncoming flood there would be a definite limit to which this set of five gates could be raised and a satisfactory hydraulic jump still form in the stilling pool below. To determine this limitation, tests were made on the model such that the elevation of the tailwater and the gate opening were determined at which the hydraulic jump would satisfactorily form.

With the reservoir of Wheeler Dam maintained at Elev. 555, or 14 feet above the crest, and the tailwater prevailing due to the headwater regulation at Wilson Dam, individual gates or sets of gates should not be completely raised for any length of time except in cases of extreme emergency.

The original plans of the Wheeler Dam provided for intermediate training walls between each set of five gates. As the inclusion of these walls was open to question in the design office, gate operating programs were studied for the conditions which would exist with or without them.

(1). <u>Gate Operating Program with Intermediate Training Walls.</u>

In the model studies of the final set-up, using the diffuser type sill, it was determined that a flood flow of 350,000 second-feet could be distributed equally through the entire sixty gates and a satisfactory jump would still form with a minimum tailwater at Elev. 505.1 which is the minimum regulated headwater at Wilson Dam. Thus one gate could be allowed to discharge 5,830 second-feet, or say 6,000 second-feet, and a set of five gates could discharge 30,000 second-feet and a satisfactory jump would be formed in the stilling pool. From the Tainter gate discharge diagram, a flow of 6,000 second-feet per gate will require an opening of 8.7 feet.

With this as a criterion, the following program is suggested in operating the Tainter gates providing the intermediate training walls are constructed: As a flood approaches, the first set of gates nearest the powerhouse should be raised to a maximum opening of 8.7 feet. With a further increase of flood flow, the adjoining set of gates can be opened a similar amount, and so on across the dam until the entire sixty gates are raised 8.7 feet which will provide a flood capacity of 360,000 second-feet. With a further increase in flood flow, all the gates can be opened completely as needed to maintain the reservoir at Elev. 555. The maximum capacity of 650,000 second-feet can be discharged with the gates completely open and the reservoir allowed to rise to Elev. 558.

(2). Gate Operating Program without Intermediate Training Wall.

A program of operation without the use of the training walls will be more complicated due to the cross-currents existing in the stilling pool and because the gate opening will be governed by these cross-currents rather than the discharge conditions at which the jump fails to form on the apron.

As previously mentioned, the model represented only a short section of the spillway and contained but 2.39 gates. With one gate closed and with the reservoir held constant at Elev. 555 and the tailwater at Elev. 505.1, the flow through the remaining gates was increased until the hydraulic conditions in the stilling pool became unbalanced such that undesirable conditions were apparent. At this point, the gages were read and the gate opening determined. The gate opening, which was found to be 3.42 feet, is the maximum amount the gates could be raised without obtaining unbalanced conditions which might needlessly endanger the structure. When the gates were raised 3.42 feet and the tailwater held at Elev. 505.1, a whirl formed on the apron downstream from the closed gate. Some erosion took place below the sill, and rock and sand were washed back on the apron. The abrasive action of this material while probably not serious would be undesirable in that it might wear the corners of the diffuser sill when it washed off the apron as the gate is opened. The flow through all sixty gates, raised 3.42 feet, was about 156,000 second-feet as determined from the discharge diagram.

To determine the amount of additional opening to provide further capacity one gate was set with an opening of 3.42 feet, the tailwater was held constant for 156,000 second-feet and the flow increased with the reservoir at Elev. 555. When undesirable conditions were again apparent, the gages were read as before. The gate opening was now 9.36 feet. The whirl on the apron was practically eliminated and no sand was apparent as when part of the gates were closed. However, with the low tailwater elevation some erosion took place immediately downstream from the sill and thus limited the amount of the second raise. The flow for all sixty gates raised 9.36 feet was 386,000 second-feet.

For the third setting, it was determined that the gates could be raised completely.

Assuming that the intermediate training walls are not constructed, the following program of gate operation is suggested: With the approach of the oncomming flood, the Tainter gates should be raised in sets of five to provide an opening of three feet. When all sixty gates are open that amount and additional capacity is required, the gates should be opened in sets of five until the opening is 8 feet. To further increase the discharge capacity, the gates can be opened completely. In the event of a small flood which would not require all the gates to be raised three feet, it is recommended that the raise be smaller and as many gates as feasible be used.

In either case the gates should be lowered in reverse order to the elevation from which they were last raised. This procedure should be followed during the receding of a flood until the initial opening has been reached, at which time each set of gates can be completely lowered as needed.

E. Tainter Gate Discharge Data.

(1) Discharge Coefficient.

When the experimental work on the model of the Wheeler Dam was first instigated, the urgency of obtaining immediate results led to the construction of what might be called a "rough" model, constructed with a wooden understructure, which, when subjected to moisture, swelled and became distorted in shape. Some idea of this distortion can be seen on Figure 26 where a comparison is made between the shape of the crest as designed and the shape as it was tested after being exposed to water.

For the sake of obtaining comparison of the discharge coefficient on such a model as compared to a carefully built model, calibration tests were made with free flow conditions (Table 3) on this so-called "rough" model.

Later as time permitted, and after a general type of scour protection had been adopted, a new model, hereafter referred to as the final model, was constructed of metal with the exception of the redwood crest and gate piers which were carefully protected by impregnating with hot linseed oil and aluminum paint. As in the case of the "rough" model, discharge measurements were made for the free flow conditions of the spillway gates on the final model (Table 4) and a comparison of results was made (Figure 27).

In the computation of the coefficient for both "rough" and final models (Tables 3 and 4) the head on the crest was corrected for velocity of approach to obtain comparable coefficient data not only between the two models but with similar data from outside sources.

The channel approach conditions in the final model were made to conform to those at the dam site by placing a false floor at Elev. 498 in the model flume so that the velocity of approach would be correct. Extensive tests were then made to obtain sufficient data with which to construct discharge coefficient curves for (1) free flow conditions with adjacent gates discharging, (2) free flow conditions with adjacent gates closed, (3) the Tainter gates with adjacent gates discharging, and (4) the Tainter gates with adjacent gates closed.

So far as is known, no experimental work has been done on a Tainter gate used in conjunction with an ogee crest. An effort was made to obtain sufficient data from the model such that a general equation for discharge through Tainter gates could be evolved. Several tests were made on the first model, which, although incomplete, have been filed with the complete data from the final model and will be used in the future as time permits in an attempt to derive a general equation. Study has already been done to some extent in this effort but the results have indicated that such an analysis will be somewhat difficult on account of the large number of variables present, such as the shape of the crest, the curvature of the gates, length of gates, position of gate hinges, gate opening and head on crest.

This apparently fruitless effort to derive a general equation did not prevent a satisfactory analysis of the observed data in its application to the particular case of the Wheeler Dam and the entire solution was approached in that light.

In the case of free flow conditions with adjacent gates open, the same data (Table 4) was used as in the comparison between the discharge coefficients on the "rough" model and the final model, except the data was not corrected for "velocity of approach" (Table 5). The effect on the coefficient of discharge (Figure 28) with the adjacent gates closed (Table 5-A) can be ascribed to the variation of the velocity of approach and the unsuppressed action with one gate only discharging.

The coefficient of discharge data (Tables 6 and 6-A) for the Tainter gates was acquired in a similar manner to that for free flow conditions. The results are more extensive due to the necessity of obtaining sufficient data to construct curves to provide for a variable reservoir surface and a variable gate opening. This was accomplished by setting the gates at a predetermined opening and measuring the flow of water required to give a certain reservoir elevation. These results were plotted (Figures 30 and 31) to obtain smooth curves so that interpolations could be made in the construction of the discharge diagram.

The characteristics of the coefficient of discharge, C_o, in the orifice formula

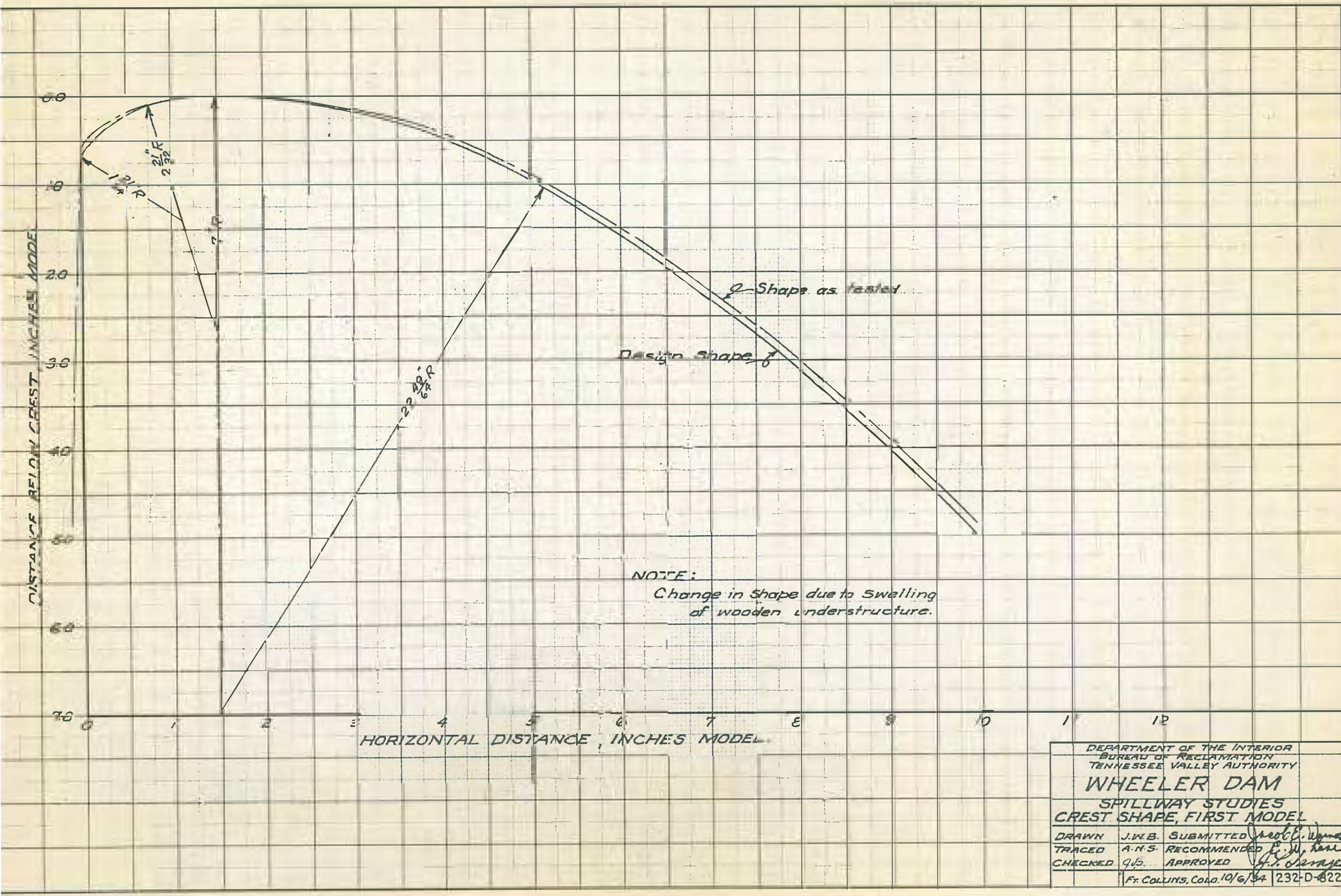
$$Q = 2/3\ C_oL\ (2g)^{\frac{1}{2}} \left(H^{3/2} - H_T^{3/2}\right)$$

are shown on Figure 29.

2. Discharge Diagram.

Tainter gate discharge diagrams for adjacent gates discharging (Figure 32) and adjacent gates closed (Figure 33) were constructed by using the coefficient of discharge curves for free flow conditions (Figure 28) and the Tainter gate discharge curves (Figures 30 and 31). In the case of the free flow conditions, the values of head and coefficient of discharge were read from the curves and the discharge quantity computed in Tables 7 and 7-A. The Tainter gate openings and discharge quantities for a given reservoir elevation, read directly from Figures 30 and 31, are recorded in Tables 8 and 8-A.

While the accuracy of the discharge diagram may be affected slightly by the relatively short section of spillway represented by the model, it is believed that the degree of accuracy is well within the limits of experimental error and as accurate as can be obtained by field tests on the prototype.



HORIZONTAL DISTANCE, INCHES MODEL
Shape as tested
Design Shape
$2\frac{21}{32}$" R
$22\frac{49}{64}$" R
NOTE:
Change in Shape due to Swelling
of wooden understructure.
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BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
CREST SHAPE, FIRST MODEL
DRAWN J.W.B. SUBMITTED
TRACED A.N.S. RECOMMENDED
CHECKED APPROVED
FT. COLLINS, COLO. 10/6/34 232-D-822

FIGURE 26

TABLE NO. 3.

SUMMARY OF DATA - WHEELER DAM (Model).

Discharge Coefficients for First Model.

1	2	3	4	5	6	7	8	9	10
Test and Run No.	Head on Crest h (ft.)	Area of Approach (sq.ft.)	Observed Quantity Q_o (c.f.s.)	Velocity in Flume (ft/sec.)	V^2	Velocity Head h_v (feet)	Total Head H (feet)	$H^{3/2}$	C
32-2	0.1139	6.1227	0.3255	0.0532	0.0028	-	0.1139	0.0385	3.176
3	.1849	6.3357	.7176	.1133	.0128	0.0002	.1851	.0797	3.378
4	.2572	6.5226	1.2279	.1883	.0355	.0006	.2578	.1309	3.519
5	.3078	6.7044	1.6445	.2453	.0602	.0009	.3087	.1715	3.598
6	.3623	6.8879	2.1646	.3152	.0994	.0015	.3638	.2194	3.701
7	.3924	6.9582	2.4701	.3550	.1260	.0020	.3944	.2477	3.741
8	.4433	7.1109	3.0245	.4253	.1809	.0028	.4461	.2980	3.808
9	.5157	7.3281	3.9000	.5322	.2832	.0044	.5201	.3751	3.901
10	.0724	5.9982	0.1569	.0262	.0007	-	.0724	.0195	3.019

TABLE NO. 4.
SUMMARY OF DATA - WHEELER DAM (Model).
Discharge Coefficients for Final Model.

1	2	3	4	5	6	7	8	9	10
Test and Run No.	Head on Crest h (ft.)	Area of Approach (sq.ft.)	Observed Quantity Q_o (c.f.s.)	Velocity in Flume (ft/sec.)	V^2	Velocity Head h_v (feet)	Total Head H (feet)	$H^{3/2}$	C
76-43	0.4598	4.9644	3.3656	0.6779	0.4595	0.0071	0.4669	0.3190	3.973
44	.4552	4.9506	3.3076	.6681	.4464	.0069	.4621	.3141	3.965
45	.3840	4.7370	2.4665	.5207	.2711	.0042	.3882	.2419	3.839
46	.3659	4.6827	2.2717	.4851	.2353	.0037	.3696	.2247	3.806
47	.3423	4.6119	2.0266	.4394	.1931	.0030	.3453	.2029	3.761
48	.3158	4.5324	1.7714	.3908	.1527	.0024	.3182	.1795	3.716
49	.2897	4.4541	1.5248	.3423	.1172	.0018	.2915	.1574	3.648
50	.2553	4.3509	1.2505	.2874	.0826	.0013	.2566	.1300	3.621
51	.2248	4.2594	1.0157	.2385	.0569	.0009	.2257	.1072	3.568
52	.1885	4.1505	0.7534	.1815	.0329	.0005	.1890	.0822	3.451
53	.1432	4.0146	0.4827	.1227	.0151	.0002	.1434	.0543	3.347
54	.0925	3.8625	0.2365	.0612	.0038	.0001	.0926	.0281	3.170
55	.3936	4.7658	2.5525	.5356	.2869	.0045	.3981	.2512	3.826
56	.1182	3.9396	0.3497	.0888	.0079	.0002	.1184	.0407	3.235
57	.0873	3.8469	0.2123	.0552	.0030	.0001	.0874	.0259	3.086
58	.4075	4.8075	2.6990	.5614	.3152	.0049	.4124	.2649	3.837
59	.4382	4.8996	3.0704	.6267	.3928	.0061	.4443	.2962	3.903
60	.4242	4.8576	2.9045	.5979	.3575	.0056	.4298	.2818	3.881
61	.4564	4.9542	3.3049	.6671	.4450	.0069	.4633	.3153	3.947
62	.3350	4.5900	1.9782	.4310	.1858	.0029	.3379	.1964	3.793
63	.2227	4.2531	1.0058	.2365	.0559	.0009	.2236	.1057	3.583
64	.1666	4.0848	0.6289	.1540	.0237	.0004	.1670	.0682	3.473
65	.1112	3.9186	0.3229	.0824	.0068	.0001	.1113	.0372	3.268

EXPLANATION OF TABLES 3 AND 4

Column 1 - Test and Run Number. Each test and each run of the test was given a number which was chronological in nature, except in cases where it was necessary to make re-runs.

Column 2 - Head on Crest, feet. The elevation of the water surface in the reservoir was measured by a float ~~gate~~ gage connected to an opening in the center of the flume, three feet upstream from the model. The reading of the gage less a correction factor for the crest elevation gave the net head on the crest. The correction factor applied was:

Test 32, Runs 2 - 10, inclusive	0.7119
Test 76, Runs 43- 65, inclusive	0.7138

Column 3 - Area of Approach to the Model, square-feet. The area through which the water approached the model was obtained by multiplying the width of the flume at the gage by the depth of water in the flume. The depth of water depending upon the head over the crest and the distance from the crest to the floor of the flume.

Column 4 - Observed Quantity, c.f.s. The flow into the model was measured by a 2-foot Cipolletti weir, the formula for which is $Q = CLH^{3/2}$, where C is a variable and depends upon the head on the weir. The value of C had previously been determined by calibration tests. L, in this case, is equal to 2.

The head on the weir was measured by a float gage and a hook gage and the results from the two gages were averaged after a correction factor for each was applied. The correction factors were:

	Hook	Float
Test No. 32 - Runs 2-10, inclusive	1.0546	7.7979
Test No. 76 - Runs 43-65 "	1.0586	7.7970

Column 5 - Velocity in Flume, feet per second. The velocity of approach was determined by dividing the observed quantity, Column 4, by the area of approach, Column 3. From the formula $Q = AV$, $V = Q/A$.

Column 6 - The square of the velocity. Obtained by squaring the results of Column 5.

Column 7 - Velocity Head, feet. The velocity head is found by substituting the value of V found in Column 5 in the formula $h_v = V^2/2g$ or dividing Column 6 by the value of 2g which is 64.29 for the Fort Collins laboratory.

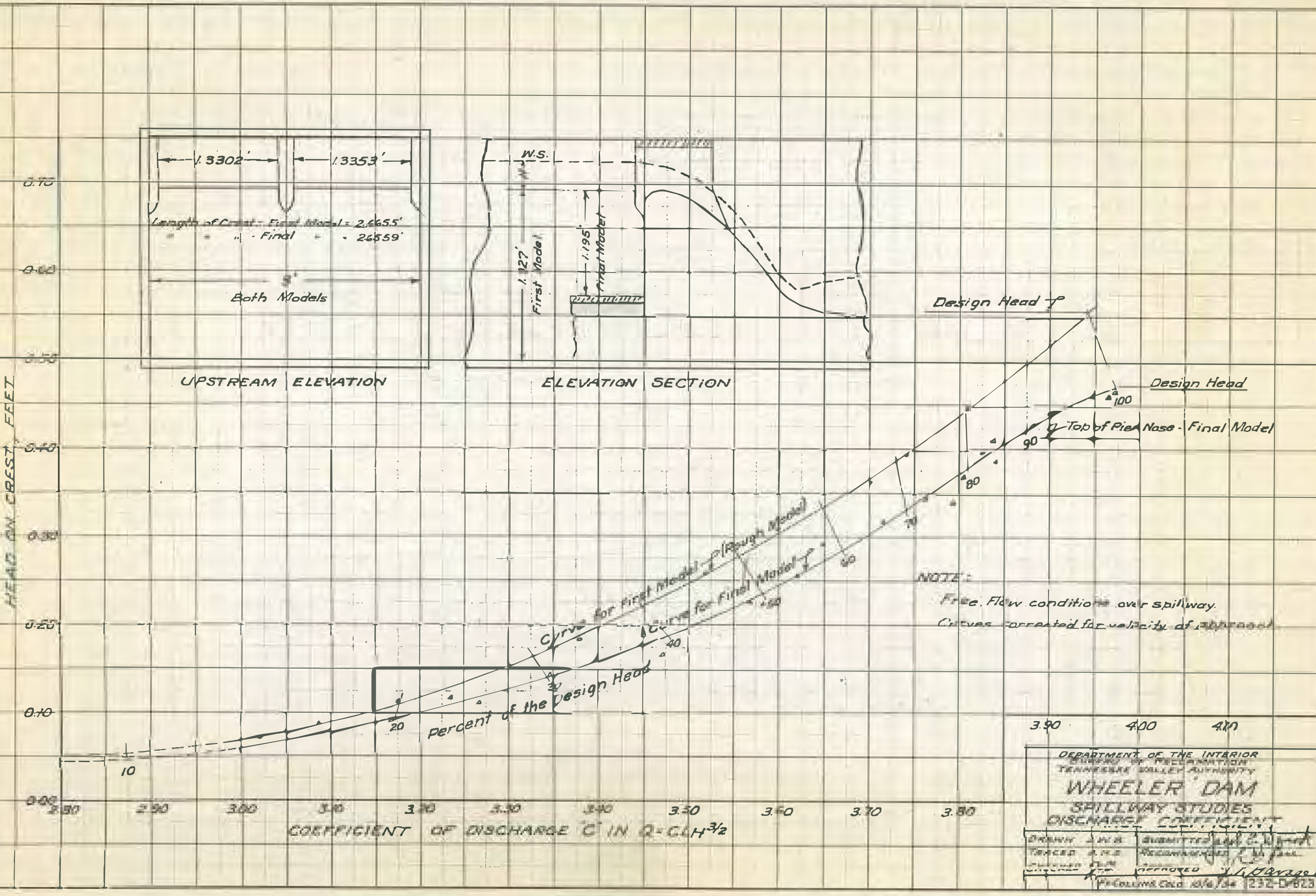
Column 8 - Total Head on Crest, feet. The total head, H, was found by adding the measured head, h, Column 2, and the computed velocity head, h_v, Column 7.

Column 9 - $H^{3/2}$. The total head, H, was raised to the three-halves power in order to use it in the general weir formula in solving for the coefficient of discharge.

Column 10 - Discharge Coefficient, C. The general weir formula $Q = CLH^{3/2}$ was used in obtaining the coefficient of discharge. It was obtained by dividing the observed quantity, Column 4, by Column 9 times the length of crest, L. The length of crest was:

Test No. 32 - Runs 2 - 10, inclusive, 2.6655 feet

Test No. 76 - Runs 43- 65, inclusive, 2.6559 feet



1.3302'
1.3353'
Length of Crest - First Model = 2.6655'
" " " - Final " = 2.6559'
8'
Both Models
UPSTREAM | ELEVATION
W.S.
1.327' First Model
1.195' Final Model
ELEVATION | SECTION
Design Head
Design Head
Top of Pier Nose - Final Model
Curve for First Model (Rough Model)
Curve for Final Model
Percent of the Design Head
NOTE:
Free Flow conditions over spillway.
Curves corrected for velocity of approach.
HEAD ON CREST, FEET
COEFFICIENT OF DISCHARGE "C" IN Q=CLH^3/2
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
2.80
2.90
3.00
3.10
3.20
3.30
3.40
3.50
3.60
3.70
3.80
3.90
4.00
4.10
10
20
30
40
50
60
70
80
90
100
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WHEELER DAM
SPILLWAY STUDIES
DISCHARGE COEFFICIENT
APPROVED
232-D-823

FIGURE 27

TABLE NO. 5.

SUMMARY OF DATA - WHEELER DAM (Model)

Discharge Coefficients - Free Flow Conditions - Adjacent Gates Discharging.

1	2	3	4	5	6	7
Test and Run No.	Head on Crest H_M(Feet)	$H^{3/2}$	Head on Crest H_P(Feet)	$LH^{3/2}$	Model Q c.f.s.	C
76-43	0.4598	0.3118	16.553	0.8281	3.3656	4.0613
44	.4552	.3071	16.387	.8156	3.3076	4.0554
45	.3840	.2380	13.824	.6321	2.4665	3.9021
46	.3659	.2213	13.172	.5878	2.2717	3.8647
47	.3423	.2003	12.323	.5320	2.0266	3.8094
48	.3158	.1774	11.369	.4712	1.7714	3.7593
49	.2897	.1560	10.429	.4143	1.5248	3.6804
50	.2553	.1290	9.191	.3426	1.2505	3.6500
51	.2248	.1066	8.093	.2831	1.0157	3.5878
52	.1885	.0819	6.786	.2175	0.7534	3.4639
53	.1432	.0542	5.155	.1439	0.4827	3.3544
54	.0925	.0282	3.330	.0749	0.2365	3.1576
55	.3936	.2469	14.170	.6557	2.5525	3.8928
56	.1182	.0406	4.255	.1078	0.3497	3.2440
57	.0873	.0258	3.143	.0685	0.2123	3.0993
58	.4075	.2602	14.670	.6911	2.6990	3.9054
59	.4382	.2901	15.775	.7705	3.0704	3.9850
60	.4242	.2763	15.271	.7338	2.9045	3.9582
61	.4564	.3083	16.430	.8188	3.3049	4.0363
62	.3350	.1939	12.060	.5150	1.9782	3.8412
63	.2227	.1051	8.017	.2791	1.0058	3.6037
64	.1666	.0680	5.998	.1806	0.6289	3.4823
65	.1112	.0371	4.003	.0985	0.3229	3.2782

TABLE NO. 5A - Adjacent Gates Closed.

1	2	3	4	5	6	7
Test and Run No.	Head on Crest H_M(Feet)	$H^{3/2}$	Head on Crest H_P(Feet)	$LH^{3/2}$	Model Q c.f.s.	C
77-1	0.3897	0.2433	14.029	0.2702	1.0051	3.7198
2	.3615	.2174	13.014	.2414	0.8939	3.7030
3	.3388	.1972	12.197	.2190	0.8040	3.6712
4	.3132	.1753	11.275	.1947	0.7142	3.6682
5	.2866	.1535	10.318	.1705	0.6169	3.6182
6	.2572	.1305	9.259	.1449	0.5195	3.5852
7	.2332	.1126	8.395	.1250	0.4419	3.5352
8	.2043	.0923	7.355	.1025	0.3577	3.4898
9	.1802	.0765	6.487	.0849	0.2875	3.3863
10	.1522	.0594	5.479	.0660	0.2190	3.3182
11*	.1372	.0508	4.839	.0564	0.1882	3.3369
12*	.1713	.0709	6.167	.0787	0.2690	3.4181
13*	.2434	.1201	8.762	.1333	0.4747	3.5611
14*	.3411	.1992	12.280	.2211	0.8070	3.6499

* 90° V-Notch Weir Used.

EXPLANATION OF TABLES 5 AND 5-A.

Column 1 - Test and Run Number. Each test and each run of the test was given a number which was chronological in nature, except in cases where it was necessary to make re-runs.

Column 2 - Head on Crest, H_M, feet. The elevation of the water surface in the reservoir was measured by a float gage connected to an opening in the center of the flume, 3 feet upstream from the model. The reading of the gage less a correction factor for the crest elevation gave the net head on the crest.

Column 3 - $H^{3/2}$. The head on the crest, Column 2, was raised to the three-halves power to determine the discharge coefficient.

Column 4 - Head on Crest, H_P, feet. The prototype head was obtained by multiplying the head on the model, column 2, by the scale ratio.

Column 5 - $(LH^{3/2})$. $H^{3/2}$ is multiplied by L, the net length of the crest, for substitution in the weir formula, $Q = CLH^{3/2}$.

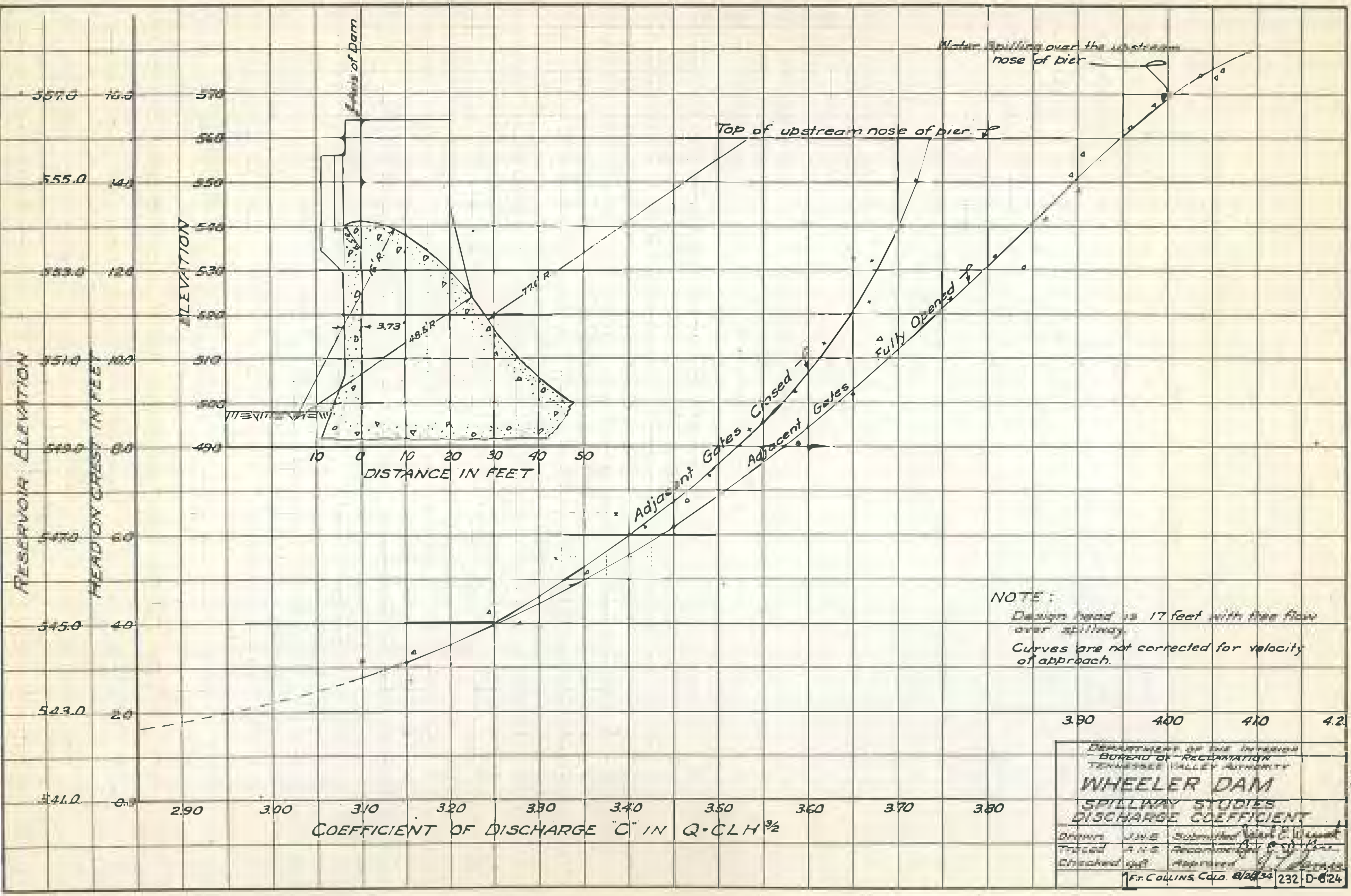
Column 6 - Observed Quantity, Q, in second-feet. The flow into the model was measured by a 2-foot Cipolletti or a 90-degree V-notch weir. The formula for the 2-foot Cipolletti weir is $Q = CLH^{3/2}$ where C is a variable and depends upon the head on the weir. The value of C had previously been determined by calibration tests. The value of L in this case was 2 feet.

The formula for the 90-degree V-notch weir is $Q = 2.49\ H^{2.48}$ and was determined by previous calibration tests.

The head on the weir was measured by a float gage and a hook gage and the results from the two gages were averaged after a correction factor for each gage had been applied.

Column 7 - Coefficient of Discharge, C. The basic expression for flow over a weir is $Q = 2/3\ CL(2g)^{\frac{1}{2}}\ H^{3/2}$, which, when the constants are combined reduces to $Q = CLH^{3/2}$. The value of C, because it contains g, is affected by locality and elevation above sea level which should be considered when the value of g for the model and prototype is different. By using the formula $g = 32.1721 - 0.08211 \cos 2a - 0.000003$*, the value of g for the Fort Collins laboratory was found to be 32.1445 and for the Wheeler Dam 32.1420, thus these values may be considered the same for all practical purposes, and the coefficient on the model and the prototype are the same. The coefficient curve for the prototype may be constructed by plotting C directly against the prototype head, Column 4.

* Mark's Engineering Handbook, Page 196 (Second Edition).


Water spilling over the upstream nose of pier
Top of upstream nose of pier
Axis of Dam
ELEVATION
DISTANCE IN FEET
77.0' R
48.5' R
3.73'
Adjacent Gates Closed
Adjacent Gates
Fully Opened
RESERVOIR ELEVATION
HEAD ON CREST IN FEET
COEFFICIENT OF DISCHARGE "C" IN Q=CLH^3/2
2.90
3.00
3.10
3.20
3.30
3.40
3.50
3.60
3.70
3.80
3.90
4.00
4.10
NOTE:
Design head is 17 feet with free flow over spillway.
Curves are not corrected for velocity of approach.
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TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
DISCHARGE COEFFICIENT
Drawn
Traced
Checked
Submitted
Recommended
Approved
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232
D-6.24

FIGURE 28

TABLE NO. 6.

SUMMARY OF DATA AND TRANSFERENCE OF RESULTS - WHEELER TAINTER GATES.

Adjacent Gates Discharging.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Test and Run No.	Head on Crest H in Feet		Reservoir Elevation	$H^{3/2}$		Gate Raised Amount (Feet) d		Head on Top of Orifice H_T (Feet)		$H_T^{3/2}$		$H^{3/2}-H_T^{3/2}$		Observed Quantity Q_o c.f.s.		Quantity per Gate c.f.s.	Orifice Coefficient C_o	
	Model	Proto.	Prototype	Model	Proto	Model	Proto	Model	Proto	Model	Proto	Model	Prototype	Model	Proto	Prototype	Model	Prototype
6-1	0.3881	13.972	554.97	0.2418	52.222	0.0279	1.004	0.3602	12.968	0.2162	46.700	0.0256	5.522	0.2547	1,980.5	828.6	0.6992	0.7017
2	.3872	13.939	554.94	.2410	52.044	.0559	2.012	.3313	11.927	.1907	41.192	.0503	10.852	0.4873	3,789.2	1,585.2	.6819	.6832
3	.3863	13.907	554.91	.2401	51.862	.0829	2.984	.3034	10.923	.1671	36.105	.0730	15.757	0.7074	5,500.7	2,301.3	.6822	.6830
4	.3892	14.011	555.01	.2428	52.446	.1109	3.992	.2783	10.019	.1468	31.715	.0960	20.731	0.9318	7,245.7	3,031.3	.6833	.6839
5	.3882	13.975	554.98	.2419	52.240	.1389	5.000	.2493	8.975	.1245	26.890	.1174	25.350	1.1436	8,892.6	3,720.3	.6864	.6864
6	.3892	14.011	555.01	.2428	52.446	.1669	6.008	.2223	8.003	.1048	22.642	.1380	29.804	1.3481	10,482.8	4,385.6	.6884	.6882
7	.3880	13.968	554.97	.2417	52.200	.1949	7.016	.1931	6.952	.0849	18.328	.1568	33.872	1.5477	12,034.9	5,034.9	.6952	.6952
8	.3888	13.997	555.00	.2424	52.365	.2219	7.988	.1669	6.009	.0681	14.727	.1743	37.638	1.7243	13,408.2	5,609.4	.6971	.6967
9	.3894	14.018	555.02	.2430	52.488	.2499	8.996	.1395	5.022	.0521	11.256	.1909	41.232	1.9080	14,836.6	6,207.0	.7040	.7037
10	.3883	13.979	554.98	.2420	52.264	.2779	10.004	.1104	3.975	.0367	7.930	.2053	44.334	2.0962	16,300.1	6,819.3	.7194	.7194
11*	.3862	13.903	554.90	.2400	51.838	.3059	11.012	.0803	2.891	.0228	4.916	.2172	46.922	2.4287	18,885.6	7,900.9	.7878	.7875
12*	.3883	13.979	554.98	.2420	52.264	.3059	11.012	.0824	2.967	.0237	5.111	.2183	47.153	2.4512	19,060.5	7,974.1	.7911	.7909
13	.3882	13.975	554.98	.2419	52.240	.0284	1.022	.3598	12.953	.2158	46.618	.0261	5.622	0.2613	2,031.9	850.1	.7050	.7072
14	.3358	12.089	553.09	.1946	42.035	.0284	1.022	.3074	11.067	.1704	36.815	.0242	5.220	0.2416	1,878.7	786.0	.7025	.7042
15	.2789	10.040	551.04	.1473	31.810	.0284	1.022	.2505	9.018	.1254	27.080	.0219	4.730	0.2192	1,704.5	713.1	.7032	.7051
16	.2216	7.978	548.98	.1043	22.532	.0284	1.022	.1932	6.956	.0849	18.344	.0194	4.188	0.1938	1,507.0	630.5	.7062	.7032
17	.1655	5.958	546.96	.0673	14.542	.0284	1.022	.1371	4.936	.0508	10.968	.0165	3.574	0.1662	1,292.4	540.7	.7091	.7075
18	.1125	4.050	545.05	.0378	8.150	.0284	1.022	.0841	3.028	.0244	5.269	.0134	2.881	0.1336	1,038.9	434.6	.7015	.7056
19	.3882	13.975	554.98	.2419	52.240	.0557	2.005	.3325	11.970	.1918	41.410	.0501	10.830	0.4928	3,832.0	1,603.1	.6926	.6923
20	.3342	12.031	553.03	.1932	41.726	.0557	2.005	.2785	10.026	.1470	31.750	.0462	9.976	0.4486	3,488.3	1,459.4	.6840	.6842
21	.2767	9.961	550.96	.1456	31.435	.0557	2.005	.2210	7.956	.1039	22.444	.0417	8.991	0.4022	3,127.5	1,308.4	.6803	.6806
22	.2216	7.978	548.98	.1043	22.532	.0557	2.005	.1659	5.973	.0675	14.599	.0368	7.933	0.3504	2,724.7	1,139.9	.6712	.6720
23	.1662	5.983	546.98	.0677	14.632	.0557	2.005	.1105	3.978	.0368	7.934	.0309	6.698	0.2928	2,276.8	952.5	.6667	.6651
24	.1109	3.993	544.99	.0370	7.979	.0557	2.005	.0552	1.987	.0130	2.801	.0240	5.178	0.2203	1,713.1	716.7	.6458	.6470
25*	.0710	2.556	543.56	.0189	4.086	.0557	2.005	.0153	0.551	.0019	0.409	.0170	3.677	0.1533	1,192.1	498.7	.6353	.6343
26	.3883	13.979	554.98	.2420	52.264	.1116	4.018	.2767	9.961	.1456	31.435	.0964	20.829	0.9278	7,214.6	3,018.3	.6784	.6777
27	.3318	11.945	552.94	.1911	41.285	.1116	4.018	.2202	7.927	.1033	22.318	.0878	18.967	0.8467	6,583.9	2,754.4	.6788	.6792
28	.2784	10.022	551.02	.1469	31.710	.1116	4.018	.1668	6.004	.0681	14.712	.0788	16.998	0.7550	5,870.9	2,456.1	.6751	.6758
29	.2226	8.014	549.01	.1050	22.686	.1116	4.018	.1110	3.996	.0370	7.988	.0680	14.698	0.6469	5,030.3	2,104.5	.6706	.6697
30	.1665	5.994	546.99	.0679	14.676	.1116	4.018	.0549	1.976	.0129	2.778	.0550	11.898	0.5201	4.044.3	1,692.0	.6655	.6651
31*	.1375	4.950	545.95	.0510	11.010	.1116	4.018	.0259	0.932	.0042	0.900	.0468	10.110	0.4547	3,535.7	1,479.2	.6838	.6843
32	.3895	14.022	555.03	.2431	52.510	.1666	5.998	.2229	8.024	.1052	22.726	.1379	29.784	1.3373	10,398.8	4,350.4	.6831	.6831
33	.3337	12.013	553.02	.1928	41.635	.1666	5.998	.1671	6.015	.0683	14.750	.1245	26.885	1.1984	9,318.8	3,898.6	.6779	.6782
34	.2782	10.015	551.02	.1468	31.695	.1666	5.998	.1116	4.017	.0373	8.051	.1095	23.644	1.0550	8,203.7	3,432.1	.6785	.6789
35	.2232	8.035	549.04	.1054	22.775	.1666	5.998	.0566	2.037	.0134	2.907	.0920	19.868	0.8939	6,951.0	2,908.0	.6848	.6845
36*	.2074	7.466	548.47	.0945	20.402	.1666	5.998	.0408	1.468	.0082	1.779	.0863	18.623	0.8746	6,800.9	2,845.2	.7138	.7145
37	.3895	14.022	555.03	.2431	52.510	.2226	8.014	.1669	6.008	.0681	14.724	.1750	37.786	1.7189	13,366.2	5,591.9	.6920	.6921
38	.3341	12.028	553.03	.1931	41.710	.2226	8.014	.1115	4.014	.0373	8.042	.1558	33.668	1.5430	11,998.4	5,019.6	.6977	.6973
39*	.2763	9.947	550.95	.1452	31.368	.2226	8.014	.0537	1.933	.0124	2.687	.1328	28.681	1.4163	11,013.1	4,607.4	.7515	.7513
40	.3919	14.108	555.11	.2453	52.990	.2776	9.994	.1143	4.114	.0387	8.344	.2066	44.646	2.1022	16,346.7	6,838.8	.7168	.7164
41	.3699	13.316	554.31	.2250	48.590	.2776	9.994	.0923	3.322	.0281	6.055	.1969	42.535	2.0402	15,864.6	6,637.1	.7298	.7298
42*	.3440	12.384	553.38	.2018	43.580	.2776	9.994	.0664	2.390	.0171	3.695	.1847	39.885	2.0306	15,789.9	6,605.8	.7742	.7746
62*	.3350	12.060	553.06	.1939	41.880	.2709	9.752	.0641	2.308	.0162	3.506	.1777	38.374	1.9782	15,382.5	6,435.4	.7839	.7843
63*	.2227	8.017	549.02	.1051	22.698	.1809	6.512	.0418	1.505	.0085	1.847	.0966	20.851	1.0058	7,821.1	3,272.0	.7329	.7339
64*	.1666	5.998	547.00	.0680	14.692	.1336	4.810	.0330	1.188	.0060	1.295	.0620	13.397	0.6289	4,890.3	2,045.9	.7145	.7142
65*	.1112	4.003	545.00	.0371	8.009	.0889	3.200	.0223	0.803	.0034	0.720	.0337	7.289	0.3229	2,510.8	1,050.5	.6736	.6740

*Free Flow Runs.

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TABLE NO. 6 A

SUMMARY OF DATA AND TRANSFERENCE OF RESULTS - WHEELER TAINTER GATES.

Adjacent Gates Closed.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Test and Run No.	Head on Crest H in Feet		Reservoir Elevation	$H^{3/2}$		Gate Raised Amount (Feet) d		Head on Top of Orifice H_T (Feet)		$H_T^{3/2}$		$H^{3/2}-H_T^{3/2}$		Observed Quantity Q_o c.f.s.		Quantity per Gate c.f.s.	Orifice Coefficient C_o	
	Model	Proto	Prototype	Model	Proto	Model	Proto	Model	Proto	Model	Proto	Model	Prototype	Model	Proto	Prototype	Model	Prototype
79-1	0.3885	13.986	554.98	0.2422	52.306	0.0278	1.001	0.3607	12.985	0.2166	46.790	0.0256	5.516	0.1101	856.1	856.6	0.7246	0.7264
2	.2780	10.008	551.01	.1466	31.660	.0278	1.001	.2502	9.007	.1252	27.028	.0214	4.632	.0935	727.1	727.6	.7360	.7347
3	.1656	5.962	546.96	.0674	14.558	.0278	1.001	.1378	4.961	.0512	11.053	.0162	3.505	.0707	549.8	550.2	.7352	.7341
4	.1111	4.000	545.00	.0371	8.000	.0278	1.001	.0833	2.999	.0240	5.193	.0131	2.807	.0568	441.7	442.0	.7305	.7364
5	.0259	0.9324	541.93	.0042	0.901	.0278	1.001	- .0019	- 0.068	.0000	0.000	.0042	0.901	.0156	121.3	121.4	.6262	.6304
6	.3879	13.964	554.96	.2416	52.180	.0278	1.001	.3601	12.963	.2161	46.675	.0255	5.505	.1106	860.0	860.6	.7310	.7312
7	.3869	13.928	554.93	.2407	51.978	.0833	2.999	.3036	10.929	.1673	36.135	.0734	15.843	.3056	2,376.3	2,377.8	.7015	.7020
8	.2780	10.008	551.01	.1466	31.660	.0833	2.999	.1947	7.009	.0859	18.556	.0607	13.104	.2524	1,962.7	1,962.4	.7007	.7004
9	.1650	5.940	546.94	.0670	14.480	.0833	2.999	.0817	2.941	.0234	5.044	.0436	9.436	.1801	1,400.5	1,401.4	.6961	.6946
10	.1030	3.708	544.71	.0331	7.140	.0833	2.999	.0197	0.709	.0027	0.597	.0304	6.543	.1269	986.8	987.4	.7033	.7058
11	.1088	3.917	544.92	.0359	7.752	.0833	2.999	.0255	0.918	.0041	0.880	.0318	6.872	.1326	1,031.1	1,031.8	.7025	.7023
12	.3876	13.954	554.95	.2413	52.124	.1389	4.997	.2487	8.957	.1241	26.808	.1172	25.316	.4857	3,776.8	3,778.4	.6983	.6981
13	.2754	9.914	550.91	.1445	31.216	.1389	4.997	.1365	4.917	.0505	10.901	.0940	20.315	.3837	2,983.7	2,985.6	.6878	.6874
14	.1742	6.271	547.27	.0727	15.704	.1389	4.997	.0353	1.274	.0066	1.438	.0661	14.266	.2818	2,191.3	2,192.7	.7183	.7188
15	.3873	13.943	554.94	.2411	52.065	.1944	6.998	.1929	6.945	.0847	18.300	.1564	33.765	.6449	5,014.7	5,017.9	.6948	.6951
15C	.3893	14.015	555.01	.2429	52.470	.1944	6.998	.1949	7.017	.0860	18.588	.1569	33.882	.6488	5,045.1	5,048.3	.6966	.6969
16	.2776	9.994	550.99	.1463	31.596	.1944	6.998	.0832	2.996	.0240	5.186	.1223	26.410	.5104	3,968.9	3,972.2	.7032	.7034
17	.2448	8.813	549.81	.1211	26.162	.1944	6.998	.0504	1.815	.0113	2.445	.1098	23.717	.4772	3,710.7	3,713.9	.7323	.7324
18	.3873	13.943	554.94	.2411	52.065	.2500	9.000	.1373	4.943	.0509	10.989	.1902	41.076	.7998	6,219.2	6,223.2	.7087	.7086
19	.3204	11.534	552.53	.1814	39.170	.2500	9.000	.0704	2.534	.0187	4.034	.1627	35.136	.7217	5,611.9	5.615.8	.7474	.7475
20	.3883	13.979	554.98	.2420	52.264	.3051	10.984	.0832	2.995	.0240	5.183	.2180	47.081	.9907	7,703.7	7,708.6	.7656	.7658
21	.3835	13.806	554.81	.2375	51.296	.3051	10.984	.0784	2.822	.0220	4.741	.2155	46.555	.9842	7,653.1	7,658.0	.7696	.7693
22	.3463	12.467	553.47	.2038	44.022	.2500	9.000	.0963	3.467	.0299	6.456	.1739	37.566	.7452	5,794.7	5,798.4	.7223	.7219

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EXPLANATIONS OF TABLES 6 AND 6-A.

Column 1 - Test and Run Number. Same as Column 1, Table 5.

Column 2 - Head on Crest, Model, feet. - See explanation of Column 2, Table 5.

Column 3 - Head on Crest, Prototype, feet - See explanation of Column 4, Table 5.

Column 4 - Reservoir Elevation, Prototype, feet. The reservour elevation was found by adding the prototype head on the crest, Column 3, to the crest Elevation (541.0).

Columns 5 and 6 - $H^{3/2}$ - See explanation of Column 3, Table 5.

Column 7 - Amount of Gate Opening, feet. To obtain calibration data it was necessary to measure the gate opening very accurately. This was done by a bar gage (Plate 1). The gage was placed near the gate and a reading taken with the bar on the crest. It was then raised the predetermined amount less the thickness of the bar with the lip of the gate lowered until it contacted the bar.

Column 8 - Amount Gate Raised, Prototype, feet. The amount was found by multiplying Column 7 by the scale ratio, 36.

Column 9 - Head on Top of Orifice, Model, feet. This value was found by subtracting the amount the gate was raised (Column 7) from the head on the crest (Column 2).

Column 10 - Head on Top of Orifice, Prototype, feet. This value was found by subtracting the amount the gate was raised (Column 8) from the head on the crest (Column 3).

Columns 11 and 12 - $H_T^{3/2}$, Model and Prototype. Same as Columns 5 and 6.

Columns 13 and 14 - $(H^{3/2} - H_T^{3/2})$, Model and Prototype. Obtained for use in the precise orifice formula by subtracting Column 11 from Column 5 and Column 12 from Column 6.

Column 15 - Quantity, Model, second-feet. See explanation Columns 6, Table 5.

Column 16 - Quantity, Prototype, second-feet. The discharge was found by multiplying the quantity on the model by the scale ratio raised to the five-halves power:

$$Q_p = Q_m \, (N)^{5/2}$$

where Q_p = Quantity in second-feet, prototype

Q_m = " " " " , model

N = Scale Ratio = 36.

Column 17 - Quantity per Gate, Prototype, second-feet. This value was obtained to assist in calculating the total flow over the spillway and for simplifying the plotting of a discharge diagram. The model contained 2.3903 gates, therefore, Column 16 divided by 2.3903 equals Column 17 or the quantity per 40-foot gate.

Columns 18 and 19 - Orifice Coefficient, Model and Prototype. The value of the coefficient was obtained by using the precise orifice formula

$$Q = 2/3\ C_o\ L\ (2g)^{\frac{1}{2}}\ (H^{3/2} - H_T^{3/2})$$

where

Q = Quantity in second-feet

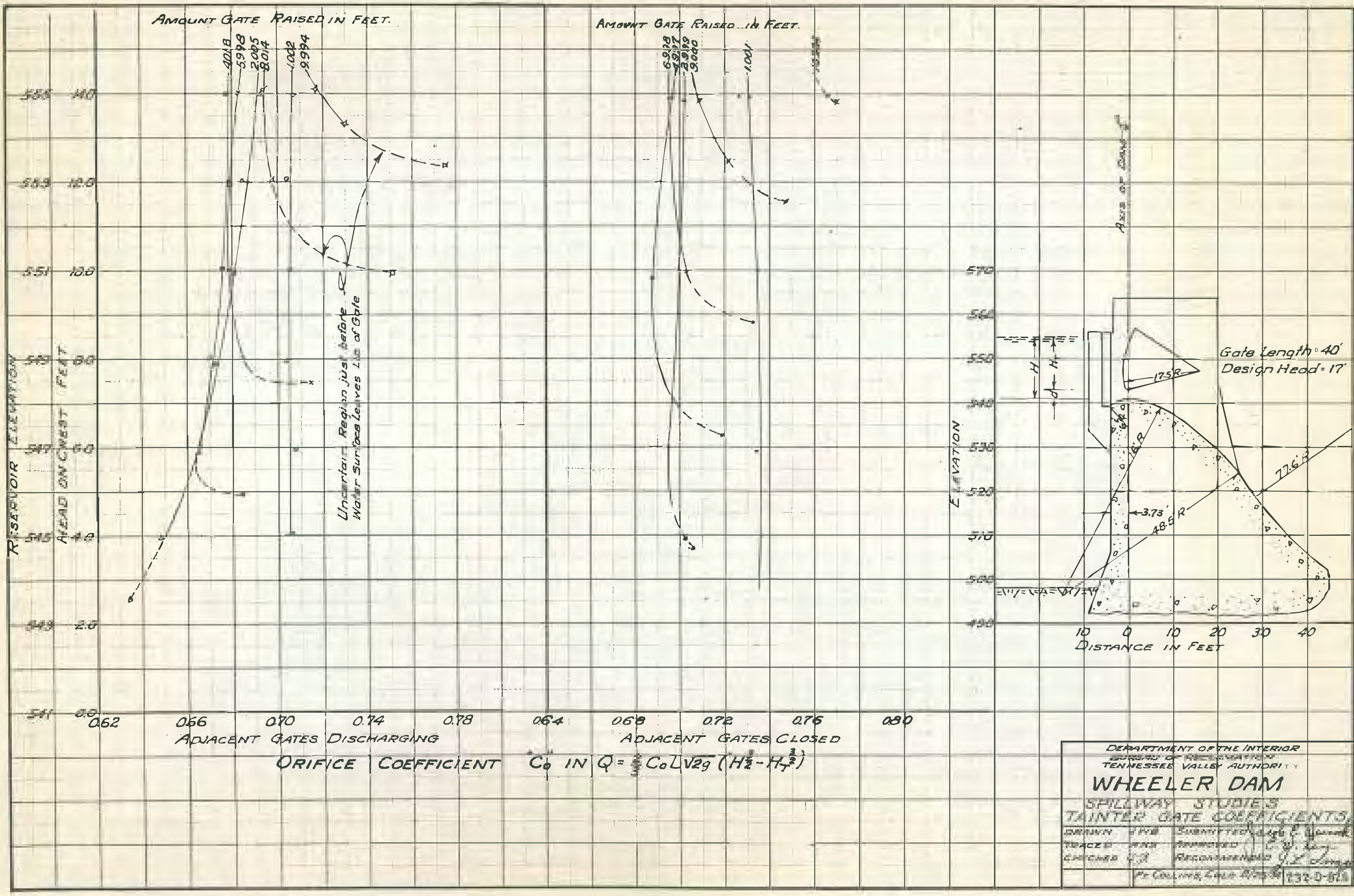
C_o = Orifice Coefficient

L = Length of Crest

g = Value of gravity = 32.1445

H = Head on bottom of Orifice (Crest)

H_T = Head on Top of Orifice (H-d)



AMOUNT GATE RAISED IN FEET.
4.018
5.998
2.005
8.014
1.002
9.994
AMOUNT GATE RAISED IN FEET.
1.001
Uncertain Region just before Water Surface Leaves Lip of Gate
RESERVOIR ELEVATION
HEAD ON CREST FEET
0.62
0.66
0.70
0.74
0.78
0.64
0.68
0.72
0.76
0.80
ADJACENT GATES DISCHARGING
ADJACENT GATES CLOSED
ORIFICE COEFFICIENT C_o IN $Q = \frac{2}{3} C_o L \sqrt{2g} (H_2^{\frac{3}{2}} - H_T^{\frac{3}{2}})$
Gate Length = 40'
Design Head = 17'
17.5'R
6'R
3.73'
48.5'R
77.6'R
ELEVATION
DISTANCE IN FEET
DEPARTMENT OF THE INTERIOR
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
TAINTER GATE COEFFICIENTS

FIGURE 29

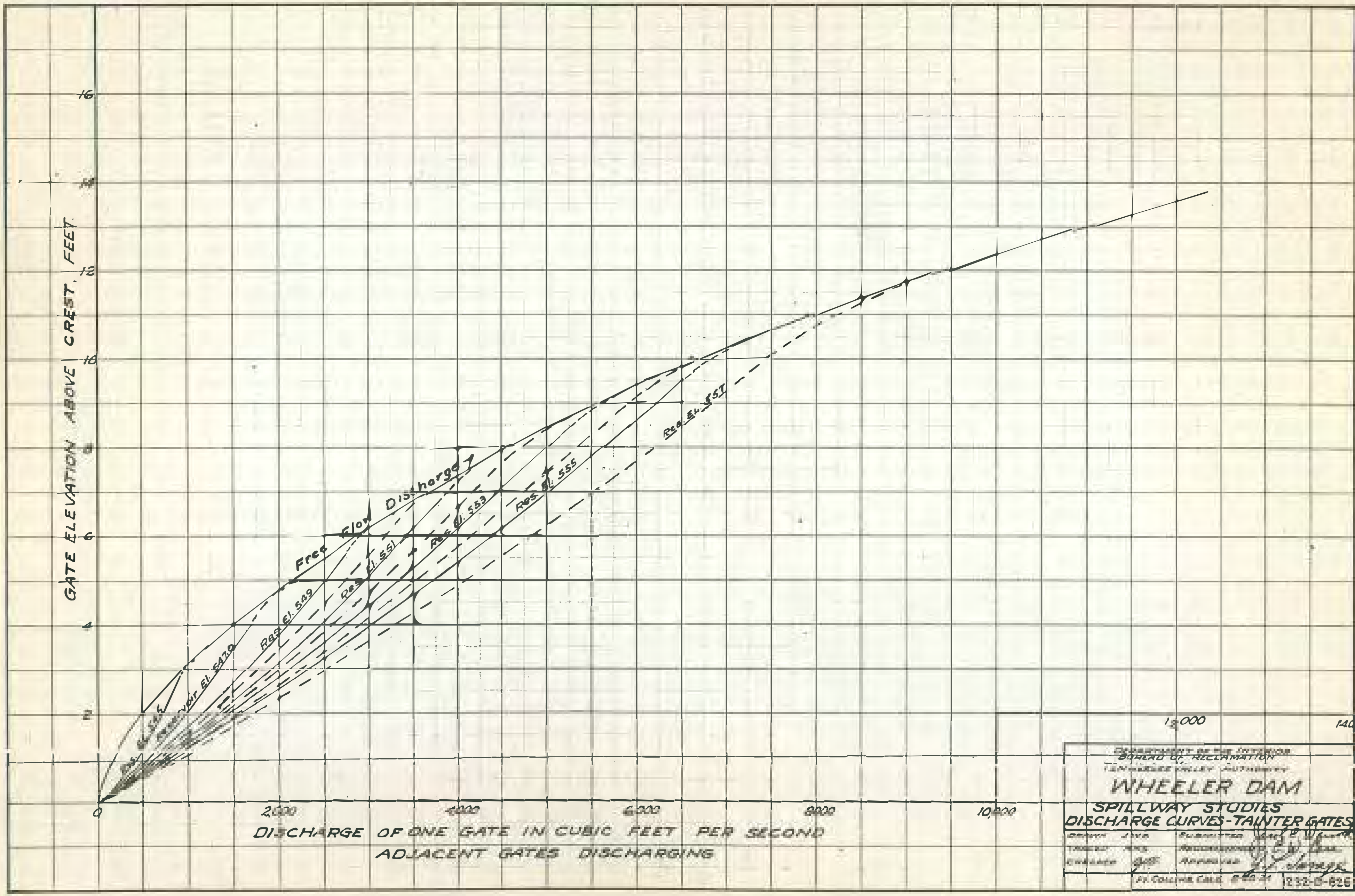
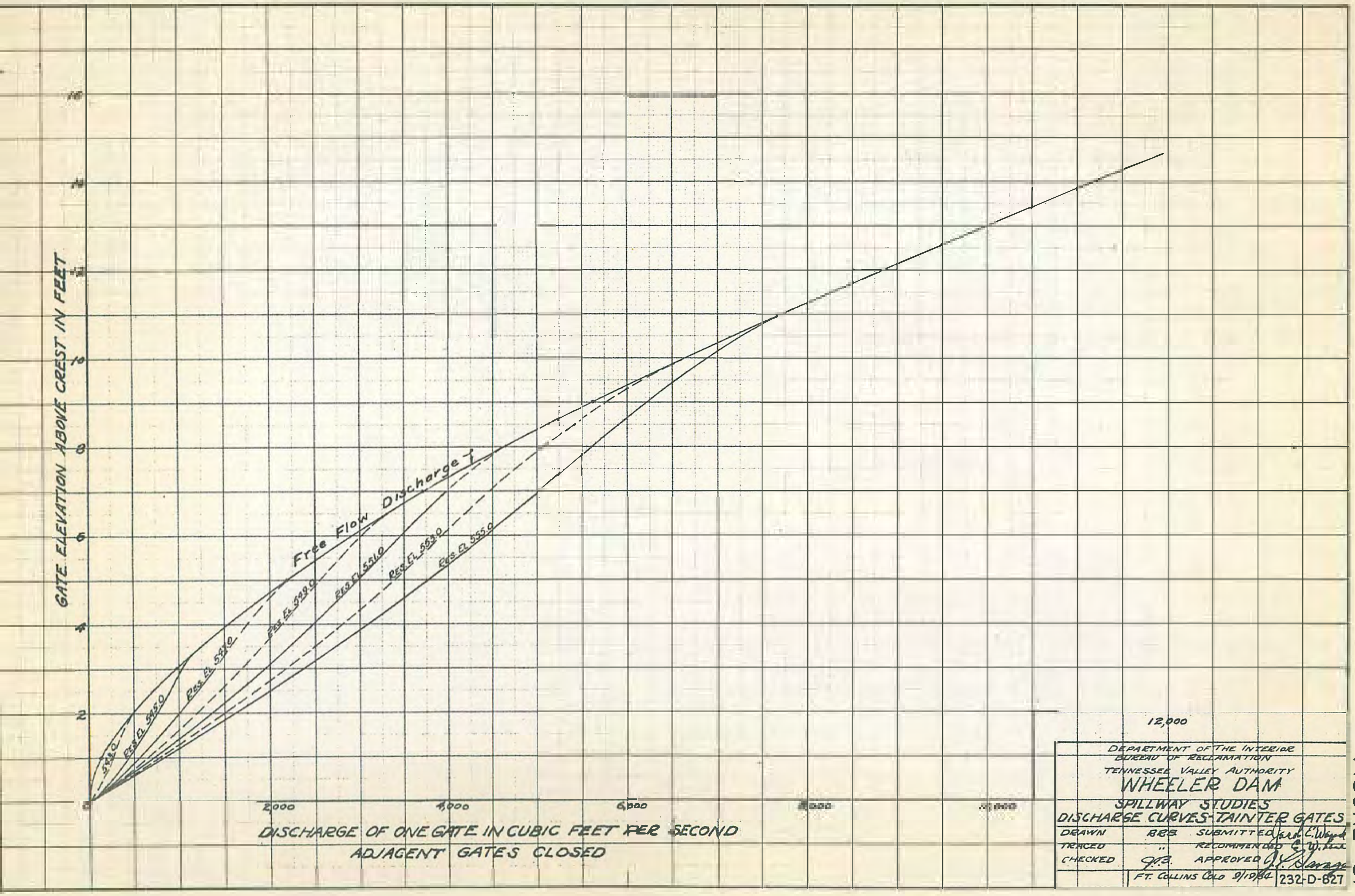

GATE ELEVATION ABOVE CREST, FEET
16
14
12
10
8
6
4
2
0
2,000
4,000
6,000
8,000
10,000
12,000
14,000
DISCHARGE OF ONE GATE IN CUBIC FEET PER SECOND
ADJACENT GATES DISCHARGING
Free Flow Discharge
Res. El. 549
Res. El. 551
Res. El. 553
Res. El. 555
Res. El. 557
BUREAU OF RECLAMATION
WHEELER DAM
SPILLWAY STUDIES
DISCHARGE CURVES-TAINTER GATES
232-D-826

FIGURE 30



GATE ELEVATION ABOVE CREST IN FEET
2
4
6
8
10
12
14
16
2,000
4,000
6,000
8,000
12,000
DISCHARGE OF ONE GATE IN CUBIC FEET PER SECOND
ADJACENT GATES CLOSED
Free Flow Discharge
543.0
Res. El. 545.0
Res. El. 547.0
Res. El. 549.0
Res. El. 551.0
Res. El. 553.0
Res. El. 555.0
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
DISCHARGE CURVES-TAINTER GATES
DRAWN BBB SUBMITTED
TRACED RECOMMENDED
CHECKED APPROVED
FT. COLLINS COLO 9/19/34
232-D-827

FIGURE 31

TABLE NO. 7

SUMMARY OF DATA - WHEELER DAM

Discharge - Free Flow Conditions - Adjacent Gates Discharging

1	2	3	4	5	6
Head in Feet Proto.	$H^{3/2}$	C	$CH^{3/2}$	Q in c.f.s. per Gate	Q in c.f.s. Total for 60 Gates
3.0	5.196	3.132	16.274	650.96	39,057.6
4.0	8.000	3.256	26.048	1,041.92	62,515.2
5.0	11.180	3.352	37.475	1,499.00	89,940.0
6.0	14.700	3.436	50.509	2,020.36	121,221.6
7.0	18.520	3.508	64.968	2,598.72	155,923.2
8.0	22.630	3.573	80.857	3,234.28	194,056.8
9.0	27.000	3.633	98.091	3,923.64	235,418.4
10.0	31.620	3.690	116.678	4,667.12	280,027.2
11.0	36.480	3.743	136.545	5,461.80	327,708.0
12.0	41.570	3.796	157.800	6,312.00	378,720.0
13.0	46.870	3.847	180.309	7,212.36	432,741.6
14.0	52.380	3.895	204.020	8,160.80	489,648.0
15.0	58.090	3.947	229.281	9,171.24	550,274.4
16.0	64.000	4.007	256.448	10,257.92	615,475.2
17.0	70.090	4.093	286.878	11,475.12	688,507.2

TABLE NO. 7 A

Discharge - Free Flow Conditions - Adjacent Gates Closed

1	2	3	4	5
Head in Feet Proto.	$H^{3/2}$	C	$CH^{3/2}$	Q in c.f.s. per Gate
3.0	5.196	3.132	16.274	650.96
4.0	8.000	3.251	26.008	1,040.32
5.0	11.180	3.331	37.241	1,489.64
6.0	14.700	3.401	49.995	1,999.80
7.0	18.520	3.464	64.153	2,566.12
8.0	22.630	3.523	79.725	3,189.00
9.0	27.000	3.572	96.444	3,857.76
10.0	31.620	3.613	114.243	4,569.72
11.0	36.480	3.648	133.079	5,323.16
12.0	41.570	3.677	152.853	6,114.12
13.0	46.870	3.702	172.513	6,900.52
14.0	52.380	3.721	194.906	7,796.24

TABLE NO. 8

SUMMARY OF DATA - WHEELER DAM.

Discharge for Tainter Gates - Adjacent Gates Discharging

Reservoir Elevation Gate Raised (Feet).	555	553	551	549	547	545	Free Flow
0.5	420	400	370	325	280	250	50
1.0	820	770	710	625	535	430	165
1.5	1220	1125	1020	900	760	580	320
2.0	1590	1460	1325	1160	960	720	500
2.5	1960	1790	1625	1400	1160	830	690
3.0	2320	2120	1910	1650	1335	960	930
3.5	2680	2425	2180	1880	1520	-	1200
4.0	3030	2750	2450	2100	1690	-	1500
4.5	3380	3030	2710	2310	1850	-	1800
5.0	3725	3330	2960	2510	-	-	2130
5.5	4060	3625	3180	2710	-	-	2490
6.0	4380	3910	3430	2920	-	-	2850
6.5	4710	4200	3660	3250	-	-	3250
7.0	5025	4470	3880	-	-	-	3660
7.5	5320	4750	4160	-	-	-	4100
8.0	5620	5025	4575	-	-	-	4575
8.5	5920	5300	-	-	-	-	5050
9.0	6220	5630	-	-	-	-	5560
9.5	6500	6100	-	-	-	-	6100
10.0	6825	-	-	-	-	-	6680
10.5	7320	-	-	-	-	-	7320
11.0	7975	-	-	-	-	-	7975

TABLE NO. 8A

Discharge for Tainter Gates - Adjacent Gates Closed.

Reservoir Elevation Gate Raised (Feet)	555	551	547	545	Free Flow
0.5	420	375	300	250	25
1.0	860	720	550	450	120
1.5	1250	1050	785	610	280
2.0	1630	1370	1000	760	480
2.5	2020	1660	1200	900	710
3.0	2380	1950	1400	1025	980
3.5	2750	2220	1570	1240	1240
4.0	3100	2480	1740	-	1530
4.5	3450	2730	1950	-	1850
5.0	3780	2980	2190	-	2190
5.5	4110	3220	-	-	2540
6.0	4420	3470	-	-	2910
6.5	4740	3720	-	-	3310
7.0	5025	3975	-	-	3720
7.5	5320	4270	-	-	4160
8.0	5610	4630	-	-	4630
8.5	5900	-	-	-	5100
9.0	6220	-	-	-	5600
9.5	6550	-	-	-	6110
10.0	6870	-	-	-	6630
10.5	7240	-	-	-	7150
11.0	7700	-	-	-	7700

EXPLANATION OF TABLES 7 AND 7-A

Column 1 - Head on Crest, Prototype, feet. Taken at a representative number of points.

Column 2 - $H^{3/2}$. Column 1 raised to the three-halves power.

Column 3 - Coefficient of Discharge. Taken directly from Figure 28.

Column 4 - $CH^{3/2}$. Multiply Column 2 by Column 3 (Used to obtain total quantity over spillway).

Column 5 - Quantity per gate. Found by multiplying Column 4 by the length of one gate, in this case 40 feet.

Column 6 - Total Quantity over Spillway. Obtained by multiplying Column 4 by 2400 or the net crest length.

EXPLANATION OF TABLES 8 AND 8-A.

From Figures 30 and 31, the Tainter gate openings and discharge quantities were read for a given reservoir elevation, then plotted on Figures 32 and 33 thus completing the discharge curve for flow through the Tainter gates of the Wheeler Dam.

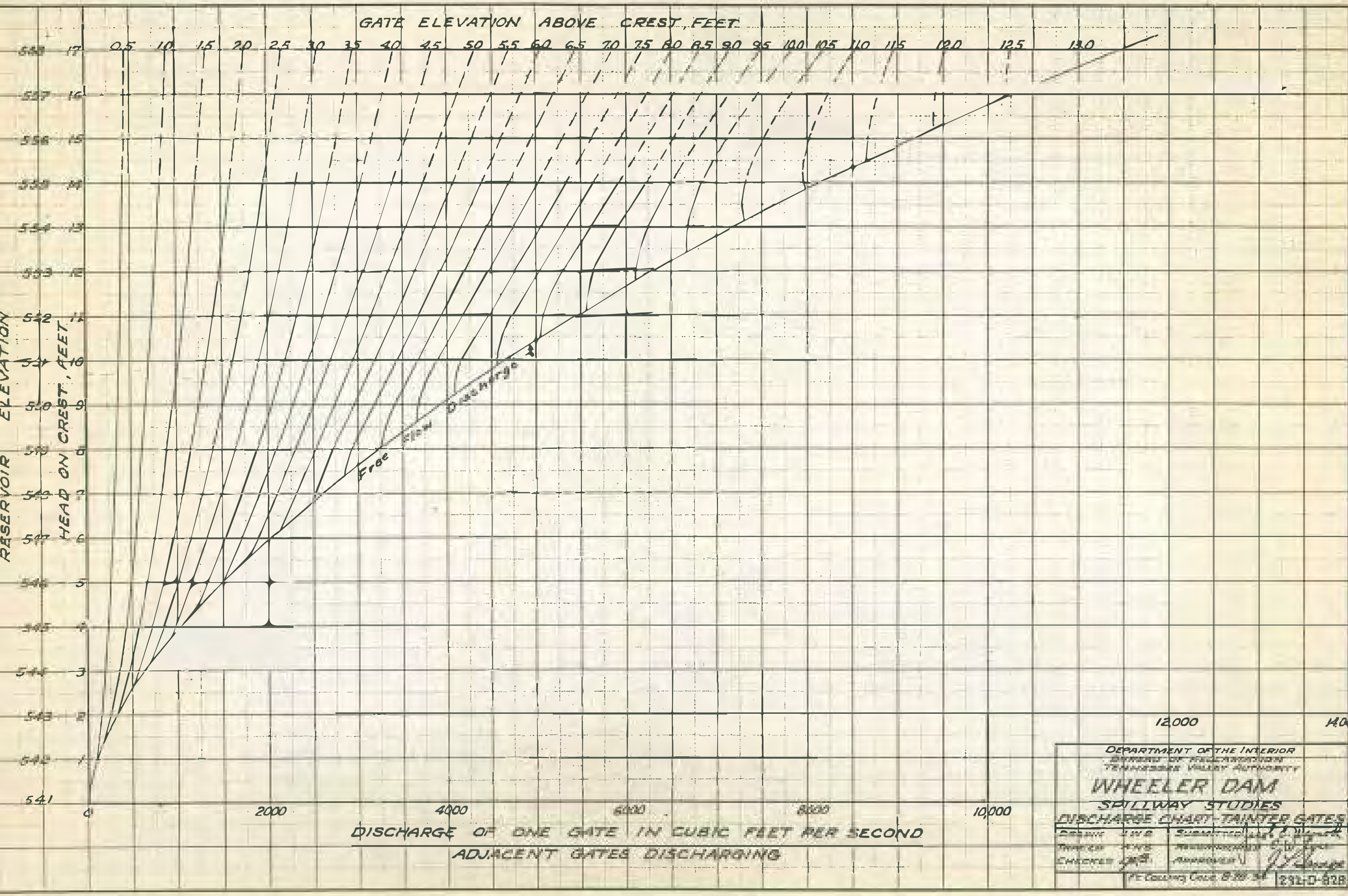

GATE ELEVATION ABOVE CREST, FEET
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
10.5
11.0
11.5
12.0
12.5
13.0
RESERVOIR ELEVATION
HEAD ON CREST, FEET
Free Flow Discharge
0
2000
4000
6000
8000
10,000
12,000
14,000
DISCHARGE OF ONE GATE IN CUBIC FEET PER SECOND
ADJACENT GATES DISCHARGING
DEPARTMENT OF THE INTERIOR
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
DISCHARGE CHART-TAINTER GATES
231-D-828

FIGURE 32

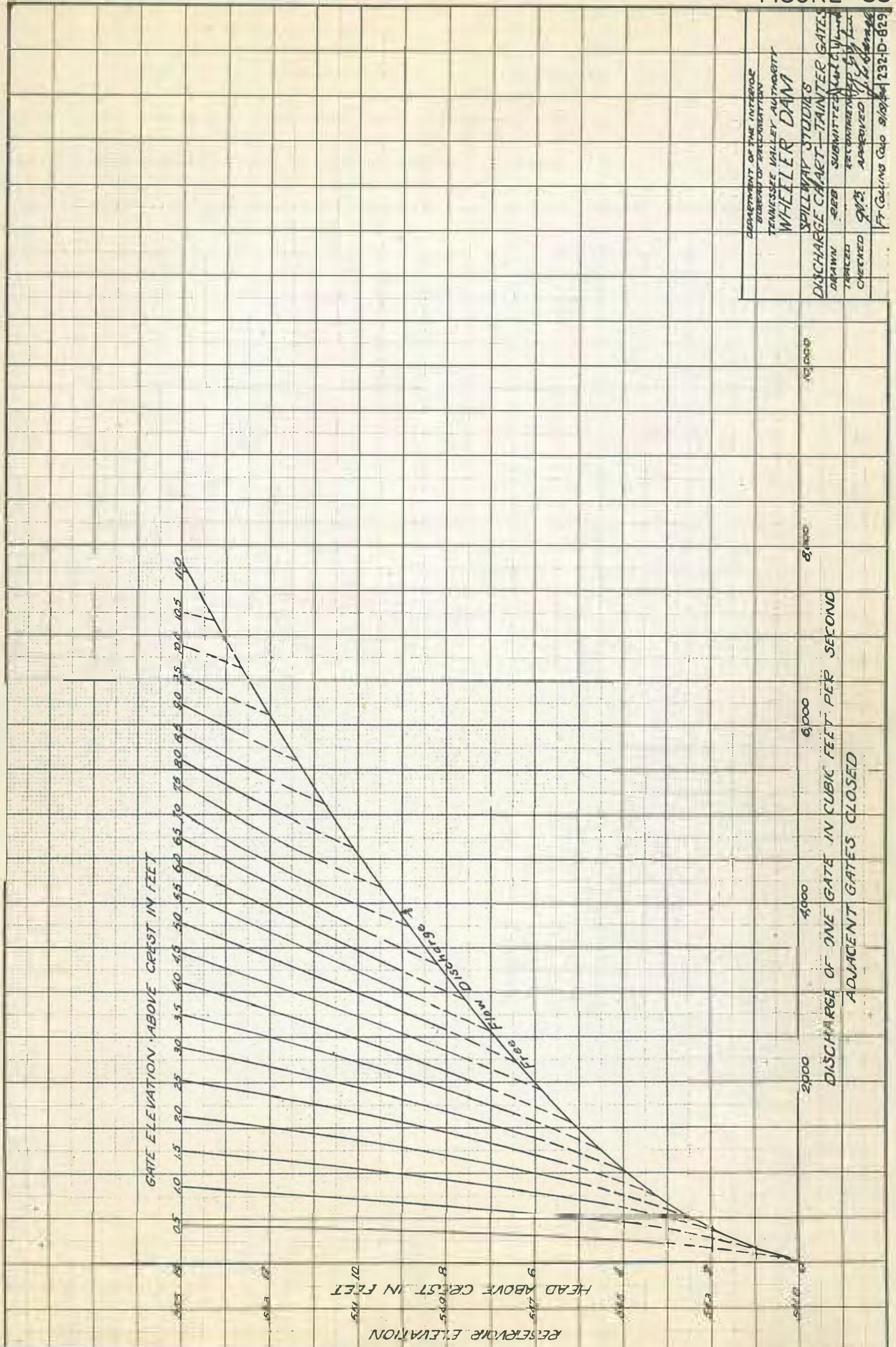

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
WHEELER DAM
SPILLWAY STUDIES
DISCHARGE CHART—TAINTER GATES
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
Ft Collins Colo
232-D-829
GATE ELEVATION ABOVE CREST IN FEET
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
10.5
11.0
Free Flow Discharge
DISCHARGE OF ONE GATE IN CUBIC FEET PER SECOND
ADJACENT GATES CLOSED
2,000
4,000
6,000
8,000
10,000
HEAD ABOVE CREST IN FEET
RESERVOIR ELEVATION
10
8
6
549.8
547

FIGURE 33

VI SUMMARY

A very satisfactory design of stilling pool, which meets all requirements as to foundation and hydraulic conditions, was developed by the experiments on the model of the Wheeler spillway. The erosion was reduced to a minimum with very little excavation necessary near the toe of the dam. The energy dissipation with the diffuser sill placed at the end of the sloping apron was sufficient to allow the handling of a flood slightly in excess of the design capacity of 650,000 second-feet without endangering the structure by excessive erosion of the river bed. It is believed that the factor of safety at the maximum discharge is such that a natural retrogression of the river bed will not affect the action of the stilling pool.

Although slight negative pressure occurred on the diffuser sill at maximum discharge only, the possibility of such a flood occurring is so remote that the condition is not considered serious.

The intermediate training wall was designed to reduce the cross-currents in the stilling pool by the unbalanced gate operation necessary on a spillway of such length as the Wheeler Dam. Some reduction of these cross-currents was made by the use of the diffuser sill.

A satisfactory design of the downstream nose of the piers was developed such that the undesirable conditions existing in the stilling pool downstream from them were reduced to a minimum.

As the inclusion of the intermediate training walls was open to question in the design office a gate operating program was evolved for the action of the stilling pool both with and without the training wall.

The discharge diagram for Tainter gates contained in this report will be helpful to the operating engineer.

APPENDIX I.

PROTECTION AGAINST SCOUR BELOW OVERFALL DAMS.

Technical Memorandum 323.

by E. W. Lane

and

W. F. Bingham

UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: PROTECTION AGAINST SCOUR BELOW OVERFALL DAMS

By E. W. LANE, RESEARCH ENGINEER

and

W. F. BINGHAM, JUNIOR ENGINEER

Under direction of

J. L. SAVAGE, CHIEF DESIGNING ENGINEER

TECHNICAL MEMORANDUM NO. 323

Denver, Colorado

Jan. 23, 1933

(Price $0.85)

HOW THEY INCREASE IN VALUE

The following table of *redemption values* of the $25 and $100 bonds illustrates the increase in value for all the bonds from the issue date to maturity:

Issue Price	$18.75	$75.00
Redemption Values After the Issue Date:		
First year	$18.75	$75.00
1 to 1½ years	19.00	76.00
1½ to 2 years	19.25	77.00
2 to 2½ years	19.50	78.00
2½ to 3 years	19.75	79.00
3 to 3½ years	20.00	80.00
3½ to 4 years	20.25	81.00
4 to 4½ years	20.50	82.00
4½ to 5 years	20.75	83.00
5 to 5½ years	21.00	84.00
5½ to 6 years	21.25	85.00
6 to 6½ years	21.50	86.00
6½ to 7 years	21.75	87.00
7 to 7½ years	22.00	88.00
7½ to 8 years	22.50	90.00
8 to 8½ years	23.00	92.00
8½ to 9 years	23.50	94.00
9 to 9½ years	24.00	96.00
9½ to 10 years	24.50	98.00
Maturity Value	$25.00	$100.00

PROTECTION AGAINST LOSS

Each bond is *registered* in the name of the owner on the books of the United States Treasury. The name of the owner is written on the face of the bond, and it

will be payable *only* to him except in case of death, when it will be payable to his heirs; in case of disability, when it will be payable to his legally accredited agent; or as a result of judicial proceedings. In accordance with laws and Treasury regulations, he may obtain a duplicate, in case of loss, theft, or destruction, by application to the Treasury Department, Division of Loans and Currency. These safeguards furnish complete protection against loss of United States Savings Bonds. If the owner desires, the Government will hold the bond in safekeeping for him and issue a receipt to him.

ON SALE AT POST OFFICES

The Post Office Department of the United States Government is the sales agency for United States Savings Bonds. They are on sale at all post offices of the first, second, and third class, and at many post offices of the fourth class. At one or more windows in each of these post offices there are one or more employees on duty to sell these bonds. They will accept payment of the purchase price, deliver the bond to the owner with his name written upon it, and transmit a record of the transaction to the United States Treasury for official registration.

If the purchaser buys the bond for himself, he should give his full name in the form in which he ordinarily uses it. This precaution will prevent any confusion as to ownership. If he buys it for another person with the plan or expectation that somebody else will collect the full amount due at maturity—one of his children, for instance—he should instruct the post-office salesman to write on the bond the name of the person to whom it is to be paid. The purchaser may then take his bond with him. If he wishes the Government to hold it for him, the post-office salesman will explain how that may be arranged. Postal Savings depositors may withdraw deposits without loss of interest to buy United States Savings Bonds.

HOW TO CASH A BOND

To obtain cash for his bond, the owner should take it—at any time after 60 days from the issue date—to any post office where the bonds are on sale, or to any incorporated bank or trust company. In the presence of a post office or bank official authorized to perform this service, he establishes his identity, signs the request for payment which appears on the back of the bond, and has the request certified. The owner then sends the bond to any Federal Reserve bank (the banker or post-office clerk will tell him the address of the nearest Federal Reserve bank) or to the Division of Loans and Currency, United States Treasury, Washington, D. C. The Government will mail him a check for the redemption value of the bond.

BUY AT ANY TIME

These bonds, which are known as Series A, United States Savings Bonds, will remain on sale until further notice,

BONDS ALWAYS REDEEMABLE

United States Savings Bonds, as their name shows, are intended to furnish a convenient means for the profitable investment of savings. The *greatest profit* and the *greatest rate of profit* are obtained if they are held for the full 10 years. But to provide for any *emergency need* of any purchaser, the Government will redeem *any bond at any time* 60 days or more after the date of issue on the request of the owner. The *price* which the Government will pay in *buying back* the bond will depend on how long the owner has held it.

For the *first year* only the issue price will be paid. A year after issue the *redemption price* will be $76 for the bond which cost $75 and which has a maturity value of $100. After 18 months the bond which cost $75 may be cashed for $77. It increases $1 in value every 6 months until it has been held 7 years. Then it will have a *redemption value* of $88. Thereafter its value increases $2 every 6 months until the 10 years are up, when the Government will pay the face value of the bond, or $100, to the owner. The increase in value of the bonds of other denominations is in the same proportion. For instance, the bond costing $18.75 will be worth $19 at the end of the first year, and $19.25 after 18 months. Thereafter it increases in value by 25 cents every 6 months until its redemption value is $22 in 7 years. At the end of 10 years the purchaser receives $25 for it.

UNITED STATES SAVINGS BONDS

THE UNITED STATES GOVERNMENT offers for sale through its post offices to the people of the United States a new form of Government security known as *United States Savings Bonds.*

These bonds are issued in denominations of $25, $50, $100, $500, and $1,000. Each bond bears the promise of the Government of the United States to pay to the owner the full amount (maturity value) stated on the bond *10 years from the date of issue.* The date of issue is the first day of the month in which the bond is purchased.

The bonds are sold at *issue prices* which are less than the *face values.* They *increase in value* regularly after the first year. A bond bought at the present issue prices and held for 10 years increases in value by exactly one-third of the purchase price. The increase is *equal to interest* on the purchase price at a rate of about 2.9 percent *compounded semiannually.* The following table of *issue prices* and *maturity values* shows *the amount the buyer pays* and the amount *he receives* in 10 years:

Issue Price		Maturity Value
$18.75	will increase in 10 years to	$25.00
$37.50	will increase in 10 years to	$50.00
$75.00	will increase in 10 years to	$100.00
$375.00	will increase in 10 years to	$500.00
$750.00	will increase in 10 years to	$1,000.00

but *the Secretary of the Treasury reserves the right to terminate the offer at any time.*

They are exempt from present and future Federal, State, and local taxation, except estate or inheritance taxes and Federal surtaxes on income.

The issue prices of this series will remain the same. *They will be on sale every business day until further notice.* A purchaser may buy one or more bonds of any denomination every week, every 2 weeks, or every month, or on any schedule that suits his budget plans. But since the bonds are intended for *investment of savings,* there is a limit on the amount any person may buy in 1 calendar year. That limit is a total of $10,000 (maturity value). He may, however, purchase $10,000 worth of bonds each separate calendar year. Except for that limitation, he may buy as many as he wishes at any time.

★

U. S. GOVERNMENT PRINTING OFFICE 2—16701

UNITED STATES SAVINGS BONDS

★ ★ ★ ★ ★ ★ ★

A New Form of Government Security

★ ★ ★ ★ ★ ★ ★

$18.75	increases in 10 years to	$25
$37.50	increases in 10 years to	$50
$75.00	increases in 10 years to	$100
$375.00	increases in 10 years to	$500
$750.00	increases in 10 years to	$1000

ON SALE AT POST OFFICES

FURTHER INFORMATION WITHIN

The United States Government Offers to the People . . .

★ ★ ★ ★ ★ ★ ★ ★

UNITED STATES SAVINGS BONDS

★ ★ ★ ★ ★ ★ ★ ★

$18.75	increases in 10 years to	$25
$37.50	increases in 10 years to	$50
$75.00	increases in 10 years to	$100
$375.00	increases in 10 years to	$500
$750.00	increases in 10 years to	$1000

ON SALE AT POST OFFICES

FURTHER INFORMATION WITHIN

A great deal of study has been given to determining the best method of protecting the stream bed below overfall dams against scour resulting from the high velocity and impact of the water flowing over them. A great variety of local conditions have been encountered which has led to many different solutions, descriptions of which are widely dispersed through engineering literature. From a study of these articles and the results of hydraulic laboratory studies on dam spillways performed by the U. S. Bureau of Reclamation, the following analysis has been developed by the writers, which classifies the various conditions and points out the general types of scour protection applicable to each.

Scour below dams results from the erosive power of the water which comes into contact with the stream bottom while moving with the high velocity which it acquires in falling over the dam. Protection is afforded by reducing the velocity of the water, by insuring that the high velocity flows do not come in contact with the bottom, or are diverted to portions of the bottom which will not endanger the structure. Usually a combination of these is used.

On many dams in the past an attempt has been made to reduce the velocity of the water passing over them by having the water fall down a series of steps on the face of the dam, dissipating its energy by contact with the successive steps and reaching the bottom with insufficient energy remaining to seriously erode the stream bed.

Probably the outstanding example of this type is the Gilboa Dam* of

*Trans., Am. Soc. C. E., p. 280, Vol. 86, 1923.

the water supply system of New York City (Plate 1-A). While the results on this dam have probably been successful, due to the carefully conducted model tests, without such studies the action on stepped weirs is apt to be different from that anticipated,"" and as the

**Civil Engineer, p. 623, Vol. 2, October, 1932.

principles of other forms of protection are becoming better understood, this form seems to be less frequently used. The present tendency is to dissipate the energy in some form of stilling-pool or to divert the high velocity stream so that it does not come into contact with the bottom where damage will result.

The Four Classes of Conditions

The most important factor in determining what form of protection should be used is the depth of water on the downstream side of the dam and its relation to the depth required to form the hydraulic jump.

In the most perfect form of hydraulic jump, the energy of the high velocity water is dissipated so thoroughly in internal impact that little energy remains to be used up in eddies and boils downstream. In the case of a dam with a well-formed jump at its toe, the velocity of the water is so quickly and uniformly reduced that little scour of bed and banks results. It is therefore desirable, whenever possible, to design the dam so that such a jump will occur, as the protection required for the bed and banks is thus reduced to a minimum.

The formula for the hydraulic jump in a horizontal channel of rectangular section is $D_2 = \frac{-D_1}{2} + \sqrt{\frac{2 V_1^2 D_1}{g} + \frac{D_1^2}{4}}$ where D_1 and D_2 are the depths upstream and downstream from the jump respectively and V_1 and V_2 are the corresponding velocities. Consider an ogee dam, Figure 1, discharging 100 second-feet per foot of length with a drop H equal to 50 feet. The velocity V_1 (neglecting friction) would then be $\sqrt{2 g \times 50} = 56.8$ feet per second and the depth D_1 would be $\frac{100}{\sqrt{2 g 50}} = 1.76$ feet. Substituting in the above formula gives a value of D_2 17.9 feet which is the height of tailwater required to form a perfect jump at the toe of the dam.

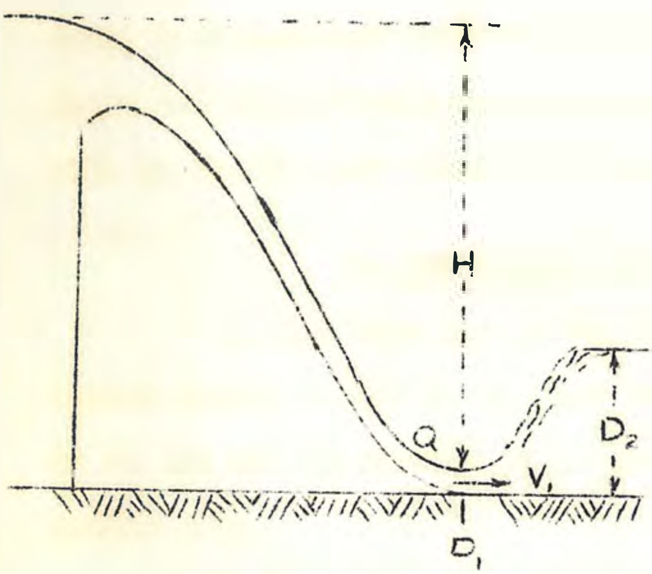


Figure 1.

The ideal condition would be to have a tailwater at such a height above the river bed for each discharge that it would form a perfect jump for the depth and velocity which would occur in the over-falling stream at the toe of the dam for that discharge. The height of the tailwater, however, is controlled by the conditions in the stream channel downstream from the dam and this ideal condition is never exactly attained. Frequently the stage-discharge or tailwater rating curve at the downstream side of the dam is as shown in Figure 2, but the curve which would be required to form the perfect jump on an apron at river bed level is as shown by the jump height curve A-B. This shows that for discharges less than 20,000 second-feet the tailwater height is greater than that required to form a perfect jump but at greater discharges the tailwater height is too low.

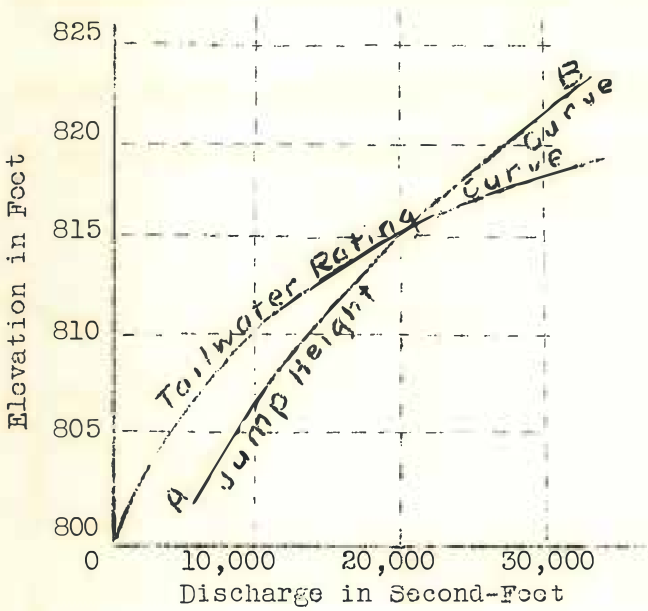


Figure 2.

The relations between the positions of these curves fall into the four following classes:

Case 1 - Jump height curve always above the tailwater rating curve.

Case 2 - Jump height curve always below the tailwater rating curve.

Case 3 - Jump height curve above the tailwater rating curve at low discharges and below at high discharges.

Case 4 - Jump height curve below the tailwater rating curve at low discharges and above at high discharges.

The best form of protection below a dam depends largely upon which of these conditions exist.

Class 1

The first class frequently occurs when a dam is placed at the head of a rapids or sudden drop in the stream bed. Under these conditions the tailwater level is low and less than the height required to form the jump. Usually, too, the bed is of solid rock which will withstand considerable scour. Under these conditions an upward curving apron is frequently put on the dam which throws the stream of high velocity water passing over the dam upward so that it strikes the stream bed some distance away from the structure. Here the energy is dissipated by impact of the water on the river bottom and adjacent water, and although some scour takes place it is too small and too far from the dam to endanger it. One of the earliest examples of this type of dam is the famous Holyoke Dam on the Merrimac River in Massachusetts (Plate 1-B). Others are the

Conowingo* and Safe Harbor dams on the Susquehanna River (Plates

*Engineering News-Record, p. 127, Vol. 108, Year 1932.

1-C and D).

At the Bull Run Dam** for the Portland, Oregon, water sup-

**Trans., Am. Soc. C. E., p. 487, Vol. 93, 1929.

ply (Plate 1-E), a wide horizontal apron was used with upward sloping deflectors at the downstream edge which directed the deflected stream so that when it fell back to the river level it was spread over a large area, and consequently did not produce as severe scour as if the impact was localized.

The conditions on the Wilson Dam on the Tennessee River (Plate 1-F) are such that it probably falls in Case 1. A wide, level apron was constructed below this dam to protect the river bottom but the depth of the tailwater was insufficient to cause the jump to form and the high velocity water passed entirely across the 200-foot apron and eroded a large hole in the solid rock at its downstream edge.*

*Engineering News-Record, p. 190, Vol. 98, Year 1927.

Experiments with a model of this dam++ have shown that piers would

++Wilsonova Prehrada na Rece Tennessee, Alabama, U. S. A. - Antonin Smrcek.

not cause the jump to form on the apron with the tailwater depth available.

When the tailwater depth is nearly sufficient to cause the jump, baffles or sills may be successfully used, but they often receive so much impact from drift or ice that they are expensive to build with a sufficiently strong anchorage. Baffles or piers do not dissipate as much energy as might be expected and they are therefore not as effective as the hydraulic jump.* Perhaps the best example

*Engineering News-Record, p. 800, Vol. 97, November 11, 1926.

of the use of baffle piers with a low tailwater level is in the Gatun spillway on the Panama Canal (Plate 1-H). This spillway is designed to discharge as much as 140,000 second-feet with a fall of nearly 75 feet. At the toe of the ogee section the water impinges directly against the flat faces of the baffle piers and much of it is thrown high into the air, reaching almost as high an elevation as the water above the dam.** When this water falls to the river level it has

**Trans., Am. Soc. C. E., p. 487, Vol. 93, 1929.
Engineering News-Record, p. 800, Vol. 97, Nov. 11, 1926.
Hydraulic Laboratory Practice, Freeman, p. 506.
International Engineering Congress, pp. 46-63, Vol. II, 1915.

nearly as great a velocity as it had at the toe of the dam. The principal effect of these baffle piers therefore is to distribute the impact over a large area rather than to dissipate the energy of the water.

If the tailwater level is not high enough to form a perfect jump it may be raised by building a low secondary dam below the main dam with a sufficient height to cause the jump to form at the foot of the main dam for all conditions of discharge. This method has been extensively used for dams on earth foundations. Plate 1-I shows a form developed by the Fargo Engineering Company. This method has also been used for a rock foundation at the Martin Dam of the Alabama Power Company. The dimensions required for such a pool are shown by the experiments recently published.*

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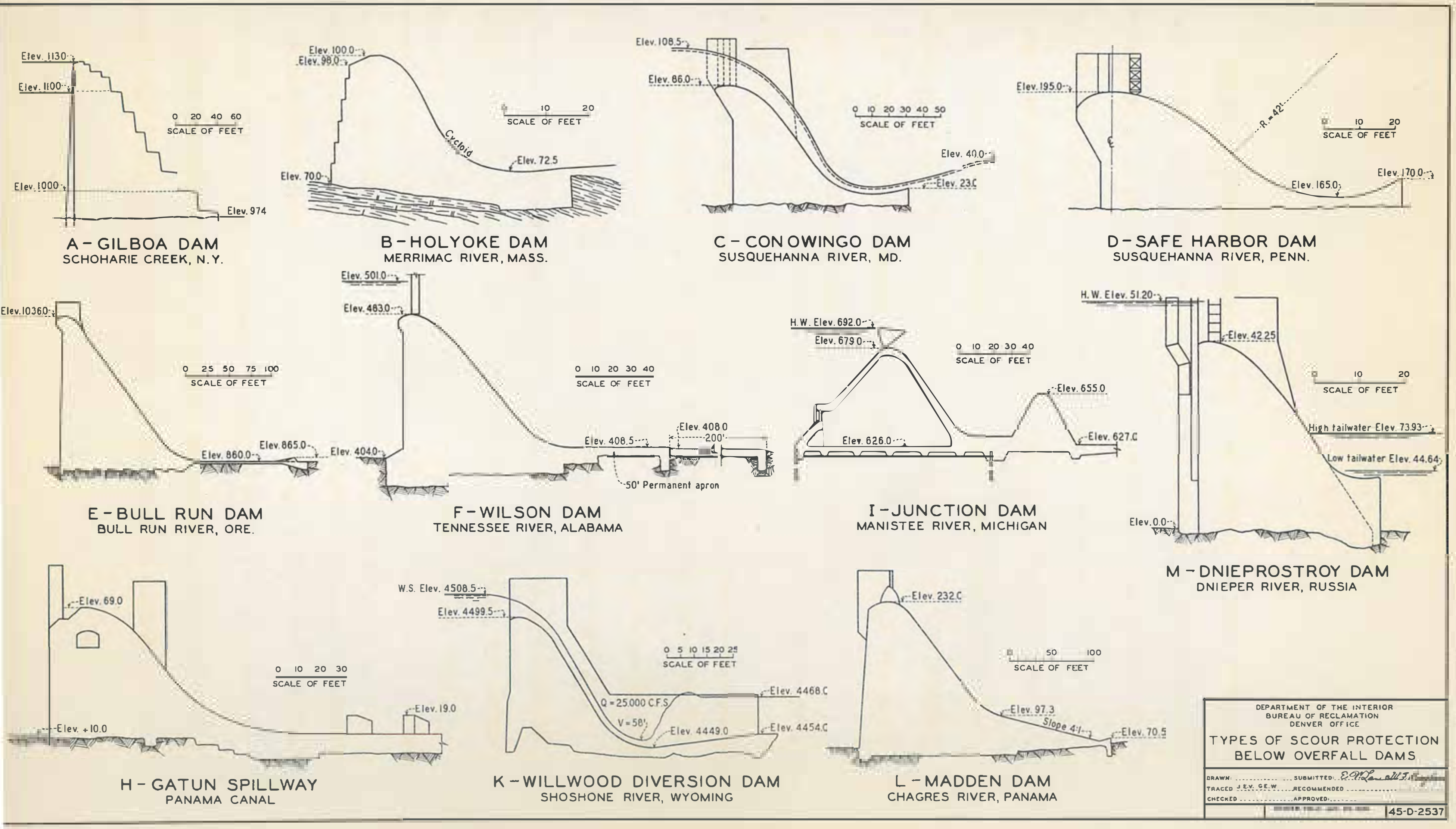
*Proceedings, Am. Soc. C. E., p. 1521, Nov., 1932.

Another method which may be suitable in some cases is to excavate a pool just below the dam to provide a depth sufficient for the formation of the jump. In this case the tailwater level is not changed, but the depth required to form the jump is provided by the lowering of the channel bottom rather than by raising the water surface. This method was used in the Wilwood Dam** (Plate 1-K).

**Engineering News-Record, p. 660, Vol. 99, October 27, 1927.

On rock foundations where the tailwater level is nearly high enough to cause the jump to form, experience indicates that if no protection is added downstream from the bucket of the dam, the bottom will be scoured out until sufficient depth is provided to permit the jump


Elev. 1130
Elev. 1100
Elev. 1000
Elev. 974
0 20 40 60
SCALE OF FEET
A - GILBOA DAM
SCHOHARIE CREEK, N.Y.
Elev. 100.0
Elev. 98.0
Elev. 70.0
Cycloid
Elev. 72.5
0 10 20
SCALE OF FEET
B - HOLYOKE DAM
MERRIMAC RIVER, MASS.
Elev. 108.5
Elev. 86.0
0 10 20 30 40 50
SCALE OF FEET
Elev. 40.0
Elev. 23.0
C - CONOWINGO DAM
SUSQUEHANNA RIVER, MD.
Elev. 195.0
R.=42'
0 10 20
SCALE OF FEET
Elev. 165.0
Elev. 170.0
D - SAFE HARBOR DAM
SUSQUEHANNA RIVER, PENN.
Elev. 1036.0
0 25 50 75 100
SCALE OF FEET
Elev. 865.0
Elev. 860.0
E - BULL RUN DAM
BULL RUN RIVER, ORE.
Elev. 501.0
Elev. 483.0
Elev. 404.0
0 10 20 30 40
SCALE OF FEET
Elev. 408.5
Elev. 408.0
200'
50' Permanent apron
F - WILSON DAM
TENNESSEE RIVER, ALABAMA
H.W. Elev. 692.0
Elev. 679.0
0 10 20 30 40
SCALE OF FEET
Elev. 655.0
Elev. 626.0
Elev. 627.0
I - JUNCTION DAM
MANISTEE RIVER, MICHIGAN
H.W. Elev. 51.20
Elev. 42.25
0 10 20
SCALE OF FEET
High tailwater Elev. 73.93
Low tailwater Elev. 44.64
Elev. 0.0
M - DNIEPROSTROY DAM
DNIEPER RIVER, RUSSIA
Elev. 69.0
0 10 20 30
SCALE OF FEET
Elev. 19.0
Elev. +10.0
H - GATUN SPILLWAY
PANAMA CANAL
W.S. Elev. 4508.5
Elev. 4499.5
0 5 10 15 20 25
SCALE OF FEET
Q = 25,000 C.F.S.
V = 58'
Elev. 4468.0
Elev. 4449.0
Elev. 4454.0
K - WILLWOOD DIVERSION DAM
SHOSHONE RIVER, WYOMING
Elev. 232.0
0 50 100
SCALE OF FEET
Elev. 97.3
Slope 4:1
Elev. 70.5
L - MADDEN DAM
CHAGRES RIVER, PANAMA
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
TYPES OF SCOUR PROTECTION
BELOW OVERFALL DAMS
DRAWN
SUBMITTED
TRACED J.E.V. GE.W.
RECOMMENDED
CHECKED
APPROVED
45-D-2537

to form and no further scour will take place.*

*Civil Engineering, p. 527, Vol. 1, 1931.

Class 2

Dams in the second class are apt to occur where the foundation rock is at a considerable depth, and where the tailwater surface is therefore higher than necessary to form the jump. This case will be more frequently encountered in the future than in the past, due to the exhaustion of the supply of good damsites. Under these circumstances, if an ogee dam with conventional bucket is used, the water flowing down the face of the dam dives under the tailwater and travels at high velocity a long distance along the bottom, forming only a very imperfect jump. The more nearly the tailwater depth corresponds to the depth required to form the jump, the shorter the distance which the high velocities extend downstream. A perfect jump could be formed for any discharge by building a level apron below the dam to give just the correct depth for the formation of the jump, but this apron would probably not be at the correct elevation for other discharges. By making the apron sloping, the various depths required for the different flows can be provided. The high velocity stream flows down the sloping apron until the depth below tailwater level is reached at which the jump can be formed for that discharge, and at this point the jump occurs. The advantages of the sloping apron have been pointed out by G. Gale Dixon.*

*Engineering News-Record, p. 696, Vol. 100, May 3, 1928.

The formula given above does not exactly apply to jumps on a sloping floor since the water enters the jump in a direction somewhat different from that in which it flows away, while the formula assumes motion in the same direction. The formula for the formation of the jump on a sloping surface has been developed by R. W. Ellms.**

**Trans., Am. Soc. M. E., Sept.-Dec., 1932, paper HYD-54-6.

A jump formed under favorable conditions is an intimate, relatively uniform mixture of air and water with a white appearance, and the velocity is reduced within a comparatively short distance. As the jump becomes less perfect the mixture is not so intimate and the velocity is not so rapidly reduced. It is questionable if a perfect jump can be formed except on a level floor. As the slope of the floor is increased, the dissipation of the energy becomes less efficient. Experiments on the model of the Cle Elum Dam spillway with floor slopes of $1\frac{1}{2}$ horizontal to 1 vertical, 2:1, 3:1 and 4:1 showed progressively less scour as the slope was flattened.

Probably the outstanding example of the use of the sloping apron for dams in the second class is the Madden Dam (Plate 1-L) now being constructed to store water for the Panama Canal. This form was developed as the result of extensive model tests.*

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*Engineering News-Record, p. 42, Vol. 109, 1932.

The height of the natural tailwater and the jump height curve for an apron at stream bed level are given on Figure 3, showing that this case falls into Class 2. The shape of the apron was determined by trial. By varying the tailwater level, a determination was made for each form of apron tested of the tailwater levels required for the various discharges, to cause the jump to form at the upstream edge of the apron. For each form a curve was plotted of tailwater level required against discharge over the dam. The agreement of this curve with the tailwater rating curve for the best apron developed is shown on Figure 4. The apron developed had a slope of 4 horizontal to 1 vertical with its upstream edge approximately 30 feet above the foundation level. At large discharges, however, the high velocities extended entirely across the apron and a small triangular sill or lip was placed on the downstream edge of the apron to deflect the swift water up off the stream bed. The sloping apron may require large volumes of concrete but in the case of the Madden Dam this was of assistance in resisting earthquake effects.

In the model experiments for the Madden Dam another form was investigated which requires less concrete and would prove advantageous in certain conditions. It may be called the high bucket

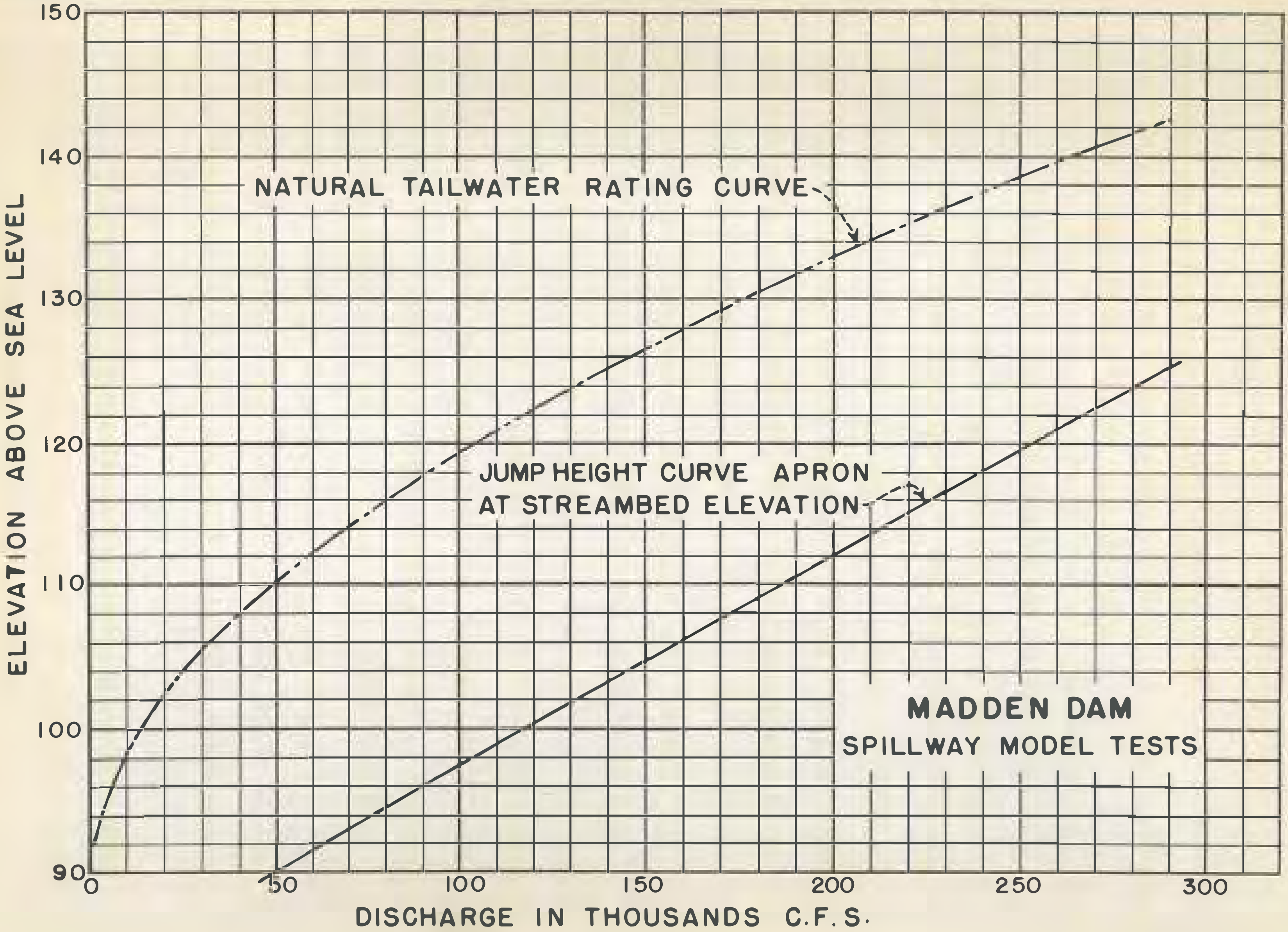

150
140
130
120
110
100
90
0
50
100
150
200
250
300
ELEVATION ABOVE SEA LEVEL
DISCHARGE IN THOUSANDS C.F.S.
NATURAL TAILWATER RATING CURVE
JUMP HEIGHT CURVE APRON AT STREAMBED ELEVATION
MADDEN DAM
SPILLWAY MODEL TESTS

FIG. 3

X-D-1186

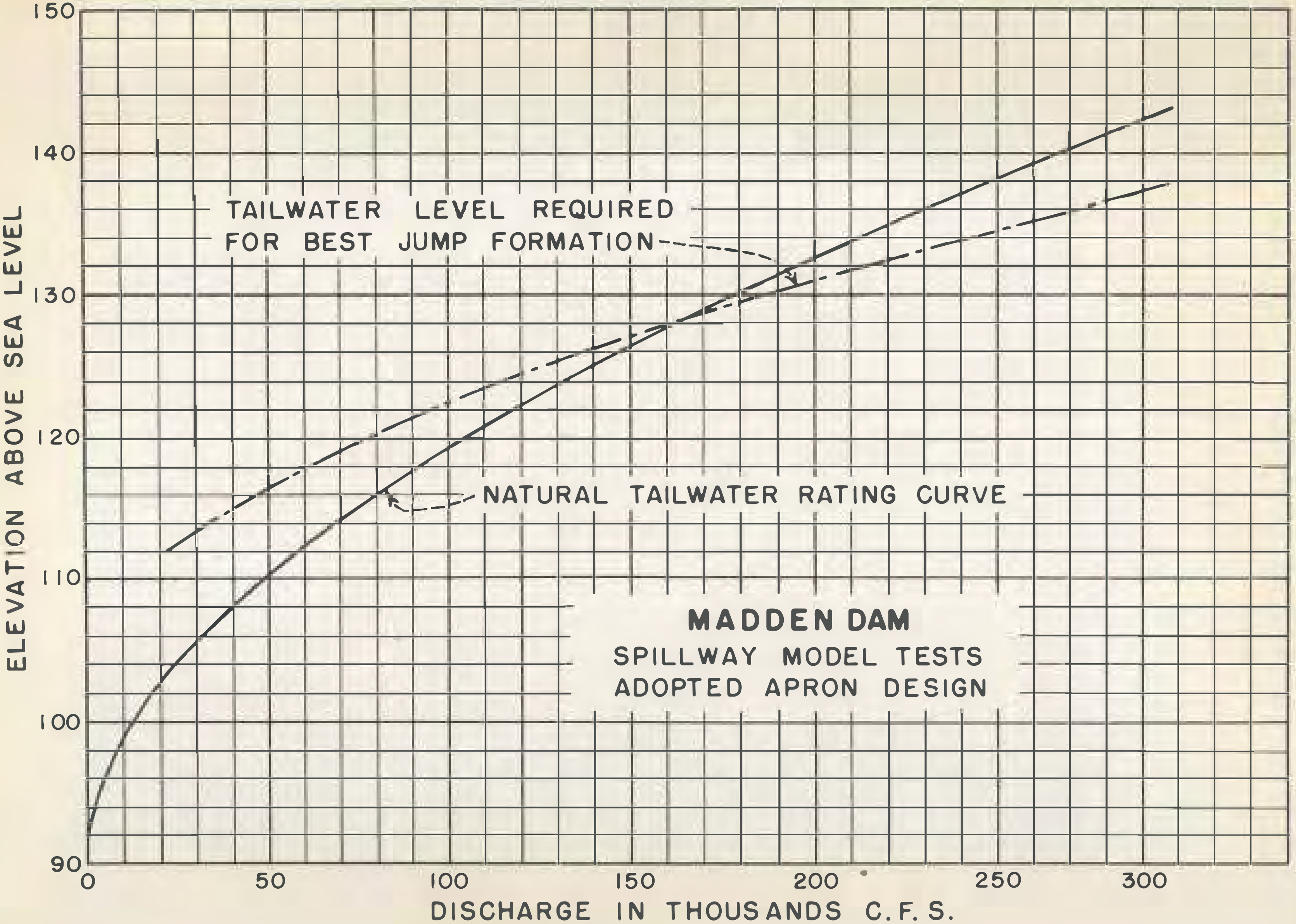

TAILWATER LEVEL REQUIRED
FOR BEST JUMP FORMATION
NATURAL TAILWATER RATING CURVE
MADDEN DAM
SPILLWAY MODEL TESTS
ADOPTED APRON DESIGN
ELEVATION ABOVE SEA LEVEL
150
140
130
120
110
100
90
0
50
100
150
200
250
300
DISCHARGE IN THOUSANDS C.F.S.

FIG. 4

X-D-1187

type. It consists of the ordinary curved bucket of the ogee weir located some distance above the river bottom, as shown on Figure 5-A.

When the tailwater level is somewhat higher above the lip of the bucket than the upper surface of the high velocity sheet at the end of the bucket, any water which might be on top of this sheet is swept away by friction with this sheet and the upper surfaces of the sheet is below the tailwater level. A pressure is then exerted on the under side of this sheet beyond the lip of the bucket as shown by the arrows in Figure 5-A, due to the pressure exerted upstream beneath the sheet from the higher tailwater level. The high velocity jet is deflected upward by these forces in a great sweep which may carry it even higher than the tailwater level, to which it falls back after the upward motion is overcome by gravity. As this water falls on top of the deep tailwater it does no damage to the river bottom and the only scour on the bottom is that due to the upstream flowing water of the eddy which forms beneath the high velocity sheet. This condition of flow occurs at flood times on the dam (Plate 1-M) of the recently completed Dnieprostroy power plant on the Dnieper River in Russia.* For somewhat higher tail-

*Engineering News-Record, p. 877, Vol. 108, June 23, 1932;
p. 90, Vol. 109, July 21, 1932.

water levels the path of the high velocity sheet tends to become nearly vertical and part of it falls back into the "valley" along

HIGH BUCKET TYPE

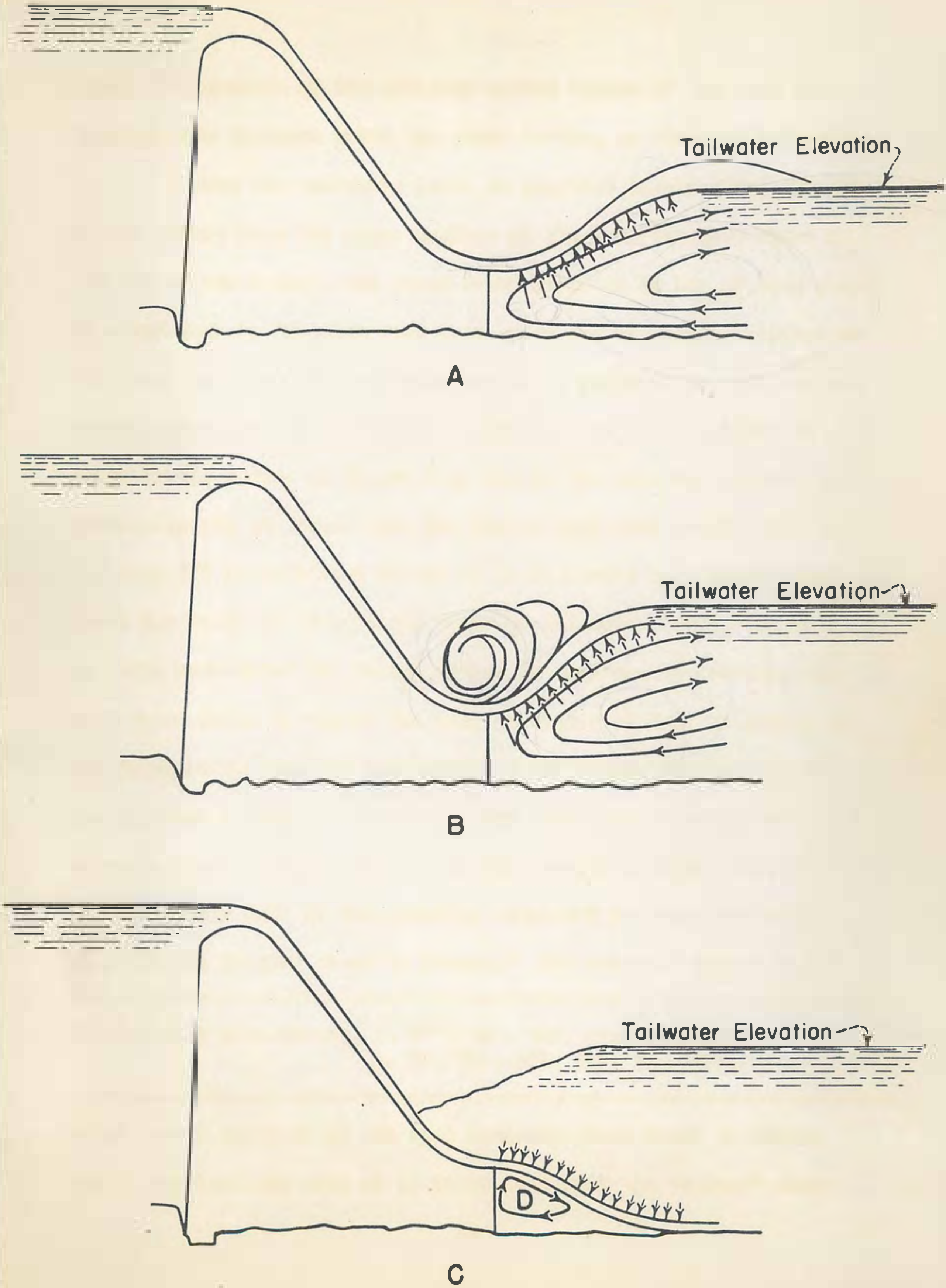
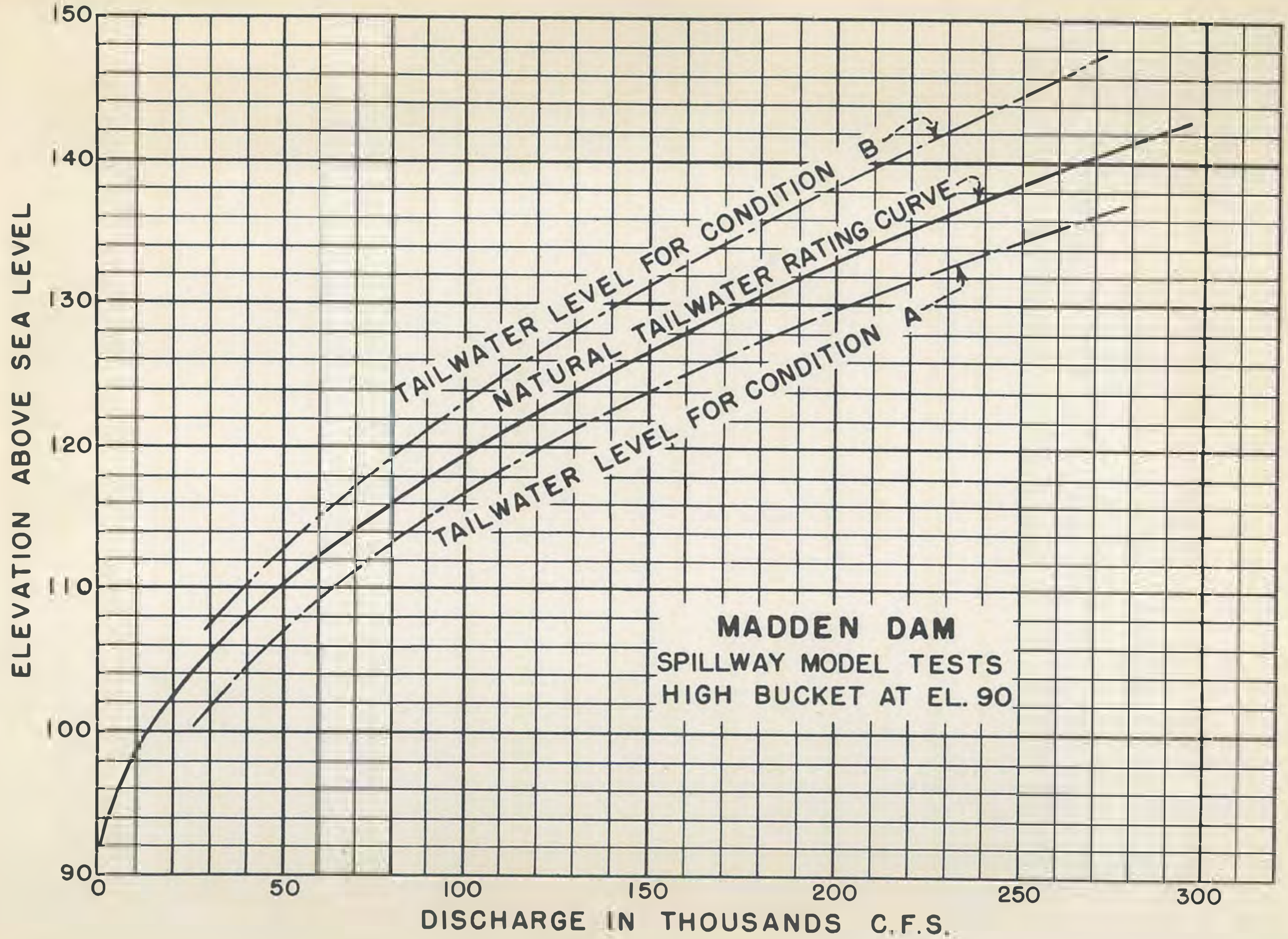


FIG. 5

X-D-1188

the face of the dam, as shown on Figure 5-B. This acts as a brake on the high velocity sheet and it does not sweep so high. The energy in this case is very thoroughly dissipated without attacking the stream bed. On the Madden Dam model the height of tailwater which would give these conditions for a bucket lip at elevation 90 was determined for the entire range of discharge. These tailwater heights were plotted against discharges and were found to be approximately parallel to the tailwater rating curve of the river as shown on Figure 6; that for condition A was roughly 3 feet below the tailwater curve of the river and that for condition B approximately 5 feet above. By lowering the bucket 5 feet the A curve would approximately coincide with the tailwater curve of the river, and a rise of 3 feet in the bucket would bring the B curve into agreement. Thus with a bucket at elevation 85 the A condition would occur at all discharges while at elevation 93 the B condition would occur at all discharges.

For the conditions at the Madden Dam, where there was on the river bed material which was sufficiently fine to be moved by the back eddy, these relations were found to be unstable, as the back eddy carried the fine material up toward the bucket and tended to build up a bar there, which cut off the back pressure from the tailwater and caused the high velocity sheet to flow more nearly horizontal. In this position the high velocity sheet tended to



TAILWATER LEVEL FOR CONDITION B
NATURAL TAILWATER RATING CURVE
TAILWATER LEVEL FOR CONDITION A
MADDEN DAM
SPILLWAY MODEL TESTS
HIGH BUCKET AT EL. 90
ELEVATION ABOVE SEA LEVEL
DISCHARGE IN THOUSANDS C.F.S.
90
100
110
120
130
140
150
0
50
100
150
200
250
300

FIG. 6

X-D-1189

carry along with it all the water in the eddy below it and built up a low pressure area at D of Figure 5, C in the place of the eddy. The force of the tailwater pressure then acted on the top side of the sheet, and deflected it downward against the bottom, which it struck with a considerable impact. With a bottom composed of solid rock or material of too large size to be moved by the back eddy, this condition would not occur.

It is believed that these flow conditions offer a method of scour protection below dams worthy of serious consideration for the condition of deep tailwater below overfall dams. Their appearance is not as satisfactory as the hydraulic jump on the sloping apron but they require much less concrete than the sloping apron and if the condition C can be avoided they bring even less scour on the bottom. In the Madden model the lip of the bucket was horizontal. If it was curved up somewhat as on the Dnieprostroy Dam it is probable that the C condition would be less likely to occur. It is not necessary to have only one of the two satisfactory conditions throughout all discharges, as one might be used for low flows and the other for high ones. This will often permit the matching of the river tailwater curve when it cannot be done with either one alone.

The conventional bucket with relatively short horizontal apron may be satisfactory for Class 2 conditions if the jump height curve (based on the apron level) is not too far below the tailwater rating curve, since the proper depth to form the jump may be reached on the slightly sloping portion of the bucket, and a relatively perfect jump be formed. If the difference between the two curves is great, the depth causing the formation of the jump will be reached on a steeply sloping surface in the bucket or on the steep face of the dam and the jump be very imperfect, with high velocity currents extending far downstream, necessitating a long apron. The length of the apron may be considerably shortened by the use of some forms of sill at its downstream end.

Class 3

The third case occurs where the tailwater depth at low discharges is insufficient to cause a jump to form on a level apron at river bottom level, but is more than sufficient at high discharges. This is a common case and the solution consists in artificially creating enough water depth to make the jump form on the apron at low discharges. This may be done by a low secondary dam or sill across near the downstream end of a level apron. This secondary dam must be high enough to cause the hydraulic jump to form upstream from it for all flows for which the natural tailwater depth is insufficient. It may need a secondary apron downstream from it for it is in fact just a second dam. The depth required may also be secured by depressing the apron, preferably by sloping it downward in a downstream direction. This method was used on the repair of the Hamilton Dam.*

*Engineering News, p. 130, Vol. 108, January 28, 1932.

This third case is usually favorable to the use of baffle piers or some form of dentated sill near the end of the apron, as these tend to break up the high velocity flow at low discharges and also to raise the tailwater, both of which actions promote the formation of the jump. They would also be advantageous at high flows since then the depth of tailwater is greater than required to form the jump and the nappe over the crest tends to dive down to the bottom of the river and flow along the apron at high velocity as previously described. When this condition occurs, the sills or piers are useful in breaking up this destructive current. For low intake dams it is not uncommon to have the height of tailwater at high flows sufficient to submerge the entire dam. In such cases the drop at the dam is slight and protection against these high flows is no problem.

Class 4

The fourth case, where the tailwater depth is sufficient at low flows but too small at high flows, can be solved by increasing the depth of tailwater sufficiently to cause the jump to form for the maximum discharge contemplated, either by a secondary dam or an excavated pool. With these of the magnitude required for the maximum flow, the tailwater depth would be more than sufficient to cause the jump to form on the apron at the lower flows but this condition would probably not be objectionable for low flows.

Effect of Changes in Dam Crest Length

To make the foregoing analysis as readily understandable as possible, it has been assumed that the stream flow was uniformly distributed over a fixed length of the dam and therefore for a given discharge there would be a fixed condition of overflow and a fixed tailwater elevation. By the proper selection of crest length, however, it may be possible to secure a closer agreement between the tailwater rating curve and the jump height curve than is secured by an arbitrarily chosen length. If the first assumed length produces a Case 1 condition, the agreement will be improved by increasing the crest length, which will cut down the discharge per foot length and consequently lower the jump height curve. Similarly, a Class 2 condition can be improved by decreasing the crest length, producing a greater discharge per lineal foot of crest and thus raise the jump height curve. Such changes may result in increases in the cost of crest gates or other features of the dam but this might be much more than offset by the reduced cost of bottom protection, and the possibilities of such a saving justify a study of this phase where the circumstances permit a choice of crest lengths.

Effect of Crest Gates

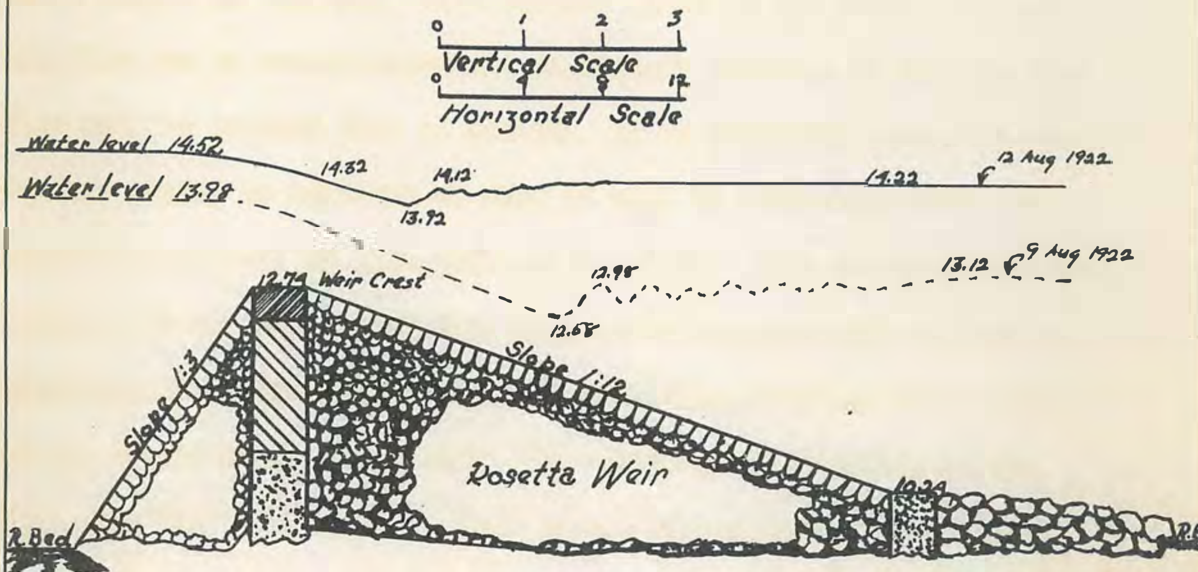
The foregoing analysis has also been simplified by assuming that the flow of the stream was uniformly distributed over the entire crest length of the dam. With control gates on the crest, however, the flow can be concentrated and at certain portions of the dam the flow will be greater than at others. To be perfectly safe, the protection should be designed so that it will be sufficient with any possible condition of gate openings and flow. As a practical matter, however, it may be assumed that reasonable judgment will be used in operating the gates, and sufficient protection provided so that no severe damage would result with the undesirable conditions acting for a limited time, such as might occur; for example, in the case of the failure of a gate to close when desired. In most cases it will be found best to have the crest gates designed to be capable of operating partially open, in order to distribute the water uniformly over the crest instead of permitting only an entirely open or entirely closed condition.

Protection for Weirs of Indian Type

The foregoing classification applies principally to protection below masonry overfall dams of the types commonly used in this country. For the broad weirs with slightly sloping aprons frequently used in India and Egypt, a somewhat different analysis is necessary. The oldest form of this type of weir consists of a

Rosetta Weir
Profile of Standing Wave

All Dimensions in Meters



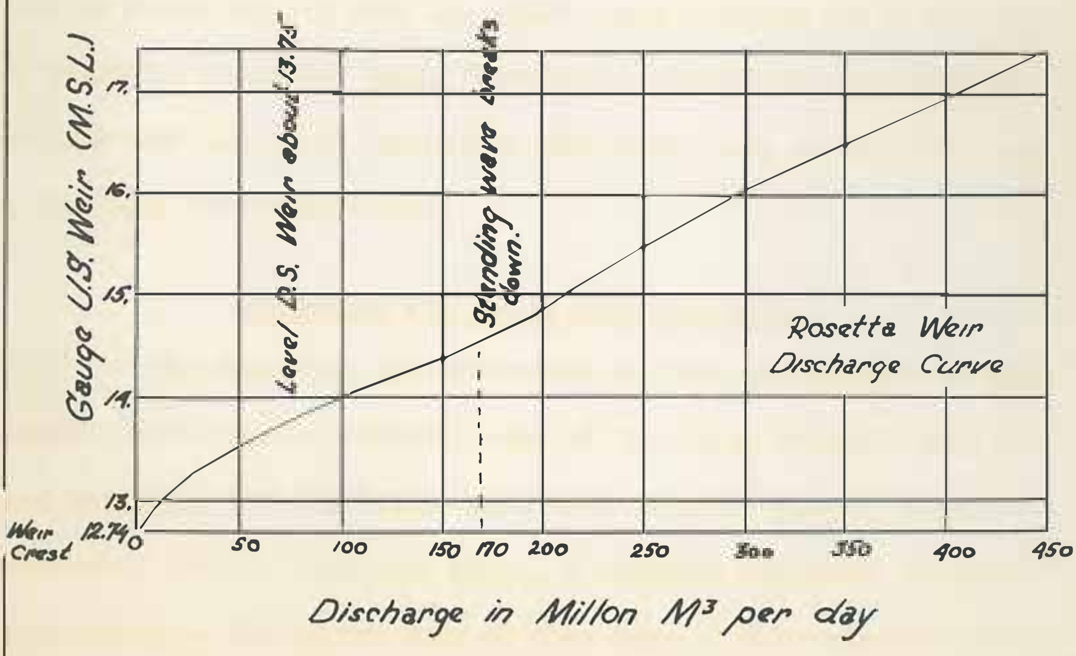


Fig. 7

X-D-1190

pile of loose rock, with or without masonry dividing walls, having a sloping upper surface paved with hand-laid rock or masonry, as shown on Figure 7. After passing over the crest of a weir of this type the water flows down the sloping apron until it encounters the tail-water, and at this point a hydraulic jump is usually formed. To prevent scour, this jump should form far enough from the downstream end of the apron that the turbulence of the jump does not reach to the unprotected river bed. To secure this result may necessitate the extension of the apron below the natural level of the river bed. It is necessary, or at least desirable, to have the point where the jump occurs move up the apron as the discharge increases, since this provides a longer length of apron to take care of the greater turbulence of the jump with the larger flows. It is also an advantage if the weir is entirely drowned out for very high flows, since this eliminates the combination of high discharge and fall which would produce so much destructive energy. These conditions exist at the Rosetta weir,* (Figure 7) which is at the head of the delta of the

*Standing Wave Weirs, by A. D. Butcher.

Nile. The surface profiles show that the position of the jump moves upstream as the discharge increases and for a flow of 170,000,000 cubic meters per day the jump is entirely drowned out. Although for many dams of this type the maintenance cost has been high, when conditions are favorable it may be very low, as indicated by the maintenance charge of 0.4% for a 15-year period at the Laguna Dam on the Colorado River,* all of which was due to a cutoff of the river just

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*New Reclamation Era, p. 189, December, 1924.

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downstream from the weir, which lowered the tailwater level 7 feet and necessitated an extension of the apron.

With this type of weir the possibilities of increasing the tailwater depth are rather limited. It is therefore unsuited to conditions having a low tailwater level and two complete failures are believed to have been due to tailwater levels so low that the hydraulic jump occurred too near to or below the downstream edge of the masonry, causing an undermining of the structure which progressed upstream and caused a breach. An adjustment of the dam to fit the tailwater conditions may sometimes be made by a proper choice of the dam length, as previously discussed.

Most of the recent dams of this type in India and Egypt have been constructed with broad, sloping masonry aprons surmounted with piers and Stoney gates. The foregoing discussion applies to this kind also, although it is complicated somewhat by the necessity of having sufficient tailwater depth to cause the jump to form on the apron with the gates in a partially open condition. To secure this it may be necessary to carry the apron below natural bed level.

If the river carries much bed load, allowance should be made in computing the position of the jump for a retrogression of the downstream river level.

Tailwater Rating Curves

Enough has been given in the foregoing to show the importance of the tailwater rating curve in the solution of this problem. One of the first steps in attacking the problem at any site is to determine the tailwater rating curve, either by observing the actual levels for a wide range of discharges or by computation. The former method should be used if possible, but fairly satisfactory results may be secured by determining the tailwater elevation for various discharges by means of backwater curves for these flows. Since the water levels at downstream points for these discharges are probably also unknown, the curves may be started at assumed elevations far enough downstream from the dam that the error in the assumed elevation will have disappeared before the curve has been computed as far upstream as the dam. That this is the case may be determined by assuming somewhat different elevations for the same discharge at the lower end of the stretch and if both assumptions give practically the same elevation at the damsite, the error introduced by an incorrect assumption is negligible.

Laboratory Experiments Necessary

It is not the intention of this article to give the impression that hydraulic laboratory tests are unnecessary in working out the best form of scour protection below dams. Such tests should be made on all important structures and will usually pay for themselves in the improvements which they bring about in the minor features of the design, entirely aside from the major improvements which they make possible. The intention of this analysis is only to point out the lines along which the best solution probably lies, in order that effort may not be wasted in unnecessary investigation.
