UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

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

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TECHNICAL MEMORANDUM NO. 406

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Charles W. Thomas

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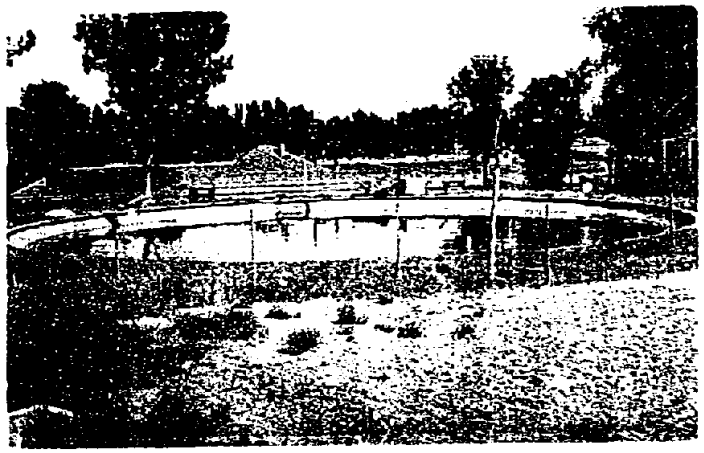
The studies described in this report were made in the laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado, by the Hydraulic Research Division of the United States Bureau of Reclamation with Jacob E. Warnock, Associate Engineer, as resident engineer. During the greater portion of the period of tests he was assisted by Grover J. Hornsby, Assistant Engineer. The tests on the model of the Norris Dam were conducted under the immediate supervision of Charles W. Thomas, Junior Engineer, and this report was prepared by him. He was assisted in the tests of the models and preparation of the report by Robert A. Goodpasture, 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.

These 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.

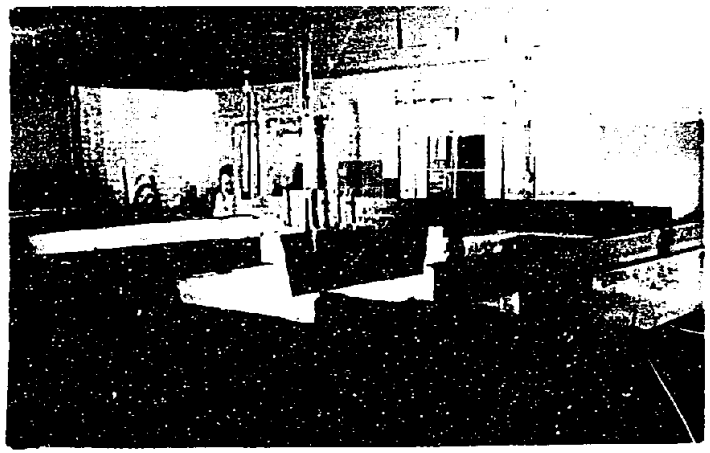
It is desired also to acknowledge the data and suggestions furnished by Mr. Carl Rohwer, Associate Engineer, Bureau of Agricultural Engineering, U. S. Department of Agriculture, on the least square adjustment of experimental data as outlined in Appendix ~~III~~ II.

It is further desired to acknowledge the cooperation of the Colorado State Board of Agriculture (governing board of 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 Norris Dam.

A. THE BUILDING.



B. THE SUPPLY RESERVOIR



C. MEASURING WEIR AND CHANNEL.

D. INTERIOR VIEW

THE HYDRAULIC LABORATORY
OF THE COLORADO AGRICULTURAL EXPERIMENT STATION.

I. SYNOPSIS

The determination of the best form of protection against scour at the toe of the Norris Dam, being built by the Tennessee Valley Authority in the Clinch River approximately 20 miles northwest of Knoxville, Tennessee, (Figs. 1 and 2) 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, for the Tennessee Valley Authority.

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

Besides the studies on the design of the stilling pool, extensive tests were made to determine the proper shape of the sluice gate outlets to provide satisfactory flow conditions in the stilling pool with the sluices operating either alone or with the spillway gates.

Piezometers installed in the upstream face of one of the hydraulic drum gates gave pressure data of value to the design office. Additional piezometric openings in the spillway training walls gave pressure data otherwise not available.

FIGURE 1

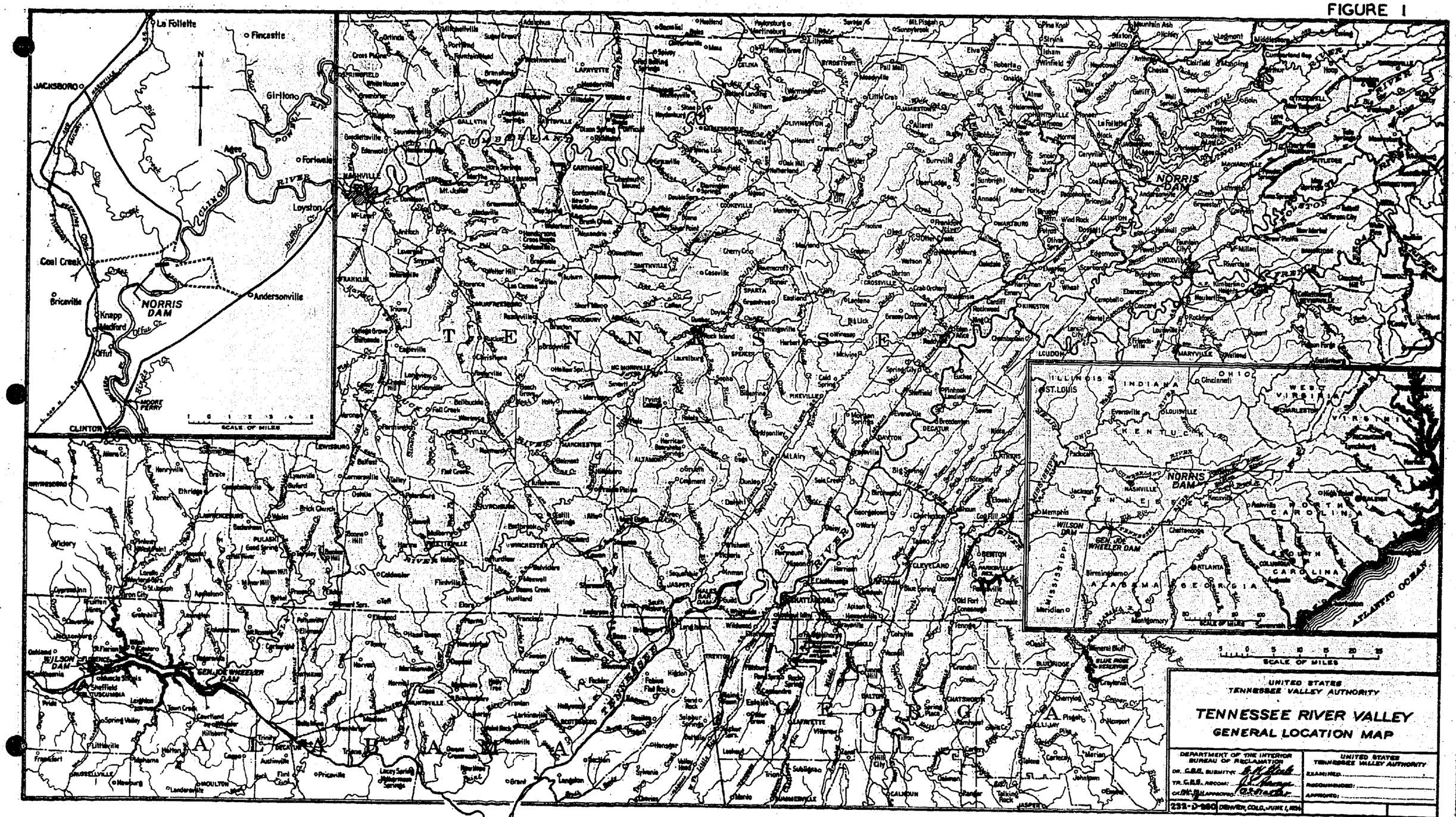
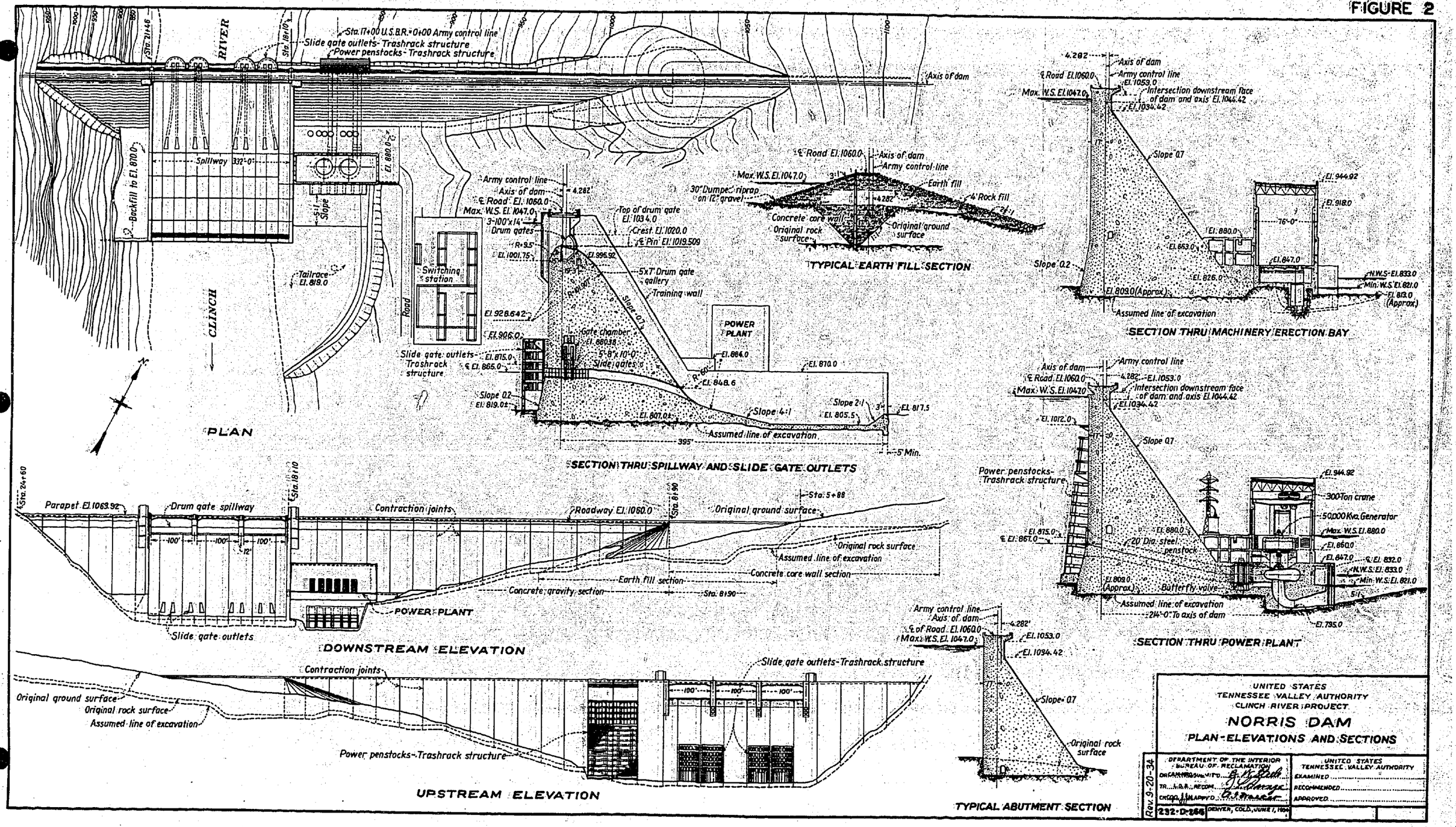

UNITED STATES
TENNESSEE VALLEY AUTHORITY
TENNESSEE RIVER VALLEY
GENERAL LOCATION MAP
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
UNITED STATES
TENNESSEE VALLEY AUTHORITY
232-D-860
DENVER, COLO., JUNE 1, 1934
SCALE OF MILES
NORRIS DAM
WILSON DAM
GEN. JOE WHEELER DAM
CHATTANOOGA
KNOXVILLE
NASHVILLE
MARYVILLE
LOUDON
CLINTON
ATLANTA
ATLANTIC OCEAN

FIGURE 2


PLAN
RIVER
CLINCH
Sta. 17+00 U.S.B.R. = 0+00 Army control line
Slide gate outlets - Trashrack structure
Power penstocks - Trashrack structure
Axis of dam
Backfill to El. 870.0
Spillway 332'-0"
Tailrace El. 819.0
Switching station
Road
Slope
Army control line
Axis of dam
℄ Road El. 1060.0
Max. W.S. El. 1047.0
3-100'x14' Drum gates
Top of drum gate El. 1034.0
Crest El. 1020.0
℄ Pin El. 1019.509
5x7' Drum gate gallery
Training wall
Gate chamber
5'-8"x10'-0" Slide gates
Slide gate outlets - Trashrack structure
POWER PLANT
Slope 0.2
Slope 0.7
Slope 4:1
Slope 2:1
Assumed line of excavation
395'
5' Min.
SECTION THRU SPILLWAY AND SLIDE GATE OUTLETS
30" Dumped riprap on 12" gravel
Concrete core wall
Original rock surface
Original ground surface
Earth fill
4' Rock fill
TYPICAL EARTH FILL SECTION
Intersection downstream face of dam and axis El. 1044.42
Slope 0.7
Slope 0.2
Assumed line of excavation
SECTION THRU MACHINERY ERECTION BAY
N.W.S. El. 833.0
Min. W.S. El. 821.0
Power penstocks - Trashrack structure
300 Ton crane
50,000 Kva. Generator
20' Dia. steel penstock
Butterfly valve
2½'-0" To axis of dam
SECTION THRU POWER PLANT
Parapet El. 1063.92
Drum gate spillway
Contraction joints
Roadway El. 1060.0
Original ground surface
Original rock surface
Assumed line of excavation
Concrete core wall section
Earth fill section
Concrete gravity section
Sta. 8+90
POWER PLANT
Slide gate outlets
DOWNSTREAM ELEVATION
Slide gate outlets - Trashrack structure
Power penstocks - Trashrack structure
UPSTREAM ELEVATION
Slope 0.7
Original rock surface
TYPICAL ABUTMENT SECTION
UNITED STATES
TENNESSEE VALLEY AUTHORITY
CLINCH RIVER PROJECT
NORRIS DAM
PLAN - ELEVATIONS AND SECTIONS
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
UNITED STATES
TENNESSEE VALLEY AUTHORITY
EXAMINED
RECOMMENDED
APPROVED
Rev. 9-20-34
232-D-266
DENVER, COLO., JUNE 1, 1934

A complete calibration of the 14 feet by 100 feet hydraulic drum gates and a study of the operation of those gates in combination with each other and with the sluice gates resulted in a gate discharge diagram and a sluice and spillway gate operating program for use by the plant operating engineer.

II. INTRODUCTION

The Norris Dam, a concrete gravity type dam 263 feet high, will provide a storage reservoir with a capacity of 3,600,000 acre-feet for the production of power, the control of destructive floods and the improvement of navigation.

The spillway of this structure will be of the overflow, ogee type with a designed capacity of 200,000 second-feet.

The flood waters, with a probable maximum discharge of 240,000 second-feet and a fall of 246 feet, could create a maximum of 4,520,000,000 horsepower. With energy of this magnitude involved, such that the foundation of the dam would be destroyed if these forces were not rendered harmless, it

was of great importance that the design of the dam and particularly the stilling pool be structurally sound. To obtain such a design it was deemed advisable to make detailed hydraulic studies on a model of the structure based on the laws of hydraulic similitude.

The design data from which this model was constructed was obtained from the design office of the U. S. Bureau of Reclamation, Denver, Colorado. Frequent reference was also made to the "Economic Height of Norris Dam" by E. B. Debler, Hydraulic Engineer, U. S. Bureau of Reclamation, a summary of which is shown in Table 1, and to House Document No. 328, 71st Congress, 2nd Session, "The Tennessee River and Tributaries, North Carolina, Tennessee, Alabama and Kentucky".

III. APPARATUS

a. Hydraulic Laboratory

The experiments for the design of the Norris Dam were conducted in the hydraulic laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado, a general layout of which is shown, together with the location of the model, on Figure 3.

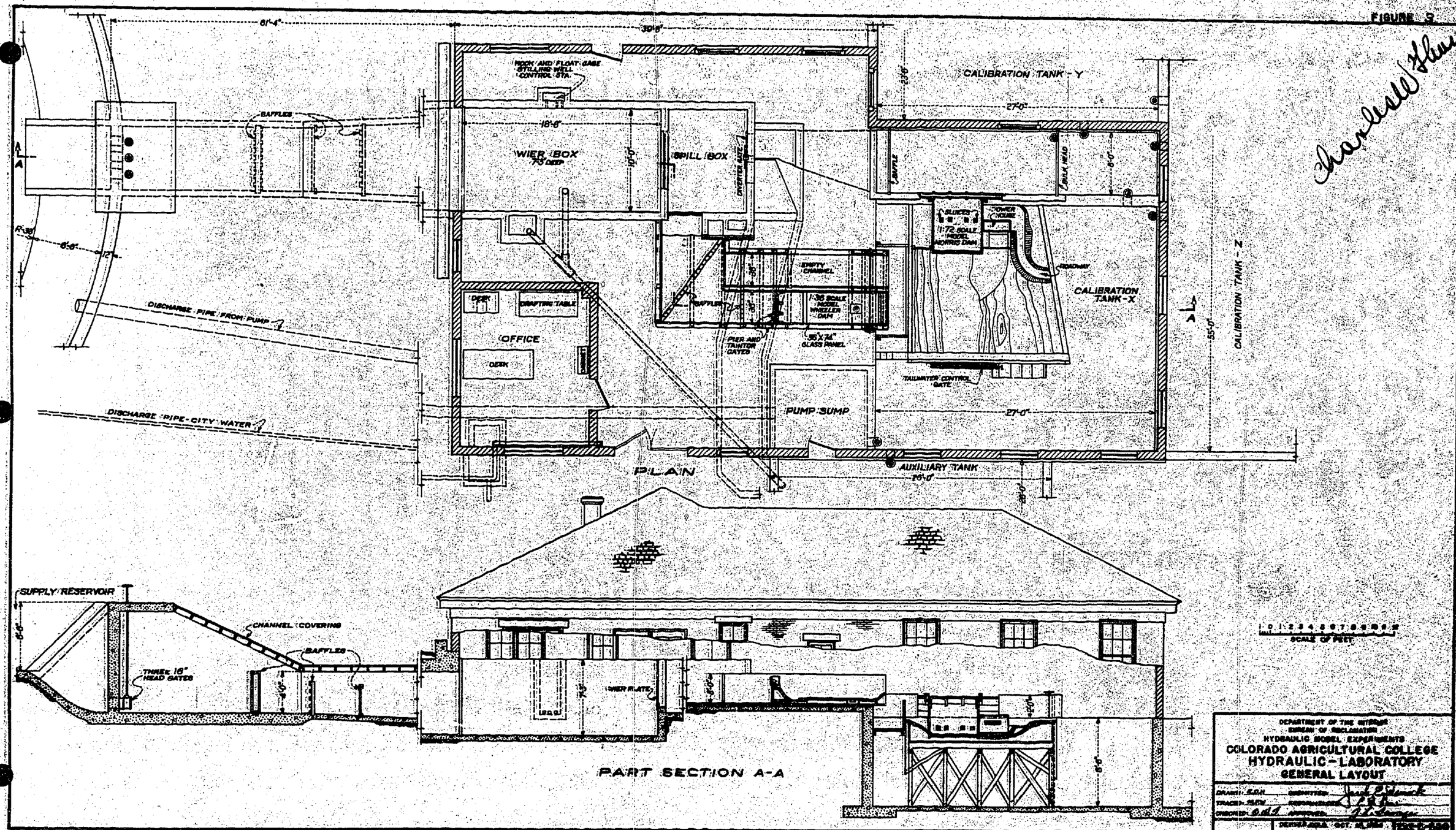
The flow of water to operate the model was obtained from a reservoir of 30,000 cubic-feet capacity, located upon a hill behind the laboratory. The flow from the reservoir,

TABLE NO. 1

NORRIS DAM PROJECT DATA

Permanent Spillway Crest	Elev. 1020
Length of Spillway	300 feet
Top of Drum Gates (Raised)	Elev. 1034
Area of Reservoir (Elev. 1050)	51,254 acres
Power Heads	120 to 203 feet
Tailwater Elevation	
10,000 Second-feet.	Elev. 828.5
240,000 Second-feet.	Elev. 880
Top of	
Dead Storage	Elev. 948
Power Storage	Elev. 1014
Ordinary Flood Storage	Elev. 1044
Extraordinary Flood Storage	Elev. 1052
Storage Capacity (acre-feet)	
Dead Storage	500,000
Power Production	1,500,000
Extraordinary Flood Control	400,000
Ordinary Flood Control	1,200,000
Total (Elev. 1052)	3,600,000
Spillway Capacity	
(Reservoir Elevation 1052)	
Gates Down (Elev. 1020)	205,000 sec.-ft.
Gates Up (Elev. 1034)	87,000 sec.-ft.
Sluice Gate Capacity	
Water Surface Elev. 1020	36,000 sec.-ft.
Drainage Area, Tributary Water Shed	2,950 sq. mi.
Annual Run-off 1,700,000 acre-feet	to 4,830,000 acre-feet.
Average Annual Run-off	3,300,000 acre-feet.
Daily Discharges, 330 sec.-ft. (1919)	to 110,000 sec.-ft.(1886)

FIGURE 3



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HYDRAULIC MODEL EXPERIMENTS

COLORADO AGRICULTURAL COLLEGE
HYDRAULIC - LABORATORY
GENERAL LAYOUT

DRAWN: S.G.H. SUBMITTED: [illegible]
TRACED: S.G.H. RECOMMENDED: [illegible]
CHECKED: [illegible] APPROVED: [illegible]
DENVER, COLO. OCT. 26, 1936 — 222-D-556

Thornell Flumes

controlled by hand-operated gates, passed through a diverging flume into a concrete weir channel, 19½ feet long, 10 feet wide and 7 feet 3 inches deep. In the side of the weir channel and 13 feet upstream from the weir was a by-pass gate and 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 with the main channel by a pipe. The weir setting and weir are illustrated in the frontispiece. The model discharge was measured over a two-foot Cipolletti weir which had been previously volumetrically calibrated in its present setting by the laboratory staffs of the Colorado Agricultural Experiment Station and Bureau of Reclamation.

From the weir the water passed through a diverting gate by means of which flow could be quickly diverted from the waste tank into the forebay of the model, which was a channel 6 feet wide, 18 feet 8 inches long and 10½ feet deep. As the direction of flow in the model was at right angles to that in this channel, baffles were necessary to prevent boils and whirls in the water over the model. After experimentation, a set of baffles was devised which eliminated practically all

of these disturbances. After passing over or through the model the water dropped to the bottom of the tank and was pumped to the supply reservoir.

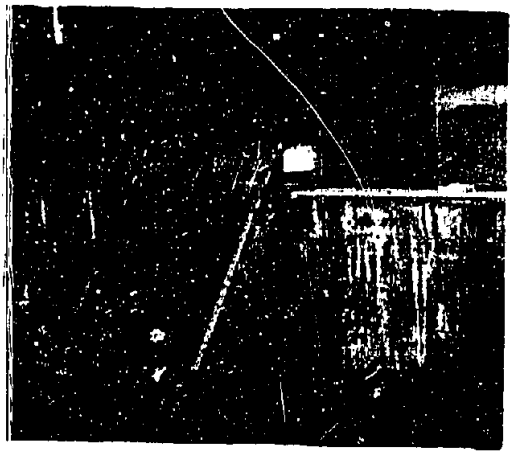
b. The Model

The model, built on a scale ratio of 1:72, was set up in a calibration tank inside the laboratory (Fig. 3). 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 = 72.

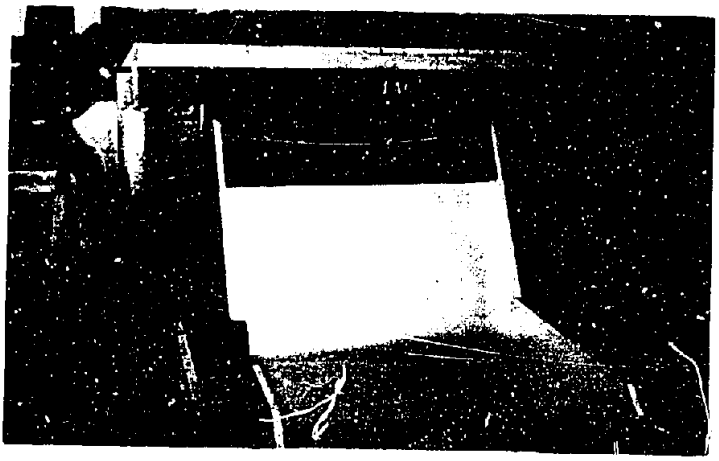
Ratio of Prototype to Model.

Quantity	Formula	Value
Linear Dimensions	N	72.
Head	N	72.
Velocity	$\sqrt{N}$	8.485
Area	N^2	5,184.
Discharge	$N^{5/2}$	43,986.24
Time	$\sqrt{N}$	8.485

The model of the dam, built with a minimum amount of wood, was located on top the concrete wall that formed one side of the approach channel (Figures 4 and 6 and Plate 1 A). The upstream face of the dam was formed by the concrete wall and a 20-gage metal plate screwed to a wooden header on the wall and iron bents supporting the model.



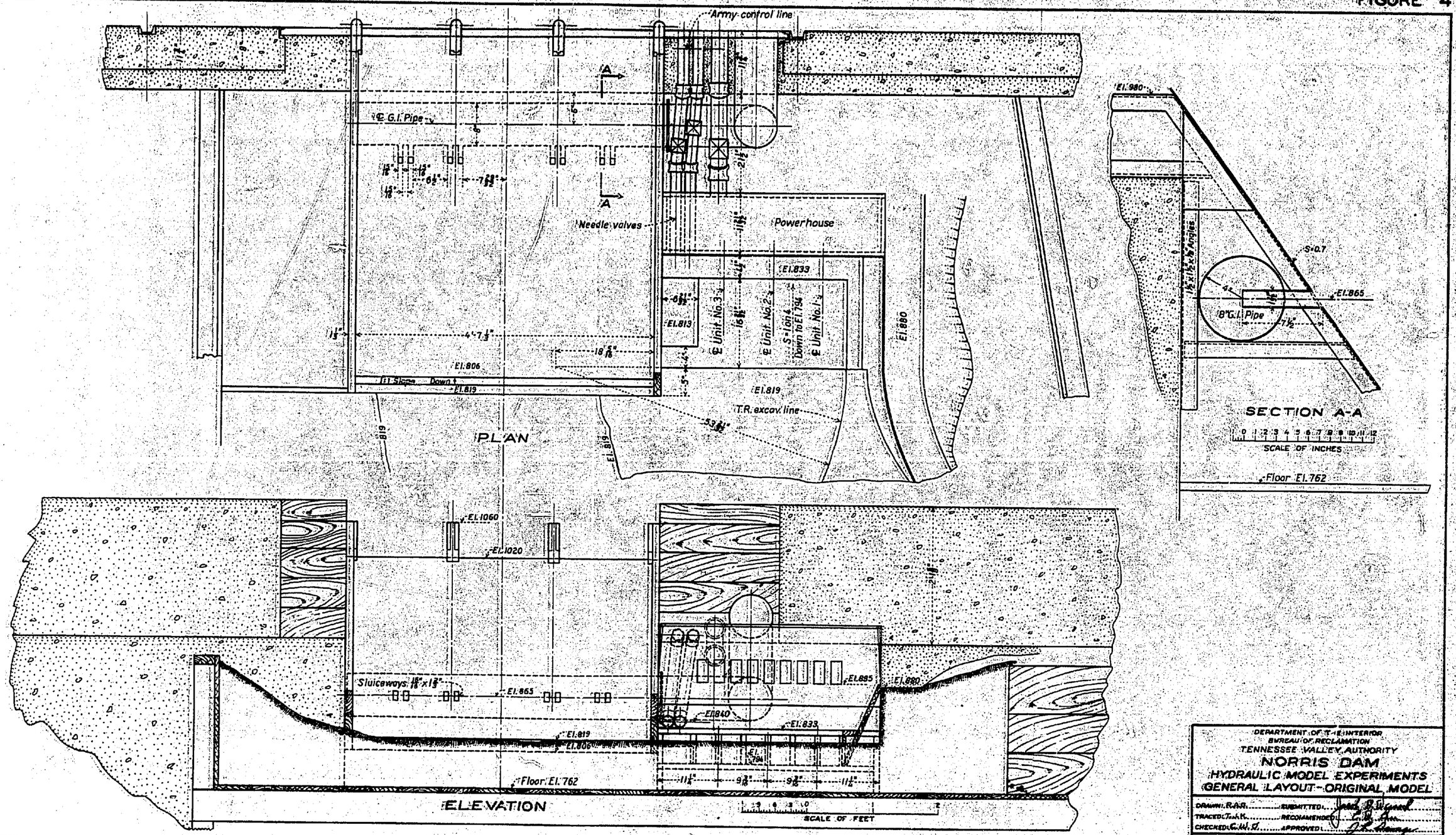
A. MODEL UNDER CONSTRUCTION.



B. COMPLETED MODEL.

SLOPING APRON TO LEVEL APRON ELEV. 810.
SLOPE SILL TO ELEV. 819 AT END OF APRON.

FIGURE 4



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

NORRIS DAM

HYDRAULIC MODEL EXPERIMENTS

GENERAL LAYOUT - ORIGINAL MODEL

DRAWN: R.A.R. SUBMITTED: [illegible]
TRACED: T.A.K. RECOMMENDED: [illegible]
CHECKED: C.W.S. APPROVED: [illegible]

DENVER, COLO., OCT. 15, 1934 232-D-694

The crest section, piers and training walls were made of redwood. The crest was surmounted by mechanically operated gates, made of 16-gage galvanized iron, simulating hydraulically operated drum gates. The downstream face and apron were of 20-gage sheet metal screwed to the angle iron supporting bents. The powerhouse superstructure and substructure were built of redwood and the former was covered with masonite. Upon completion, the entire model was carefully finished and painted with extra bright aluminum paint.

The rock topography downstream from the dam was reproduced in a wooden tank supported on 4-inch by 4-inch columns, by inserting 27-gage galvanized iron cross-sections cut to conform to the topography. Wet sand tamped between the sections to within an inch of the top formed the base for the concrete coating. The overburden was formed of sand and fine gravel. The tailwater was regulated by a set of fixed and movable louvers (Plate 2 A and B).

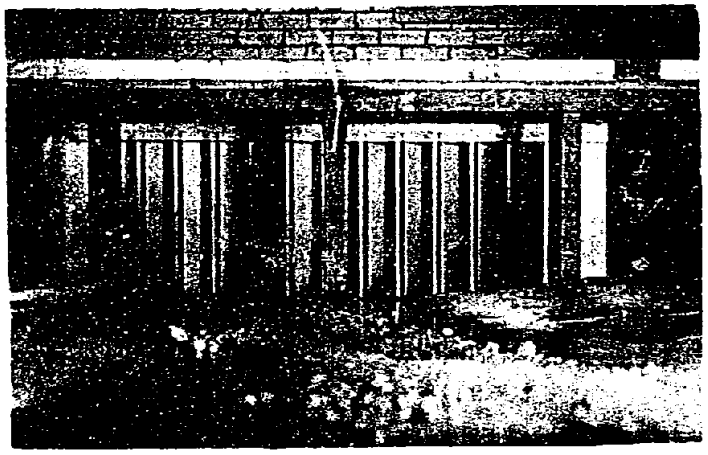
IV. PROCEDURE

a. General

The investigations on the model of the Norris Dam are divided into three main subjects; spillway studies, studies of auxiliary passages and architectural studies.

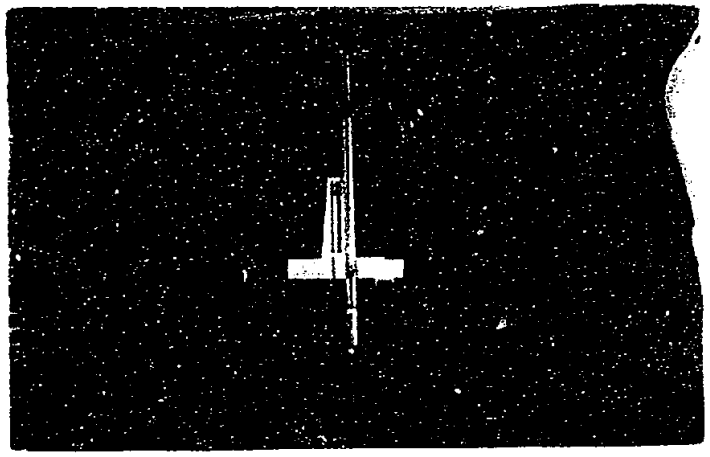
The spillway studies were divided into: stilling pool characteristics, calibration of drum gates, pressures

A. OPEN POSITION.

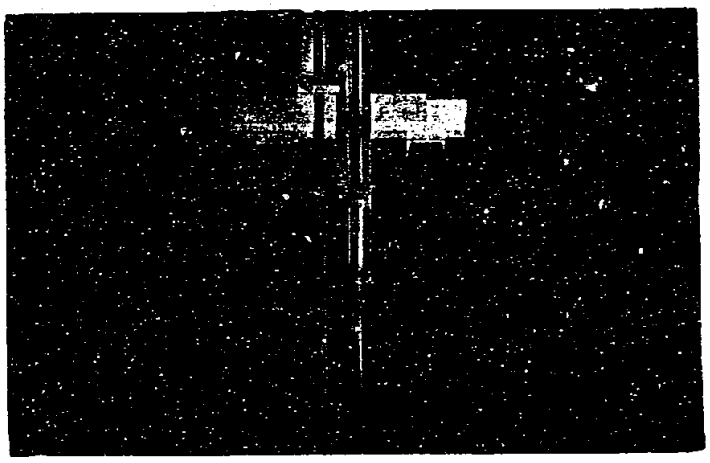


B. CLOSED POSITION.

TAILWATER REGULATOR.



C. ENTIRE INSTRUMENT.



D. CLOSE-UP.

PITOT TUBE.

TAILWATER REGULATOR AND PITOT TUBE.

PLATE 2

on drum gates, measurements of velocities on apron, design of spillway training walls and gate operating program.

The studies of auxiliary passages consisted of studies of sluiceways through the dam with an operating program, needle valves through the powerhouse and the powerhouse.

The architectural studies on the piers and highway bridge across the spillway were carried on in conjuntion with the other work.

b. Spillway Studies

1. Stilling Pool Characteristics.

It has been mentioned previously that the design of the stilling pool for the Norris Dam is of vital importance to the safety of the structure due to the vast amount of energy that must be dissipated.

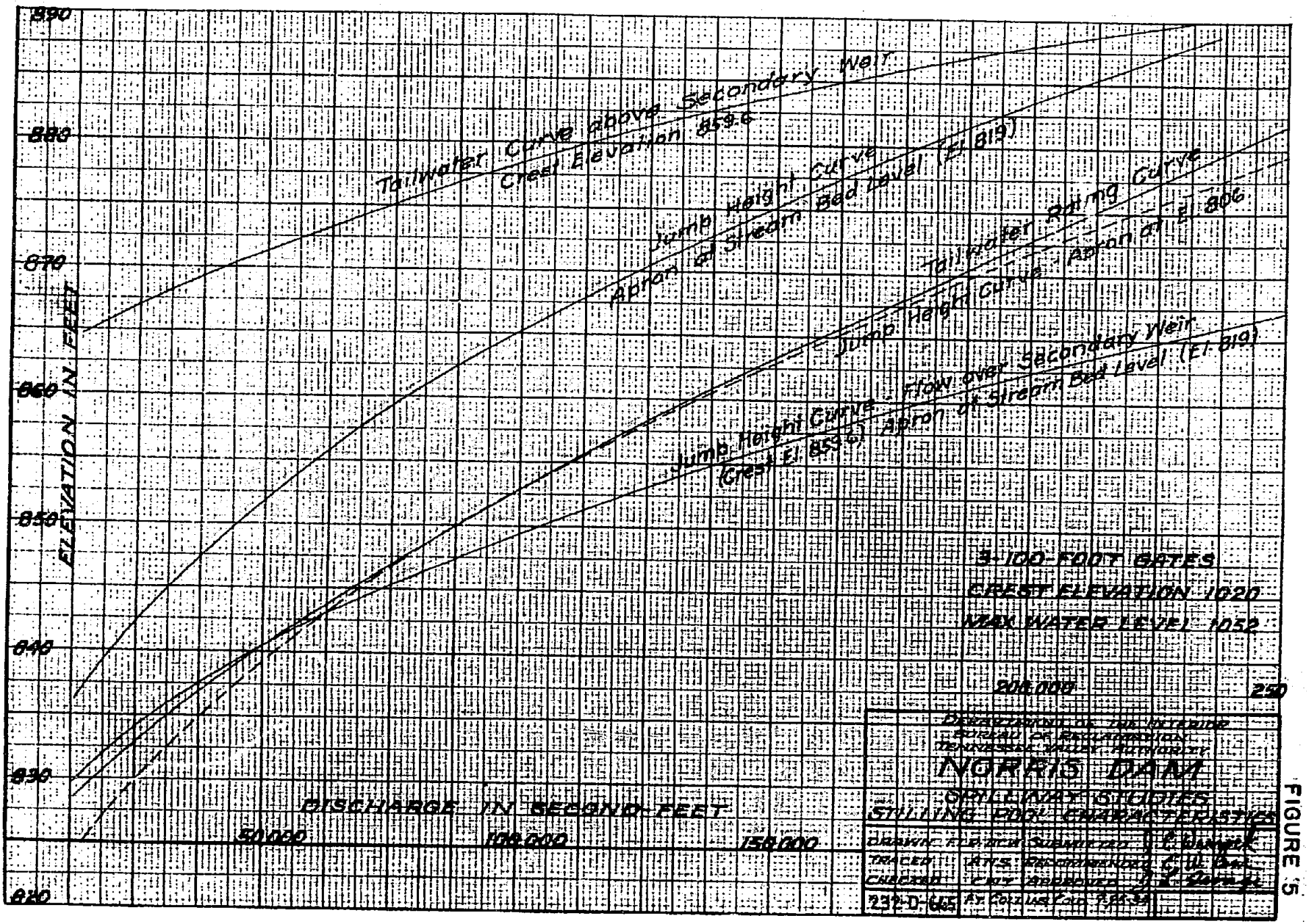
Until recently the principles of protection against scour below dams has not been completely understood. A determination for each individual case by means of model testing has rapidly grown into use, but the principles involved were not in all cases well considered and the results of these tests were therefore not as reliable as otherwise might have been the case. As a result of the studies by the U. S. Bureau of Reclamation in connection with the Cle Elum, Madden 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 are applicable to the conditions at a given dam site, but the classification of principles narrows the field which it 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" a copy of which is attached to this report as Appendix I. Briefly stated, the type of scour protection depends upon the relationship, throughout the entire range of discharge, of the tailwater elevation 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 and the jump-forming height curve are given in Appendix I.

The conditions at Norris Dam are very favorable to the protection from scour by simple means. The rating curve for the tailwater below the dam (Figure 5) was determined by Mr. Frank Campbell, under the direction of Mr. E. B. Debler, according to the methods suggested by Prof. S. M. Woodward. The height of water necessary to form the jump on an apron at stream bed level (Figure 5) is based on the assumption that all the gates are held at the same elevation.

As will be seen by Figure 5 the jump height curve for the apron at stream bed level is considerably above the tailwater rating curve for all discharges. This dam, therefore falls into Class I described in 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. The first mentioned of these solutions with the stratified rock bottom and the very high heads acting at the Norris Dam would produce severe scour in the river bed downstream from the dam and pile up debris in the stream channel below the dam which would probably obstruct the flow from the powerhouse. Since a very good solution is possible by another one of the



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NORRIS DAM

SPILLWAY DISCHARGE CHARACTERISTICS

DRAWN	R.E.R.	SUBMITTED	
TRACED		RTS RECOMMENDED	
CHECKED		CWT APPROVED	
232-D-665		AS POSTULATED 7-25-34	

FIGURE 5

four methods, this method, which would have undesirable results was not given serious consideration. Satisfactory protection from the hydraulic standpoint could undoubtedly be secured by means of the tailwater weir mentioned in the second solution. The secondary weir, when used, must be high enough to produce the required tailwater level for all discharges. Figure 5 shows that a weir with crest at Elev. 859.6 would be required to hold the water to the height necessary for a discharge of 250,000 second-feet. Such a dam would be about 40 feet high and would require training walls 70 feet high along both sides of the pool. Moreover, the pool would have to be of considerable length, $4\frac{1}{2}$ to 5 times the height of the jump as this is the length which is required to produce satisfactory conditions where the height of the secondary weir is an appreciable part of the pool depth. Some protection against scour would also be required below the secondary weir, but since (as shown on Figure 5) the natural tailwater curve is nearly high enough to produce a jump below the secondary weir on an apron at stream bed level, little in addition to the horizontal apron below the secondary weir would be required. The cost of the 40-foot tailwater weir with its apron and 70-foot retaining walls, however, is so great that this form of protection although

undoubtedly satisfactory from the hydraulic standpoint would be entirely too expensive.

The use of baffles of the ordinary forms to dissipate the energy of the water passing over the Norris Dam is not favored because of the unprecedented velocities which would act on them and the difficulty of building them strong enough to resist the blows to which they might be subjected by logs. The large difference in height between the tailwater rating curve and the curve for height of jump on an apron at stream bed level indicates that the energy which would have to be dissipated by any form of baffle would be large and therefore they would be subjected to large impacts. Several forms of baffles were tested, however, in an attempt to shorten the length of depressed apron downstream. The best form of protection is secured by deepening the pool by excavation, placing the pool bottom at Elev. 806, or 13 feet below the stream bed level. For this condition the depth of pool is sufficient to form a jump at all flows. The close agreement between the jump height curve and the tailwater curve indicates that nearly perfect hydraulic jump would be formed in which the energy would be efficiently dissipated.

The jump height curve is computed on the assumption that no friction exists to retard the velocity of the water passing over the dam. This, of course, is not true, but as

the amount of this friction would be uncertain, it is best to make the action sure, whatever its magnitude. The existence of the friction will provide a factor of safety in case the actual tailwater level proves to be somewhat lower than predicted. An additional factor of safety is provided by the slope up from pool bottom to river bottom level, which tends to deflect the currents upward and raise the water surface at that point, producing the same effect as a higher tailwater level. If the velocity of the water approaching this sill at the end of the pool increases, the rise of the water surface over it also increases and the tendency of the pool to be swept out by the inflowing water is more strongly opposed.

The action of the pool as shown on Figure 5 is based on the assumption that the gates are kept at the same elevation. For the lower discharges this is not so important, since the pool is deeper than necessary for this condition and some unbalancing of the gate discharge would be permissible.

From this preliminary analysis of the conditions, the range of model tests was narrowed appreciably. Although the theoretical analysis showed an apron at Elev. 806 necessary to form the jump, the first apron tried was placed at Elev. 810 and top of sill at end of apron at Elev. 819.

Throughout the series of tests on various aprons, a representative number of runs were made on each test. Model flows representing 20,000, 40,000, 80,000, 120,000, 160,000, 200,000 and 240,000 second-feet on the prototype were used. The flows were accurately measured by means of the two-foot Cipolletti weir and held constant throughout the run. The head over the crest was regulated by means of the drum gates in all except the first six tests, which were made before the drum gates were installed in the model. With the exception of the last few tests, the flows were regulated as follows: 20,000, 40,000, and 80,000 second-feet with all gates held at Elev. 1034; 120,000 and 160,000 second-feet with all gates set so as to hold the reservoir at Elev. 1052; 200,000 second-feet with all gates at Elev. 1020; 240,000 second-feet with all gates at Elev. 1020, flow divided 200,000 over spillway and 40,000 through sluiceways. The last tests were made by allowing 40,000 to pass through the sluiceways at all flows and regulating the flow over the spillway to 20,000, 40,000 and 80,000 second-feet with all gates held at Elev. 1034; 120,000 and 160,000 second-feet with all gates set at the correct elevation to hold the reservoir at Elev. 1052; 200,000 second-feet with all gates at Elev. 1020, making total quantities of 40,000 (sluices only discharging), 60,000, 80,000, 120,000, 160,000, 200,000 and 240,000 second-feet respectively. This change was made, as will be explained later, to follow the operating program for

the project as recommended from the model tests.

The tailwater was regulated by means of the movable louvers (Plate 2) to the correct height for the quantity passing the dam as indicated by the tailwater curve on Figure 5 and held constant throughout the run. With the tailwater set at the correct elevation as indicated by the float gage connected to a piezometer opening in the river bed, the action of the stilling pool was recorded pictorially and by point gage readings. The point gage consisted of a single horizontal bar located above the center line of the spillway and perpendicular to the axis of the dam. On this bar was a movable slide arranged to traverse the length of the spillway apron from the downstream face of the dam to a point well beyond the end of the apron. This slide was fitted with a graduated staff, reading in feet, tenths and hundredths, and sliding vertically. By estimating the thousandth fraction, this staff was used to obtain the water surface on the model to the nearest thousandth of a foot. The results were transferred to the prototype values for plotting. The 40,000, 80,000, 120,000, 160,000 and 200,000 second-feet runs were the only ones profiled by means of this point gate. Pictures or notes were used as records for the other flows.

For the first six tests the powerhouse, drum gates and sluiceways through the dam were omitted from the model and a sand box somewhat wider than the spillway model and extending 10 feet (model) downstream from the end of the apron was used instead of the regular topography. These omissions and substitutions were made in order to expedite the preliminary tests on the model and to determine the relative amount of scour of the river bed downstream from the end of apron for each set-up. After the powerhouse and sluiceways were installed they were changed from time to time as the designs were altered either by the Denver design office or hydraulic laboratory staff.

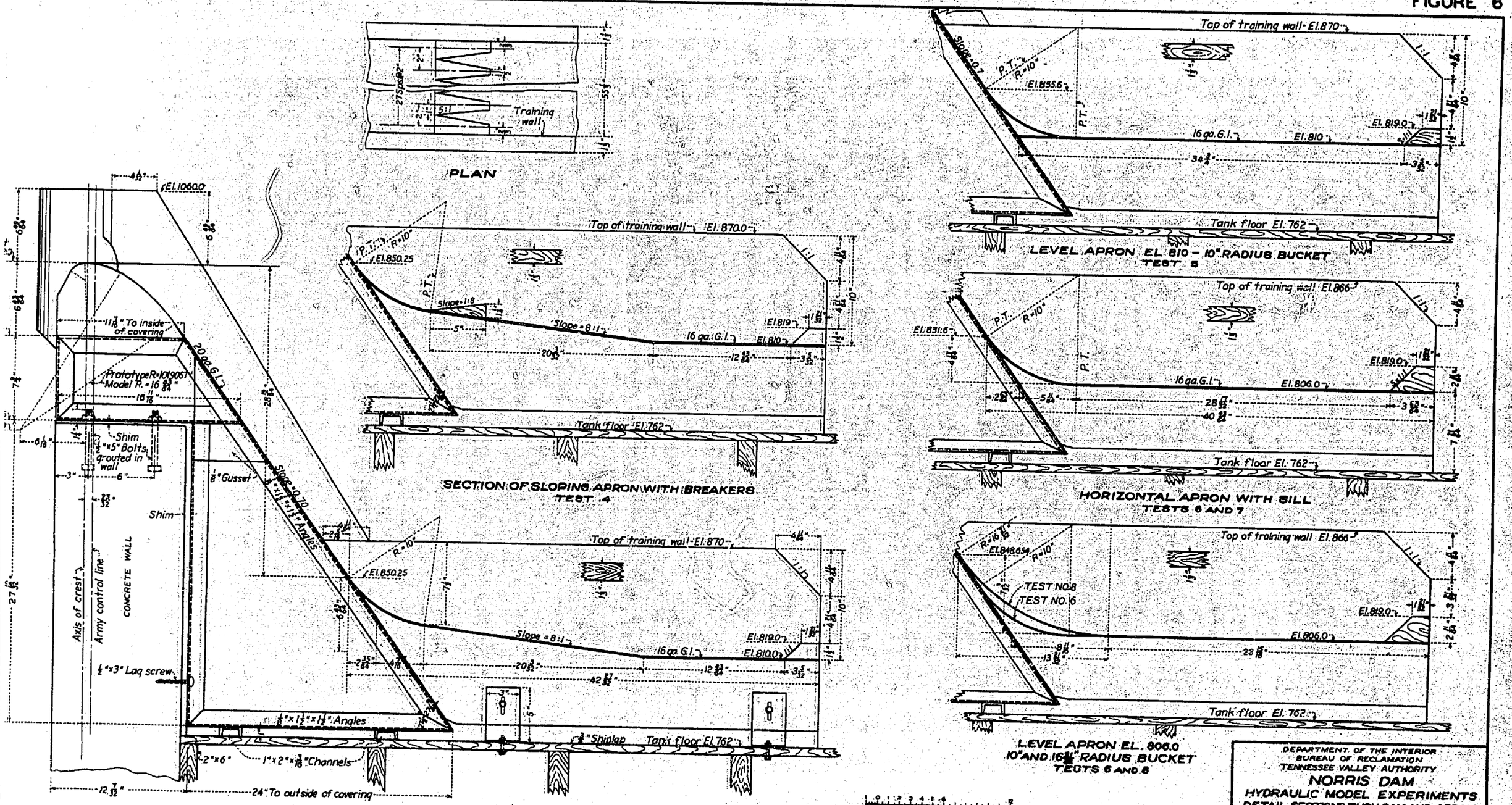
The apron as originally installed consisted of a flat section at Elev. 810 and a sloping portion on an 8:1 slope connected to the face of the dam by means of a 60-foot radius bucket. A detailed layout is shown on Figure 6, Tests 1, 2 and 3.

Before the 8:1 slope was removed from the apron a set of dentated breaker blocks (Figure 6, Test 4) was placed on the slope and tested.

The sloping portion of the apron was next removed and the 60-foot radius bucket (Figure 6, Test 5) moved down to connect with a flat apron at Elev. 810.

All three of the foregoing set-ups were tested with tailwater held at the normal elevation and also with the tailwater held at a level 4 feet above normal which simulated

FIGURE 6


PLAN
Training wall
SECTION OF SLOPING APRON WITH BREAKERS
TEST 4
Top of training wall El. 870.0
Tank floor El. 762
Slope = 8:1
16 ga. G.I.
Slope = 0.70
1/8" Gusset
1" x 1" x 1/8" Angles
Shim
1/2" x 3" Lag screw
Axis of crest
Army control line
CONCRETE WALL
1/8" x 1 1/2" x 1 1/2" Angles
3/8" Shiplap
2" x 6"
1" x 2" x 3/16" Channels
24" To outside of covering
11 1/16" To inside of covering
Prototype R = 10'9 1/8"
Model R = 16 63/64"
20 ga. G.I.
El. 1060.0
SECTION THRU CREST SHOWING STEEL BENTS, TRAINING WALL AND SLOPING APRON
TESTS 1, 2 AND 3
LEVEL APRON EL. 810 - 10" RADIUS BUCKET
TEST 5
HORIZONTAL APRON WITH SILL
TESTS 6 AND 7
LEVEL APRON EL. 806.0
10" AND 16 3/4" RADIUS BUCKET
TESTS 6 AND 8
TEST NO. 8
TEST NO. 6
Scale of Inches
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
NORRIS DAM
HYDRAULIC MODEL EXPERIMENTS
DETAIL SECTIONS THRU DAM AND APRON
DRAWN J.E.W. SUBMITTED
TRACED P.W.R. RECOMMENDED
CHECKED G.H.G. APPROVED
DENVER COLO., OCT. 15, 1934
232-D-698

lowering the apron 4 feet.

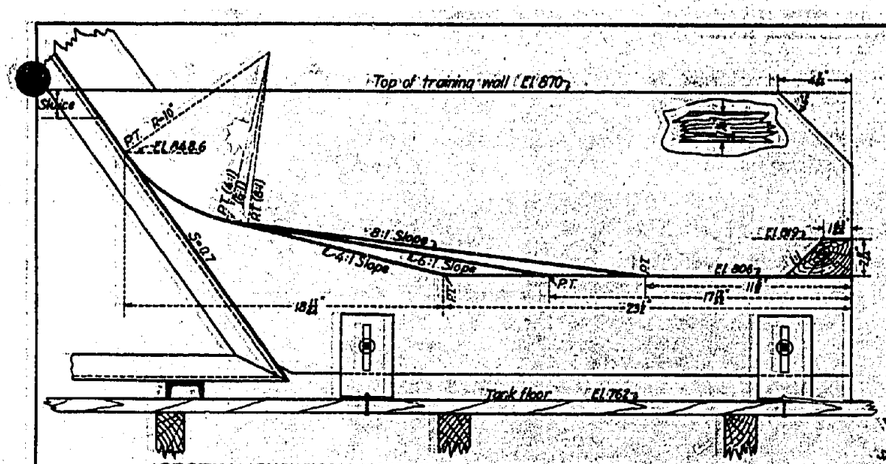
An improvement was indicated by holding the tailwater at the higher level. Accordingly, the flat apron was lowered to Elev. 806 and the 60-foot radius bucket maintained (Figure 6, Tests 6 and 7). The bucket was next changed to 100-foot radius in order to provide a greater thickness of concrete at the toe of the dam to allow for possible scour at that point (Figure 6, Test 8). It was hoped that the lower part of the large radius bucket would simulate the sloping apron.

The possibilities of the sloping apron were very apparent, so the testing reverted to developing a sloping portion of apron from the curved bucket to the flat apron section in order to improve the jump conditions at low flows, reduce excavation in the pool and give added thickness of apron next to the dam.

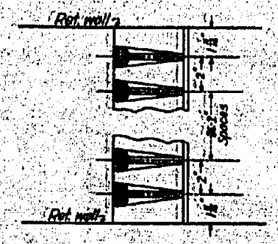
Slopes of 4:1, 6:1 and 8:1 keeping the upper tangent point of the bucket fixed and varying the length of slope were tested for the sloping portion of apron (Figure 7, Tests 9, 10 and 11). Results showed the 4:1 slope (Test 9) most advantageous and it was adopted for further studies.

After fixing the elevation of the flat portion of the apron and the rate of slope for the sloping portion of apron, a series of tests were conducted to determine the possibility of shortening the length of horizontal apron downstream or improve flow conditions by additions and alterations in sills with the same length of apron.

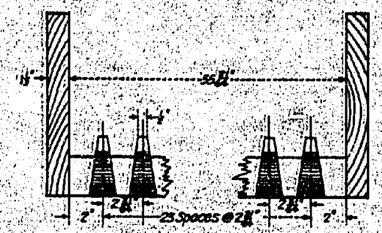
FIGURE 7



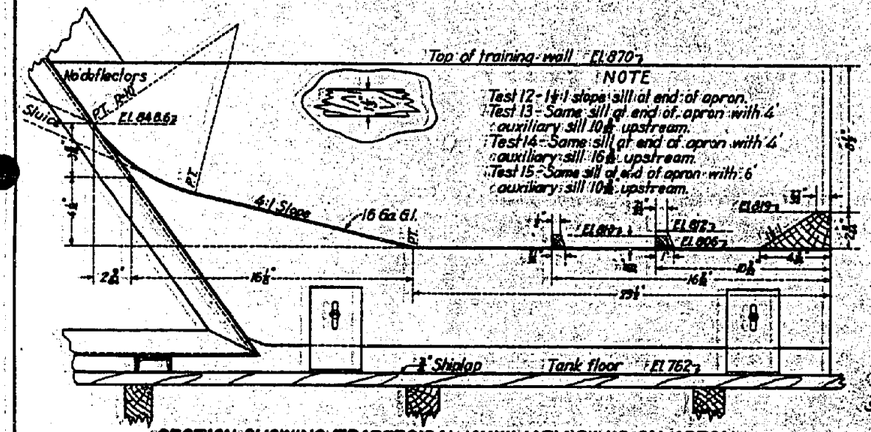
SECTION SHOWING 4:1, 6:1 AND 8:1 SLOPING APRON TO ELEV. 806.0
TESTS 9-10 & 11 RESP



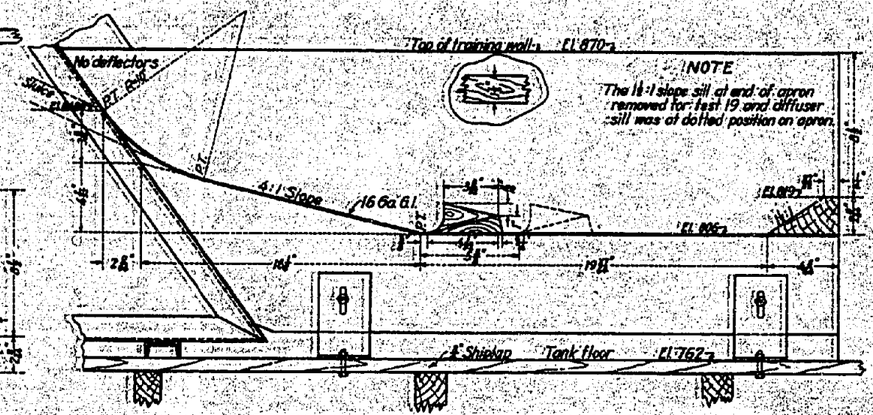
PLAN



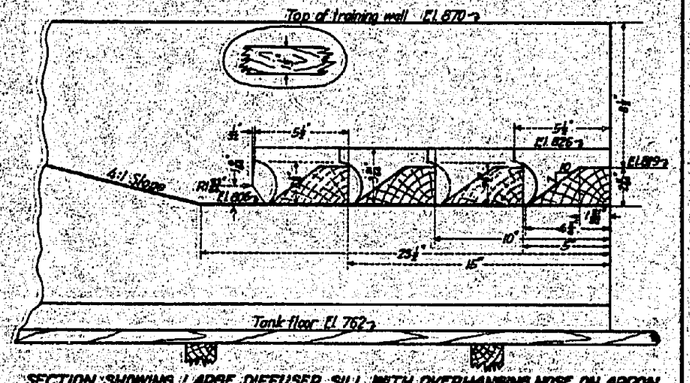
UPSTREAM ELEVATION



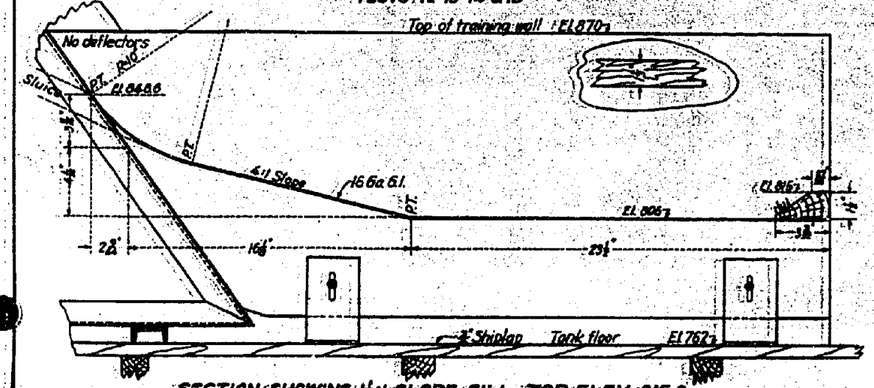
SECTION SHOWING TRAPEZOIDAL AUXILIARY SILLS ON APRON
TESTS 12-13-14 & 15



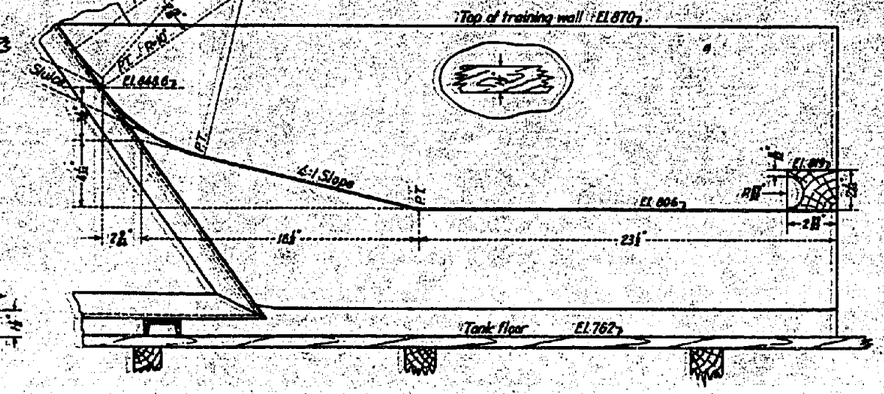
SECTION SHOWING DIFFUSER SILL ON APRON
TESTS 18 & 19



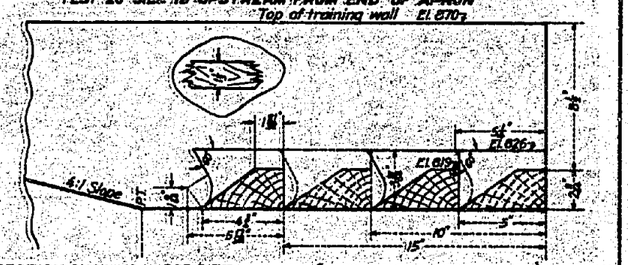
SECTION SHOWING LARGE DIFFUSER SILL WITH OVERHANGING NOSE ON APRON
TEST 25 SILL AT END OF APRON
TEST 26 SILL 5' UPSTREAM FROM END OF APRON
TEST 27 SILL 10' UPSTREAM FROM END OF APRON
TEST 28 SILL 15' UPSTREAM FROM END OF APRON



SECTION SHOWING 1½:1 SLOPE SILL. TOP ELEV. 815.0
TEST 16



SECTION SHOWING CONCAVE UPSTREAM FACE SILL ON APRON
TEST 23



SECTION SHOWING LARGE DIFFUSER SILL (OVERHANGING NOSE REMOVED) ON APRON
TEST 29 SILL 15' UPSTREAM
TEST 30 SILL 10' UPSTREAM
TEST 31 SILL 5' UPSTREAM
TEST 32 SILL AT END OF APRON

SCALE OF INCHES

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

N.O.R.R.I.S. DAM

HYDRAULIC MODEL EXPERIMENTS
DETAIL SECTION OF APRONS AND SILLS

DRAWN: C.W.E. & R.H. SUBMITTED: Jack C. Vigrank
TRACED: L.L.G. RECOMMENDED: C. W. Lee
CHECKED: G.W.A. APPROVED: J. E. Savage

DENVER, COLORADO, OCT. 15, 1934

232-D-697

The first change was the slope of the face of the sill at the end of the apron. The original slope was 1:1 from Elev. 806 to Elev. 819 and the top width 10 feet. This was changed to a $1\frac{1}{2}$:1 slope with a top width of 5 feet (Figure 7, Test 12).

With this altered sill at the end of the apron, at the suggestion of Mr. Charles H. Paul, an auxiliary sill six feet high was tested at one position on the apron and an auxiliary sill four feet high was tested at two positions on the apron. Figures 7, Tests 13, 14 and 15 show details and locations of these sills.

When the tests on the model were started, the data on the elevation of the stream bed at the end of the apron was incomplete. The sills had previously been designed for an average stream bed elevation of 819. Data specifically requested for that purpose led to the design of a sill with a $1\frac{1}{2}$:1 slope and top width of 5 feet at Elev. 815 (Figure 7, Test 16).

A low diffuser sill suggested by G. J. Hornsby, Assistant Engineer, U. S. Bureau of Reclamation, was tested at two positions on the apron, primarily to spread the jets from the sluice openings and secondly to aid in improving flow conditions from the spillway. This sill was tested at the upstream position with the main sill at the end of the apron and at the downstream position with the main sill removed from the end of the apron, Figure 7, Tests 18 and 19 show the positions on the apron and the details of the sill.

A sill with a concave upstream face, suggested by Mr. J. L. Savage, was tested at the downstream end of the apron. See Figure 7, Test 23 for details.

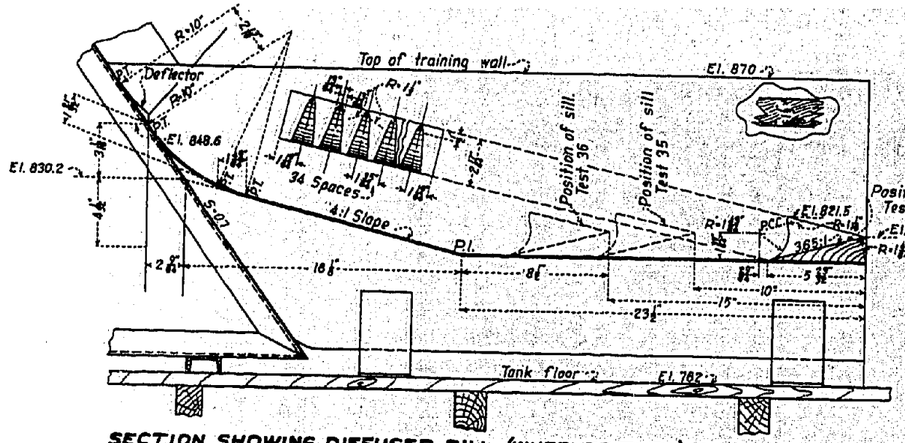
A large diffuser sill with an overhanging nose on the blocks was tested at end of apron and several positions upstream. Later the overhanging noses were sawed off and the sill tested at the same positions. Figure 7, Tests 25, 26, 27, and 28 show the details and positions of the sill before the noses were cut off and Tests 29, 30, 31 and 32 show the details and positions after the noses of the blocks were removed.

A Wheeler type low diffuser sill was tested at three positions on the apron to improve flow conditions. The details and locations of this sill can be seen on Figure 8, Tests 34, 35 and 36.

A sill with constant top width of 3 feet and conforming to the elevation of the rock line at the end of the apron, thus varying at top elevation from 816 to 820, having a variable slope on the upstream face and a constant bottom width of 24 feet was tested (Figure 8, Test 37).

From the results of the testing on this sill there was evolved a set of sills with top at Elev. 817.5, a top width of 3 feet and front face with slopes of $1\frac{1}{2}$:1, 2:1, $2\frac{1}{2}$:1 and 3:1. These were tested at the end of apron. Figure 8, Tests 39, 40, 41 and 43 show details of sills.

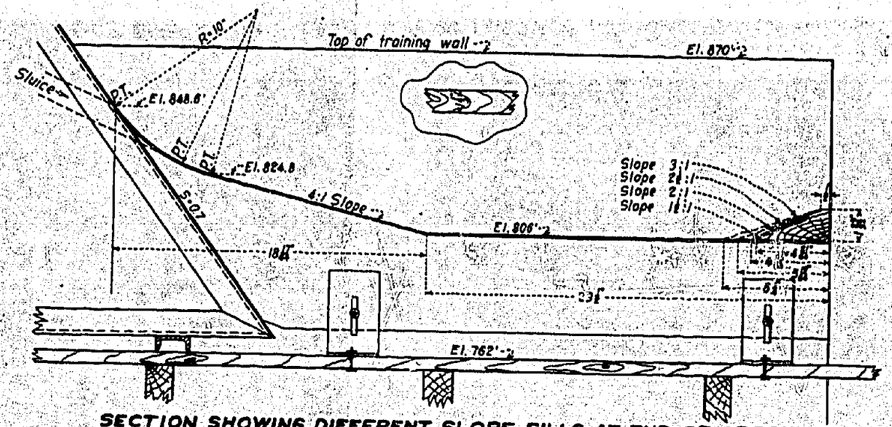
FIGURE 8



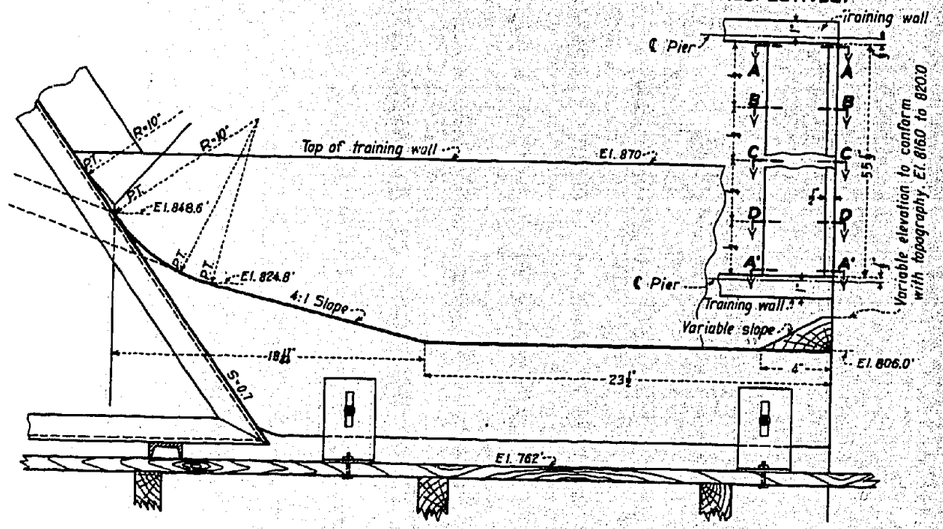
SECTION SHOWING DIFFUSER SILL (WHEELER TYPE) ON APRON
TESTS 34 35 AND 36 - SILL AT END OF APRON, 10" AND 15" UPSTREAM RESPECTIVELY

MINIMUM SECTION

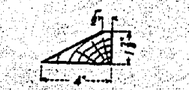
MAXIMUM SECTION
SECTION A-A AND A'A'



SECTION SHOWING DIFFERENT SLOPE SILLS AT END OF APRON
TEST 39 - 2½:1 SLOPE SILL
TEST 40 - 2½:1 SLOPE SILL
TEST 41 - 3:1 SLOPE SILL
TEST 43 - 1½:1 SLOPE SILL



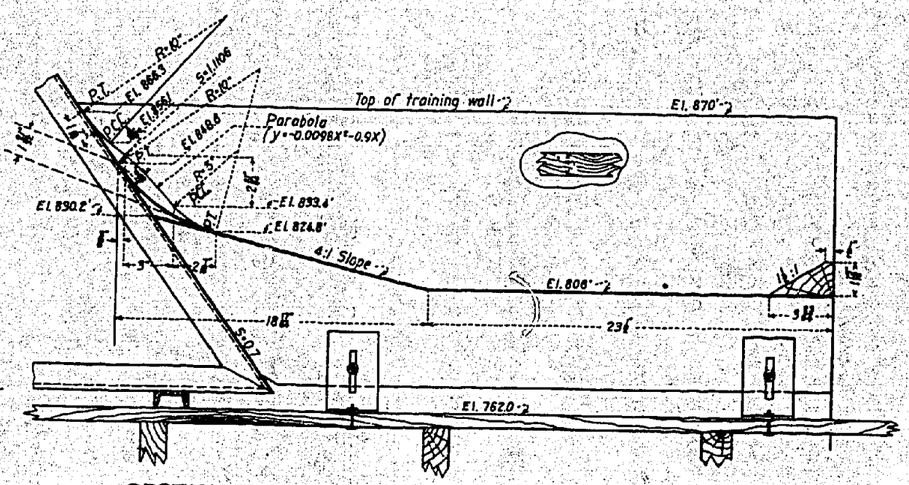
SECTION SHOWING VARIABLE SECTION TRAPEZOIDAL SILL AT END OF APRON
TEST 37



SECTION B-B

SECTION C-C

SECTION D-D



SECTION SHOWING PARABOLIC HUMP OVER SLUICE OPENINGS
TEST 42

SCALE OF INCHES

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

NORRIS DAM

HYDRAULIC MODEL EXPERIMENTS
DETAIL SECTIONS THROUGH SILLS AND APRONS

DRAWN: E.D.H. SUBMITTED:
TRACED: R.E.P. RECOMMENDED:
CHECKED: C.W.J. APPROVED:

DENVER, COLORADO, OCT. 18, 1934 232-D-699

The parabolic hump over the sluice openings shown on Figure 8, Test 42 was tested in conjunction with the flow over the spillway.

To identify the experimental data, each alteration or set-up of the model was assigned to a test number and each run of a test was given a run number. In like manner, each picture made was given a picture number. Thus 10-B-6 is interpreted as Test or Set-up 10, Picture B of Run 6.

The flow conditions in each test were recorded (1) by still and motion pictures, (2) by gages and instruments and (3) by notes from observation.

The construction program for the Norris project necessitated the building of two cofferdams at different locations on the spillway apron. To prevent sliding, permanent concrete keys were incorporated in the spillway apron, and the cofferdams constructed over them. After these cofferdams are removed the keys will remain on the apron.

After the testing of the model was completed, but before it was dismantled, some concern was felt as to what effect the keys would have on the flow conditions. They were installed at their proper location on the apron (Frontispiece and Plate 67 A and B) and visual tests were made of the flow conditions from the spillway and sluiceways.

The parabolic hump over the sluice openings shown on Figure 8, Test 42 was tested in conjunction with the flow over the spillway.

To identify the experimental data, each alteration or set-up of the model was assigned to a test number and each run of a test was given a run number. In like manner, each picture made was given a picture number. Thus 10-B-6 is interpreted as Test or Set-up 10, Picture B of Run 6.

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2. Calibration of Drum Gates

Prior to the actual calibrating of the drum gates, a tentative schedule of operation was worked out for the gates to cover the range of gate elevations from 1020 to 1034. These settings were fixed at Elev. 1020, 1023.5, 1027, 1031.5 and 1034.

For each of these gate settings a range of heads over the gates was determined. These heads were not the same for each gate setting, but were tentatively determined so that the minimum head was fixed by the elevation of the gates and the maximum head was that obtained when the elevation of the water surface of the reservoir was approximately one foot above the maximum of Elev. 1052.

The quantities dealt with in determining the crest coefficients on a model are relatively small, therefore, great care was taken in determining them. A small error in the model quantity results in a large error when the results are transferred to the prototype. The drum gates on the crest [redacted] of the model were set by means of a hook gage and light spirit level to the desired elevation before the runs

were started and checked after the runs were completed. In this way the elevation of the model gates could be accurately determined and referenced to a common datum in respect to the forebay gage. The settings used in the tests were not the exact settings previously decided upon, but the variations were small and, as the exact elevation was determined, it was not deemed necessary to spend additional time to set the gates at an even figure in regard to elevation.

Representative heads, measured from the highest portion of the gate were run with all gates set as nearly as possible to the same elevation. During the runs a series of readings were taken simultaneously on the measuring weir gages and the forebay gage at equal intervals of time.

The length of spillway crest was carefully measured by a micrometer caliper and checked from time to time to assure constant conditions.

As the results of the first tests were analyzed, it was deemed necessary to test intermediate gate elevations and the same careful procedure was followed.

The original settings used were Elev. 1020, 1023.51, 1027.11, 1031.54 and 1033.91; the intermediate settings run later were Elev. 1022.06, 1025.03, 1028.93 and 1031.41.

Check runs were made throughout the series to assure consistency of results.

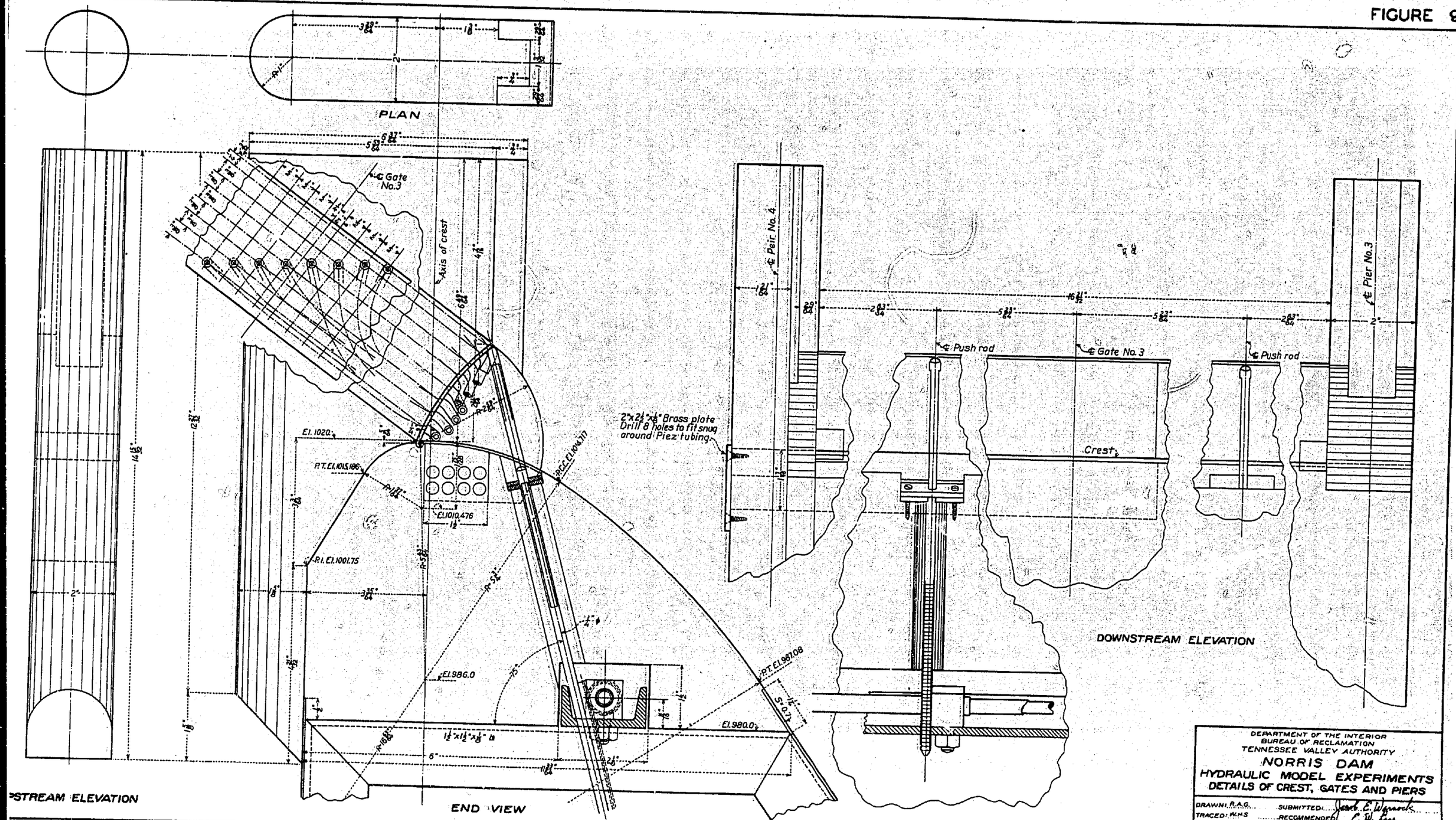
3. Pressures on Drum Gates

Eight piezometer openings were installed in one of the drum gates and connected to glass tubes by means of rubber tubing. These piezometer openings were referenced to the gage board in such a manner that the capillary attraction in the glass tube was taken into account. Figure 9 shows the locations of the piezometer openings in the gate. The pressures were read in feet of water directly on the glass gage tubes. While the above mentioned calibration runs were in progress, a series of readings on the gage tubes were observed for each gate elevation and head over the gate. These results were transferred to prototype and plotted as will be explained later.

4. Tests on Training Walls

From visual observations of the upper nappe of the jet along the training walls on the downstream face of the dam, it was felt that these walls might be modified to contain less yardage of concrete and improve their architectural appearance. Upper nappe measurements were made with a point gage reading from the top of the training wall perpendicularly to the water surface with all gates at Elev. 1034 and the reservoir surface at Elev. 1052; and all gates set so as to hold the reservoir at Elev. 1052 when quantities equal to 120,000, 160,000 and 180,000 second-feet were

FIGURE 9


PLAN
℄ Gate No.3
Axis of crest
E.1.1020
P.T. E.1.1015.186
E.1.1010.476
P.I. E.1.1001.75
P.C.C. E.1.1004.711
2"x2¼"x⅛" Brass plate
Drill 8 holes to fit snug around Piez tubing
E.1.986.0
P.T. E.1.987.08
E.1.980.0
1½"x1½"x⅛" ∟
END VIEW
℄ Peir No. 4
℄ Pier No.3
℄ Push rod
℄ Gate No. 3
℄ Push rod
Crest
DOWNSTREAM ELEVATION
STREAM ELEVATION
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
NORRIS DAM
HYDRAULIC MODEL EXPERIMENTS
DETAILS OF CREST, GATES AND PIERS
DRAWN: R.A.G.
TRACED: W.H.S
CHECKED: G.W.J
SUBMITTED:
RECOMMENDED:
APPROVED:
DENVER COLO., OCTOBER 15, 1934
232-D-695

passing the spillway. The results were transferred to prototype and plotted (Figure 33) along a profile of the training walls.

It was also desired to obtain pressures on the sides of the training wall on the powerhouse side of the apron to assist in the design of this wall. Twenty-five piezometer openings were placed on the spillway side of the training wall and connected to glass gage tubes by means of rubber tubing. Figure 36 shows the location of the piezometer openings. Runs of 20,000, 40,000, 60,000, 80,000, 120,000, 160,000, 200,000 and 240,000 second-feet were made and a series of observations made on the gage tubes for each run. The water surface on the powerhouse side of the wall was observed by means of the tailwater float gage and the water surface on the apron in front of the piezometer openings was measured with a point gage. The results were transferred to prototype and plotted.

5. Velocity Studies

While the model tests were in progress the question arose as to the relative coefficient of friction on the model and the prototype. In order to check the model results, measurements of the velocity on the apron near the toe of the model dam were made with a pitot tube.

The tube was mounted in a fixed position on the center line of the spillway at a point two inches (model) downstream from the point of tangency of the 60-foot radius bucket and the level apron at Elev. 806. The pitot tube was of the single tube type,

reading total head directly in feet, tenths and hundredths. The thousandth fraction of a foot was estimated. The static head was measured by a point gage over the tube opening (Plate 2).

Runs of 40,000 (Gates at Elev. 1034), 120,000 (Reservoir at Elev. 1052), and 200,000 second-feet (Gates at Elev. 1020) were made with the tailwater completely removed or in other words with no hydraulic jump on the apron. This was done so the hydraulic jump would not affect the measurements of the velocity on the model, as the loss of velocity in the jump was not taken into account when the prototype velocity was calculated. A traverse from apron to water surface was made during each run and a series of readings taken at each depth setting. The opening in the tube was accurately located for each setting.

The prototype velocities were computed at the same point as the velocities were measured on the model using Kutter's N equal to 0.013.

6. Gate Operating Program

For the purpose of determining a gate operating program a series of runs were made with the gates set at different elevations from the fixed crest. All combinations of one gate at Elev. 1020 and the other two at Elev. 1034 were run with the reservoir elevation held at Elev. 1034 and also at 1052. Similarly all combinations of two gates at Elev. 1020 and the one at Elev. 1034 were run with the reservoir held constant at Elev. 1034 and also at Elev 1052. Pictures and notes were made of the stilling pool conditions for each setting.

c. Auxiliary Passages

1. Sluices

As the model of the dam was located over a concrete wall which was part of the permanent building, the sluiceways could not be built directly through the dam. Instead the water was brought under the model through an eight-inch pipe and carried through the dam parallel to its axis. The sluiceways were opened out of this header pipe perpendicular to the center line and carried through the downstream face of the dam. A single slide gate at the entrance from the forebay was the only control provided at first.

As originally installed each of the eight sluices was carried through the model with the center line level at Elev. 865 (Figure 4, Section A-A). Visual observations and pictures were made of the action in the stilling pool with all sluices wide open and the pond held at Elev. 1034; with the sluices closed and various flows over the spillway; and with all sluices open and 200,000 second-feet over the spillway.

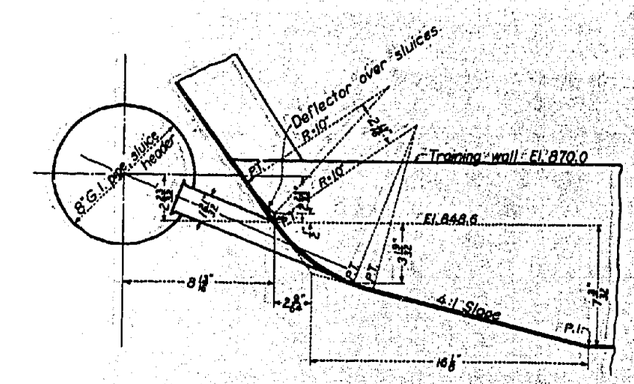
In the next set-up, the center line of the sluice gates remained at Elev. 865 and the invert of the sluice passages followed a parabola from the gates and connecting with the tangent to the 60-foot radius bucket at Elev. 828.5, or just above the point of tangency of the 60-foot radius bucket and 4:1 slope (Figure 2). To reproduce this on the model would have necessitated cutting into the concrete wall, so the invert of the sluice was made on a slope emerging from the face of the dam at a point of tangency at the same

point as the P.C.C. of the parabola and 60-foot radius (Figure 10, Tests 12 to 32 inclusive). The procedure of testing followed closely that given above except the various sills and diffuser baffles used on the apron were also tested in conjunction with the sluices to see if a more even distribution of velocities across the apron and sill could be obtained. Also controls were made for each sluice so they could be operated singly or in combinations. However, the sluice was either completely open or closed at all times, as this is the method of operation expected on the prototype.

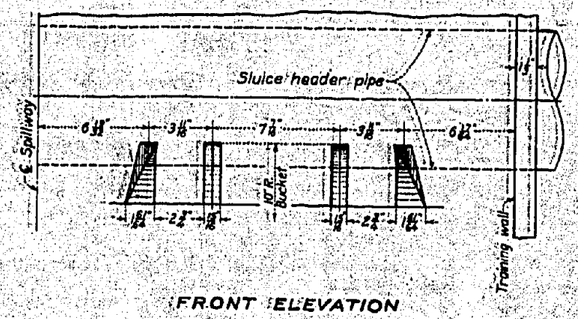
In the original design, the sluices were brought through the dam in pairs. As a result, the jets impinging in the stilling pool caused unsatisfactory conditions. It was hoped to remedy these conditions by spreading the outlets more evenly across the bucket. Accordingly, four sluices were held with their center lines perpendicular to the axis of the dam and the other four (one of each pair) were diverged at an angle of eleven degrees from the other one in each pair. The invert was held at the same slope as before. The half plan on Figure 10 shows the details of this set-up as well as a plan of the parallel sluice set-up. Procedure similar to above was followed in testing the set-up.

Tests were also made on this set-up to determine the pressures existing in the sluiceways when water was flowing over

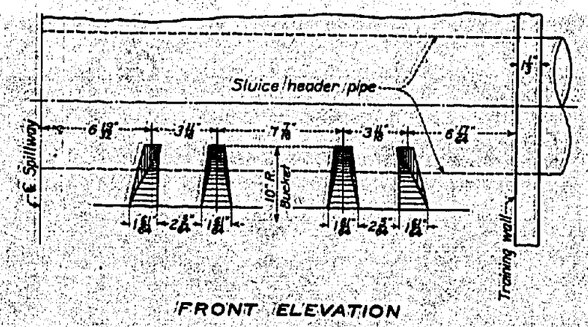
FIGURE 10



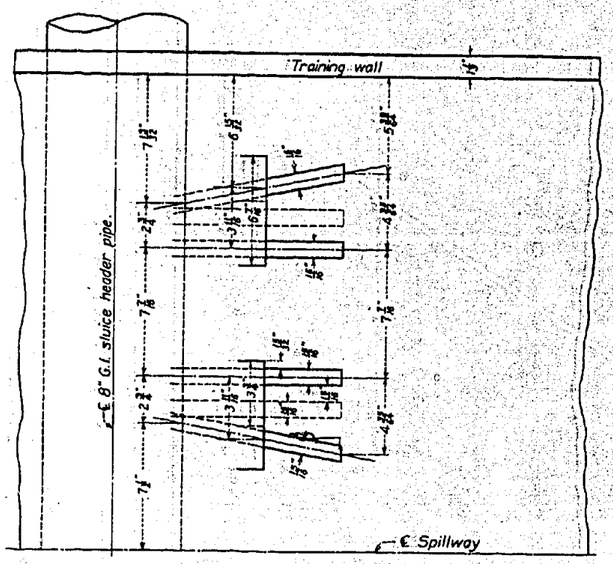
SECTION



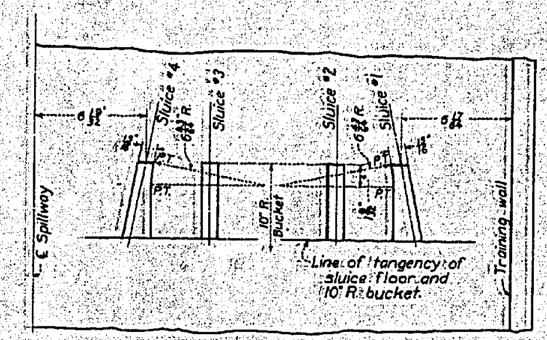
FRONT ELEVATION



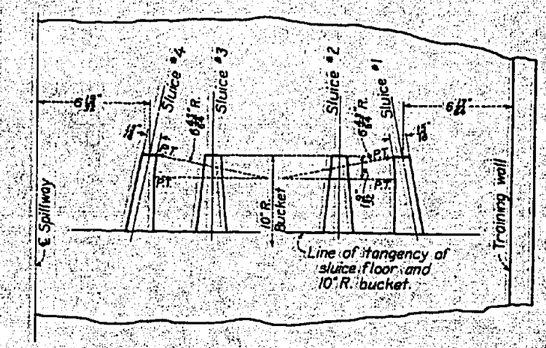
FRONT ELEVATION



HALF PLAN
SHOWING
PARALLEL SLUICES, TESTS 12 TO 32 INCL.
DIVERGING SLUICES, TESTS 33 TO 46 INCL.
3 3/16" DEFLECTORS, TESTS 23 TO 32 INCL.
6 11/16" DEFLECTORS, TESTS 34 TO 37 INCL.



PLAN
SHOWING
STRAIGHT SLUICES NO. 2 AND 3
DIVERGING SLUICES NO. 1 AND 4
TEST 44



PLAN
SHOWING
DIVERGING SLUICES NO. 1, 2, 3 AND 4
TEST 46

SCALE OF INCHES

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

NORRIS DAM
HYDRAULIC MODEL EXPERIMENTS
DETAILS OF SLUICEWAYS

DRAWN: E.D.H. SUBMITTED: Jack E. Casserd
TRACED: L.H.H. RECOMMED: S. E. W. Hoge
CHECKED: C.W.J. APPROVED: J. L. Savage

DENVER, COLO., OCT. 15, 1934. 232-D-691

the dam and none through the sluice openings. To do this, two sluices were sealed from the header and tube openings inside each sluice were connected to glass gages by rubber tubing. Runs were made with flows of 20,000, 40,000, 80,000, 120,000, 160,000 and 200,000 second-feet and a series of observations on the gage taken for each sluice on each run.

As a means of decreasing the negative pressure found in the sluices a parabolic hump was incorporated into the dam extending from the 0.7 slope down to the 60-foot radius bucket through which the sluices opened. See Fig. 8, Test 42 and Plate 54, A and B. Pressures in the sluices, action of sluices etc., were determined as described above.

Various types of hoods on the face of the dam above the sluice openings were tested on the model previous to installing the parabolic hump. Pressures in the sluiceways were measured as above on the best form of these hoods.

The action of the sluiceways was improved by the diverging set-up described above, but neither the action of the water flowing over the dam and past the outlets nor the general appearance of the dam was satisfactory. To correct these factors the shape of the outlets through the face of the dam and bucket was changed.

The first change was to maintain the sides of the straight sluices parallel to their center lines. In the diverging

the dam and none through the sluice openings. To do this, two sluices were sealed from the header and tube openings inside each sluice were connected to glass gages by rubber tubing. Runs were made with flows of 20,000, 40,000, 80,000, 120,000, 160,000 and 200,000 second-feet and a series of observations on the gage taken for each sluice on each run.

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The first change was to maintain the sides of the straight sluices parallel to their center lines. In the diverging

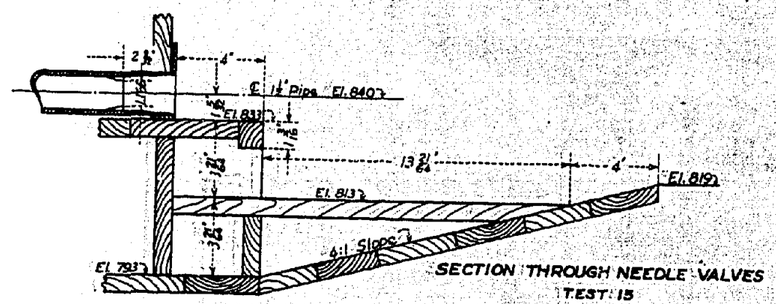
sluices the side farthest from the straight sluice was made parallel to the center line of the diverging sluice. The side of the outlet next to the straight sluice was made parallel to the center line of the straight sluice (Figure 10, Plate 61 B). An operating program for the sluiceways was developed. Studies were made of the action of the sluices with and without the spillway discharging and the spillway discharging with and without the sluices in operation. This change showed decided improvement but the symmetrical appearance of the dam was disturbed (Plate 61 B).

To improve the appearance of the openings and further spread the discharge jets, the sides of the outlets of the straight sluices were diverged with respect to their center lines an amount such that the width of the floor at the opening was equal to the floor width at the opening of the diverging sluiceways (Plates 56 A and 61 C).

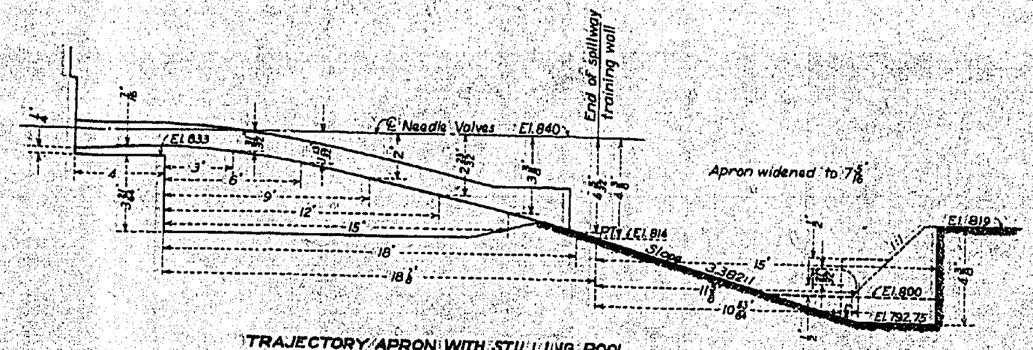
2. Needle Valves

The original design of the Norris Dam included two 72-inch needle valves installed in the river end of the powerhouse and discharging into the tailrace next to the spillway training wall. These were incorporated in the model as shown on the original design (Figure 11, Test 15). Operation disclosed immediately that impingement of the jets of water in the tailrace caused un-

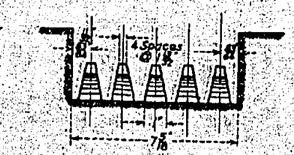
FIGURE 11



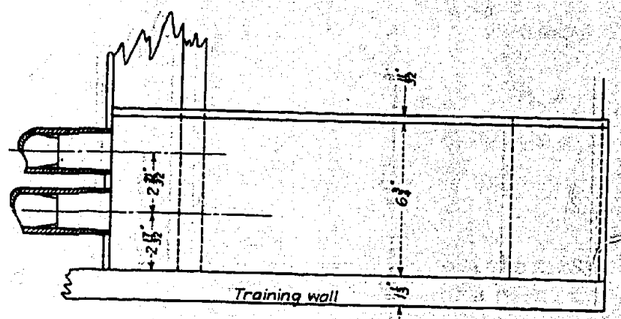
SECTION THROUGH NEEDLE VALVES
TEST 15



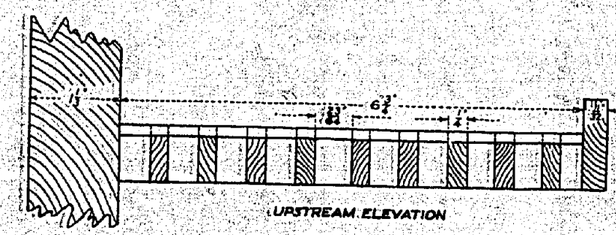
TRAJECTORY APRON WITH STILLING POOL
SECTION FOR TEST 21
FLOOR ELEVATION 800 AND DIFFUSER SILL
FOR TEST 22



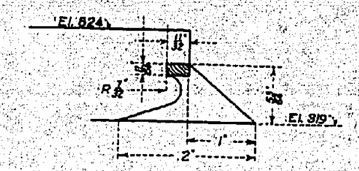
UPSTREAM ELEVATION OF DIFFUSER SILL



PLAN

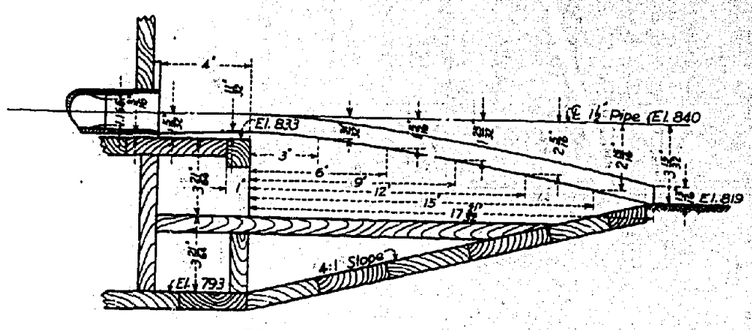


UPSTREAM ELEVATION

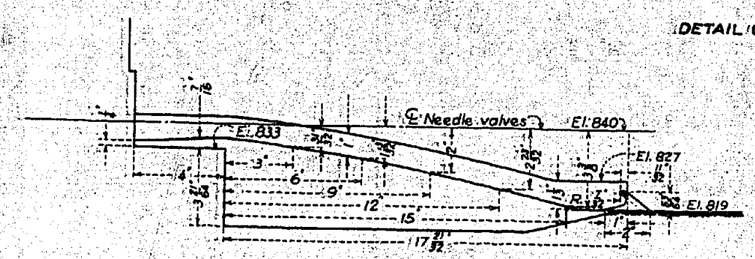


SIDE ELEVATION

DETAIL OF ENSIGN BAFFLES



TRAJECTORY APRON SECTION
TEST 17



TRAJECTORY APRON WITH ENSIGN BAFFLES
SECTION FOR TEST 20

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

NORRIS DAM
HYDRAULIC MODEL EXPERIMENTS
DETAIL SECTIONS THRU NEEDLE VALVES

DRAWN: R.A.G. SUBMITTED: Jack E. Warnock
TRACED: W.C.W. RECOMMENDED: E. D. Tam
CHECKED: C.W.T. APPROVED: J. L. Savage
DENVER, COLORADO OCT. 15, 1936 232-D-687

satisfactory conditions. A satisfactory solution was first sought by building an apron from the exit of the valves to the end of the paving in the tailrace. This apron, 40½ feet in width, was joined to the spillway training wall on one side and had a training wall three feet in height on the other side. The longitudinal section of this apron was a parabola. This parabola was flatter than the theoretical trajectory of the jets from the valves and was rotated about the focus so as to raise the downstream end into the path of the trajectory (Figure 11, Test 17). This was done to spread the jets and cause the water to enter the tailrace in a comparatively thin sheet so that a hydraulic jump would form. Runs were made with the head on the valves ranging from 150 to 194 feet and a tailwater corresponding to the combined flow through the valves and powerhouse or valves alone.

The first apron tested indicated the possibility of further rotating the downstream end upward to cause more spreading. Such a procedure was followed.

A type of Ensign baffle on the apron was tested (Figure 11, Test 20). The baffle tested was very inefficient in the dissipation of the energy. A structure of this type large enough to be efficient would not be economical or practical.

An excavation was made in the tailrace to form a stilling pool. By trying different lengths of this pool, a more satisfactory solution of the problem was obtained (Figure 11, Test 21). To further improve this set-up, a diffuser baffle was incorporated in the bottom making possible a shorter and shallower pool (Figure 11, Test 22).

Observations on needle valves were made visually and by pictures, thus making the studies more qualitative than quantitative. The heads on the orifices used to simulate valves were measured by a manometer.

3. Powerhouse

The powerhouse and tailrace were not built on the initial model, but were added after the first six tests. The first designs included three turbine units and draft tubes. The powerhouse was constructed to form a header for the six openings simulating the turbines discharging into the draft tubes. The correct flow was maintained through the powerhouse and the tailwater held at the height corresponding to the flow and visual observations were made.

A later design of the powerhouse included only two units and some changes in draft tubes. These changes were incorporated in the model and tests were run similar to the previous ones.

The only other changes in the powerhouse were the removal of the needle valves and the bridge in front of their exits.

d. Architectural Studies

The only changes made for purely architectural studies were on the spillway bridge and piers. The original bridge as constructed was of the through truss type in 100-foot spans from pier to pier. This was replaced by a pony type truss and finally by a plate girder bridge. The latter necessitated changes in pier design, which, when tested produced no undesirable results. In the final set-up the elevator towers at each end of the spillway were built.

Architectural studies of other features of the model were made in conjunction with the hydraulic studies as in the case of the sluiceway openings.

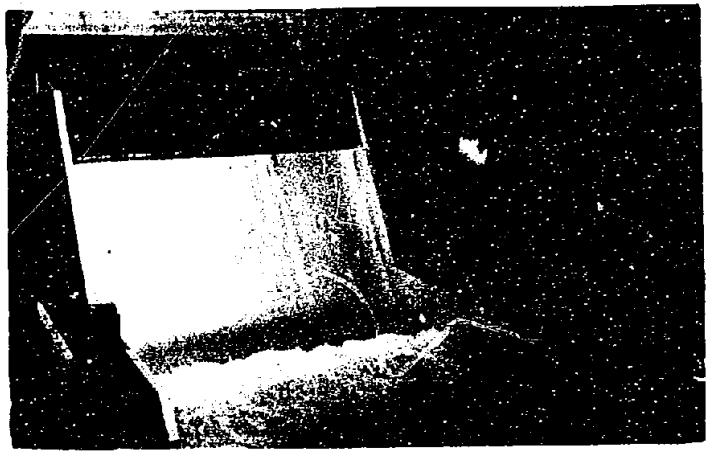
V. RESULTS

b. Spillway Studies

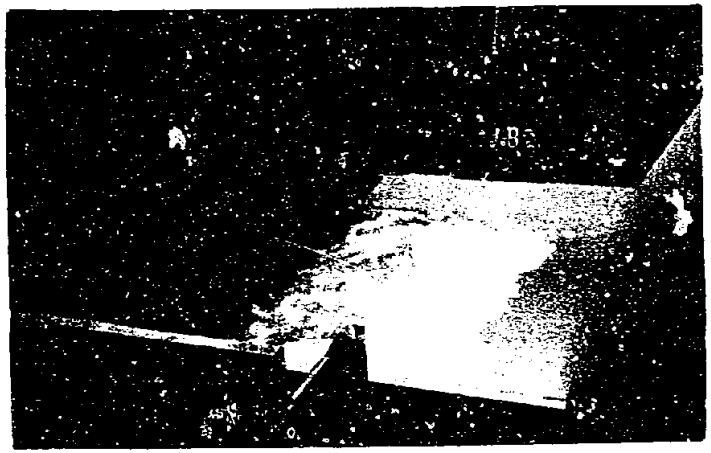
1. Stilling Pool Characteristics

Little is known of the characteristics of the hydraulic jump on a sloping section of apron, but from the results of tests made on structures previous to the testing of the Norris model, the behavoir was found to suit certain conditions. In the original design of the Norris Dam much thought was given to possible scour on the apron at the toe of the dam. In order to provide for this condition an additional thickness of apron at this point was deemed necessary. The sloping apron in addition to lessening the excavation in the pool would provide for this added thickness. Also the tailwater rating curve and the jump-forming height curve for a level depressed apron tend to draw apart at their lower ends (Figure 5) showing the pool to be too deep at low flows. The sloping apron, by causing the jump to form on the slope, would tend to bring these two curves together, thus giving better conditions at low flows.

The first slope was made to conform to the natural dip of the rock strata or an 8:1 slope downstream with a level apron at Elev. 810 (Plate 1 B). It was found that such an arrangement did not give a sufficient length of deep pool for the formation of the jump at low flows, although the action at higher flows was fairly good (Plates 3 and 4). The higher discharges showed the water

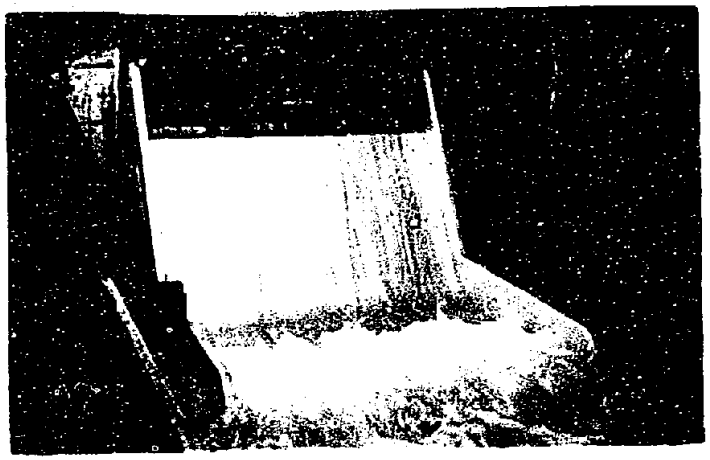


A. LOOKING UPSTREAM.

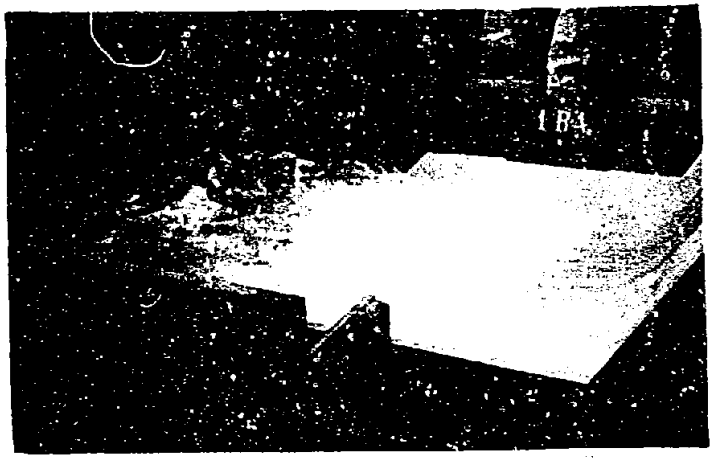


B. SIDE VIEW.

DISCHARGE 40,000 SECOND-FEET.



C. LOOKING UPSTREAM.

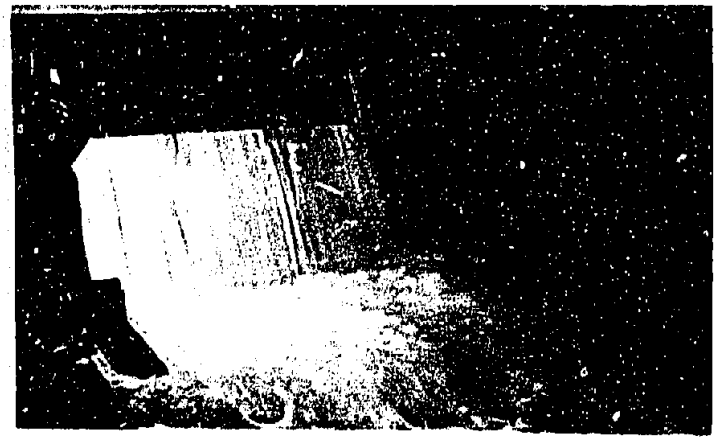


D. SIDE VIEW.

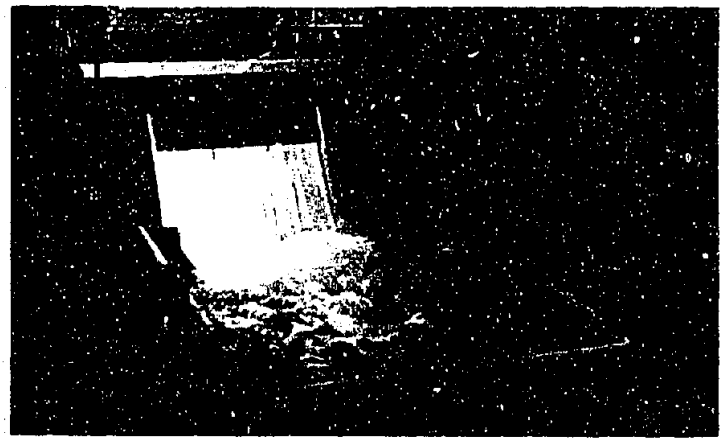
DISCHARGE 120,000 SECOND-FEET.

8:1 SLOPING APRON TO LEVEL APRON ELEV. 810.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

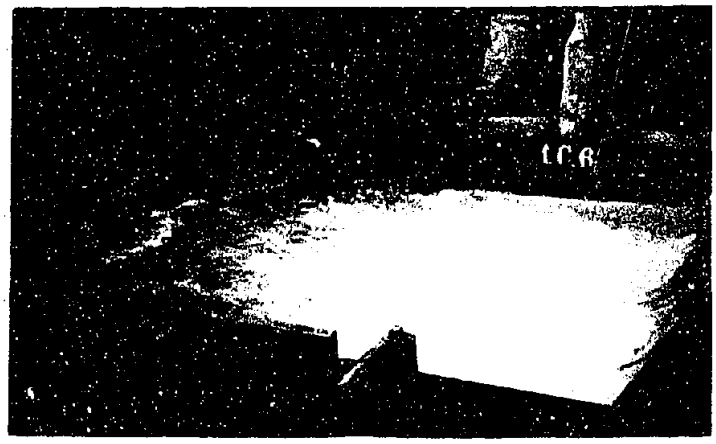
PLATE 3



A. DISCHARGE 200,000 SECOND-FEET.



B. DISCHARGE 200,000 SECOND-FEET.

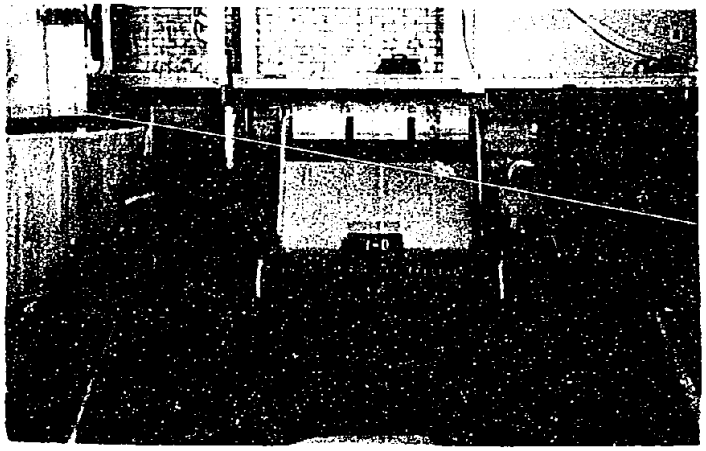


C. DISCHARGE 200,000 SECOND-FEET.

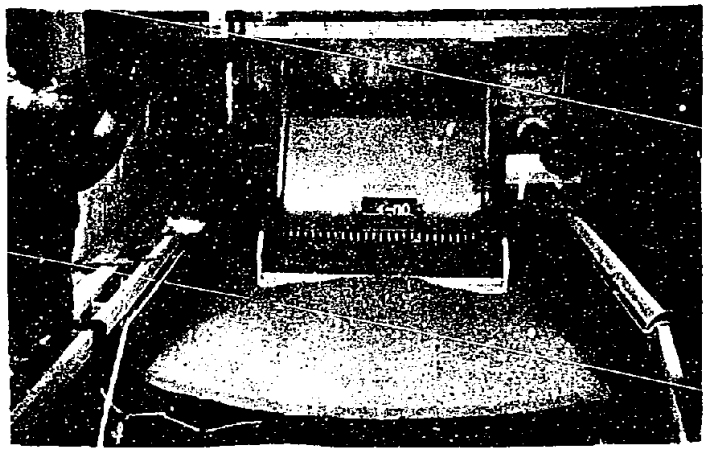
8:1 SLOPING APRON TO LEVEL APRON ELEV. 810.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

surface low over the entire apron and a slight tendency for the pool to sweep as can be seen on Figure 12, Test 3. Considerable scour below the apron was noted. When the tailwater was raised 4 feet, better results were obtained, thus showing the possibilities of a sloping section of apron with the level apron at Elev. 806.

By adding a set of breaker blocks at the point of tangency between the 60-foot radius bucket and the 8:1 slope across the entire width of the apron, it was hoped that the jet of water over the dam might be broken up sufficiently to cause more water to remain on the apron and decrease the velocities passing the sill. The results, (Figure 12, Test 4) show very little additional water retained on the apron and a very slight reduction of velocities passing the sill. Very undesirable conditions were set up at low flows by the disturbance caused by the blocks (Plate 5 C and D). The disturbance was not so obvious at higher flows and the water surface downstream was quieted as can be seen on Plate 6. There was less scour downstream from the end of the apron than in the previous test but the amount was still appreciable (Plate 5 A and B). The benefits derived from the blocks in reducing the scour and quieting the water surface at high flows were more than offset by the bad conditions at lower flows and the high cost of initial

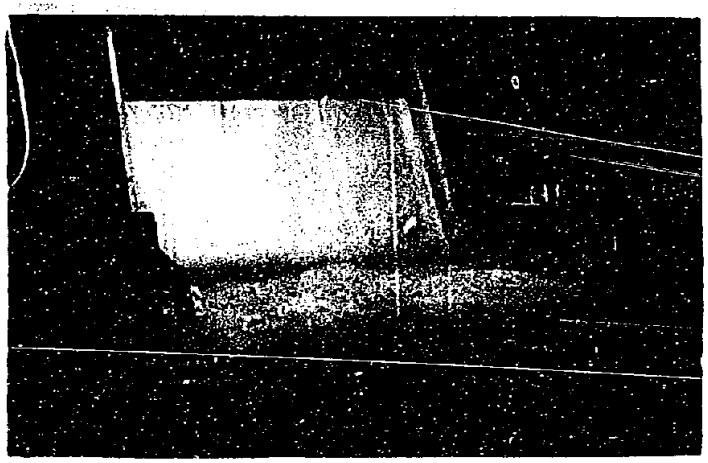


A. BEFORE RUN.

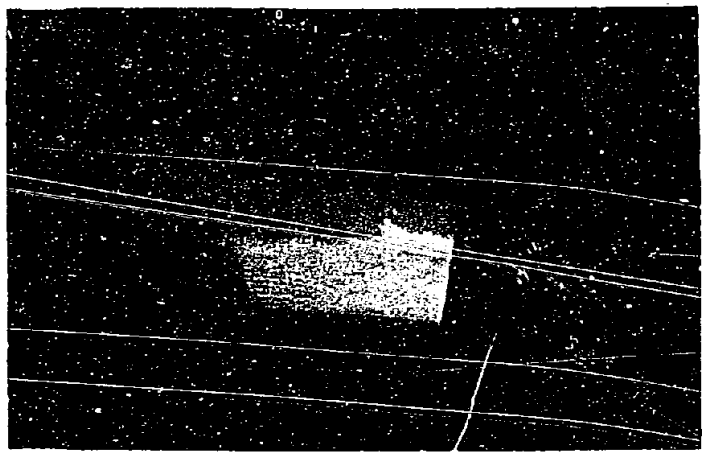


B. AFTER RUN.

NO FLOW.

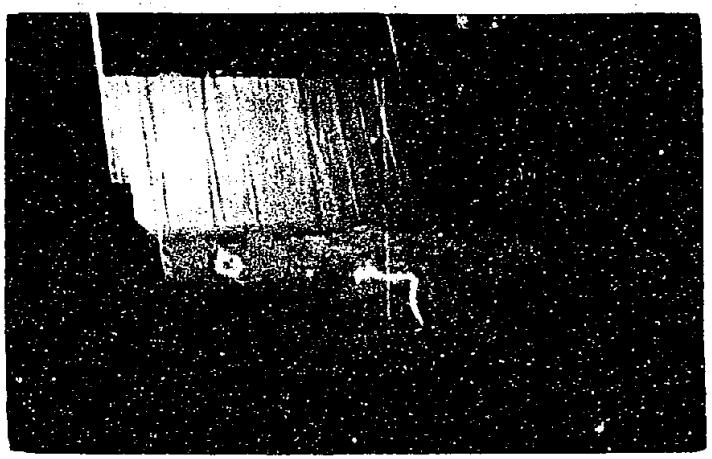


C. DISCHARGE 40,000 SECOND-FEET.

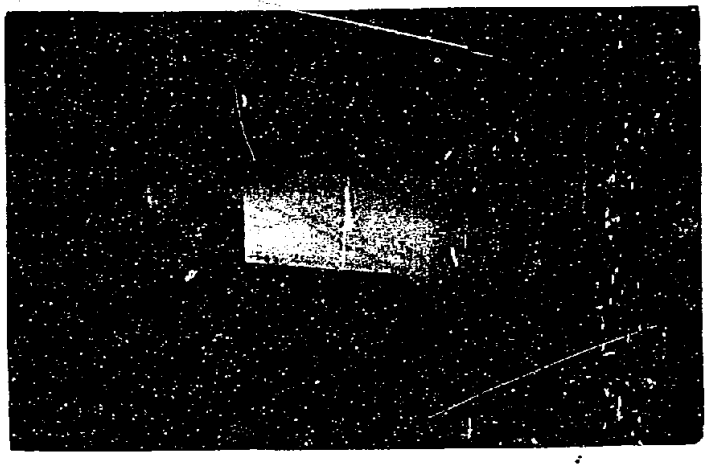


D. DISCHARGE 40,000 SECOND-FEET.

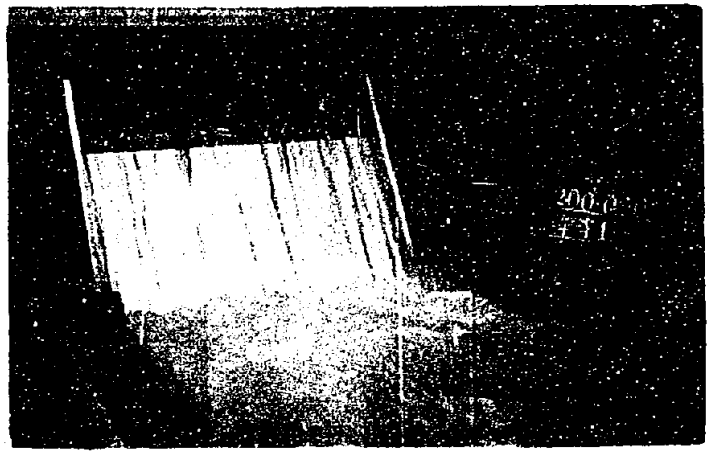
BLOCKS ON 8:1 SLOPING APRON. LEVEL APRON ELEV. 810.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.



A. DISCHARGE 120,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.



C. DISCHARGE 200,000 SECOND-FEET.

D. DISCHARGE 200,000 SECOND-FEET.

BLOCKS ON 8:1 SLOPING APRON. LEVEL APRON ELEV. 810.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

PLATE 6

FIGURE 12

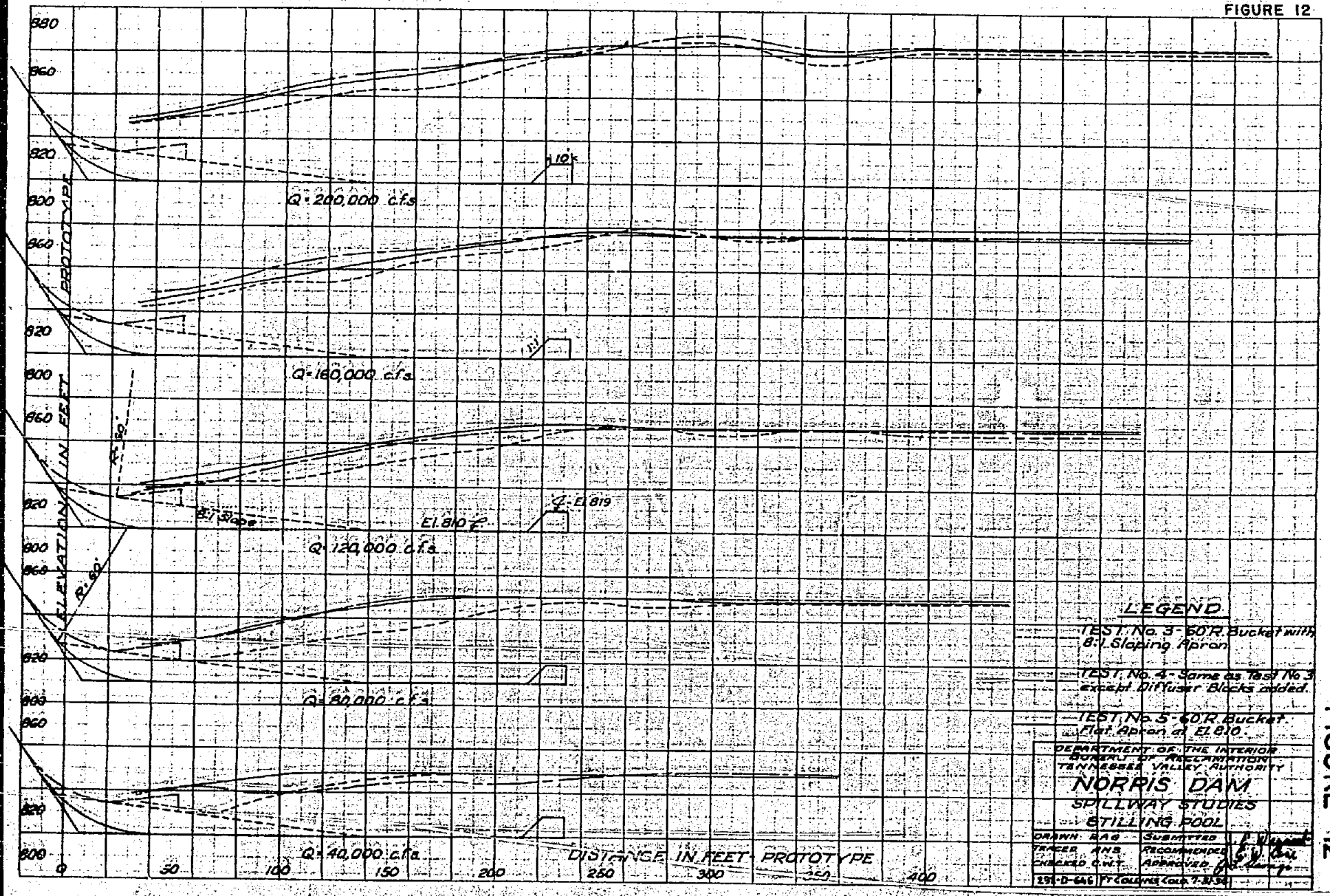


FIGURE 12

installation.

The results of the tests on the full-length flat apron at Elev. 810 showed insufficient tailwater at the intermediate and high flows to insure the formation of a hydraulic jump (Figure 12, Test 5) with sufficient depth at low flows. Less scour downstream resulted from this set-up than either of the previous tests. By raising the tailwater four feet, the conditions were somewhat improved, although the jump was drowned out to a certain extent at low flows.

In keeping with the indications from previous tests and the action predicted on Figure 5, the flat apron was lowered to Elev. 806 and connected to the 0.7 slope by a 60-foot radius bucket. The slope up to river bed level at the end of the apron was 1:1 with the top of the sill at Elev. 819. The results verified the assumptions made from Figure 5 and good jump conditions were obtained at all except low flows (Plates 7 and 8, and Figure 13, Test 6). The scour conditions with this set-up did not prove serious (Plate 7 A and B).

As soon as the drum gates on the crest were installed and in operating condition, they were used in regulating the reservoir elevations. A comparison of the stilling pool conditions with gates raised and without gates raised can be made by reference to

A. BEFORE RUN.

B. AFTER RUN.

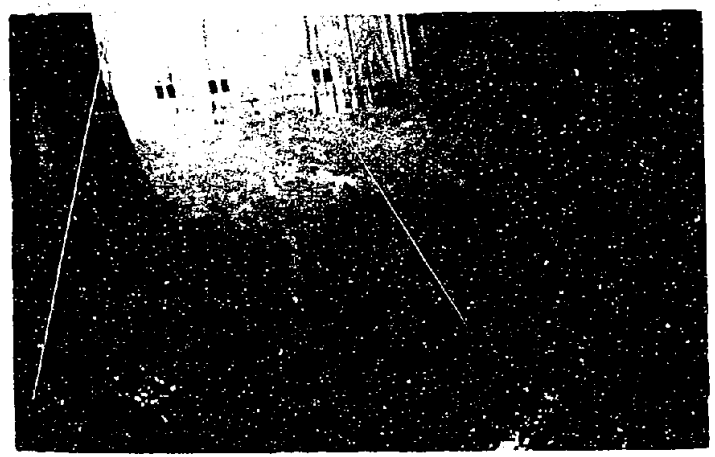
NO FLOW.

C. DISCHARGE 40,000 SECOND-FEET.

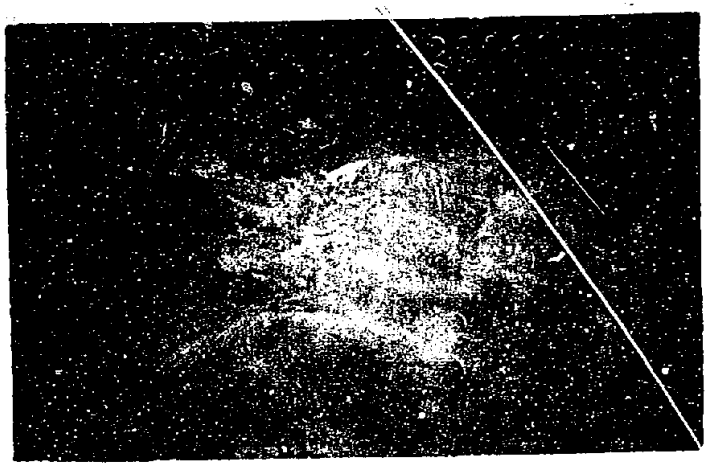
D. DISCHARGE 40,000 SECOND-FEET.

DRUM GATES NOT OPERATED. LEVEL APRON ELEV. 806 - 60' RADIUS BUCKET.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

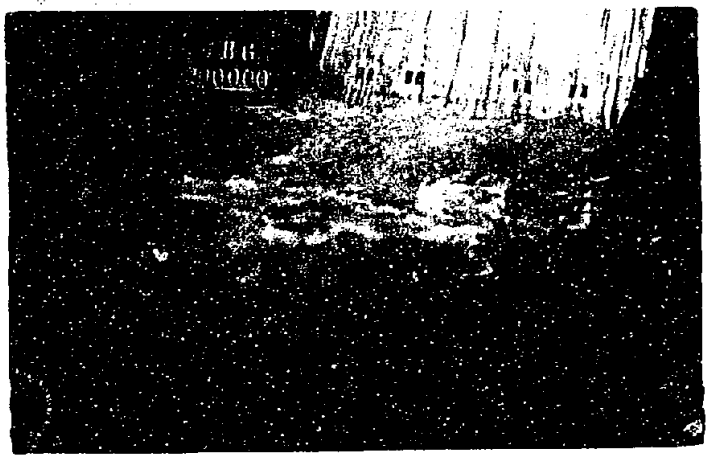
PLATE 7



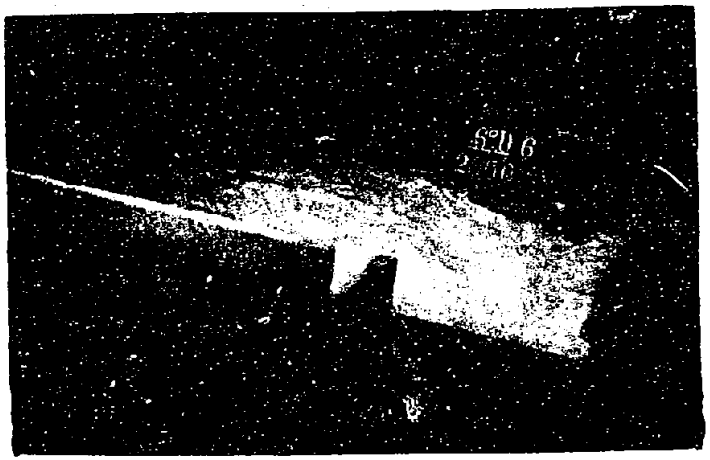
A. DISCHARGE 120,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.



C. DISCHARGE 200,000 SECOND-FEET.



D. DISCHARGE 200,000 SECOND-FEET.

DRUM GATES NOT OPERATED. LEVEL APRON ELEV. 806 - 60'RADIUS BUCKET.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

PLATE 8

FIGURE 13

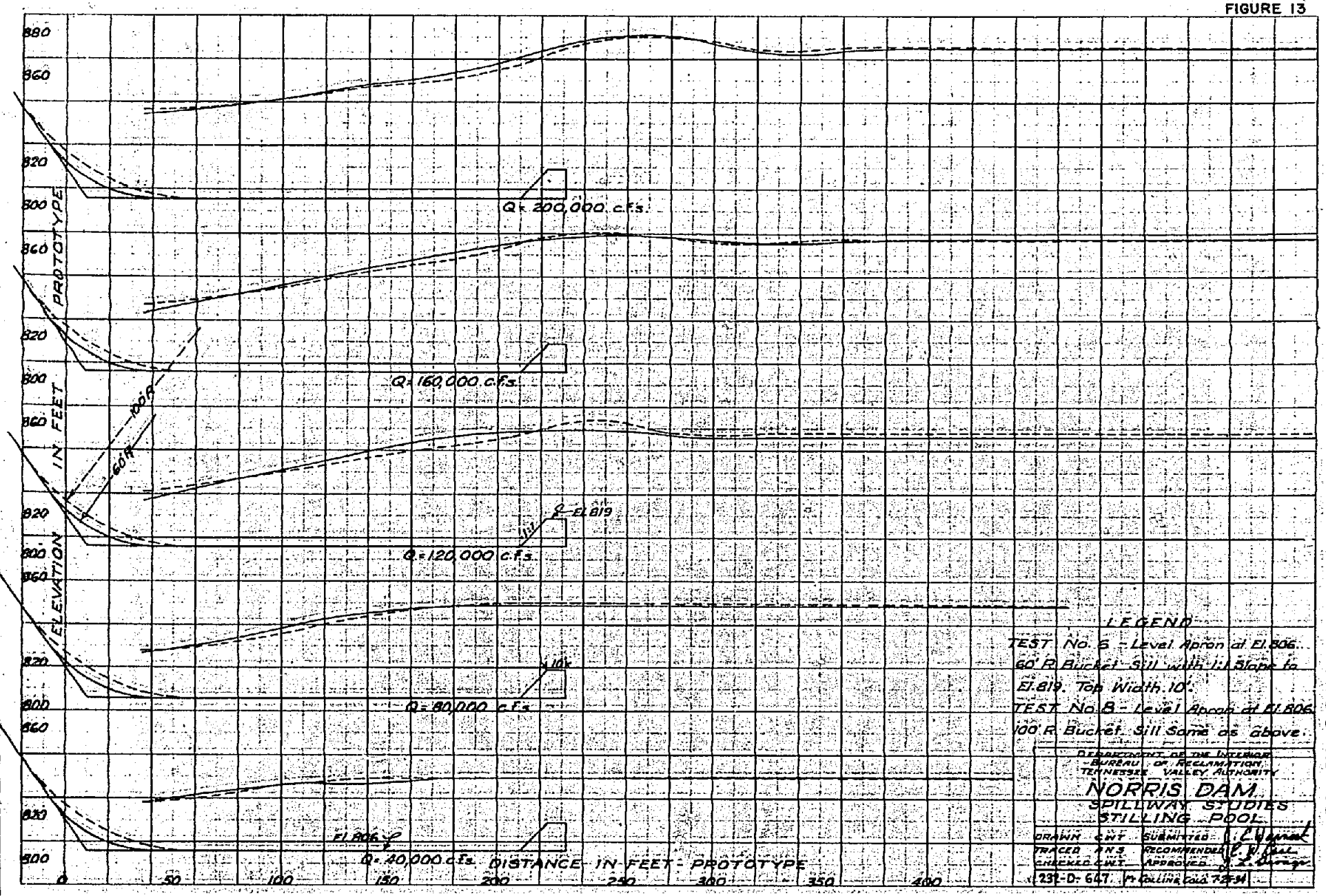


FIGURE 13

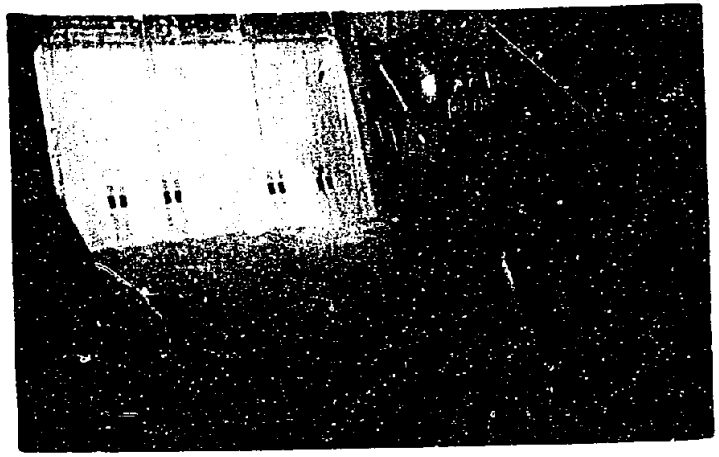
KEUFFEL & ESSER CO. N.Y.

Plates 7, 8, 9 and 10.

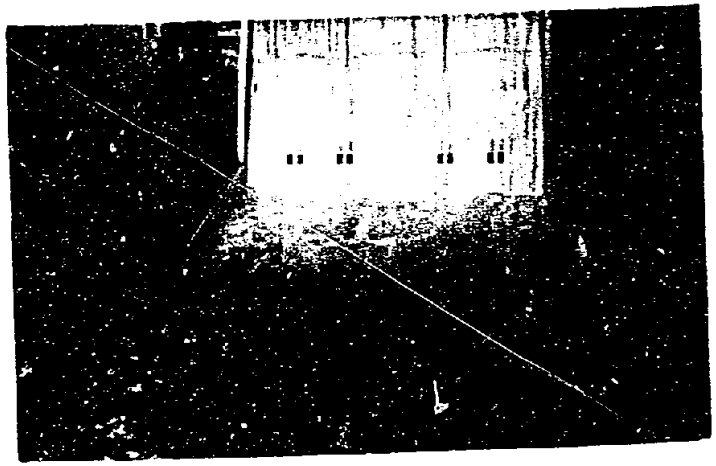
With the radius of the bucket increased from 60 to 100 feet, the flow conditions were somewhat improved, but the results were not as satisfactory as was anticipated. The profile of the water surface showed very little change from conditions with the 60-foot radius bucket (Figure 13). The flow conditions are shown on Plates 11 and 12.

The possibilities of using a combination sloping and flat apron were kept in mind, and, when the desired results of the larger radius bucket did not materialize, different slope portions of apron and 1:1 slope sill to Elev. 819 at end of apron were tested and the sections plotted as are shown on Figure 14. The 8:1 and 6:1 slopes did not allow sufficient length of flat apron at Elev. 806 to form a good jump, but the 4:1 slope gave good conditions throughout the range of flows and much better conditions at the low flows than either the flatter slopes or the level apron. The 4:1 slope was adopted for further testing.

The results of changing the slope on the face of the sill at the end of the apron from 1:1 to $1\frac{1}{2}$:1 and decreasing the top width can be seen by comparing curves marked Test 9 with those marked Test 12 on Figure 15. Less water was held on the apron with the 1:1 slope sill, but more stable condition of the water

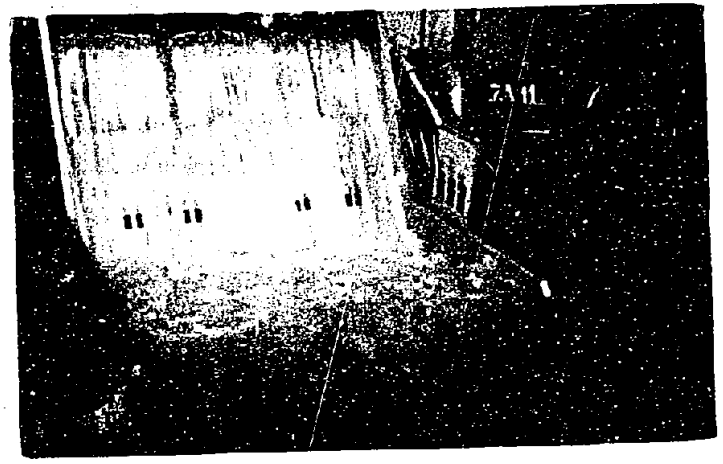


A. DISCHARGE 40,000 SECOND-FEET.

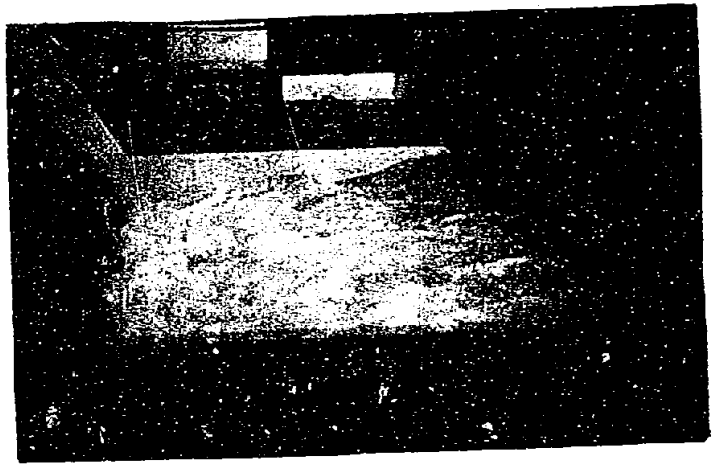


B. RESERVOIR ELEVATION 1052.

GATES ELEV. 1034.



C. DISCHARGE 120,000 SECOND-FEET.

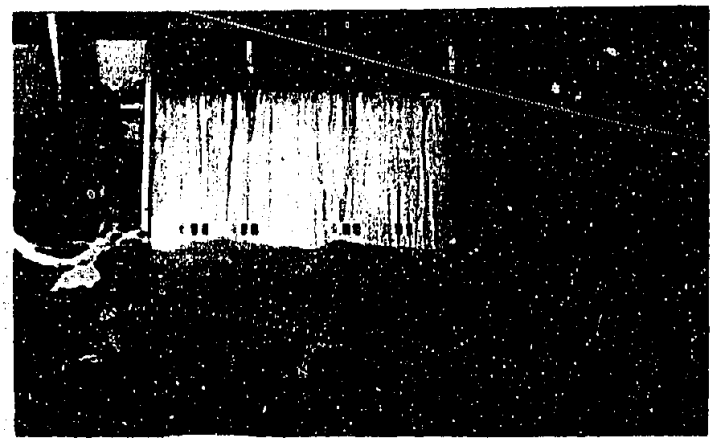


D. DISCHARGE 120,000 SECOND-FEET.

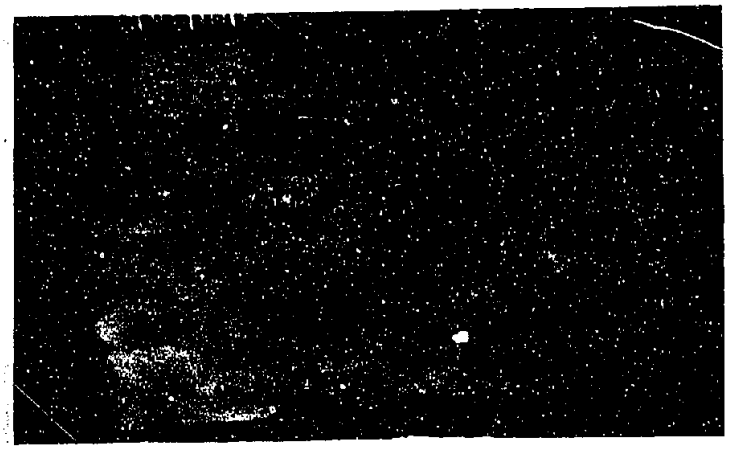
RESERVOIR ELEVATION 1052.

DRUM GATES OPERATED. LEVEL APRON ELEV. 806 - 60' RADIUS BUCKET.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

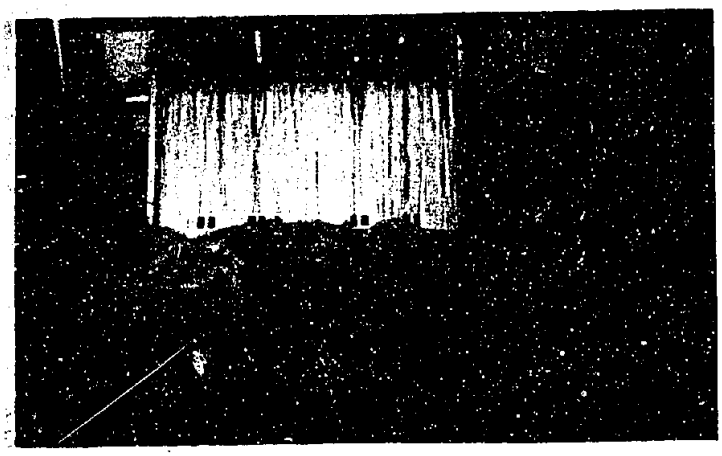
PLATE 9



A. DISCHARGE 200,000 SECOND-FEET.



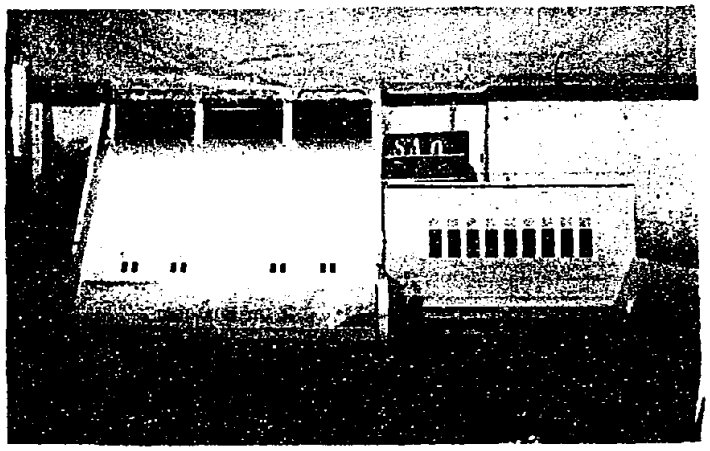
B. DISCHARGE 200,000 SECOND-FEET.



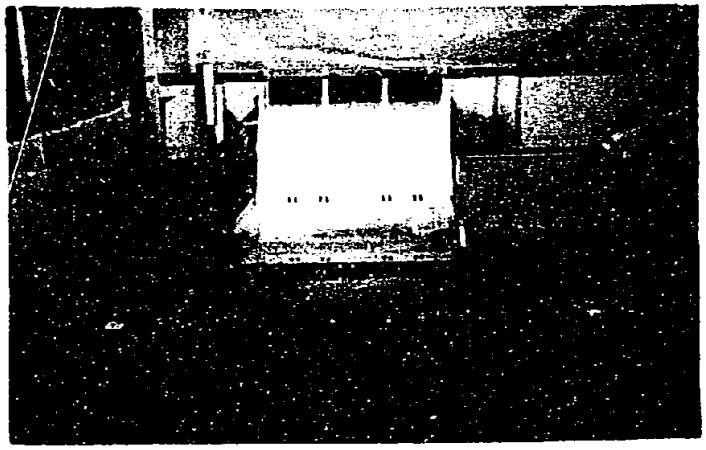
C. DISCHARGE 240,000 SEC-FT. SLUICES OPEN.

GATES ELEV. 1020

DRUM GATES OPERATED. LEVEL APRON ELEV. 806 - 60' RADIUS BUCKET.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

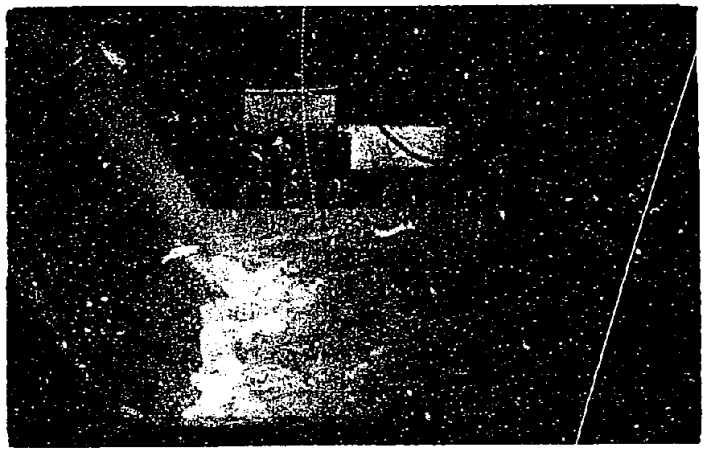


A. BEFORE RUN.

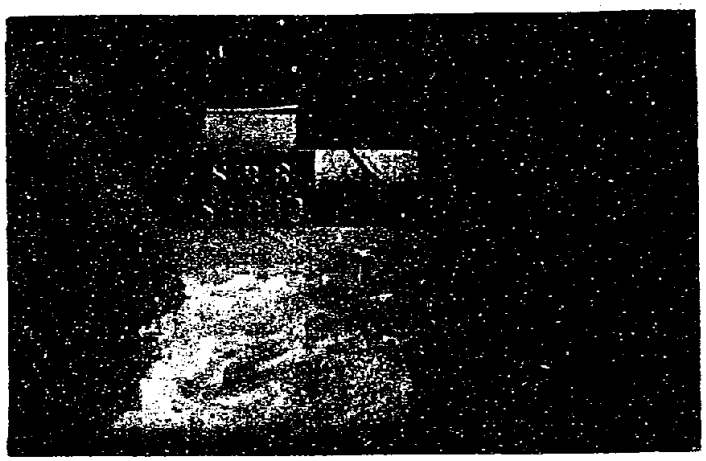


B. AFTER RUN.

NO FLOW.



C. DISCHARGE 40,000 SECOND-FEET.



D. DISCHARGE 80,000 SECOND-FEET.

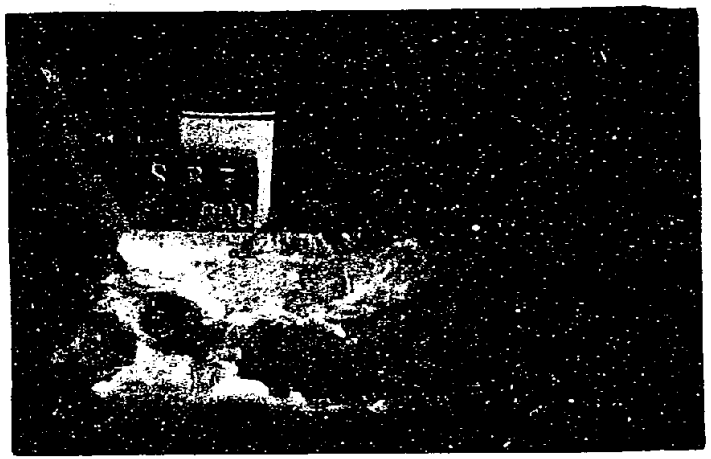
LEVEL APRON ELEV. 806 - 100' RADIUS BUCKET.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

PLATE 11

A. DISCHARGE 120,000 SECOND-FEET.

B. DISCHARGE 160,000 SECOND-FEET.

C. DISCHARGE 200,000 SECOND-FEET.



D. DISCHARGE 240,000 SECOND-FEET.

LEVEL APRON ELEV. 806 - 100' RADIUS BUCKET.
1:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

PLATE 12

FIGURE 14

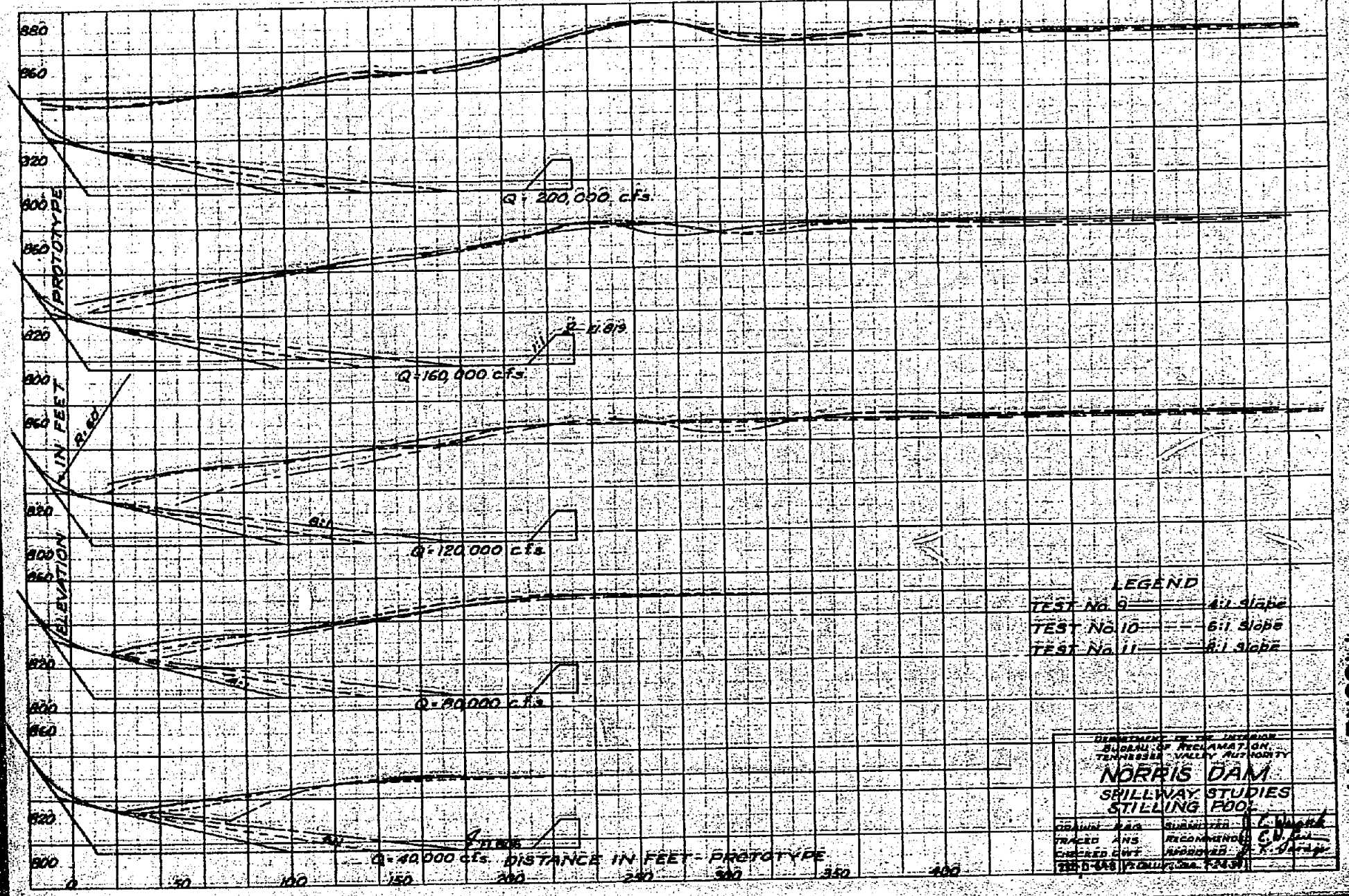


FIGURE 14

FIGURE 15

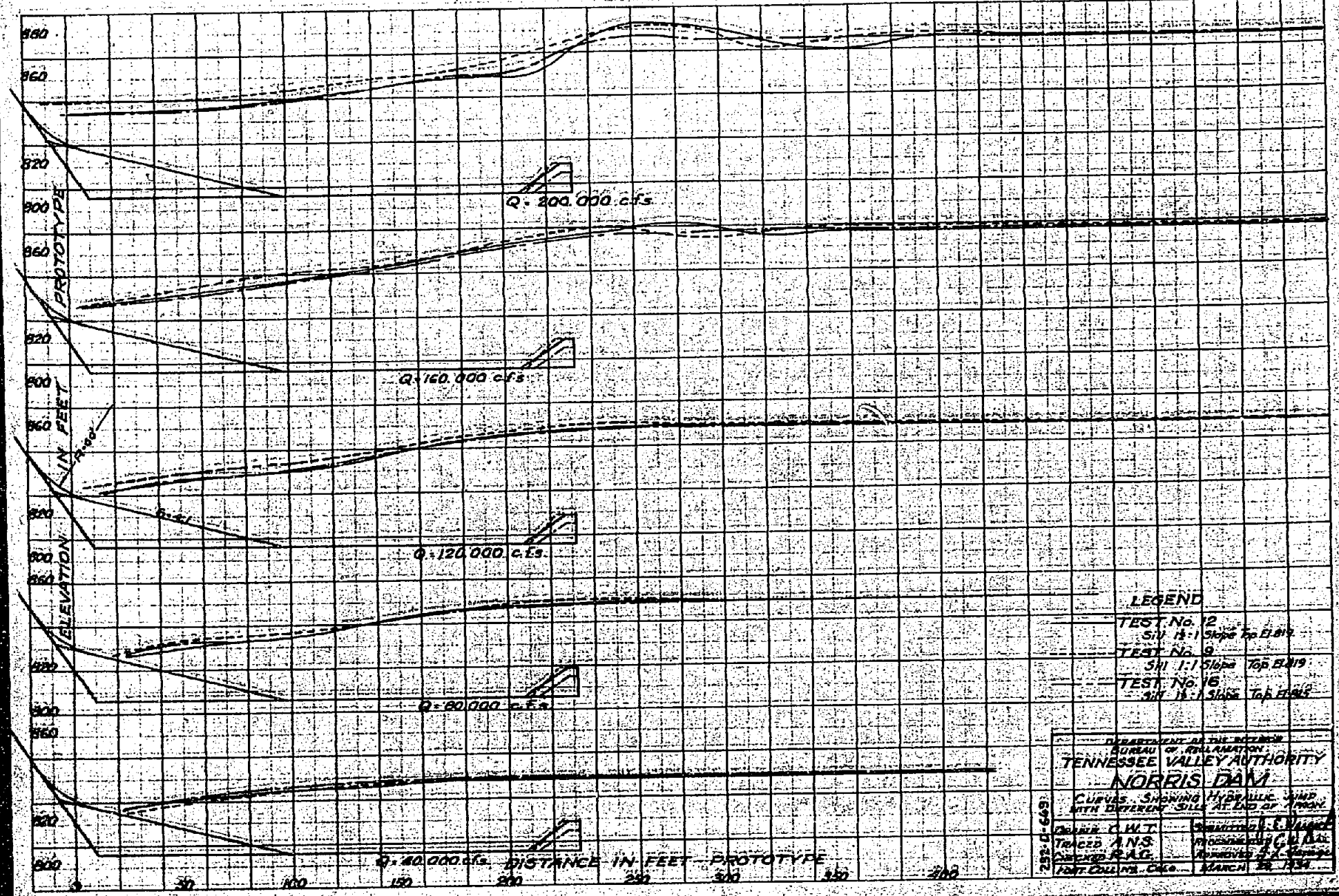


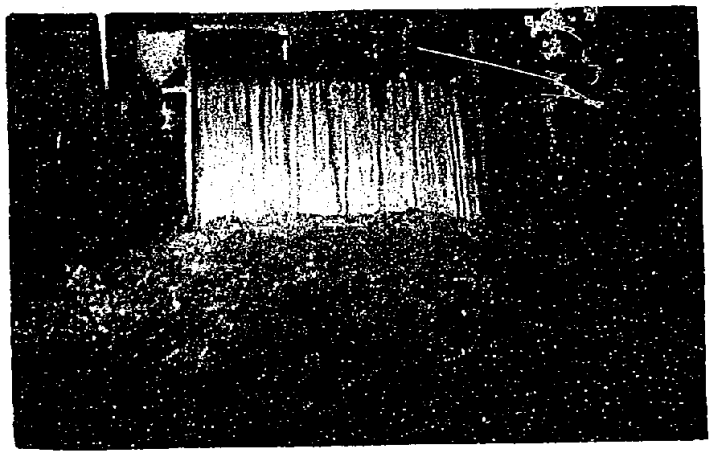
FIGURE 15

surface was noted (Plates 13 and 14).

To effect a saving in the yardage of concrete in the sill at the end of the apron, the top was lowered to Elev. 815, but the results showed higher velocities passing the sill and the appearance of the water surface (Plate 15) was not so good. The profile of the water surface through the jump is shown as Test 16, Figure 15, and is compared with the 1:1 slope sill and the $1\frac{1}{2}$:1 slope sill with the top at Elev. 819.

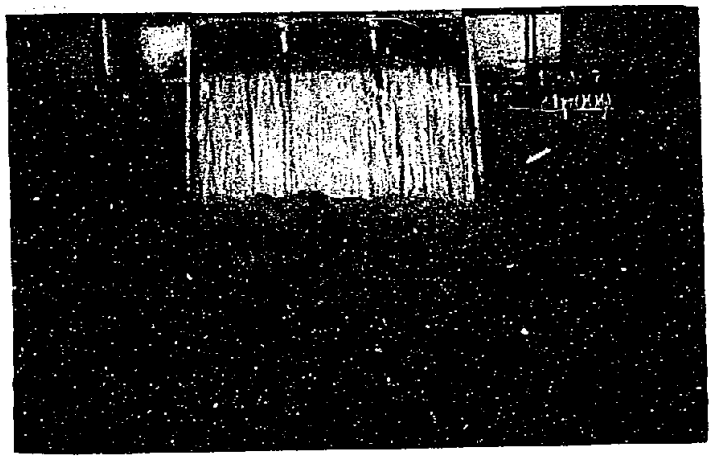
The tests on the small auxiliary trapezoidal sills suggested by Mr. Charles H. Paul showed more water retained on the apron at most flows (Figure 16), but disturbances set up on the apron (Plates 16, 17 and 18C) caused a very undesirable condition in the stilling pool. The sills were also subjected to severe impact by the water flowing at high velocity down the apron, thereby making it highly probable that the sills would suffer considerable scour. The sudden rise in water surface shown on Plates 16C, 17C and 18C would also likely increase the uplift on the apron. The action caused by the sills at low flows was not undesirable, as can be seen from Plates 16 A, 17 A and 18 A. A summary of results, however, showed a generally undesirable condition.

The low diffuser type sill tested at two positions on

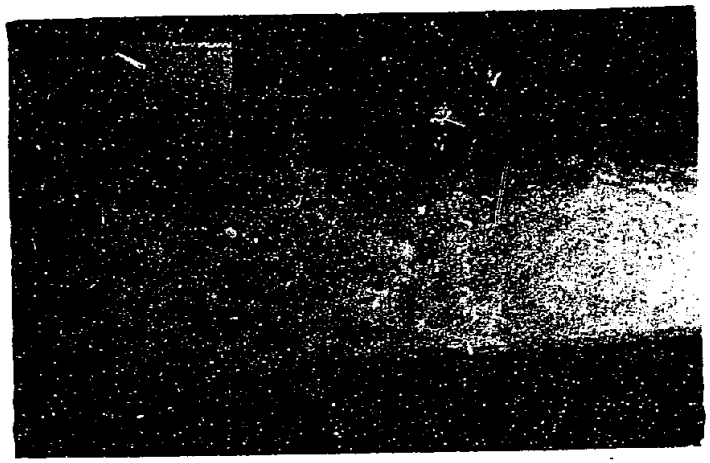


A. DISCHARGE 200,000 SECOND-FEET.

B. DISCHARGE 200,000 SECOND-FEET.



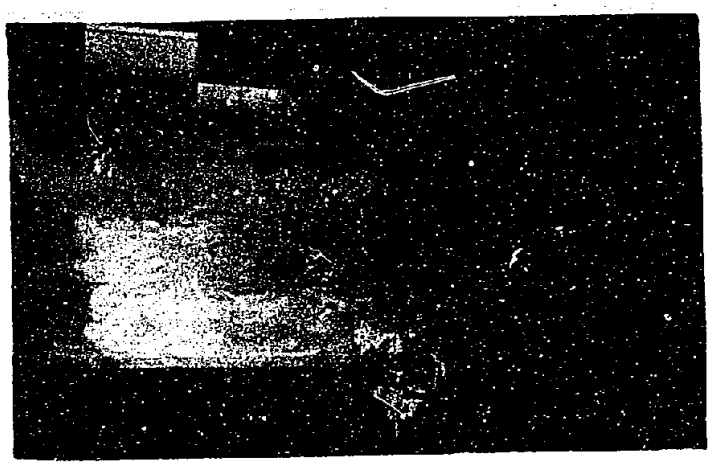
C. DISCHARGE 240,000 SECOND-FEET.



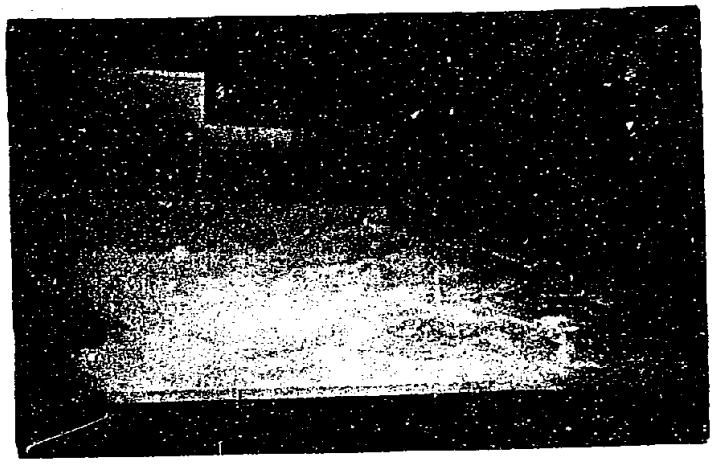
D. DISCHARGE 240,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
$1\frac{1}{2}$:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.

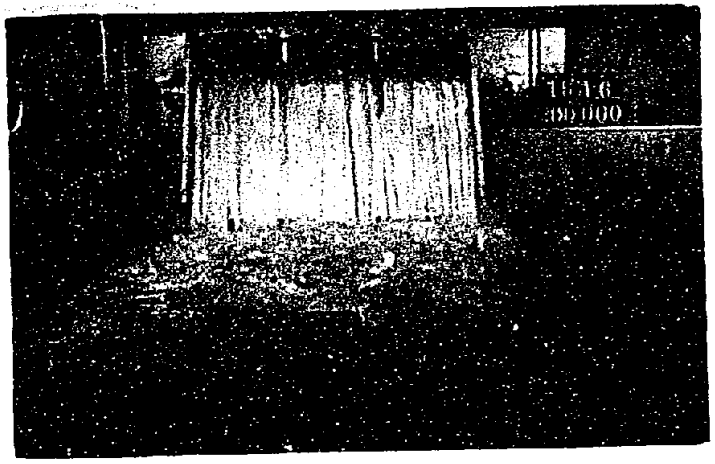
PLATE 14



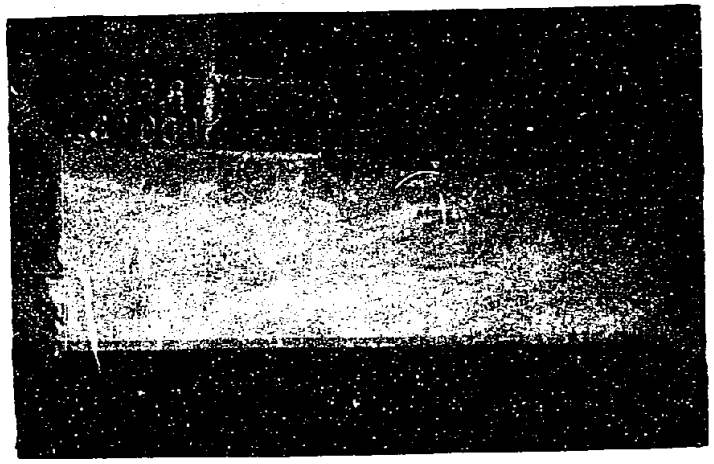
A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.



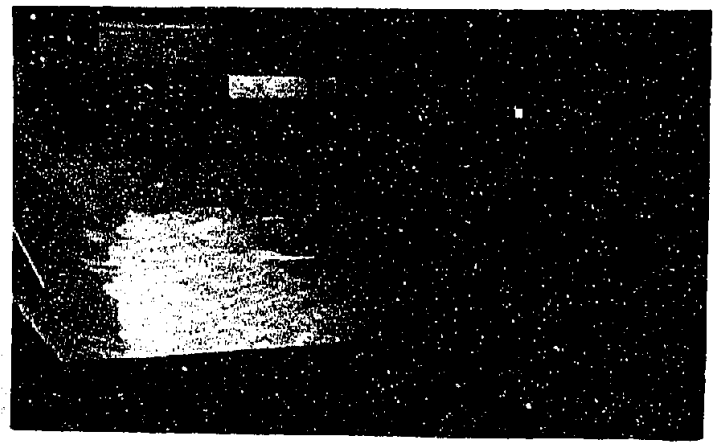
C. DISCHARGE 200,000 SECOND-FEET.



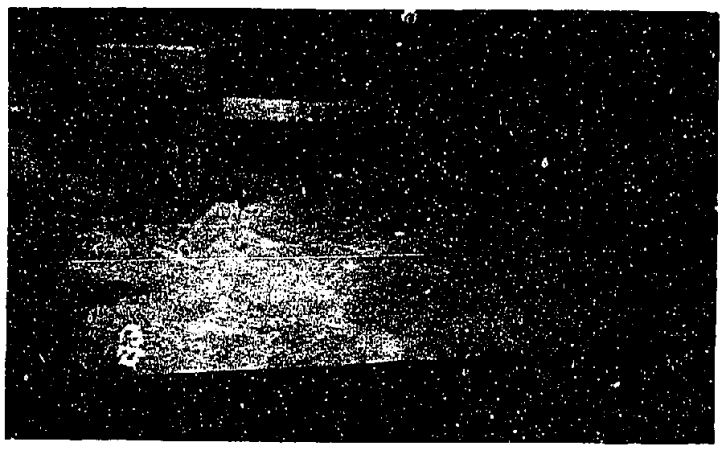
D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
1½:1 SLOPE SILL TO ELEV. 815 AT END OF APRON.

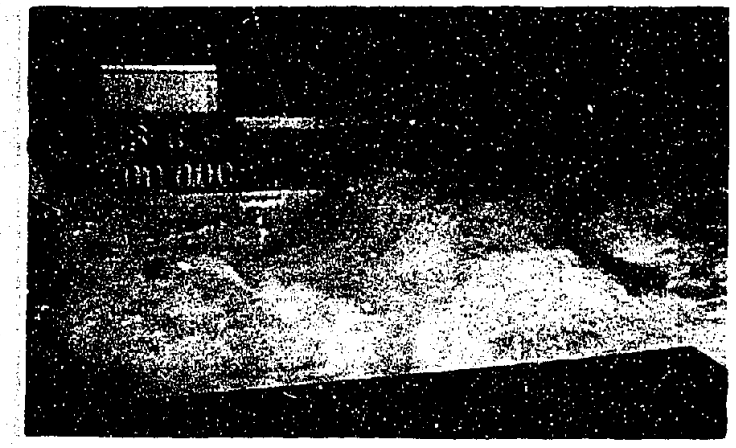
PLATE 16



A. DISCHARGE 40,000 SECOND-FEET.

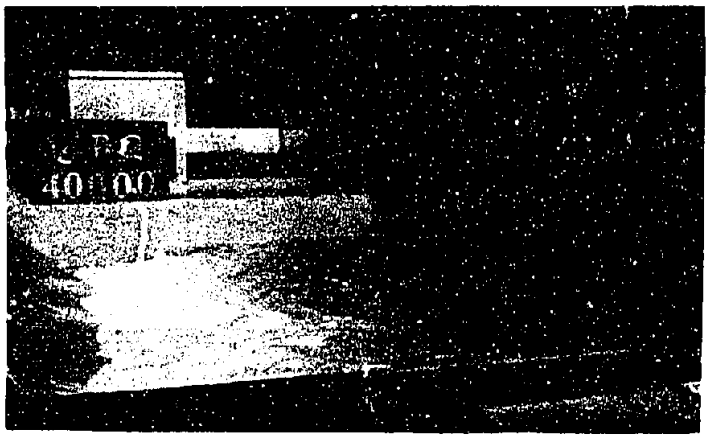


B. DISCHARGE 120,000 SECOND-FEET.

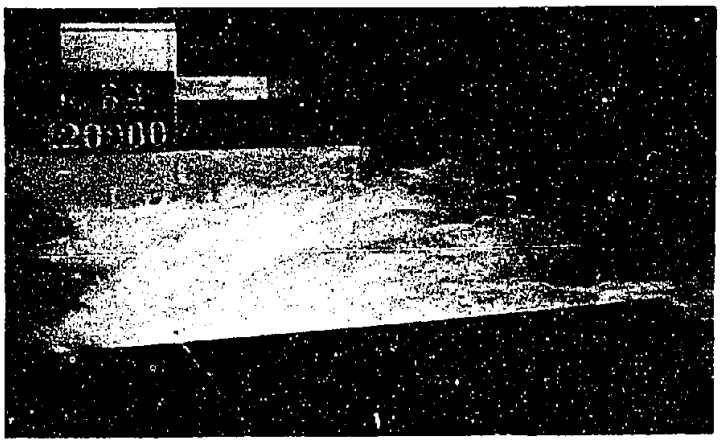


C. DISCHARGE 200,000 SECOND-FEET.

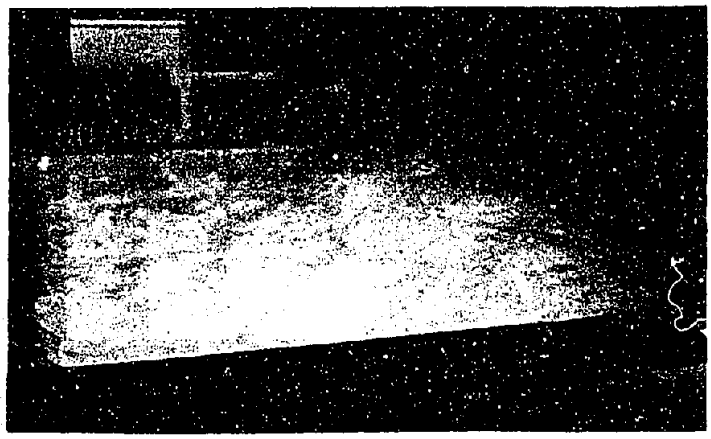
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
1½:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.
6' AUXILIARY SILL 80.7' DOWNSTREAM FROM END OF 4:1 SLOPE.



A. DISCHARGE 40,000 SECOND-FEET.

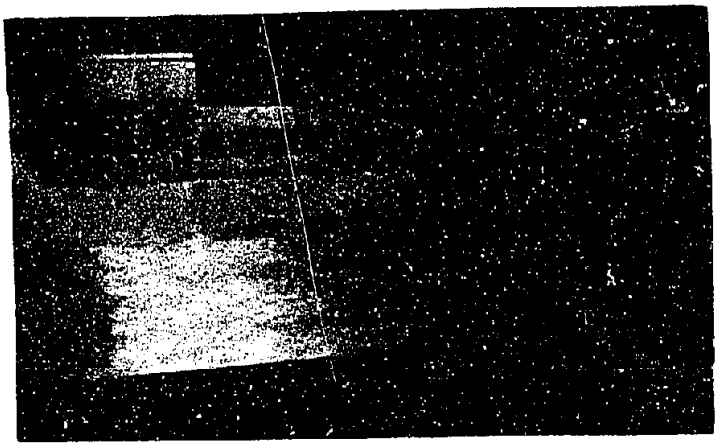


B. DISCHARGE 120,000 SECOND-FEET.

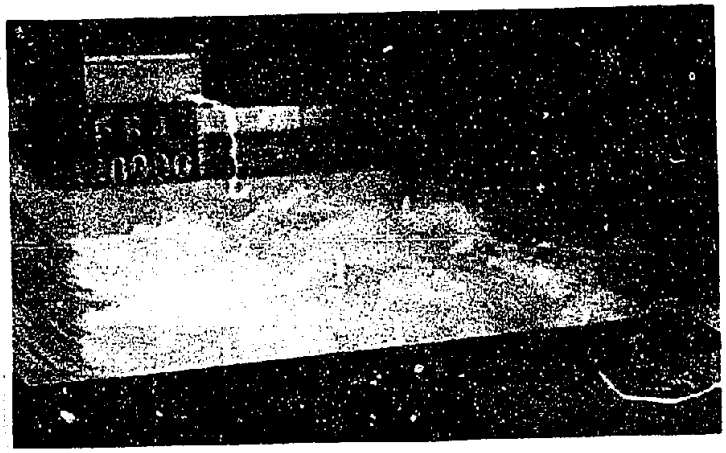


C. DISCHARGE 200,000 SECOND-FEET.

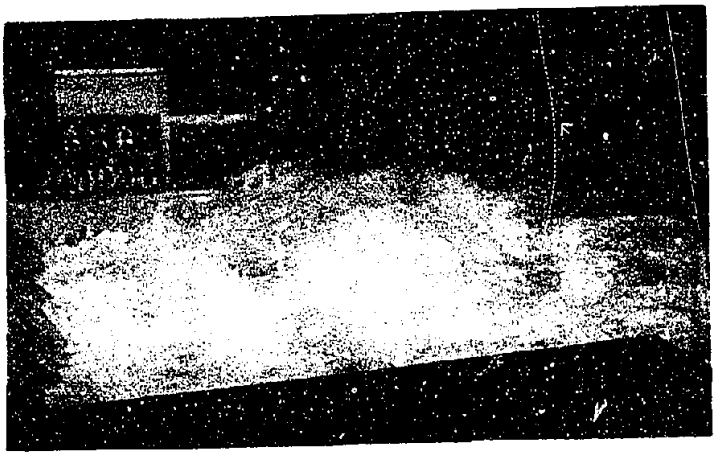
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
1½:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.
4' AUXILIARY SILL 80.7' DOWNSTREAM FROM END OF 4:1 SLOPE.



A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.



C. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
1½:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.
4' AUXILIARY SILL 44.7' DOWNSTREAM FROM END OF 4:1 SLOPE.

FIGURE 16

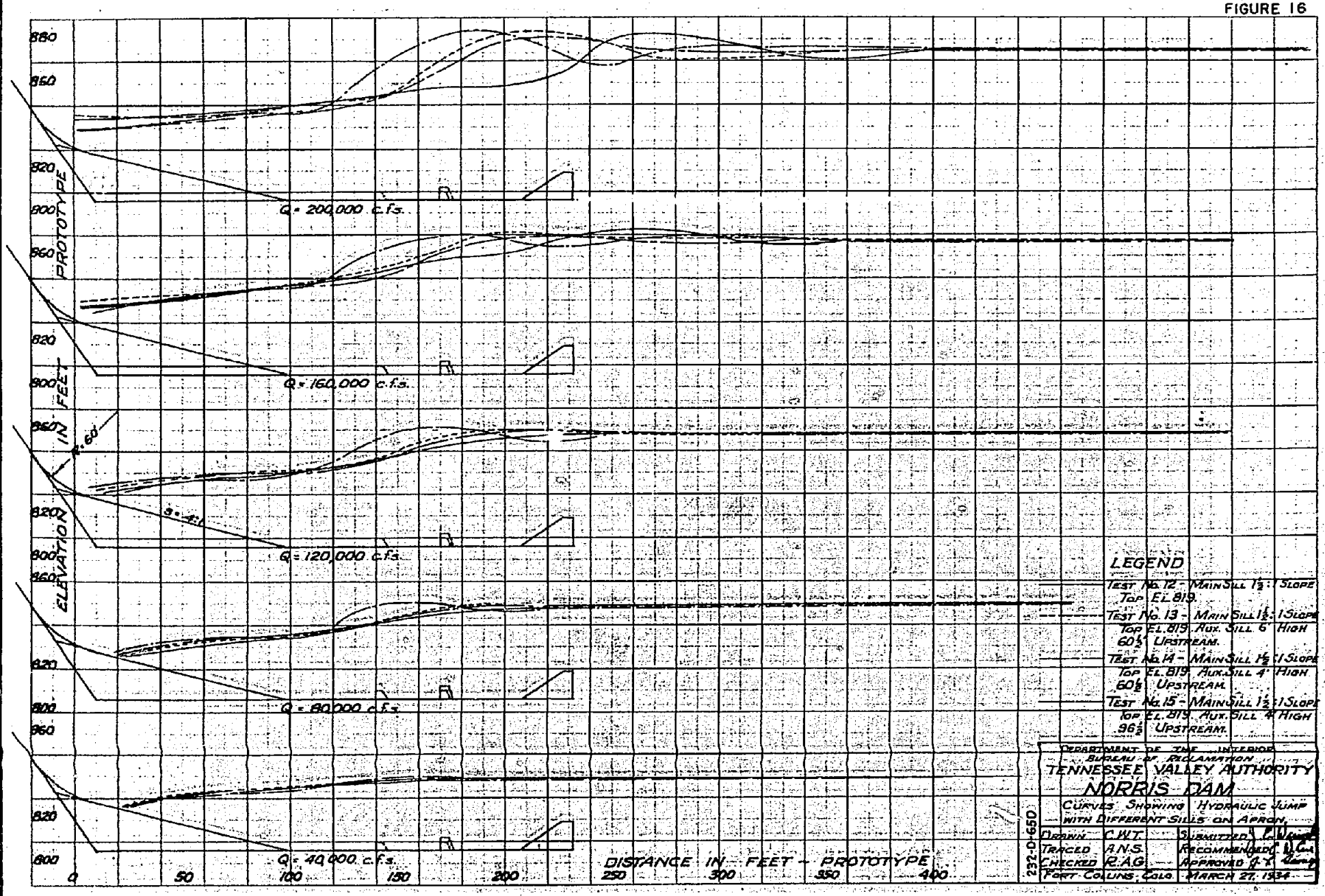
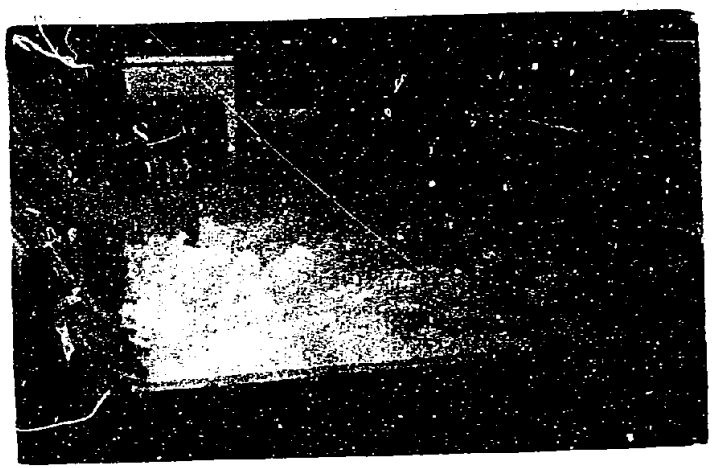


FIGURE 16

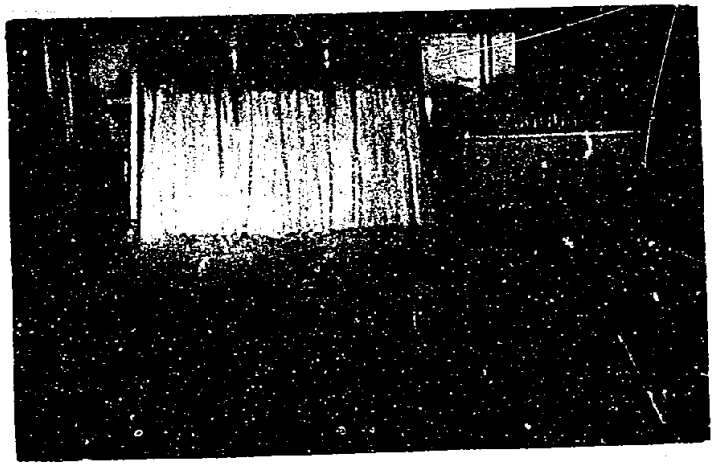
the apron in conjunction with the $1\frac{1}{2}$:1 slope sill, top Elev. 819, at end of apron was installed primarily to spread the flow of water from the sluiceways. The flow conditions for the spillway operating were slightly improved, but the blocks were too small to effect a material change in flow conditions, although the surface conditions were improved. The comparison of the profiles of the water surface with the $1\frac{1}{2}$:1 sill, top Elev. 819, with and without the diffuser sill on the apron is given on Figure 17. The flow conditions for the two positions of the sill on the apron are shown on Plates 19 and 20. Very good flow conditions were obtained at the low and intermediate discharges as can be seen in Illustrations A and B, Plates 19 and 20. The cost of this type of baffle would be high, but a saving might be effected by shortening the apron and thus render possible the use of this type of sill at little additional cost. However, the sill did not solve the sluice problem and was therefore abandoned.

It was anticipated by using the sill with curved upstream face, suggested by Mr. Savage, that the sheet of high velocity water along the apron would be turned upstream and impinge on the incoming jet. The results showed the jet to be of such depth by the time it reached the sill that it completely filled the concave face and formed in effect a sill with a vertical upstream face. This caused disturbances in the pool over the sill and

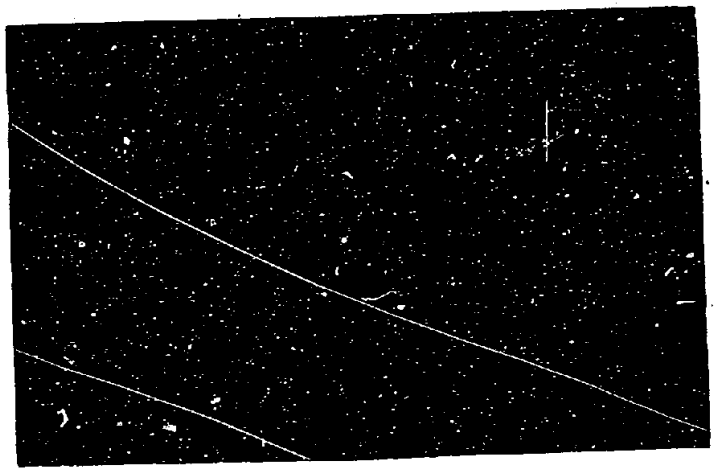
A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.



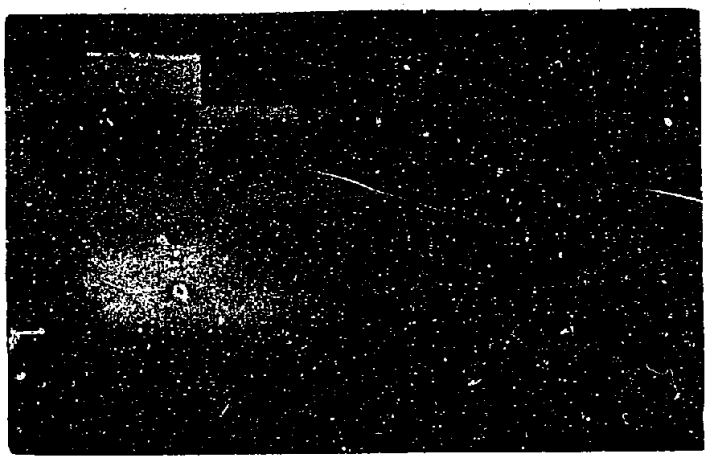
C. DISCHARGE 200,000 SECOND-FEET.



D. DISCHARGE 200,000 SECOND-FEET.

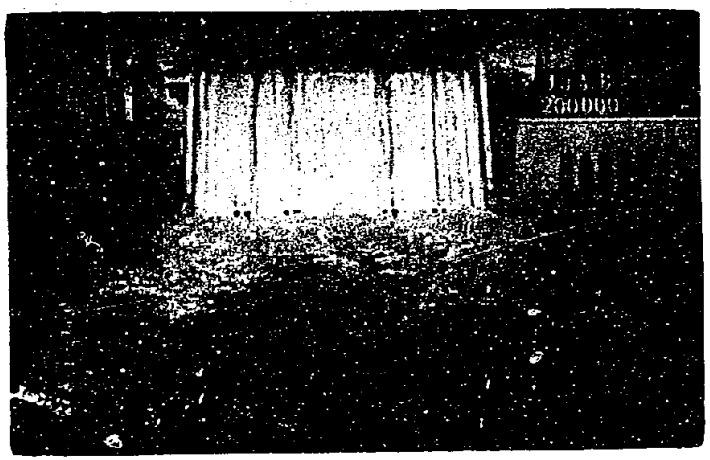
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
1½:1 SLOPE SILL TO ELEV. 819 AT END OF APRON.
LOW DIFFUSER SILL, FRONT EDGE 2¼' DOWNSTREAM FROM END OF 4:1 SLOPE.

PLATE 19

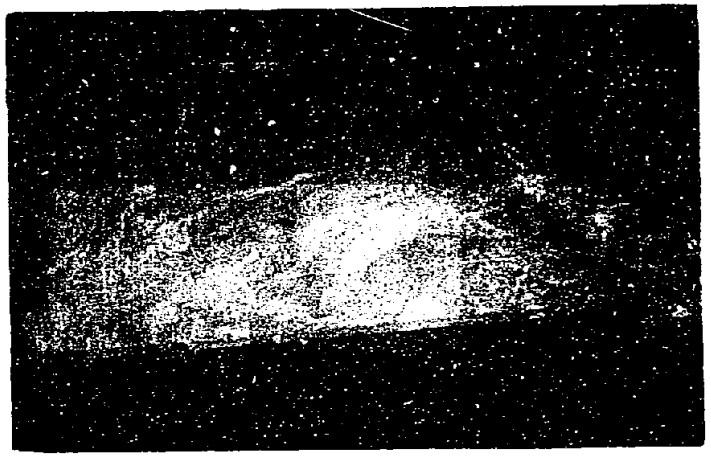


A. DISCHARGE 40,000 SECOND-FEET.

B. DISCHARGE 120,000 SECOND-FEET.



C. DISCHARGE 200,000 SECOND-FEET.



D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806. NO SILL AT END OF APRON.
LOW DIFFUSER SILL, FRONT EDGE 32¼' DOWNSTREAM FROM END OF 4:1 SLOPE.

FIGURE 17

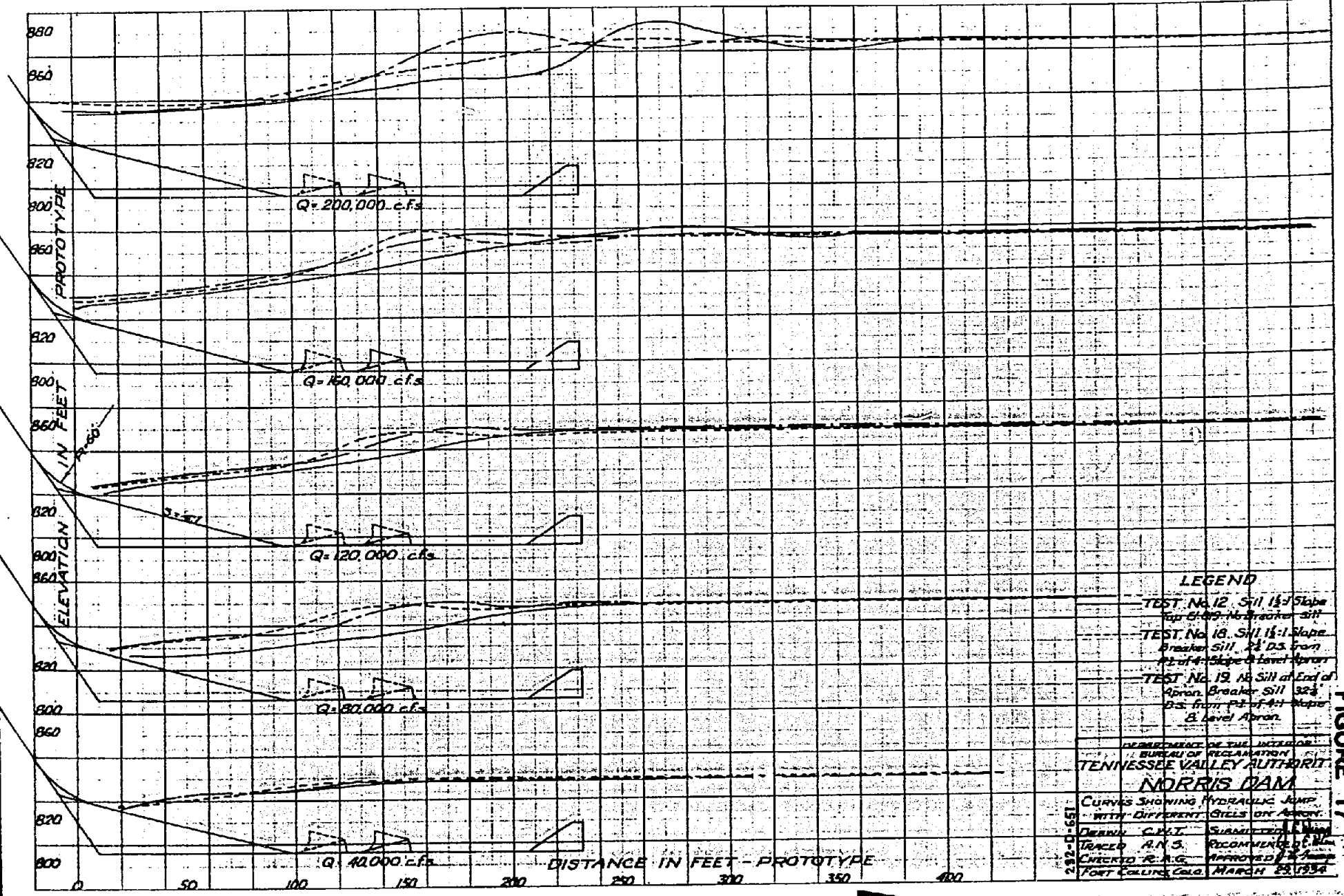
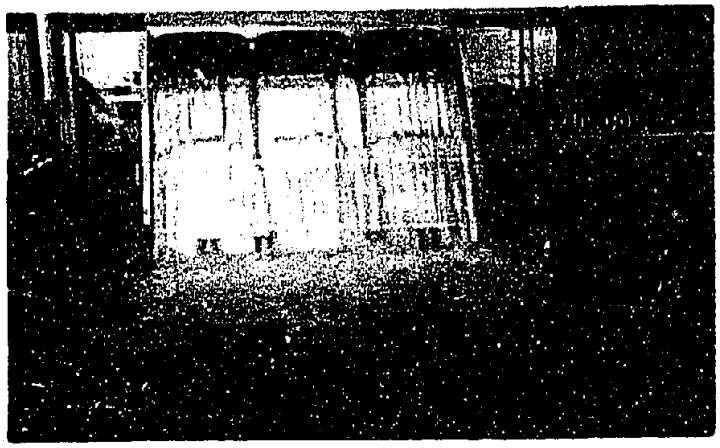


FIGURE 17

very rough surface conditions in the entire pool as can be seen on Plates 21 and 22. The profiles of the water surface in the pool is shown by Test 23, Figure 18. If the radius of the curve could have been increased considerably so that the entire high velocity jet would strike the lower half of the curve the predicted action might have taken place, but this would have necessitated a structure of such size as to be unreasonable.

The above solution offered possibilities in that the curved upstream face and overhang might be applied to the diffuser type blocks. Such a sill was tested at different positions on the flat portion of apron, thus simulating a shortening of the apron. The results were satisfactory. Smoother conditions of the water surface were obtained and the velocities beyond the end of the apron were very evenly distributed. Paint tests disclosed that part of the jet was turned back on the apron by the curved upstream face of the block thus causing an ellipsoidal roll to form on the apron ahead of the sill. Another portion of the water passed between the blocks and was deflected upward by the sloping portion of the sill and allowed to spread due to the increase in area between the blocks as the water moved upward and outward. The high velocity jets flowing between the blocks were further

A. DISCHARGE 40,000 SECOND-FEET.

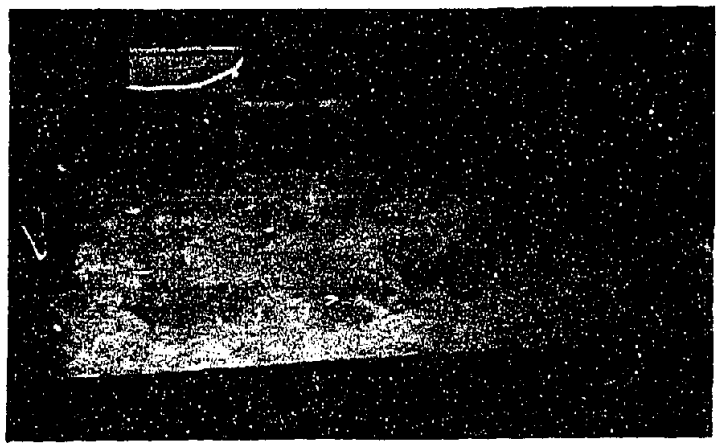


B. DISCHARGE 120,000 SECOND-FEET.

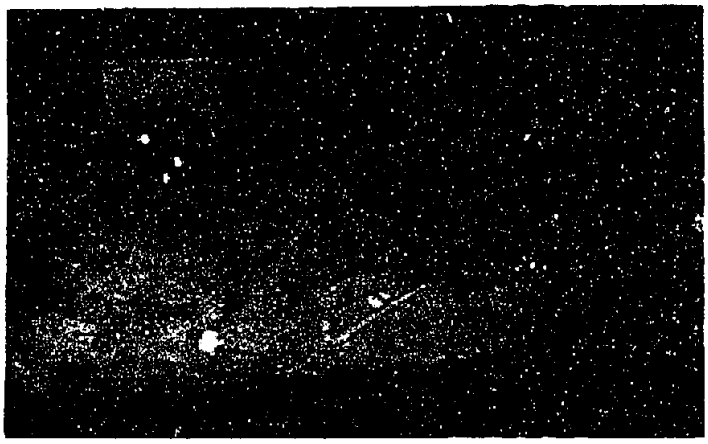
C. DISCHARGE 120,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
CURVED UPSTREAM FACE SILL TOP ELEV. 819 AT END OF APRON.

A. DISCHARGE 200,000 SECOND-FEET.



B. DISCHARGE 200,000 SECOND-FEET.



C. DISCHARGE 240,000 SECOND-FEET.

(6) SLOPING APRON TO LEVEL APRON ELEV. 804.
CURVED UPSTREAM FACE SILL TOP ELEV. 819 AT END OF APRON.

FIGURE 18

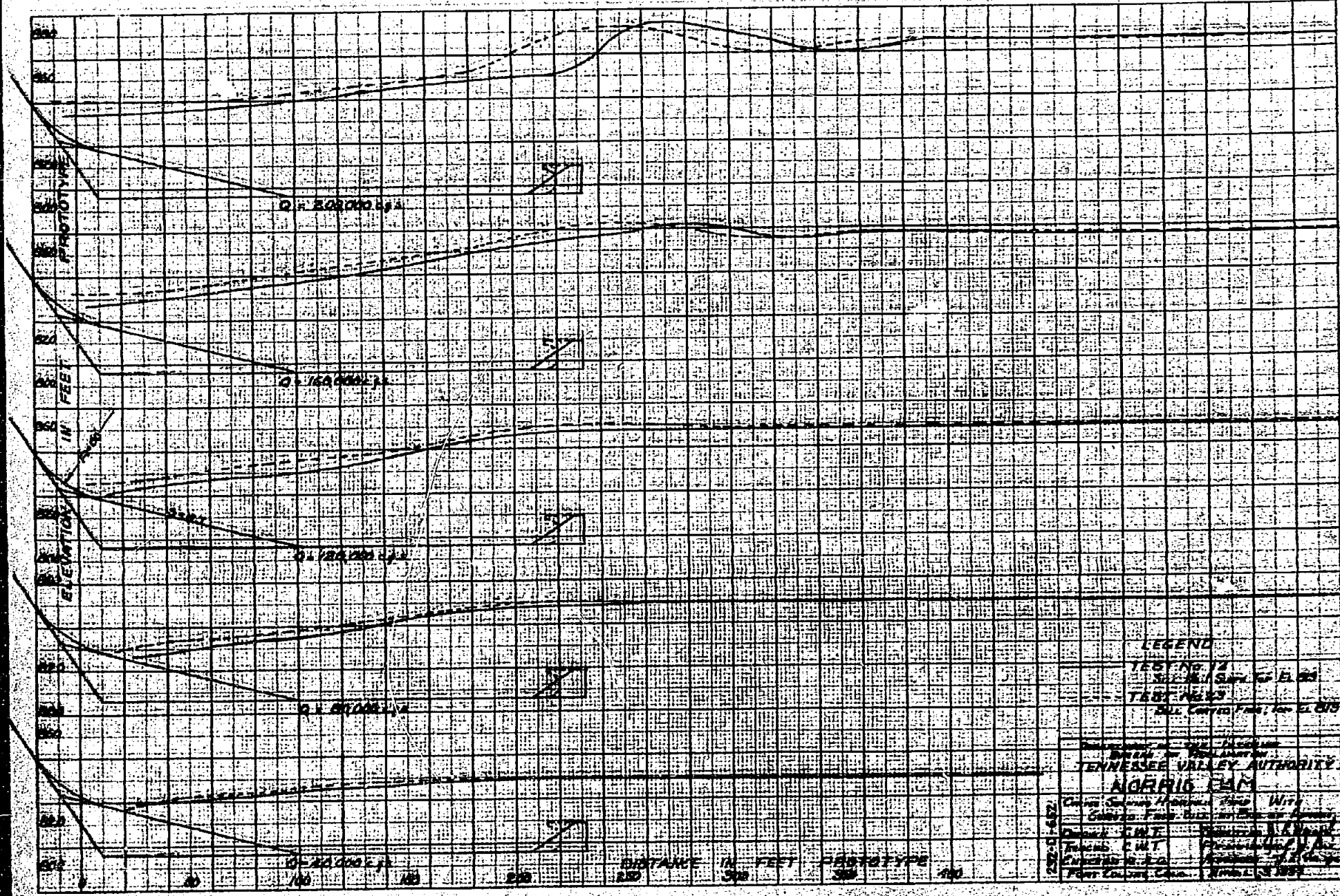


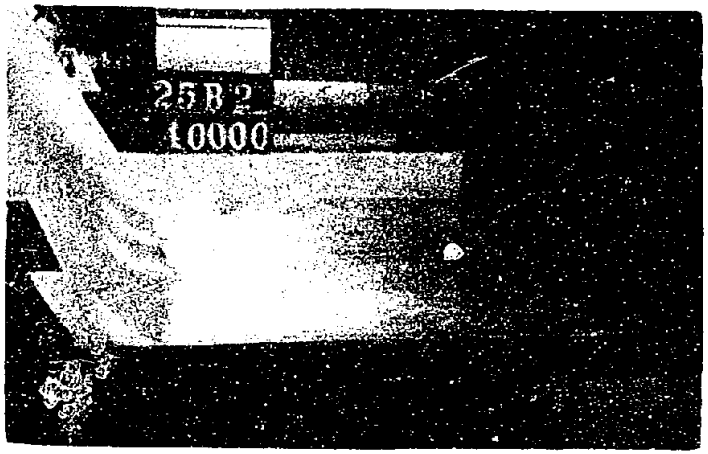
FIGURE 18

broken up and the velocity diminished by the effect of the water spilling off the face of the blocks and into the chamber. The velocity remaining in the water after it passes the sill is moving in an upward direction away from the river bed thus diminishing the possibility of scour downstream from the apron. There was very little scour noted and a tendency existed to deposit material at the end of the apron.

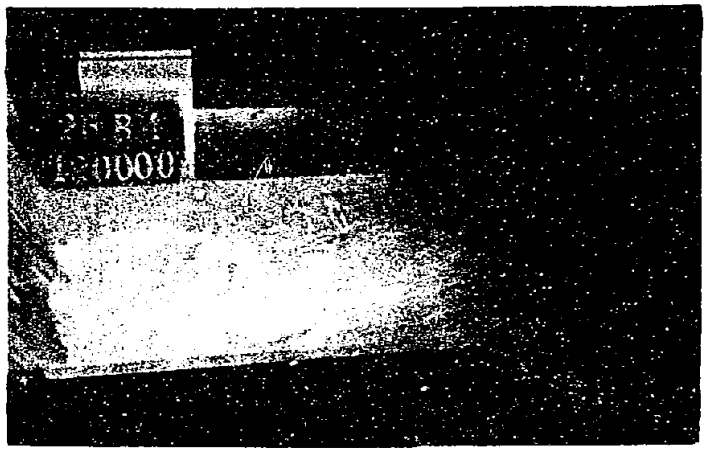
Good results were obtained at all positions of the sill on the apron, although the best results were found when the apron was shortened 60 feet. The profiles of the water surface and the locations at which the sill was tested are shown on Figure 19. The flow conditions are shown on Plates 23 to 27 inclusive.

By use of this sill the apron and training walls could have been shortened 60 feet and better flow conditions obtained than with the longer apron. The high initial cost combined with the probability of damage due to excessive erosion of the sill or impact of logs on the dentates made the use of this sill inadvisable.

The cost of construction and probability of erosion of the sill were lessened by removing the overhanging noses from the blocks thus making the faces of the blocks tangent to the curve at thirty degrees with the vertical. The results (Figure 20 and

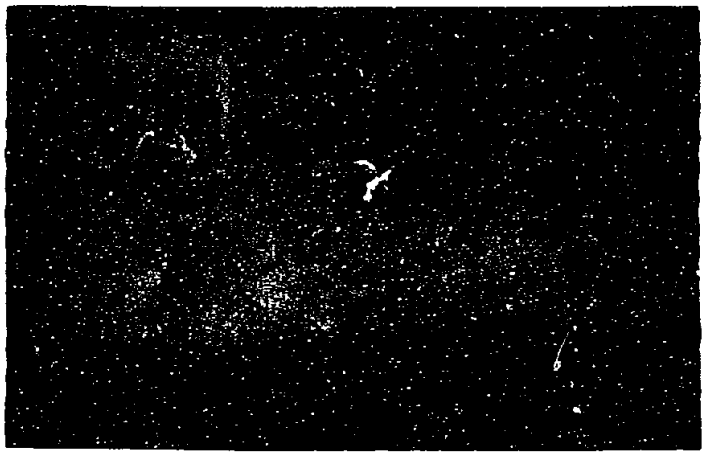


A. DISCHARGE 40,000 SECOND-FEET.



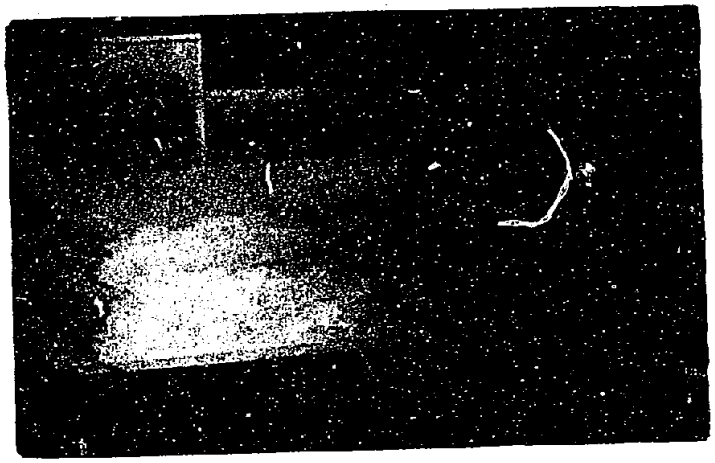
B. DISCHARGE 120,000 SECOND-FEET.

C. DISCHARGE 200,000 SECOND-FEET.

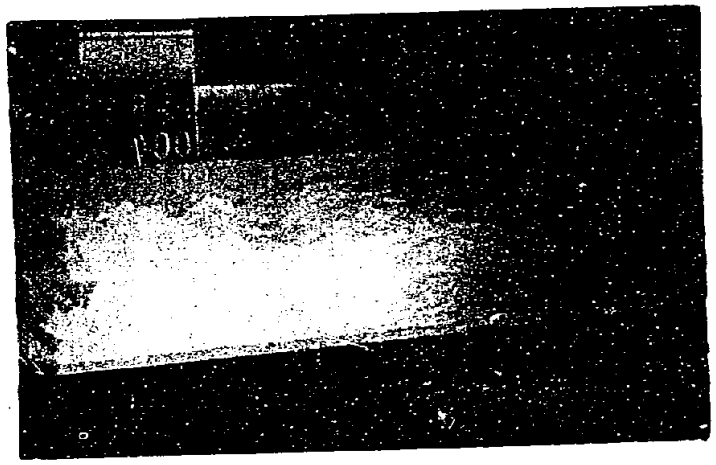


D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, WITH OVERHANG, AT END OF APRON.

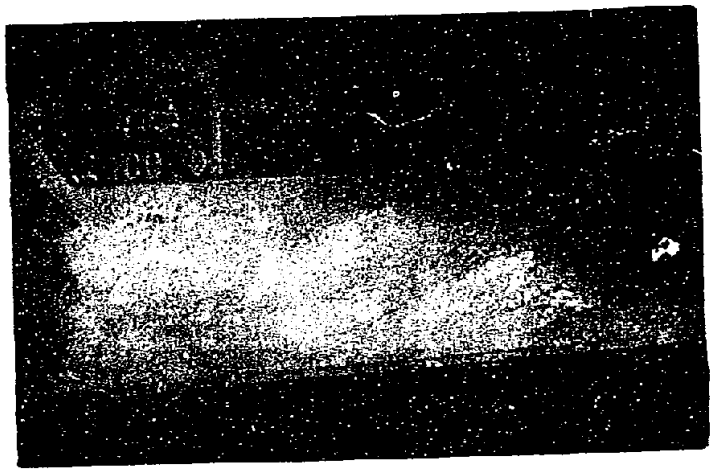


A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.

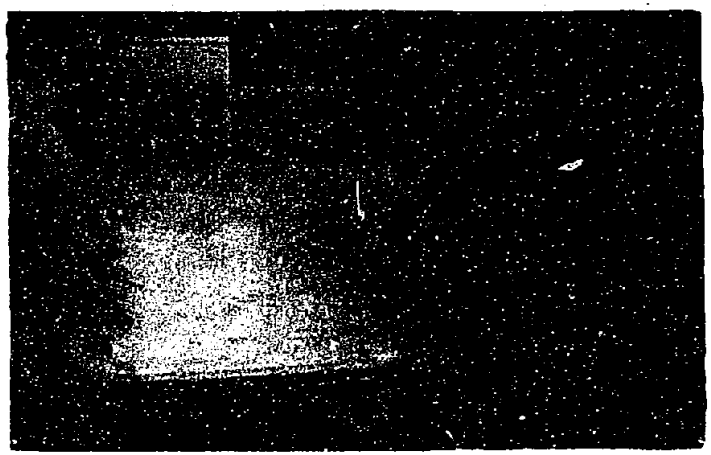
C. DISCHARGE 200,000 SECOND-FEET.



D. DISCHARGE 200,000 SECOND-FEET.

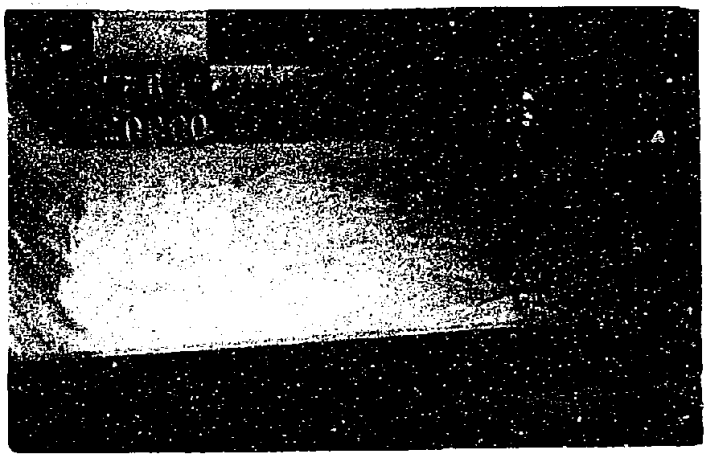
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, WITH OVERHANG, 30' UPSTREAM FROM END OF APRON.

PLATE 24



A. DISCHARGE 40,000 SECOND-FEET.

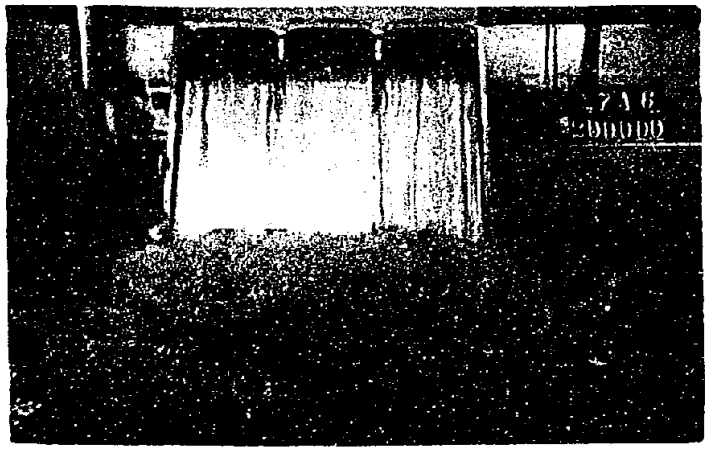
B. DISCHARGE 80,000 SECOND-FEET.



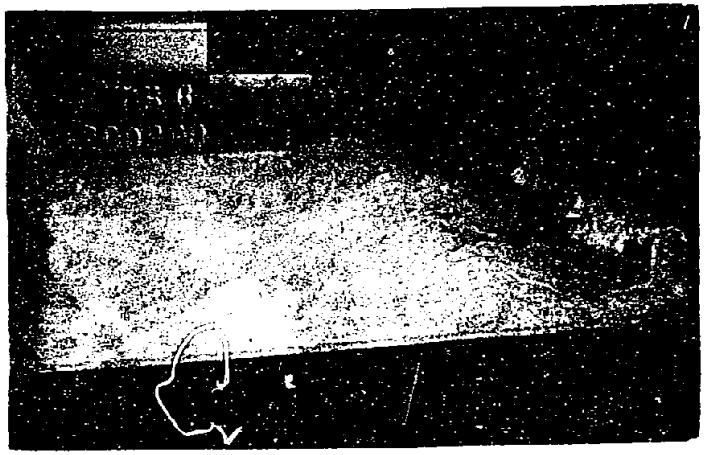
C. DISCHARGE 120,000 SECOND-FEET.

D. DISCHARGE 160,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, WITH OVERHANG, 60' UPSTREAM FROM END OF APRON.

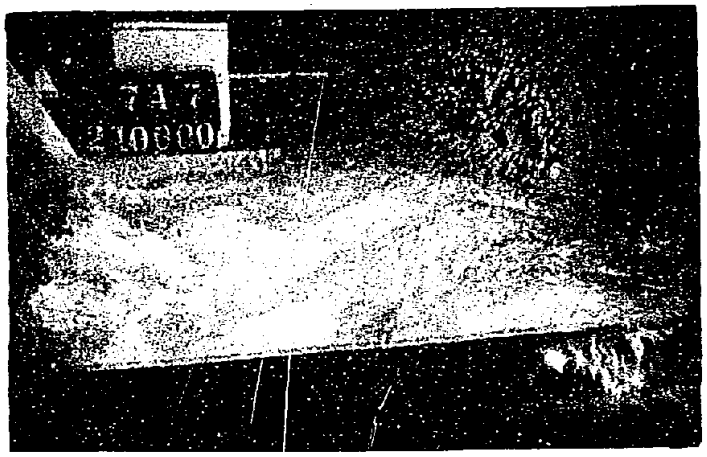


A. DISCHARGE 200,000 SECOND-FEET.



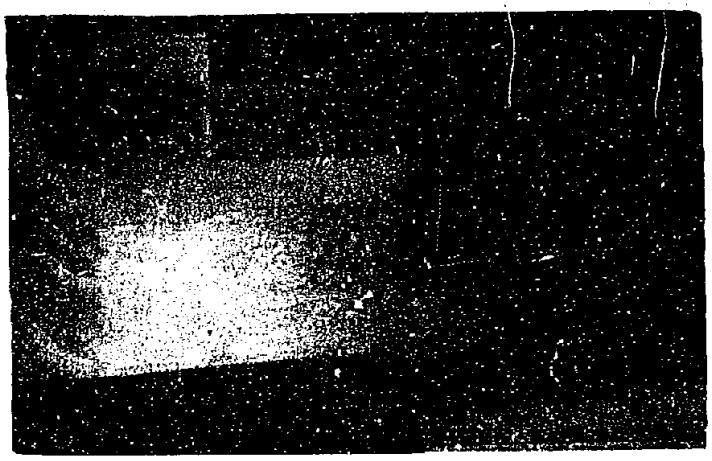
B. DISCHARGE 200,000 SECOND-FEET.

C. DISCHARGE 240,000 SECOND-FEET.

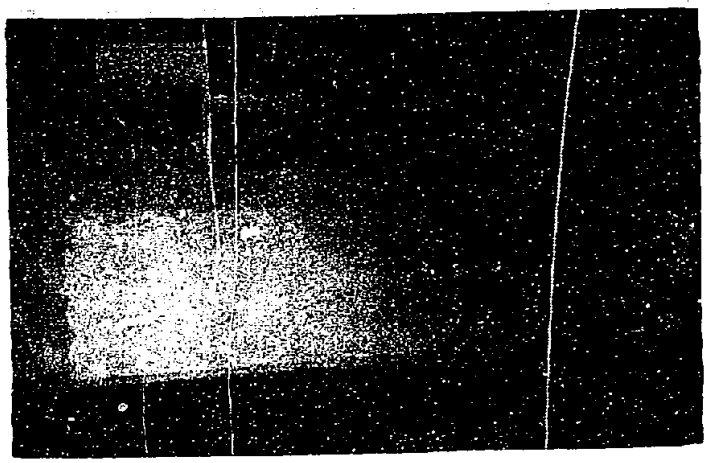


D. DISCHARGE 240,000 SECOND-FEET.

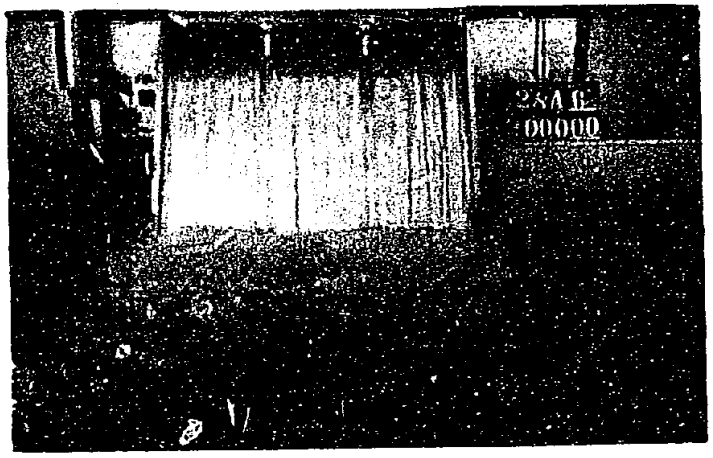
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, WITH OVERHANG, 60' UPSTREAM FROM END OF APRON.



A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.



C. DISCHARGE 200,000 SECOND-FEET.

D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, WITH OVERHANG, 90' UPSTREAM FROM END OF APRON.

FIGURE 19

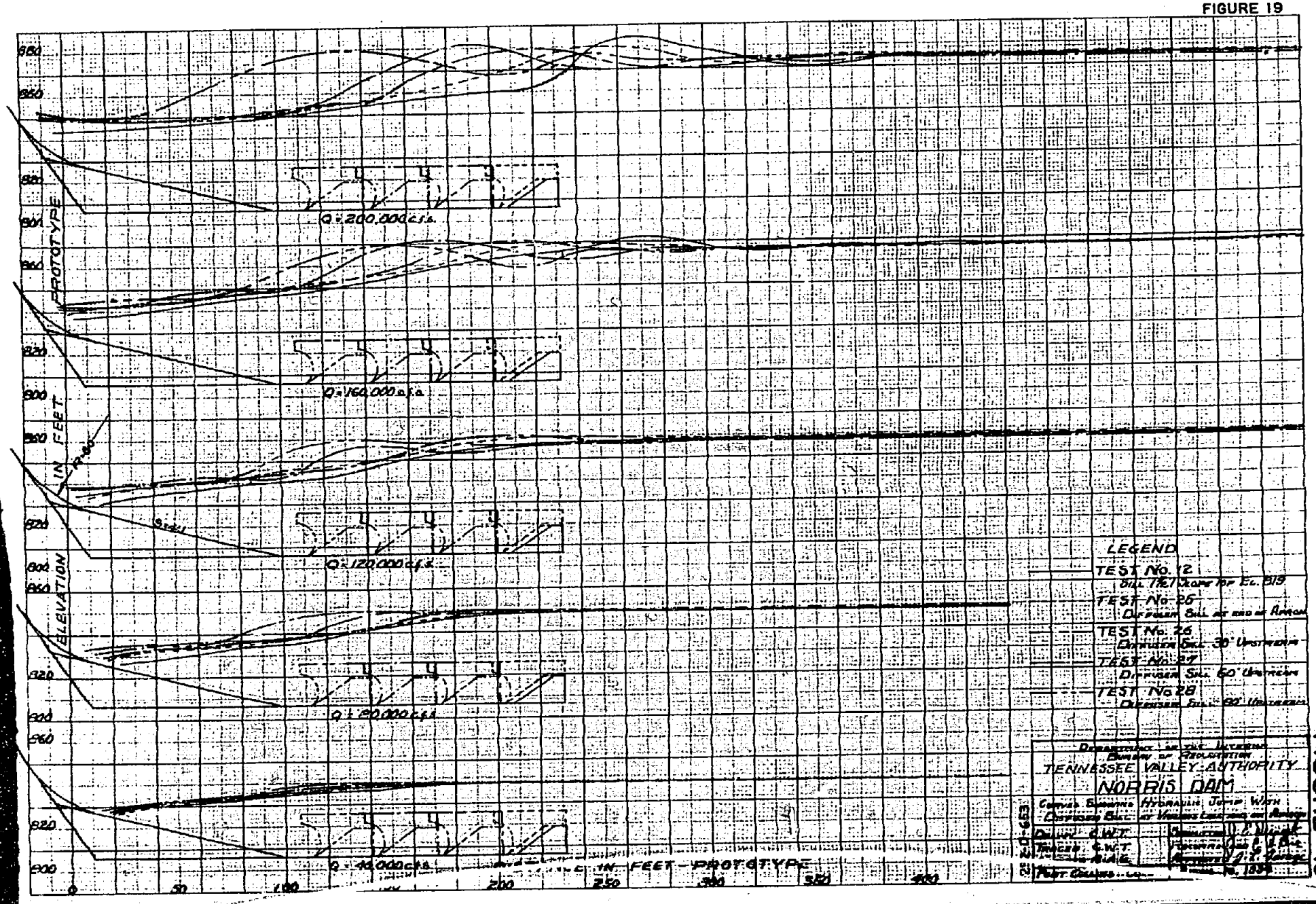
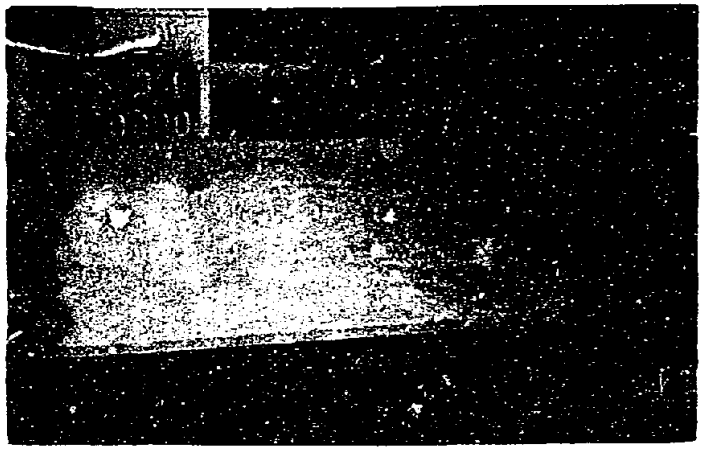


FIGURE 19

Plates 28 to 32 inclusive) of the tests at different positions on the apron were in effect the same as the results of the preceding tests except a possible improvement of conditions in the water surface in the pool. Although it was thought that the additional cost of construction of this sill could have been borne by the saving in shortening of the apron and training walls, the bad features of the sill were embodied in the very grave probability of erosion on the sill and the resulting necessary maintenance.

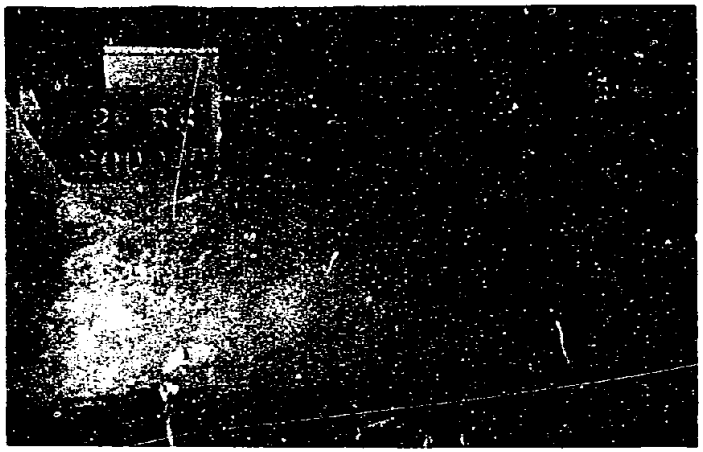
A type of sill similar to the one above was used as the final design on the model of the Wheeler Dam. A sill of the Wheeler type was tested on the Norris apron at three different positions. The results showed an improvement in flow conditions in the pool for low and medium flows and good results for sluices only operating, but at high flows over the spillway very little improvement was noticed except in surface conditions. The velocities downstream from the end of the apron were much higher than in other set-ups. Figure 21 shows the profiles of the water surface for the sills in the different positions. The effect of lowering the water surface over the sill produces higher velocities leaving the apron. With the sill at the end of the apron entirely removed the surface conditions agreed closely with the conditions with the diffuser sill at the end of the apron. Plates 33 to 36 inclusive

A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.

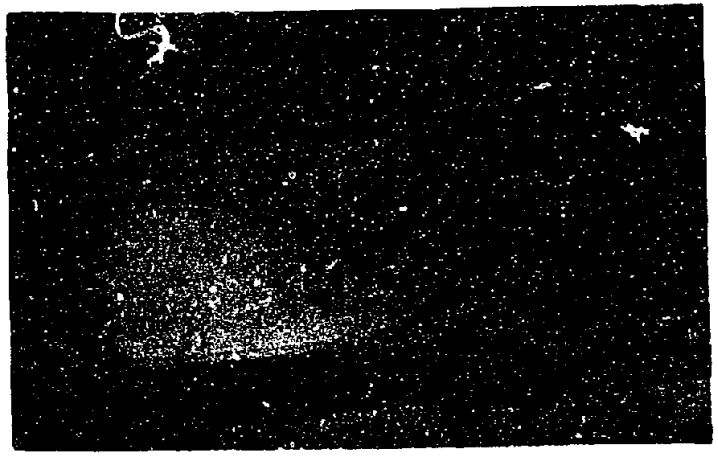
C. DISCHARGE 200,000 SECOND-FEET.



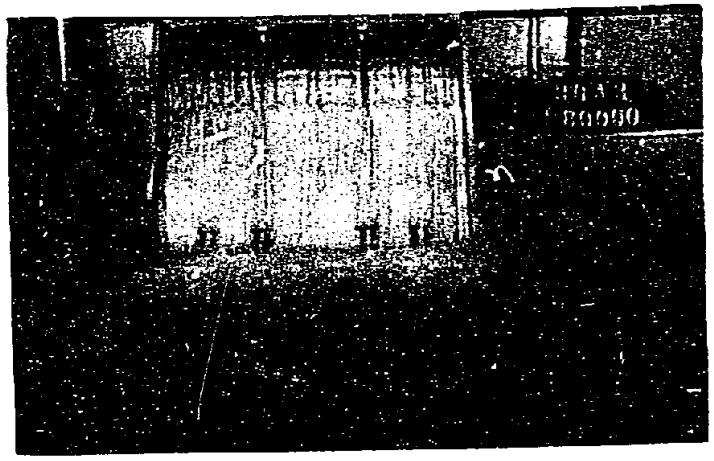
D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, OVERHANG REMOVED, 90' UPSTREAM FROM END OF APRON.

PLATE 28

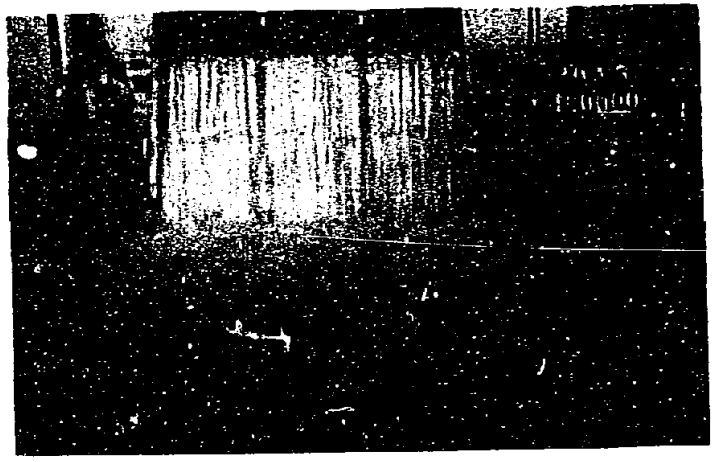


A. DISCHARGE 40,000 SECOND-FEET.



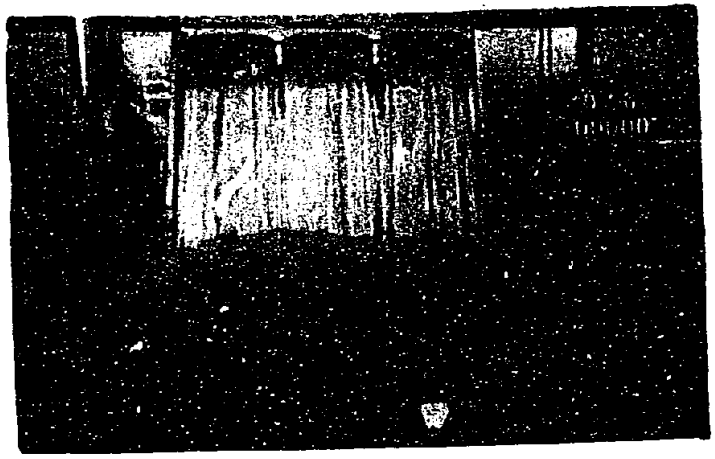
B. DISCHARGE 80,000 SECOND-FEET.

C. DISCHARGE 120,000 SECOND-FEET.

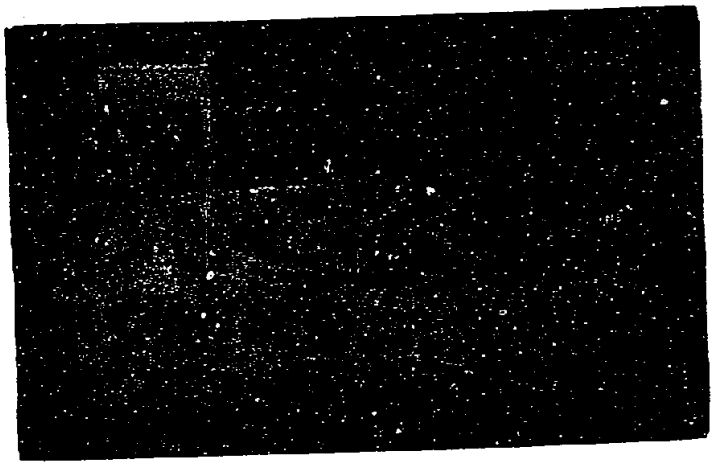


D. DISCHARGE 160,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, OVERHANG REMOVED, 60' UPSTREAM FROM END OF APRON.

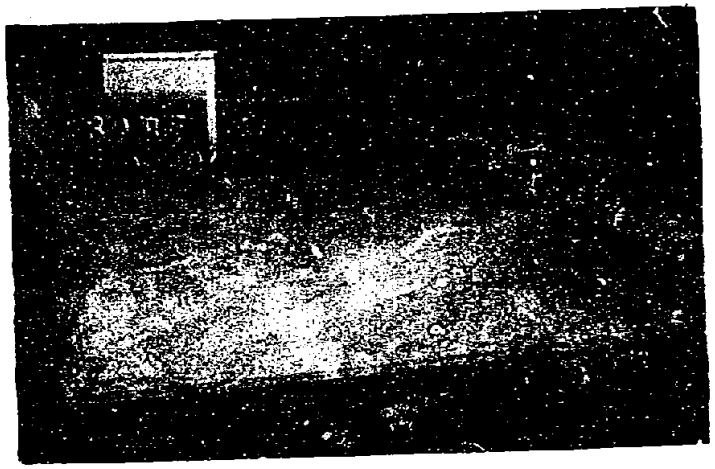


A. DISCHARGE 200,000 SECOND-FEET.



B. DISCHARGE 200,000 SECOND-FEET.

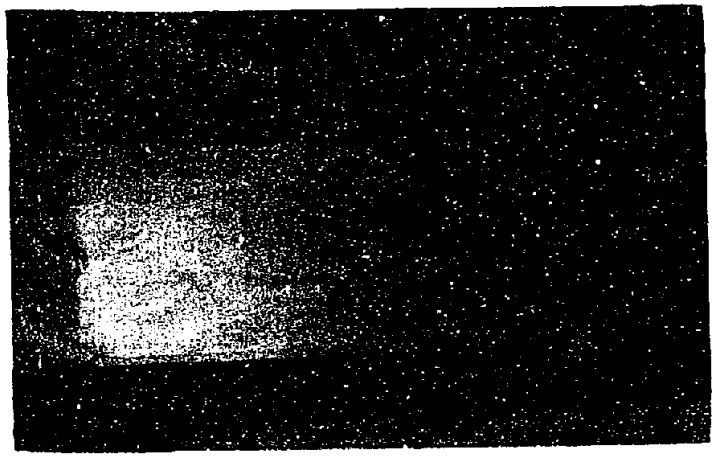
C. DISCHARGE 240,000 SECOND-FEET.



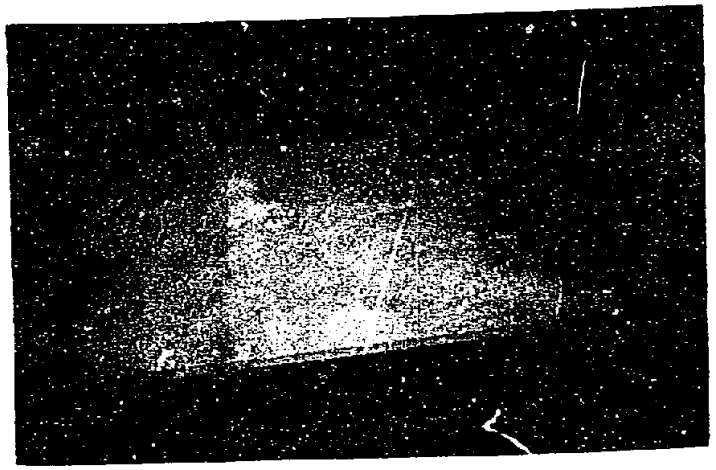
D. DISCHARGE 240,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, OVERHANG REMOVED, 60' UPSTREAM FROM END OF APRON.

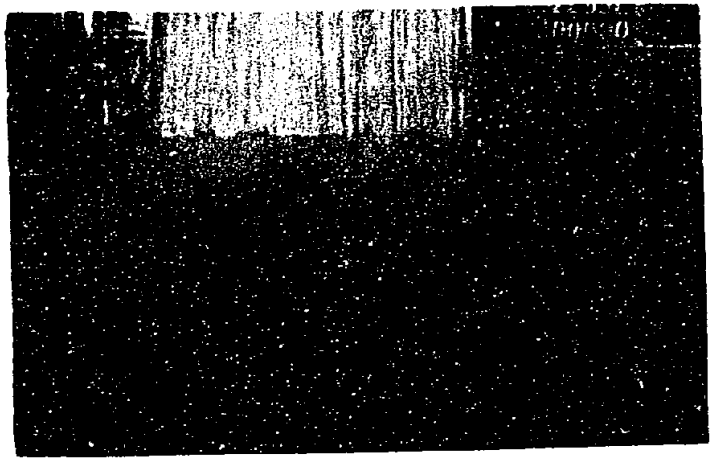
PLATE 80



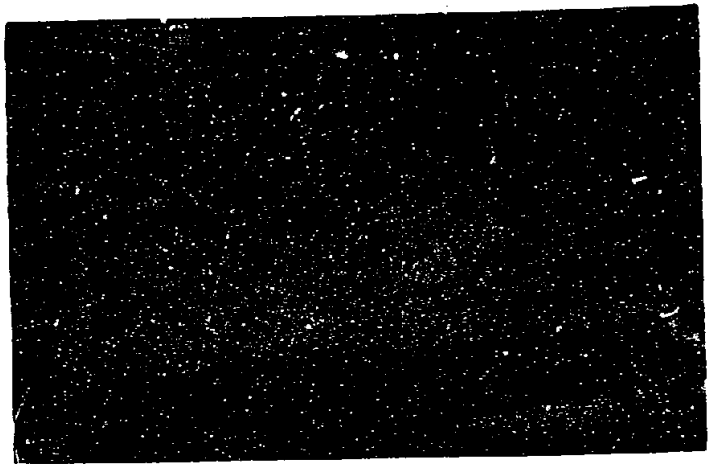
A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.

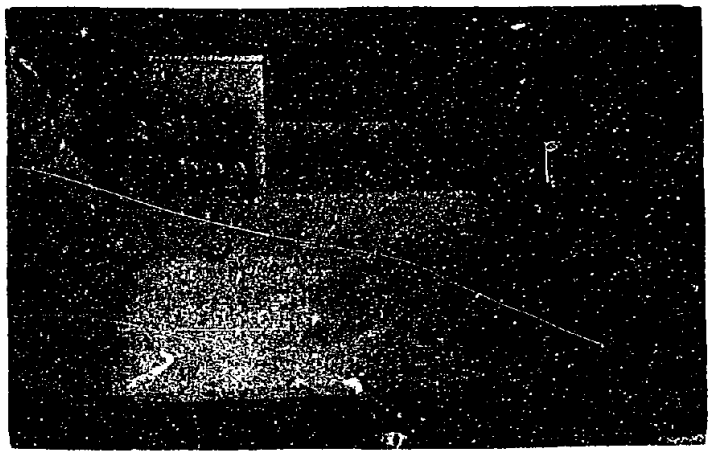


C. DISCHARGE 200,000 SECOND-FEET.



D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, OVERHANG REMOVED, 30' UPSTREAM FROM END OF APRON.



A. DISCHARGE 40,000 SECOND-FEET.

B. DISCHARGE 120,000 SECOND-FEET.

C. DISCHARGE 200,000 SECOND-FEET.

D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LARGE DIFFUSER SILL, OVERHANG REMOVED, AT END OF APRON.

FIGURE 20

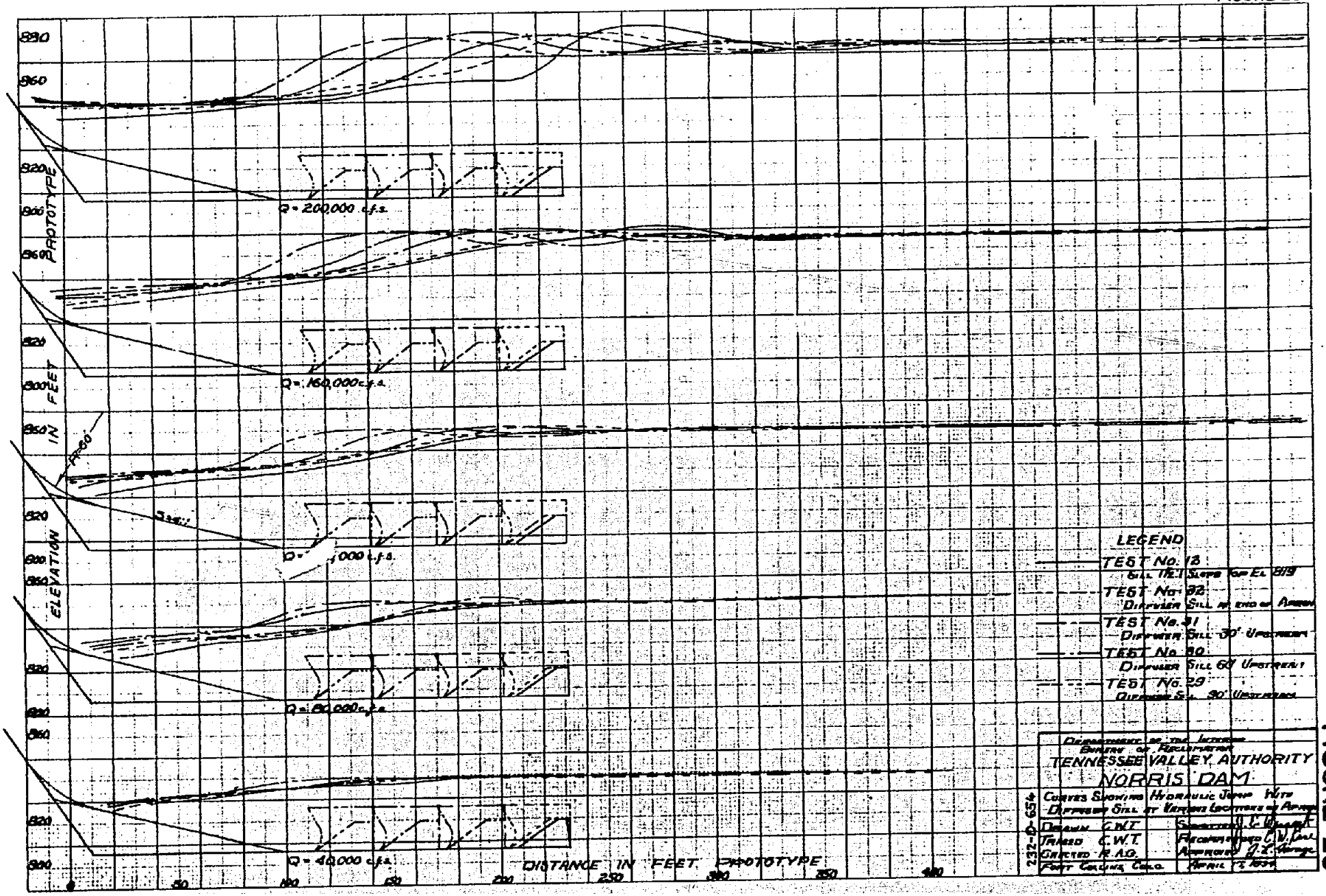


FIGURE 20

FIGURE 21

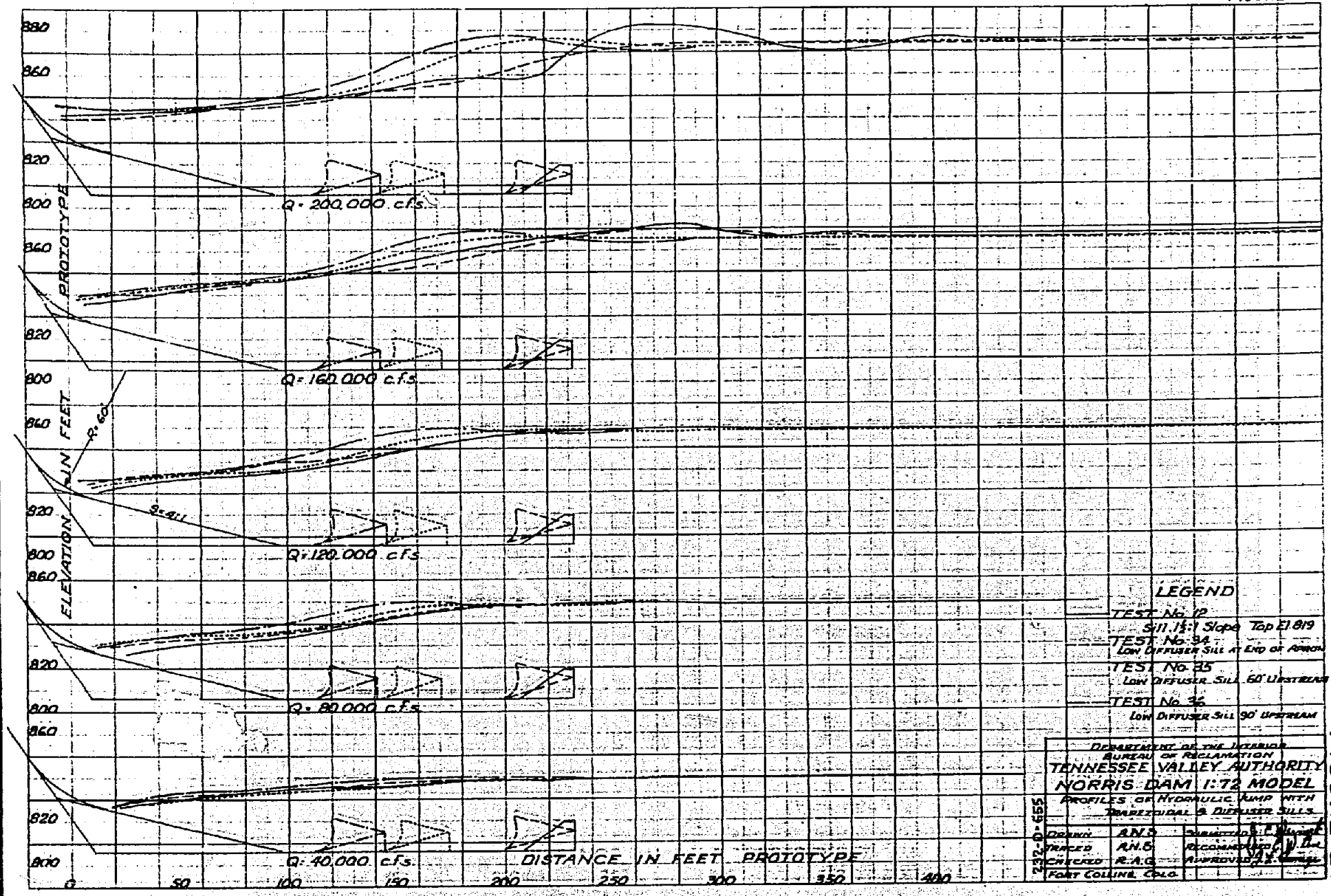
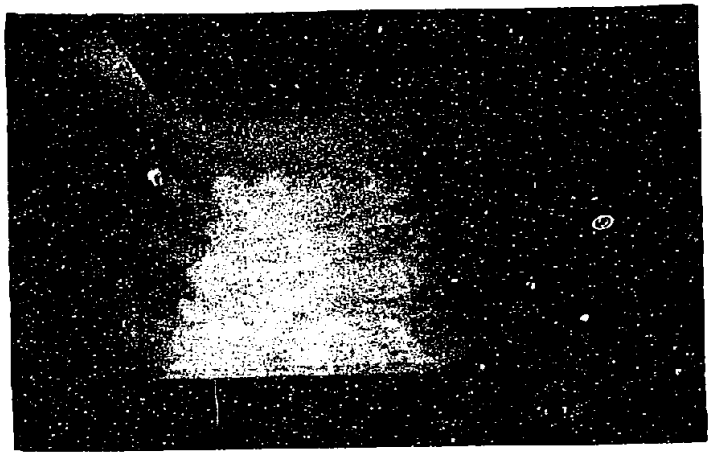


FIGURE 21

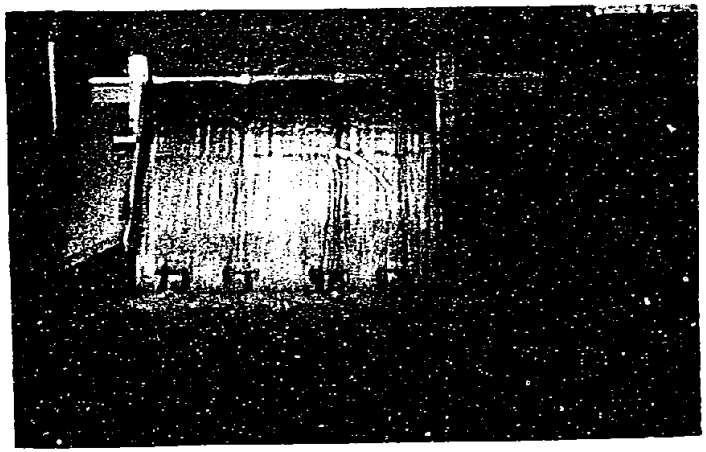
show the flow conditions. The sill located 60 feet upstream from the end of the apron produced the best results.

From later field data an accurate cross-section of the stream bed at the end of the apron was determined and a sill built with a constant top width of three feet and conforming to the contour of the stream bed for the width of the spillway. The bottom width was held constant at 24 feet and the slope of the upstream face varied accordingly. The velocity was concentrated near the center of the apron as the water crossed the sill and continued downstream (Plates 37 and 38). This was caused by the lower sill section at the center of the apron and the consequent flatter slope. The higher sill sections at the sides of the spillway apron caused disturbances which resulted in higher water surfaces. This condition is not shown on Figure 22 as the sections are on the center line of apron.

From the aforementioned field data, the average elevation of the stream bed was determined to be Elev. 817.5 instead of 819. A set of sills with their tops at that elevation, a constant top width of three feet and upstream face slopes of $1\frac{1}{2}$:1, 2:1, $2\frac{1}{2}$:1 and 3:1 were tested. The results of these tests (Figures 23 and 24) indicated that the $1\frac{1}{2}$:1 slope sill was the best from the hydraulic standpoint as well as being less expensive

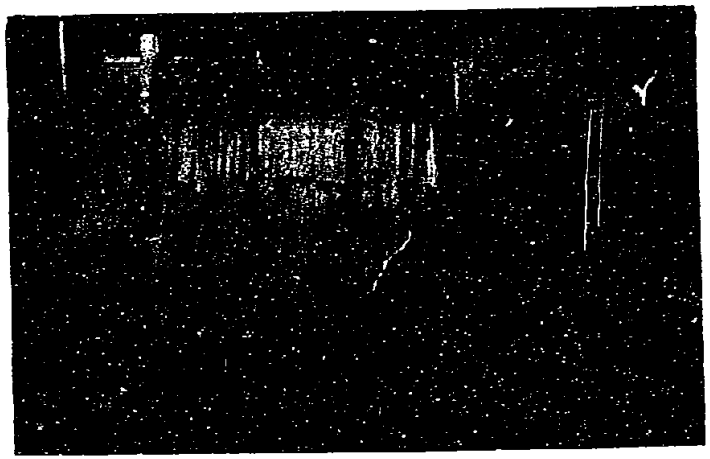


A. DISCHARGE 40,000 SECOND-FEET.



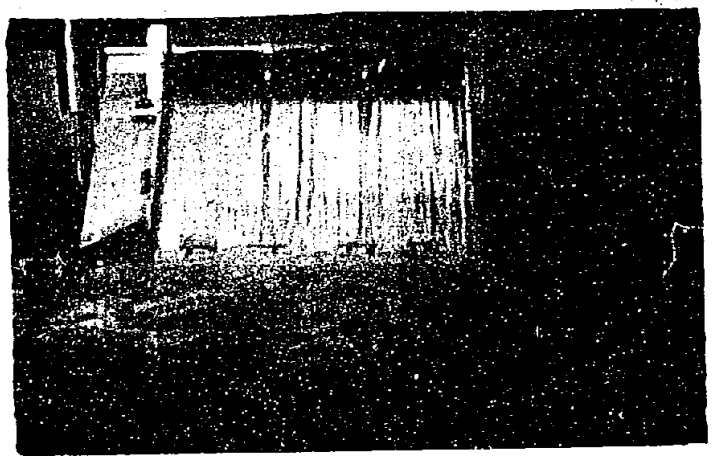
B. DISCHARGE 80,000 SECOND-FEET.

C. DISCHARGE 120,000 SECOND-FEET.

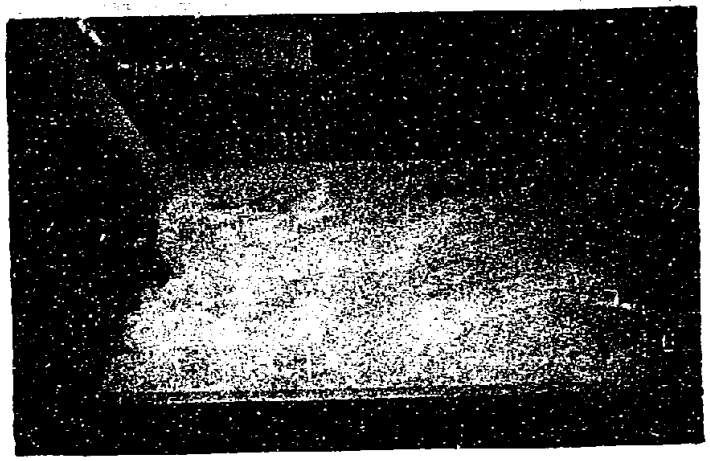


D. DISCHARGE 160,000 SECOND-FEET.

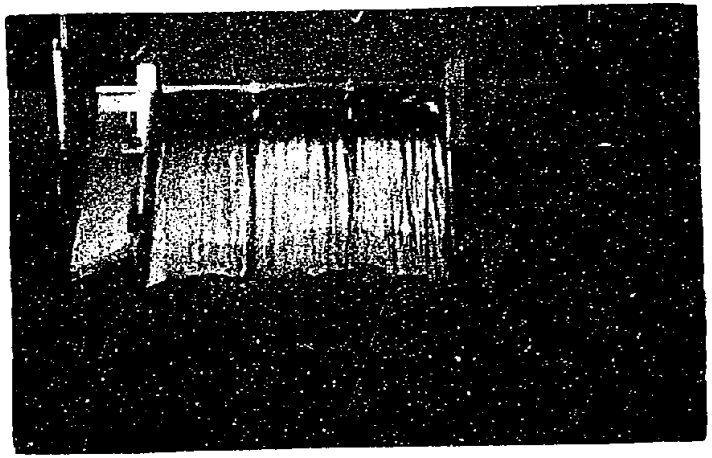
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LOW DIFFUSER SILL (WHEELER TYPE) 60' UPSTREAM FROM END OF APRON.



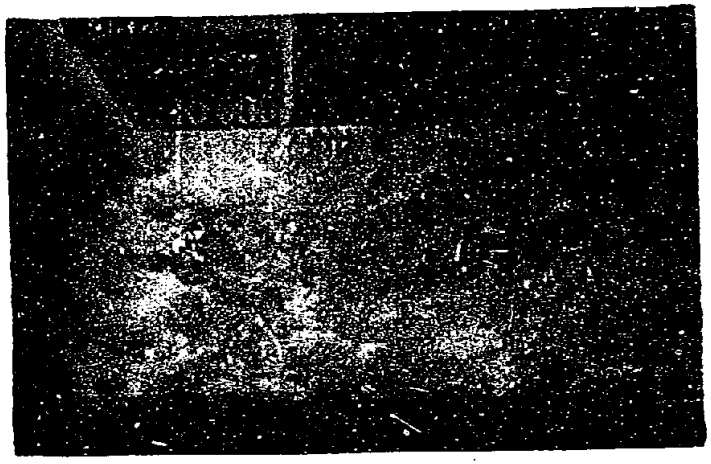
A. DISCHARGE 200,000 SECOND-FEET.



B. DISCHARGE 200,000 SECOND-FEET.



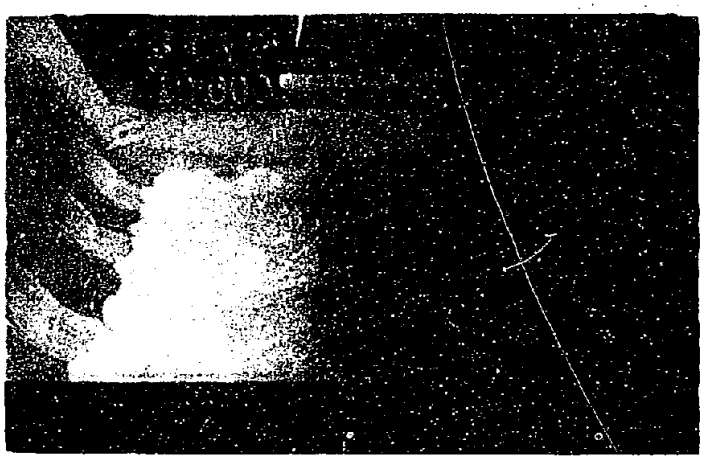
C. DISCHARGE 240,000 SECOND-FEET.



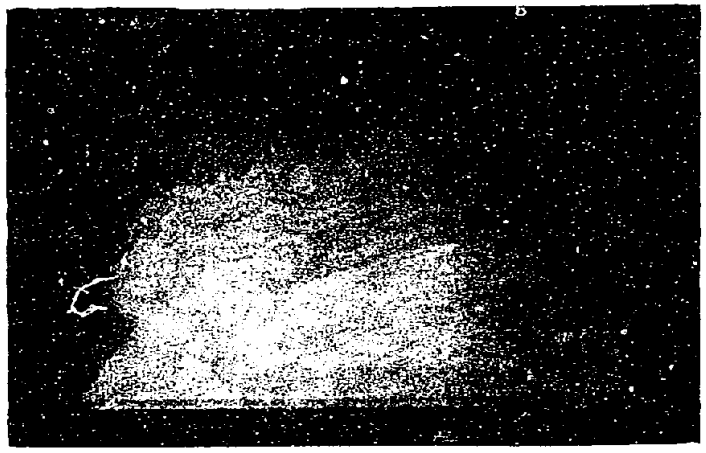
D. DISCHARGE 240,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LOW DIFFUSER SILL (WHEELER TYPE) 60' UPSTREAM FROM END OF APRON.

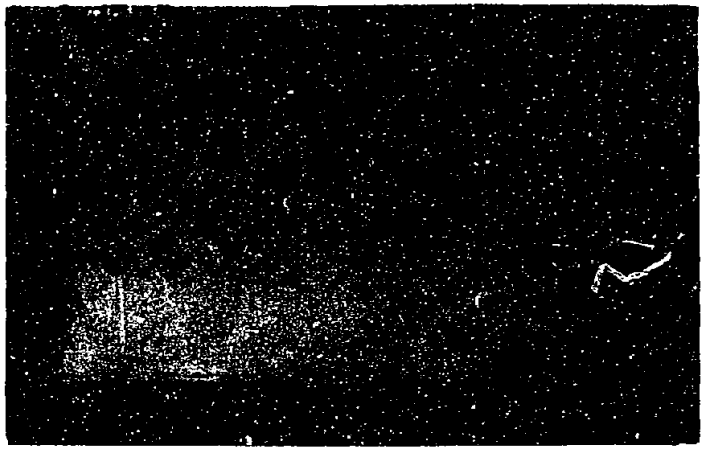
PLATE 36



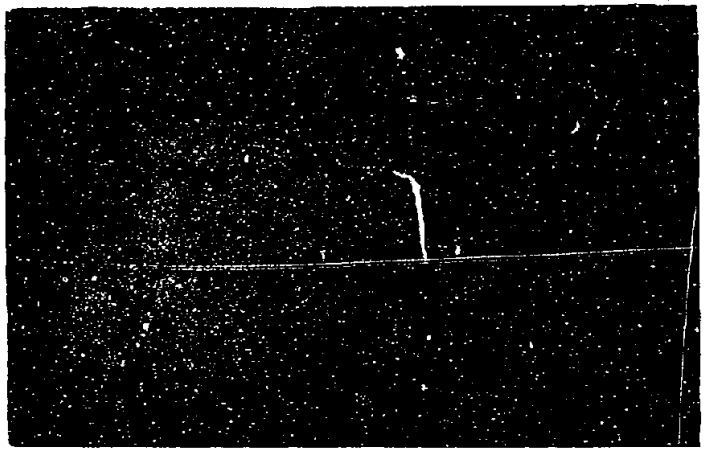
A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 120,000 SECOND-FEET.



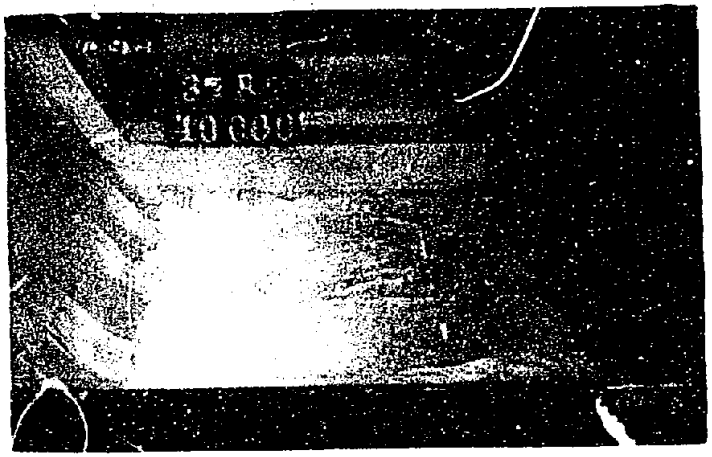
C. DISCHARGE 200,000 SECOND-FEET.



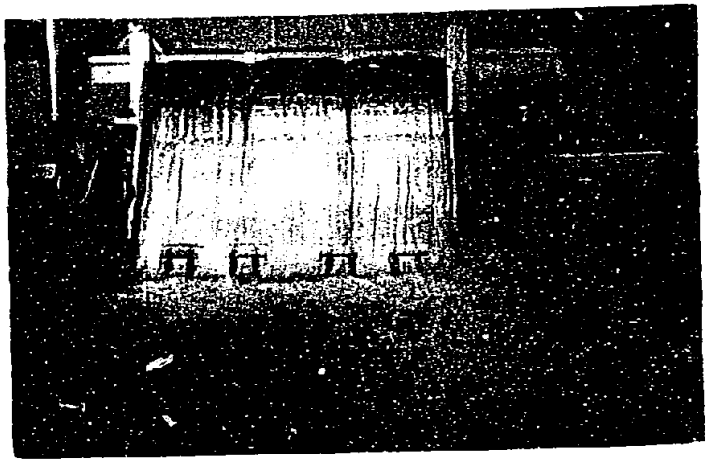
D. DISCHARGE 240,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
LOW DIFFUSER SILL (WHEELER TYPE) 90' UPSTREAM FROM END OF APRON.

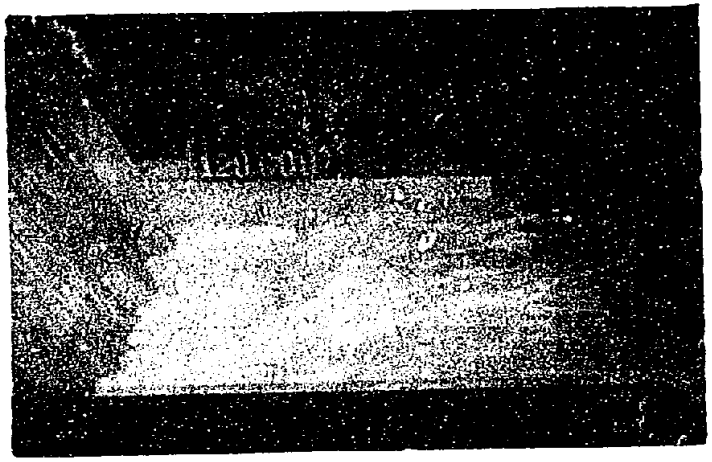
PLATE 26



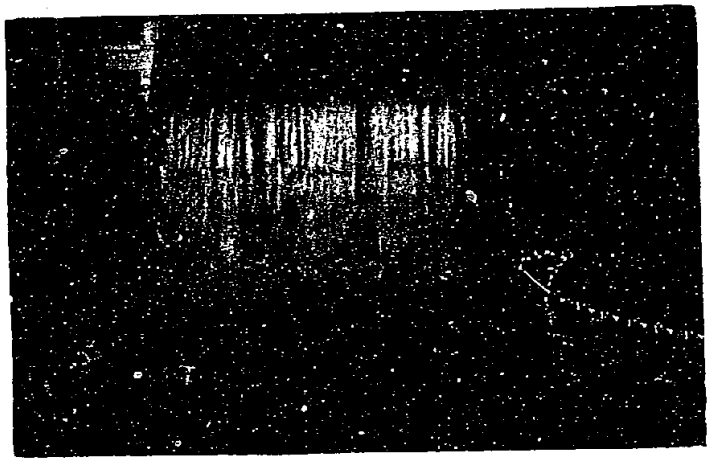
A. DISCHARGE 40,000 SECOND-FEET.



B. DISCHARGE 80,000 SECOND-FEET.

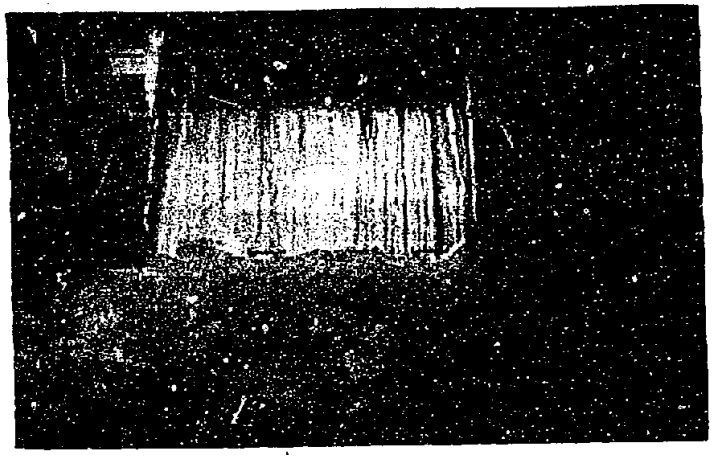


C. DISCHARGE 120,000 SECOND-FEET.

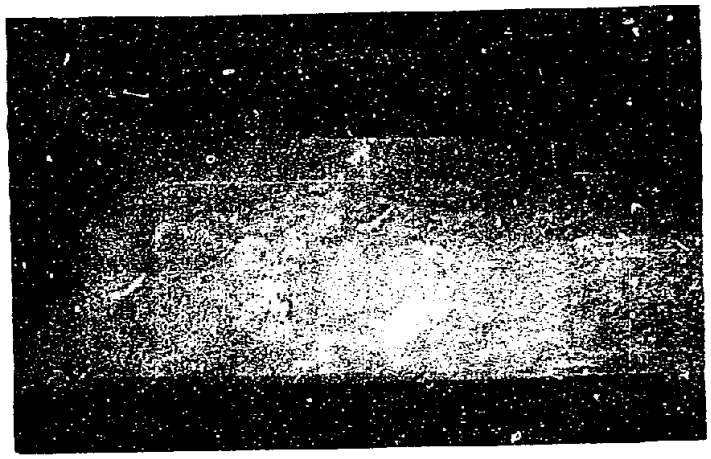


D. DISCHARGE 160,000 SECOND-FEET.

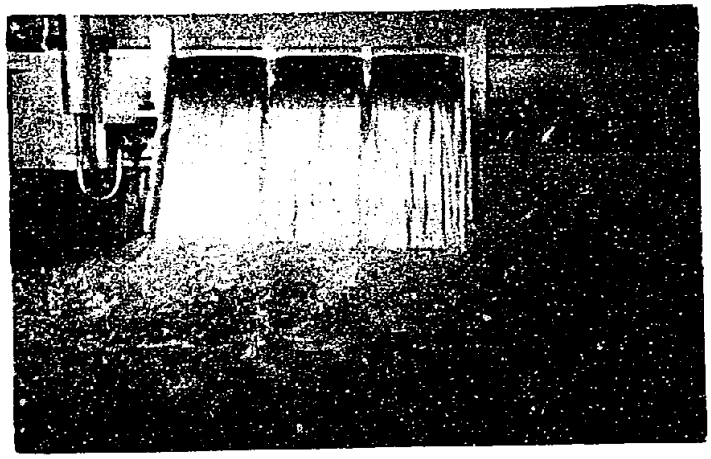
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
VARIABLE SECTION TRAPEZOIDAL SILL AT END OF APRON.



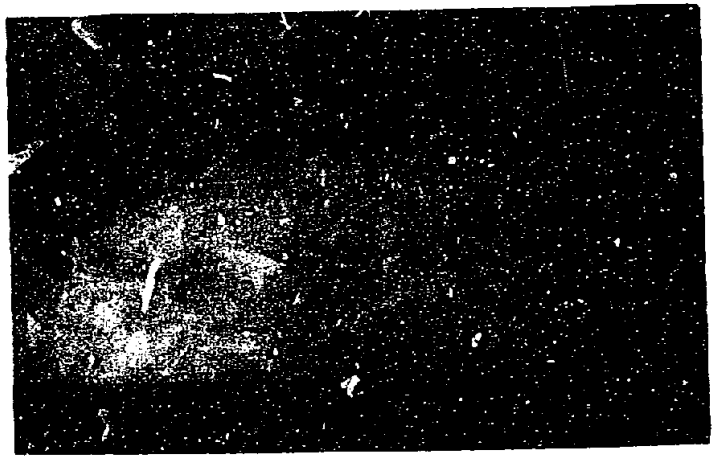
A. DISCHARGE 200,000 SECOND-FEET.



B. DISCHARGE 200,000 SECOND-FEET.



C. DISCHARGE 240,000 SECOND-FEET.



D. DISCHARGE 240,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
VARIABLE SECTION TRAPEZOIDAL SILL AT END OF APRON.

FIGURE 22

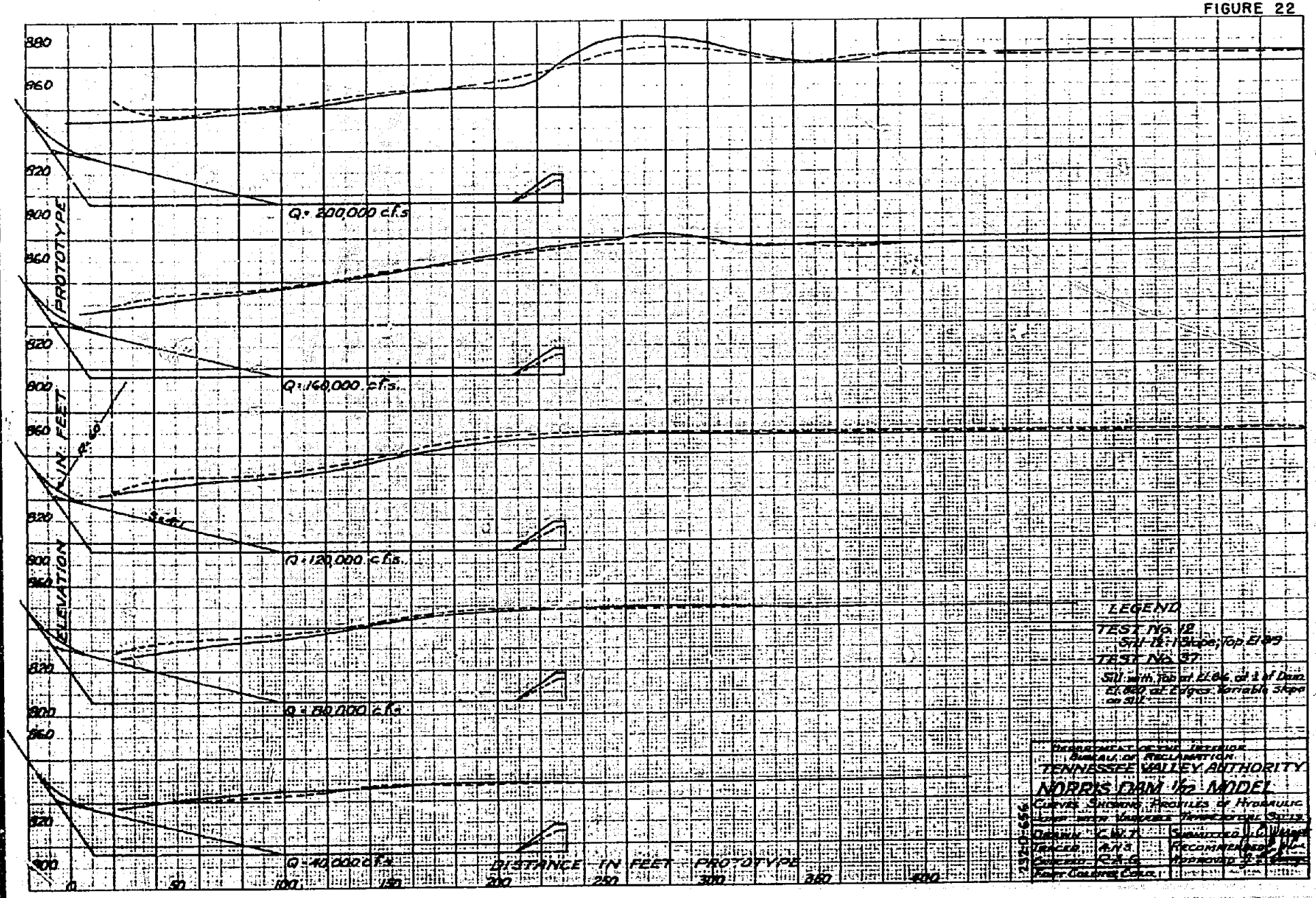

ELEVATION IN FEET PROTOTYPE
880
860
840
820
800
780
760
740
720
700
Q = 200,000 c.f.s.
Q = 160,000 c.f.s.
Q = 120,000 c.f.s.
Q = 80,000 c.f.s.
Q = 40,000 c.f.s.
DISTANCE IN FEET PROTOTYPE
0
50
100
150
200
250
300
350
400
LEGEND
TEST No. 12
Sill 1½:1 Slope; Top El 99
TEST No. 37
Sill with Top at El 64, at ½ of Dam
El 89 at Edges, Variable Slope on Sill
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY
NORRIS DAM 1/50 MODEL
CURVES SHOWING PROFILES OF HYDRAULIC JUMP WITH VARIOUS TEMPORARY SILLS
DRAWN C.W.T.
TRACED A.N.S.
CHECKED R.A.G.
SUBMITTED
RECOMMENDED
APPROVED
FIGURE 22

FIGURE 23

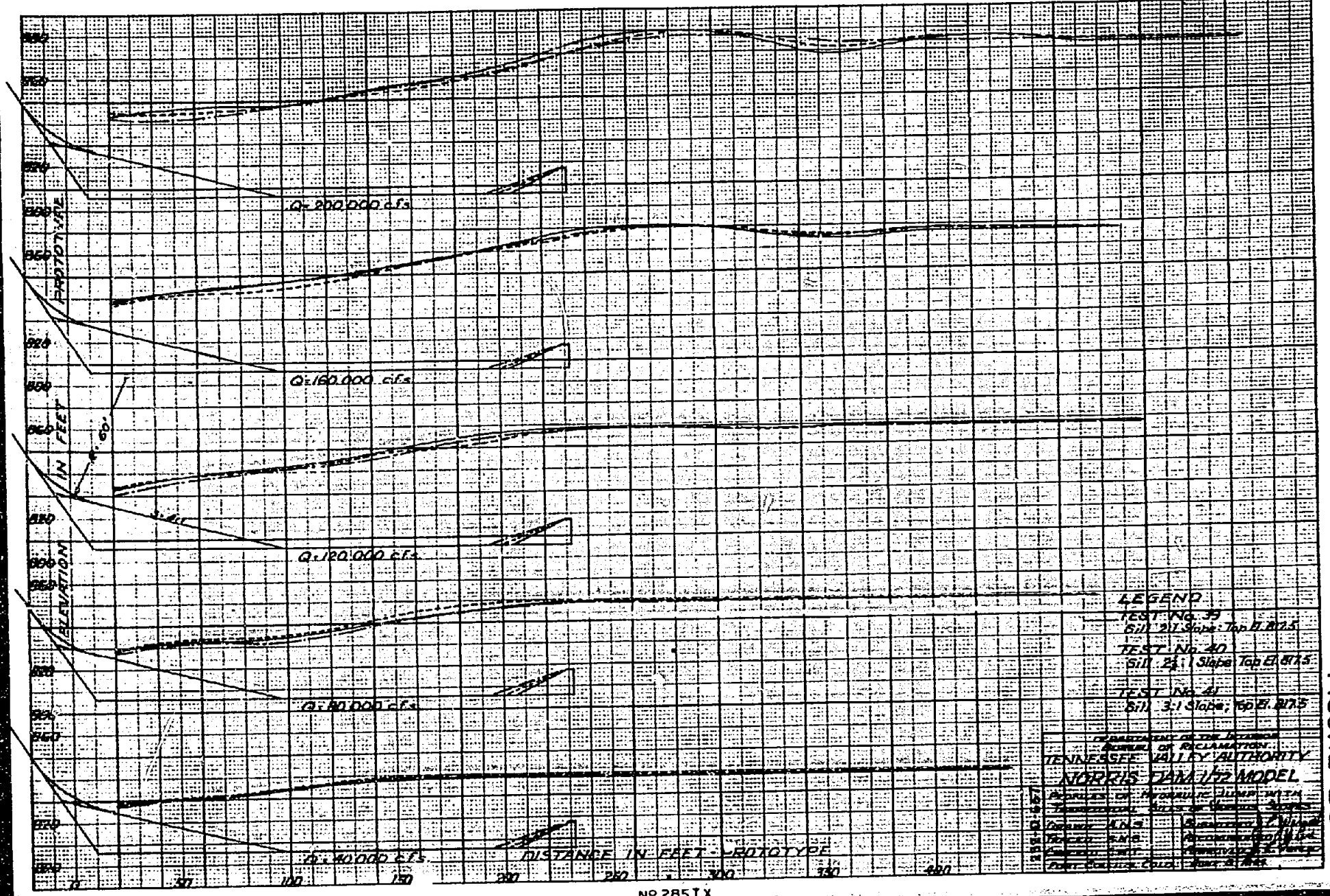


FIGURE 23

Nº 285 TX

FIGURE 24

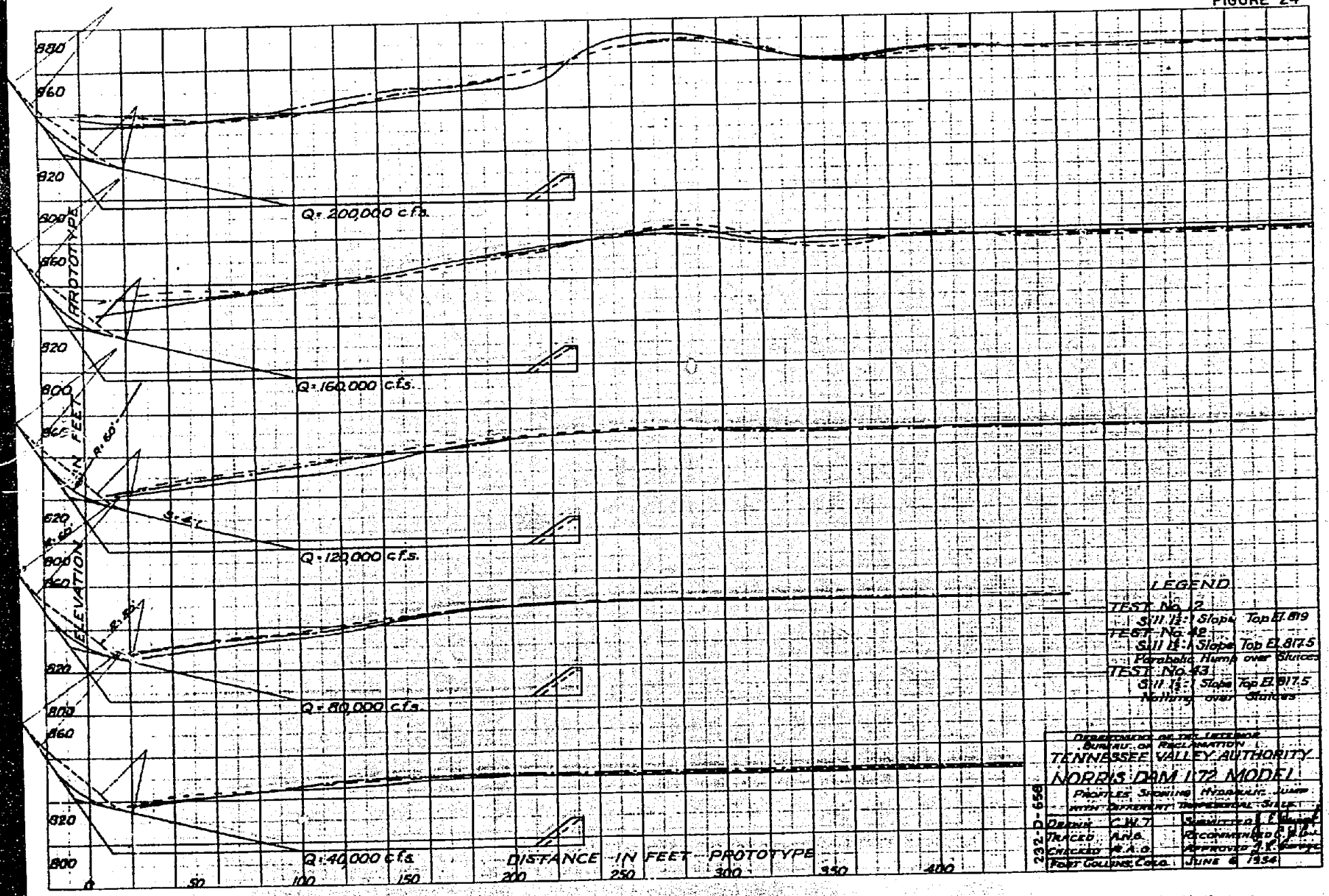


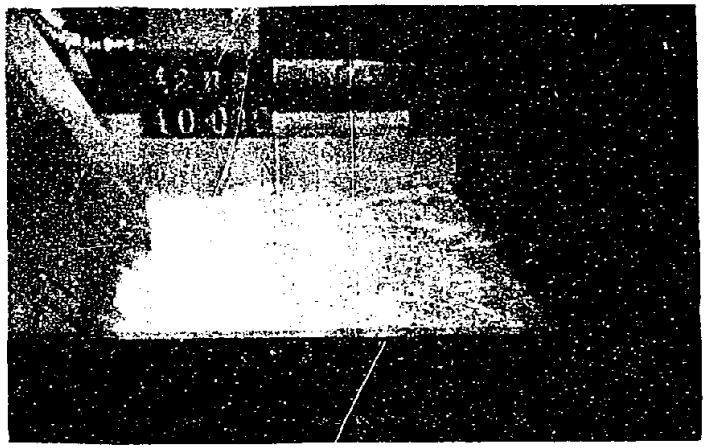
FIGURE 24

to build. The water surface conditions were improved as compared to the higher sill of the same slope (Figure 24), and no additional erosion was noted at the end of the apron. The modifications of the sluice outlets made their operation satisfactory with this sill. It was therefore ~~adopted~~ recommended for the final design.

The parabolic hump on the dam over the sluice openings had very little effect on the flow over the spillway as can be seen by referring to figure 24, Test 42 and Plates 39 and 40.

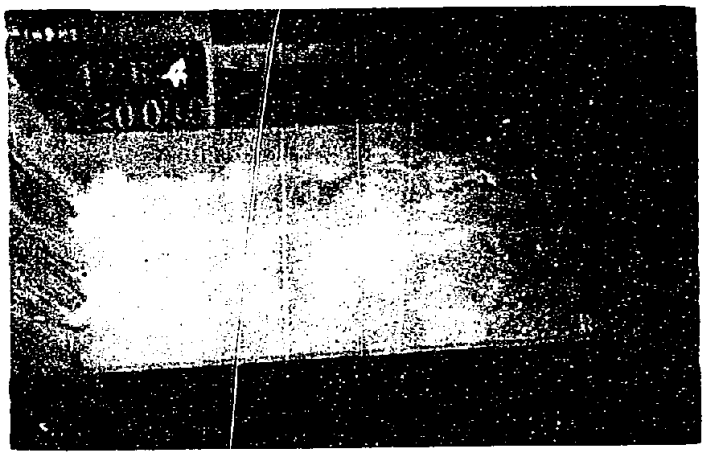
The problem of designing the stilling pool was complicated throughout the tests by the necessity of meeting the requirements of three following conditions: (1) sluices discharging singly or in combinations (2) spillway discharging and (3) sluices and spillway discharging into the pool. In many instances the set-up would fulfill one or more of these requirements, but would be objectionable for the remainder.

From the tests on the sluiceways made in conjunction with the tests of the spillway operation, the most satisfactory operating conditions were obtained by first opening the sluiceways as a flood flow approached the reservoir and as the flood flow increased and the reservoir elevation raised, allow the spillway to discharge with the sluiceways still in operation. As the flood subsided, the sluiceways should continue to discharge until after the spillway gates have been raised and discharge over them



A. DISCHARGE 40,000 SECOND-FEET.

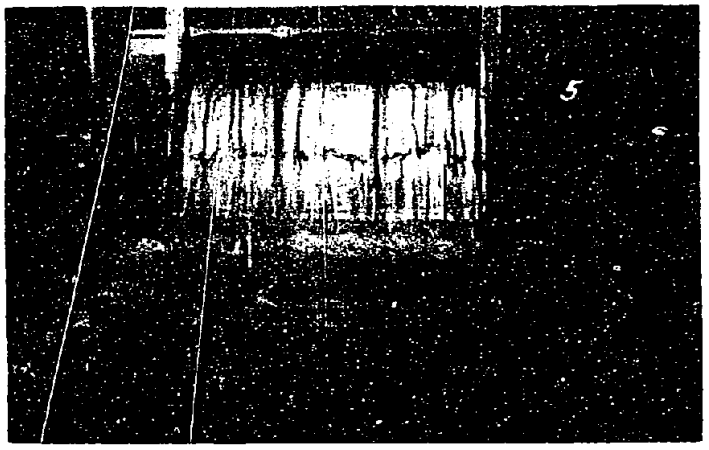
B. DISCHARGE 40,000 SECOND-FEET.



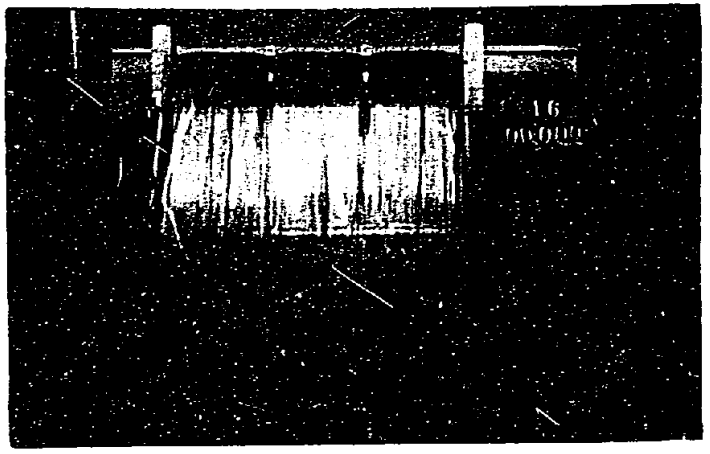
C. DISCHARGE 120,000 SECOND-FEET.

D. DISCHARGE 120,000 SECOND-FEET.

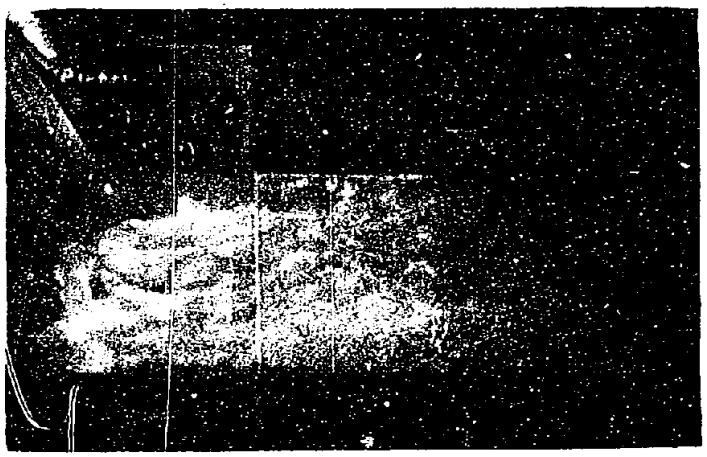
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON.
PARABOLIC HUMP OVER SLUICE OPENINGS.



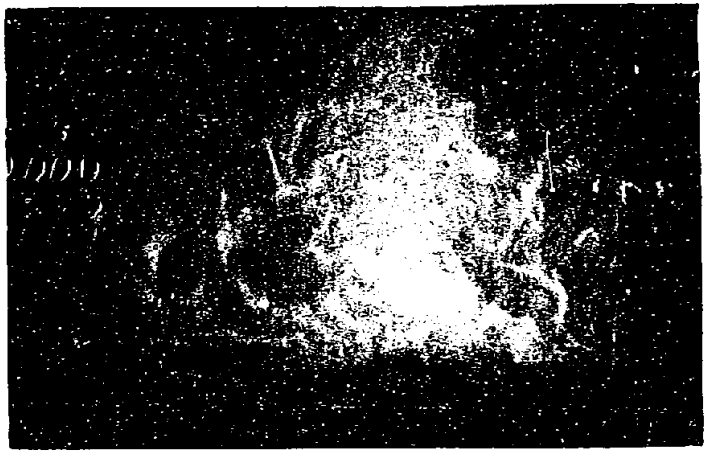
A. DISCHARGE 160,000 SECOND-FEET.



B. DISCHARGE 200,000 SECOND-FEET.



C. DISCHARGE 200,000 SECOND-FEET.



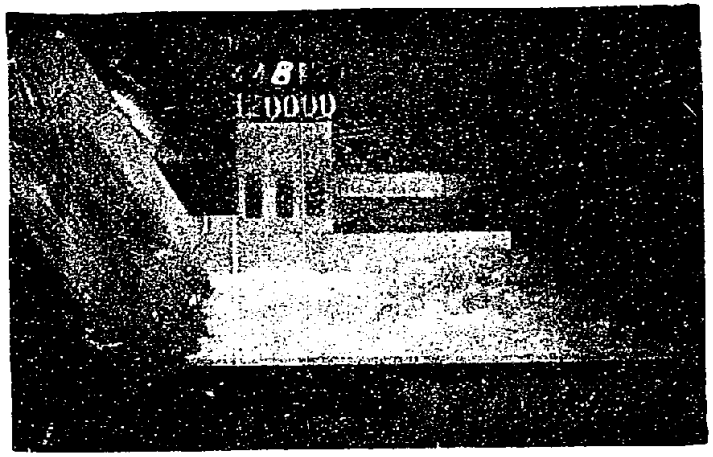
D. DISCHARGE 200,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON.
PARABOLIC HUMP OVER SLUICE OPENINGS.

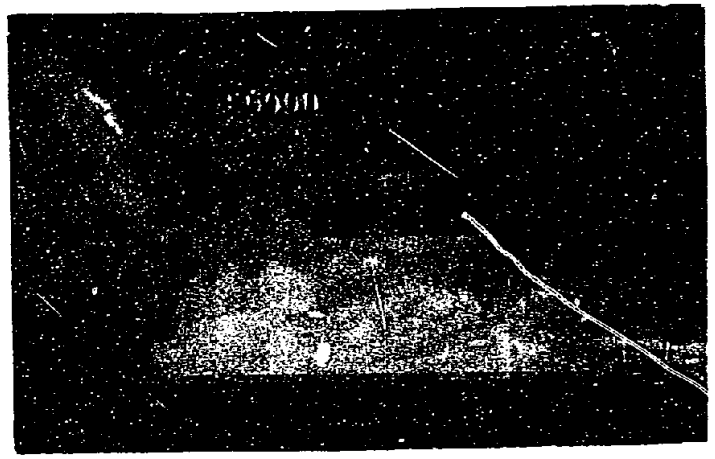
stopped. Thus at maximum flow conditions, the sluiceways would discharge 40,000 second-feet and the spillway 200,000 second-feet. In Figure 25, Test 44 made under these conditions is compared to Test 43 made with the sluiceways closed until the spillway discharge reached 200,000 second-feet, then opening the sluiceways until the maximum capacity of 240,000 second-feet was passing the dam. Flow conditions for the former condition are shown on Plate 41.

All the figures referred to in the foregoing discussion showing the profile of water surface, were made from data obtained by means of the point gage previously described. These sections show conditions on the center line of the apron and are plotted from a reference datum at Elev. 806. The curves show comparative results and most of them are plotted in conjunction with the curve of the water surface when the 4:1 sloping apron to the level apron, Elev. 806, and a $1\frac{1}{2}$:1 slope sill to Elev. 819 at the end of the apron was used. This was done to obtain a basis of comparison.

Check runs made from time to time showed results of the s illing pool to be very consistent.

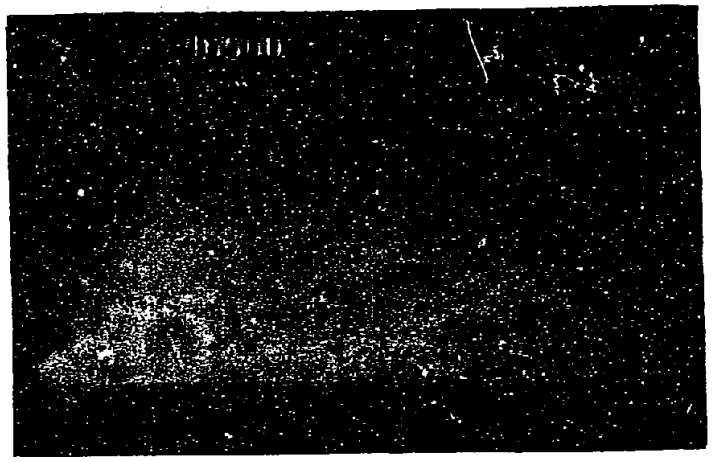


A. DISCHARGE 120,000 SECOND-FEET.



B. DISCHARGE 160,000 SECOND-FEET.

C. DISCHARGE 200,000 SECOND-FEET.



D. DISCHARGE 240,000 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806. DIVERGING OUTLETS FOR SLUICES.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON.

PLATE 41

FIGURE 25

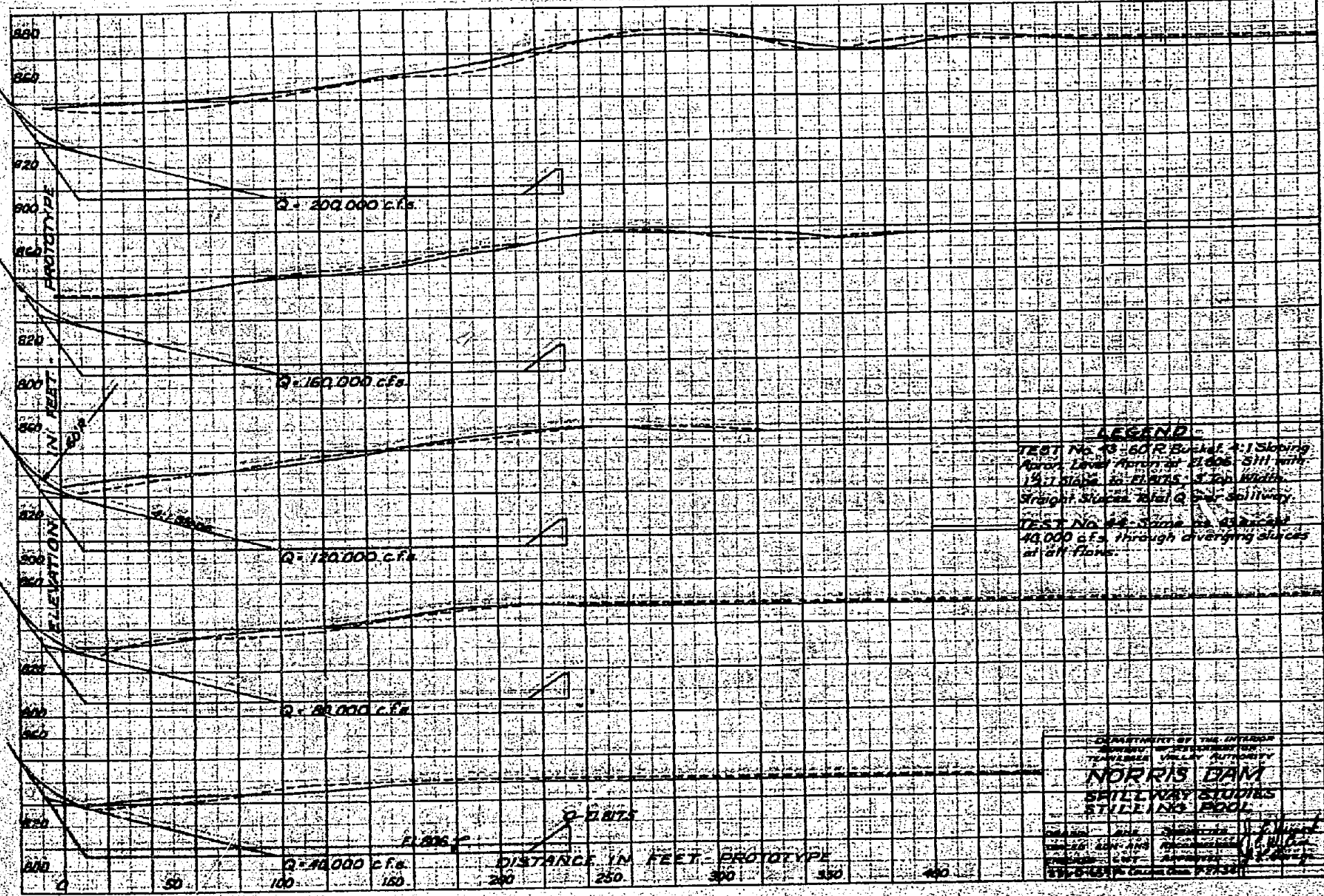


FIGURE 25

2. Calibration of Drum Gates.

In the original analysis of the discharge data for the spillway drum gates, the formula $Q = CLH^{3/2}$ was used and the value of the coefficient C was computed in terms of model values for Q, L and H and with the gates set at Elev. 1020, 1023.51, 1027.11, 1031.54 and 1033.91 respectively. When the values of C were plotted on cross-section paper against the values of H, the resulting curves appeared very erratic, so that logarithmic plotting was tried in an effort to straighten the data. The logarithmic plotting produced a good curve for each gate elevation. Difficulty was experienced in this method of analysis due to the short range of values of coefficient C at some gate elevations which made it extremely difficult to read values from the log curves to plot on cross-section paper.

A summary of these results are shown in Table 2. Columns 1 and 2 identify the runs. Column 3 gives the gate elevation in feet (prototype) above sea level datum, as determined by averaging the elevation of all three gates of the model with reference to the fixed crest elevation of 1020 when the gates were completely lowered. Column 4 shows the discharge over the gates of the model as measured with a two-foot Cipolletti weir, previously calibrated in place, and a hook gage. Readings on a float gage in the same gage well served as a check. Column 5 gives the average

TABLE NO. 2

NORRIS DAM SPILLWAY

SUMMARY OF DATA.

Coefficient of Discharge of Spillway Drum Gates.

1	2	3	4	5	6
Test No.	Run No.	Gate Elevation	Q	Head on Model Crest	Model C in $Q=CLH^{3/2}$
6	1	1020.000	0.431300	0.1011	3.2236
	2	1020.000	0.918000	.1609	3.4152
	3	1020.000	1.838800	.2464	3.6076
	4	1020.000	2.725200	.3136	3.7235
	5	1020.000	3.624600	.3759	3.8049
	6	1020.000	4.555500	.4245	3.9231
	7	1020.000	7.901400	.5096	3.9834
	8	1020.000	4.571100	.4317	3.8662
	9	1020.000	4.104700	.4035	3.8423
	10	1020.000	3.146800	.3424	3.7662
	11	1020.000	6.257200	.5275	3.9165
	12	1020.000	7.462300	.5914	3.9314
7	7A	1033.908	0.433781	.0894	3.8979
	7X	1033.908	0.226314	.0578	3.9007
	8A	1033.908	0.913597	.1476	3.8658
	9A	1033.908	1.841458	.2381	3.8031
	9X	1033.908	1.342832	.1916	3.8418
	9Y	1033.908	1.348603	.1918	3.8528
	10A	1033.908	2.725303	.3083	3.8199
	10B	1033.908	3.677393	.3761	3.8246
	10C	1033.908	3.162368	.3424	3.7868
	10D	1033.908	3.154194	.3392	3.8301
	10E	1033.908	3.671152	.3748	3.8392
	10X	1033.908	2.285645	.2749	3.8050
	21	1031.412	0.218380	.0573	3.8189
	22	1031.412	0.433440	.0892	3.9095
	23	1031.412	0.910530	.1465	3.8940
	24	1031.412	1.836850	.2349	3.8715
	25	1031.412	2.733680	.3060	3.8740
	21X	1031.561	0.323450	.0725	3.9796
	22A	1031.561	0.429780	.0873	3.9935
	22B	1031.561	0.435370	.0850	4.0021
	22C	1031.561	0.434720	.0889	3.9418
	22X	1031.561	0.941460	.1501	3.8837
	22Y	1031.561	0.680540	.1203	3.9108
	23A	1031.561	0.913380	.1468	3.8938
	23X	1031.561	1.380472	.1930	3.9057
	24A	1031.561	1.843140	.2339	3.9089
	24X	1031.561	2.229120	.2648	3.9255
	25A	1031.561	2.715530	.3025	3.9154
	25B	1031.561	2.715530	.3021	3.9229
	26	1026.909	0.441000	.0953	3.5964
	27	1026.909	0.915750	.1529	3.7088
	28	1026.909	1.828340	.2277	3.7851
	29	1026.909	2.723990	.3057	3.8101
	30	1026.909	3.635150	.3725	3.8362
	26A	1027.113	0.433971	.0919	3.7372
	27A	1027.113	0.925072	.1505	3.8004
	28A	1027.113	1.838450	.2360	3.8489

1	2	3	4	5	6
Test No.	Run No.	Gate Elevation	Q	Head on Model Crest	Model C in $Q=CLH^{3/2}$
7	29A	1027.113	2.732409	0.3047	3.8985
	29B	1027.113	2.746624	.3080	3.8559
	29C	1027.113	3.613752	.3666	3.9065
	30A	1027.113	3.629348	.3698	3.8714
	30B	1027.113	3.615777	.3727	3.8127
	30X	1027.113	4.532255	.4304	3.8505
	30Y	1027.113	4.521115	.4243	3.9244
	30Z	1027.113	4.517326	.4280	3.8707
	37	1031.542	0.227686	.0679	3.9131
	38	1031.542	0.396173	.0837	3.9310
	39	1031.542	0.608774	.1119	3.9001
	40	1031.542	0.841669	.1404	3.8390
	41	1031.542	1.113727	.1690	3.8447
	42	1031.542	1.404295	.1950	3.9131
	43	1031.542	1.705270	.2225	3.8980
	44	1031.542	2.080311	.2536	3.9079
	45	1031.542	2.270069	.2690	3.9042
	46	1020.000	0.458898	.1026	3.3465
	47	1020.000	0.915325	.1586	3.4818
	48	1020.000	1.846162	.2455	3.6416
	49	1020.000	2.745885	.3144	3.7370
	50	1020.000	3.654899	.3695	3.9040
	51	1020.000	4.530627	.4303	3.8512
	52	1020.000	5.349420	.4785	3.8782
	53	1020.000	6.218486	.5243	3.9308
	50A	1020.000	3.715939	.3775	3.8473
	51A	1020.000	4.521995	.4340	3.7955
	52A	1020.000	5.381154	.4753	3.9405
	47A	1020.000	0.928425	.1610	3.4484
	46X	1020.000	0.228705	.0661	3.2217
	50B	1020.000	3.616586	.3731	3.8082
	54	1028.934	0.437988	.0902	3.8757
	55	1028.934	0.929103	.1493	3.8640
	56	1028.934	1.852252	.2359	3.8802
	57	1028.934	2.723454	.3039	3.9008
	58	1025.028	0.924052	.1544	3.6577
	59	1025.028	1.850856	.2427	3.7146
	60	1025.028	2.752995	.3125	3.7821
	61	1025.028	3.631884	.3752	3.7952
	62	1025.028	0.438395	.0952	3.5819
	63	1022.061	0.436361	.1006	3.2833
	64	1022.061	0.922894	.1815	3.4129
	65	1022.061	1.469610	.2187	3.4960
	66	1022.061	3.600043	.3795	3.6962
	67	1022.061	4.526398	.4330	3.8129
	68	1023.507	0.441013	.0997	3.5620
	69	1023.507	0.921902	.1594	3.4765
	70	1023.507	1.336908	.2459	3.6154
	71	1023.507	3.659937	.3832	3.7130

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observed head over the model crest. This head was measured from the highest point of the gates to the water surface at a point three feet upstream from the face of the model dam and was used in the calculation of the coefficient C in the equation $Q = CLH^{3/2}$. Q from Column 4 and L, measured between piers with a micrometer caliper, were used for the other terms in the equation.

Since the above method of analysis has the decided disadvantage of being unable to obtain smooth curves and interpolation between observed data, the values of Q as abscissa and H as ordinate were then plotted on logarithmic sheets and with the fundamental equation $Q = KLH^n$ as a basis, the values of K_m and n were obtained from the intercepts and slopes of the straight lines through the data for each gate elevation. Very satisfactory results were produced by this method and to further justify it, additional runs were made with gates set at Elev. 1022.06, 1025.03, 1028.93 and 1031.41 respectively. A decided advantage of this method of analysis was that fewer runs were required to fix the position of a particular curve.

After summarizing the data obtained from the logarithmic plottings, it was found that the values of K_m and n must be fixed

with greater accuracy than is possible on logarithmic plottings due to the slight irregularities of the printing of the paper and the impossibility of interpolating to the necessary degree of accuracy.

Trial computations using the least square method of adjustment of observed data produced a much smaller percentage of error such that it was adopted for handling the coefficient data.

A discussion of the least square method of adjusting experimental data has been written by Mr. Carl Rohwer, Associate Engineer, Bureau of Agricultural Engineering, Department of Agriculture. The methods set forth in this paper have been applied to the data obtained from the above tests. A copy of this treatise is attached to this report as Appendix II.

Table No. 3 gives the summary of results of smoothing the data by means of the method of least squares.

Columns 1 and 2 identify the tests and runs. Column 3 gives the gate elevation in prototype feet, sea level datum, as determined by averaging the elevations of the highest part of the curve of the gate for all three gates of the model with reference to the fixed crest elevation of 1020, when the gates were completely lowered. Column 4 shows average observed head over the model crest measured at all times from the highest portion of the curve of the gate to the water surface at a

TABLE NO. 3

NORRIS DAM SPILLWAY

SUMMARY OF DATA.

Adjustment by Least Square Method.

1 Test No.	2 Run No.	3 Gate Elevation	4 Model Head on Crest	5 Model Q	6 Colog H	7 Log Q	8 $(\text{Log }H)^2$	9 Log H x Log Q
6	1	1020.000	0.1011	0.4813	-0.99528	-0.31628	0.99052	+0.315488
	2	1020.000	.1609	0.5150	-.79344	-.03716	.62954	+.029484
	3	1020.000	.2464	1.0888	-.60836	+.25553	.37010	-.160932
	4	1020.000	.3136	2.7225	-.50352	+.43497	.25343	-.219641
	5	1020.000	.3737	3.5246	-.42734	+.53926	.18263	-.230589
	7	1020.000	.4096	7.9014	-.21496	+.89770	.04632	-.192970
	8	1020.000	.4317	4.0711	-.36482	+.66008	.13309	-.240788
	9	1020.000	.4035	4.1047	-.39414	+.61327	.15534	-.241730
	10	1020.000	.3424	3.1468	-.46547	+.49787	.21668	-.231740
	11	1020.000	.5275	6.2572	-.27770	+.79636	.07716	-.221220
	12	1020.000	.5914	7.4583	-.22811	+.87239	.05204	-.198965
					-5.27322	+6.19394	3.10637	-1.533401
		11 A = 5.27322 n + 5.19394 1.533401 + 3.20637 n = 5.27322 A						
		A = 1.2475815 = Log K_m L n = 1.617505						
		K_m L = 17.68404 L = 4.166802 K_m = 4.2439						
7	46	1020.000	0.1026	0.486698	-0.96865	-0.33028	0.97782	+0.32481
	46X	1020.000	.0661	0.328703	-1.17960	-0.64072	1.39193	+0.75593
	47	1020.000	.1666	0.914926	-0.79970	-.03787	0.63955	+0.03028
	47A	1020.000	.1610	0.928825	-0.79317	-.03216	0.62912	+0.02309
	48	1020.000	.2486	1.644152	-0.60465	+.21602	0.37204	-0.16242
	49	1020.000	.3144	2.745865	-0.50252	+.43868	0.25253	-0.22044
	50	1020.000	.3695	3.484579	-0.43239	+.54204	0.18696	-0.24350
	50A	1020.000	.3773	3.726939	-0.42331	+.57008	0.17919	-0.24130
	50B	1020.000	.3721	3.616686	-0.42917	+.55830	0.18433	-0.23700
	51	1020.000	.4303	4.330467	-0.36623	+.63616	0.13412	-0.24030
	51A	1020.000	.4640	4.621996	-0.33351	+.66533	0.11141	-0.23756
	52	1020.000	.4788	5.349480	-0.32012	+.72830	0.10246	-0.23316
	52A	1020.000	.4758	5.381104	-0.32293	+.73086	0.10435	-0.23610
	53	1020.000	.5243	6.218424	-0.28042	+.79369	0.07864	-0.22257
					-7.81017	+4.91136	5.38343	-1.12998
		14 A = 7.81017 n + 4.91136 5.38343 n + 1.12998 = 7.81017 A						
		A = 1.2432486 = Log K_m L n = 1.598705						
		K_m L = 17.50006 L = 4.166802 K_m = 4.2016						
7	63	1022.061	0.1003	0.426361	-0.99740	-0.34018	0.99481	+0.33922
	64	1022.061	.1616	0.921696	-.79165	-.03466	.62677	+.02750
	65	1022.061	.2167	1.449610	-.66416	+.16729	.44108	-.11110
	66	1022.061	.3790	3.600648	-.42075	+.55630	.17706	-.23409
	67	1022.061	.4350	4.526396	-.36151	+.65575	.13114	-.23677
					-3.23767	+0.98433	2.37206	-0.19674
		5 A = 3.23767 n + 0.98433 0.19674 + 2.37206 n = 3.23767 A						
		A = 1.2322884 = Log K_m L n = 1.396973						
		K_m L = 17.07064 L = 4.16696 K_m = 4.0967						
7	68	1023.807	0.0997	0.441013	-1.00130	-0.35556	1.00260	+0.35603
	69	1023.807	.1674	0.921902	-0.77781	-.03522	0.60602	+0.02617
	70	1023.807	.2439	1.626900	-0.60926	+.26429	0.37117	-0.16089
	71	1023.807	.3532	3.669927	-0.45197	+.56465	0.17353	-0.25532
					-2.82462	+0.43767	2.18332	-0.01198
		4 A = 2.82462 n + 0.43767 2.18332 n + 0.01198 = 2.82462 A						
		A = 1.2219826 = Log K_m L n = 1.578411						
		K_m L = 16.67068 L = 4.16696 K_m = 4.0027						
7	58	1026.028	0.1344	0.924088	-0.81155	-0.03430	0.66059	+0.02788
	59	1026.063	.2457	1.830866	-0.61495	+.26750	0.37814	-0.16443
	60	1026.028	.3125	2.782955	-0.50515	+.44445	0.25518	-0.22515
	61	1026.028	.3758	3.631884	-0.42574	+.56014	0.18125	-0.23847
	62	1026.028	.0952	0.432395	-1.02134	-.36418	1.04318	+0.37198
					-3.37033	+0.87490	2.51604	-0.22413
		5 A = 3.37[?]6 n + 0.87490 2.8160 n + 0.1[?]48 = 3.37653 A						
		A = 1.2[?]7828 = Log K_m L n = 1.54280						
		K_m L = 16.80137 L = 4.16696 K_m = 3.9601						
7	26A	1027.113	0.0919	0.433971	-1.04223	-0.36256	1.07471	+0.37694
	27A	1027.113	.1406	0.920072	-0.82246	-.03617	0.67644	+0.02777
	28A	1027.113	.2460	1.838460	-0.65709	+.26446	0.39056	-0.16463
	27A	1027.113	.3047	2.732409	-0.51612	+.43656	0.26639	-0.22532
	29B	1027.113	.3080	2.746424	-0.51146	+.43880	0.26160	-0.22442
	27C	1027.113	.3666	3.617783	-0.43581	+.55796	0.18993	-0.24316
	30A	1027.113	.3690	3.329848	-0.43203	+.52243	0.18665	-0.24184
	30B	1027.113	.3787	3.418777	-0.42164	+.55820	0.18173	-0.23927
	30X	1027.113	.4304	4.030265	-0.36615	+.60532	0.13405	-0.24020
	30Y	1027.113	.4243	4.021118	-0.37233	+.60344	0.13863	-0.24397
	30Z	1027.113	.4220	4.017326	-0.36986	+.60452	0.13684	-0.24136
					-5.92721	+4.38592	3.66119	-1.56158
		11 A = 5.92721 n + 4.38592 3.66119 n + 1.56158 = 5.92721 A						
		A = 1.2178338 = Log K_m L n = 1.62269						
		K_m L = 16.51329 L = 4.16695 K_m = 3.9628						

1 Test No.	2 Run No.	3 Gate Elevation	4 Model Head on Crest	5 Model Q	6 Colog H	7 Log Q	8 $(\text{Log }H)^2$	9 Log H x Log Q
7	54	1028.934	0.0902	0.457890	+1.04379	-0.33954	1.09159	+0.37440
	55	1028.934	.1398	0.939100	0.82894	+0.02196	0.68218	+0.02630
	56	1028.934	.2359	1.632251	0.62727	-0.24770	0.39347	-0.16792
	57	1028.934	.3009	2.723451	0.51727	-0.43512	0.26757	-0.22507
					3.01527	-0.31234	2.43481	+0.00779
		4 A = 3.01527 n + 0.31234 2.43481 n - 0.00779 = 3.01527 A						
		A = 1.2119095 = Log K_m L n = 1.50411						
		K_m L = 16.2895 L = 4.16695 K_m = 3.9092						
7	21	1031.412	0.0578	0.218980	-1.24185	-0.66098	1.54219	+0.82076
	22	1031.412	.0892	0.529447	-1.04964	-0.26307	1.10174	+0.28109
	23	1031.412	.1465	0.910880	-0.83418	-0.04070	0.69582	+0.03398
	24	1031.412	.2349	1.836850	-0.62912	+0.26407	0.39579	-0.16613
	25	1031.412	.3060	2.735680	-0.51428	+0.43578	0.26448	-0.22411
					-4.26905	-0.26390	4.00002	+0.84258
		5 A = 4.26905 n - 0.26390 4.00002 n - 0.84308 = 4.26905 A						
		A = 1.21289146 = Log K_m L n = 1.50810						
		K_m L = 16.30310 L = 4.16690 K_m = 3.9125						
7	37	1031.842	0.0279	0.227686	-1.55738	-0.64264	1.55094	+0.79810
	38	1031.848	.0837	0.396173	-1.07727	-0.40212	1.16051	+0.43318
	39	1031.842	.1119	0.60877	-0.95117	-0.21558	0.90472	+0.20502
	40	1031.842	.1404	0.841689	-0.85263	-0.07486	0.72698	+0.06363
	41	1031.842	.1690	1.113757	-0.77211	+0.04678	0.59615	-0.03612
	42	1031.842	.1980	1.406295	-0.70777	+0.14766	0.50406	-0.10449
	43	1031.842	.2224	1.704270	-0.65367	+0.23305	0.42698	-0.15245
	44	1031.842	.2532	2.080311	-0.59658	+0.31812	0.35604	-0.18986
	45	1031.842	.2690	2.270069	-0.57023	+0.35604	0.32519	-0.20302
					-7.41984	-0.23471	6.52959	+0.61234
		9 A = 7.41984 n - 0.23471 6.52959 n - 0.61234 = 7.41984 A						
		A = 1.2077597 = Log K_m L n = 1.494725						
		K_m L = 16.13440 L = 4.16696 K_m = 3.8721						
7	21X	1031.561	0.0728	0.323450	-1.13946	-0.49020	1.29885	+0.55864
	21A	1031.561	.0878	0.429780	-1.05659	-0.36678	1.11646	+0.38839
	22B	1031.561	.0860	0.435570	-1.06552	-0.36114	1.11412	+0.38119
	22C	1031.561	.0859	0.434180	-1.03110	-0.36179	1.10481	+0.38023
	22X	1031.561	.1001	0.903480	-0.88362	-0.02620	0.67826	+0.02188
	22Y	1031.561	.1203	0.680540	-0.91972	-0.16715	0.84590	+0.15373
	23A	1031.561	.1468	0.915390	-0.83327	-0.03838	0.69434	+0.03279
	23X	1031.561	.1930	1.383472	-0.71444	+0.14808	0.51045	-0.10063
	24A	1031.561	.2358	1.843160	-0.63077	+0.26556	0.39812	-0.16756
	24X	1031.561	.2648	2.252120	-0.57708	+0.34618	0.33302	-0.20070
	25A	1031.561	.3025	2.715830	-0.51927	+0.43396	0.26964	-0.22525
	25B	1031.561	.3021	2.715630	-0.51986	+0.43384	0.27034	-0.22554
					-9.84350	-0.19114	8.62826	+0.99727
		12 A = 9.84350 n - 0.19114 8.62826 n - 0.99727 = 9.84350 A						
		A = 1.204529 = Log K_m L n = 1.49632						
		K_m L = 16.0238 L = 4.16696 K_m = 3.8449						
7	7A	1033.910	0.0896	0.423781	-1.04666	-0.36279	1.09551	+1.42704
	7X	1033.910	.0678	0.525516	-1.22607	-0.24480	1.50321	+0.77898
	8A	1033.910	.1476	0.913697	-0.83091	-0.03924	0.69041	+0.03260
	9A	1033.910	.2061	1.341462	-0.68594	+0.12416	0.39945	-0.14896
	9X	1033.910	.1816	1.342632	-0.71780	+0.12806	0.51495	-0.09190
	9Y	1033.910	.1816	1.342632	-0.71718	+0.12865	0.51430	-0.09214
	10A	1033.910	.3063	2.725803	-0.51108	+0.43548	0.26115	-0.22261
	10B	1033.910	.3761	3.677293	-0.42470	+0.56554	0.18037	-0.24018
	10C	1033.910	.3414	3.162369	-0.46647	+0.50000	0.21644	-0.23274
	10D	1033.910	.3392	3.164194	-0.46954	+0.50027	0.22047	-0.23485
	10E	1033.910	.3748	3.671158	-0.42626	+0.56480	0.18144	-0.24072
	10X	1033.910	.2749	2.325440	-0.56092	+0.36641	0.31463	-0.20784
					-8.02340	+2.39949	6.11843	-0.81013
		12 A = 8.02340 n + 2.39949 6.11843 n + 0.81013 = 8.02340 A						
		A = 1.1930494 = Log K_m L n = 1.456447						
		K_m L = 15.66929 L = 4.16698 K_m = 3.7604						

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point three feet upstream from the face of the model dam. Column 5 shows the quantity of water passing over the model gates, calculated from observations of the head passing the two-foot Cipolletti weir.

Column 6 gives the colog of the values of H. As H is always less than unity, the log H is always less than zero. The log of a number less than unity is the minus colog of that number.

Column 7 shows the log of Q. In instances where Q is less than unity, the minus colog is shown as explained above. Column 8 gives the square of the value given in Column 6. The values shown in Column 9 are the products of values in Columns 6 and 7.

The runs were divided into groups, made with the gates at the same elevation. At the bottom of each of these groups the calculations for the values of K_m and n for that gate elevation are shown.

The algebraic sums of the columns were used to form the two equations of A and n as set forth in Appendix II. These two equations are given in the table immediately below the algebraic sums. Solving the two equations, the values of n and A are obtained. Since A is the log of $K_m L_m$, with a known value of L_m obtained by measurement as described above,

a value of K_m was obtained. These values of K_m and n are the most probable values for the series of observations.

It will be noted that some runs were omitted from this table. As described in Appendix II, the percent of deviation of a single observation can be obtained. If this deviation exceeds a certain limit, as set forth in the discussion, it is reasonable to believe that the observation is subject to some outside irregularity and can be eliminated from the series. The deviation and the allowable deviation were calculated for each run and if the deviation exceeded the allowable, the run was omitted from the final calculation and also from the table.

An explanation of the laws of hydraulic similitude and the transference of results from model to prototype is given by Alton C. Chick in John R. Freeman's "Hydraulic Laboratory Practice", Page 796.

$$Q_p = Q_m N^{5/2} \quad (1)$$

where Q_p = prototype quantity in c.f.s.

Q_m = model quantity in c.f.s.

N = scale ratio of model (in this case 72)

In the case of the quantity of water flowing over a dam

$$Q_p = K_p L_p H_p^n \quad (2)$$

For the model

$$Q_m = K_m L_m H_m^n \quad (3)$$

From (1) and (3)

$$Q_p = K_m L_m H_m^n N^{5/2} \quad (4)$$

Equating (2) and (4)

$$K_p L_p H_p^n = K_m L_m H_m^n N^{5/2} \quad (5)$$

where

K_m = coefficient of discharge on the model

K_p = coefficient of discharge on the prototype

L_m = length of crest on the model

L_p = length of crest on the prototype

but

$L_p = N L_m$

$H_p = N H_m$

Substituting these values in (5)

$$K_p N L_m (N H_m)^n = K_m L_m H_m^n N^{5/2}$$

Dividing through by $L_m N H_m^n$

$$K_p N^n = K_m N^{3/2}$$

Whence

$$K_p = K_m N^{3/2-n} \quad (6)$$

With a value of model K and n for each gate setting a table of values for prototype K and n were obtained. This table is shown as Table No. 4.

Values of K_p were calculated using Equation 6 and the values of K_m and n from Table No. 3. The values of n are the same whether model or prototype. Table No. 4 gives

TABLE NO. 4

NORRIS DAM SPILLWAY

SUMMARY OF DATA.

Calculations of Prototype Coefficient of Discharge

from Observed Data.

$K_{proto} = K_{model} \times N^{1.5-n}$

where N = 72 and Log N = 1.857333

1	2	3	4	5	6	7
Gate Elevation	n	1.5-n	Log n x (1.5-n)	$N^{1.5-n}$	K Model	K Proto.
1020.000	1.6175	-0.1175	-0.218237	0.60500	4.2439	2.5676
1020.000	1.5997	- .0997	- .185176	0.65285	4.2018	2.7431
1022.061	1.5990	- .0990	- .183876	0.65481	4.0967	2.6826
1023.507	1.5754	- .0754	- .140043	0.72436	4.0007	2.8979
1025.028	1.5429	- .0429	- .079680	0.83238	3.9601	3.2963
1027.113	1.5227	- .0227	- .042161	0.90748	3.9629	3.5963
1028.934	1.5041	- .0041	- .007634	0.98258	3.9092	3.8411
1031.412	1.5051	- .0051	- .009472	0.97843	3.9125	3.8281
1031.542	1.4967	+ .0033	+ .006129	1.01421	3.8721	3.9271
1031.561	1.4883	+ .0117	+ .021731	1.05131	3.8469	4.0443
1033.908	1.4865	+ .0135	+ .025074	1.05943	3.7604	3.9839

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the calculated results. Column 1 gives the elevation of the gates. Column 2 gives the values of n taken from Table No. 3. Column 3 gives values of 3/2 minus n. Column 4 gives product of values in Column 3 and log of 72 which is 1.857333. Column 5 gives the values of $N^{1.5-n}$. Column 6 gives the values of K_m taken from Table 3 and Column 7 gives values of the prototype coefficient for the different gate elevations as calculated from Equation 6.

The results from this table were plotted on cross-section paper K_p vs Gate Elevation, K_m vs Gate Elevation, K_p vs n and n vs Gate Elevation. These curves are shown on Figure 26.

These results show that as the gates are raised the prototype coefficient of discharge increases and the model coefficient decreases. Also as the gates are raised the n decreases for both model and prototype.

All the curves on this plate are interdependent. For instance, if the prototype coefficient curve is changed slightly, then the other three curves must be changed accordingly in order for the given values to check. All the plotted points are not given the same weight on the curves as there are more observations on some than on others.

FIGURE 26

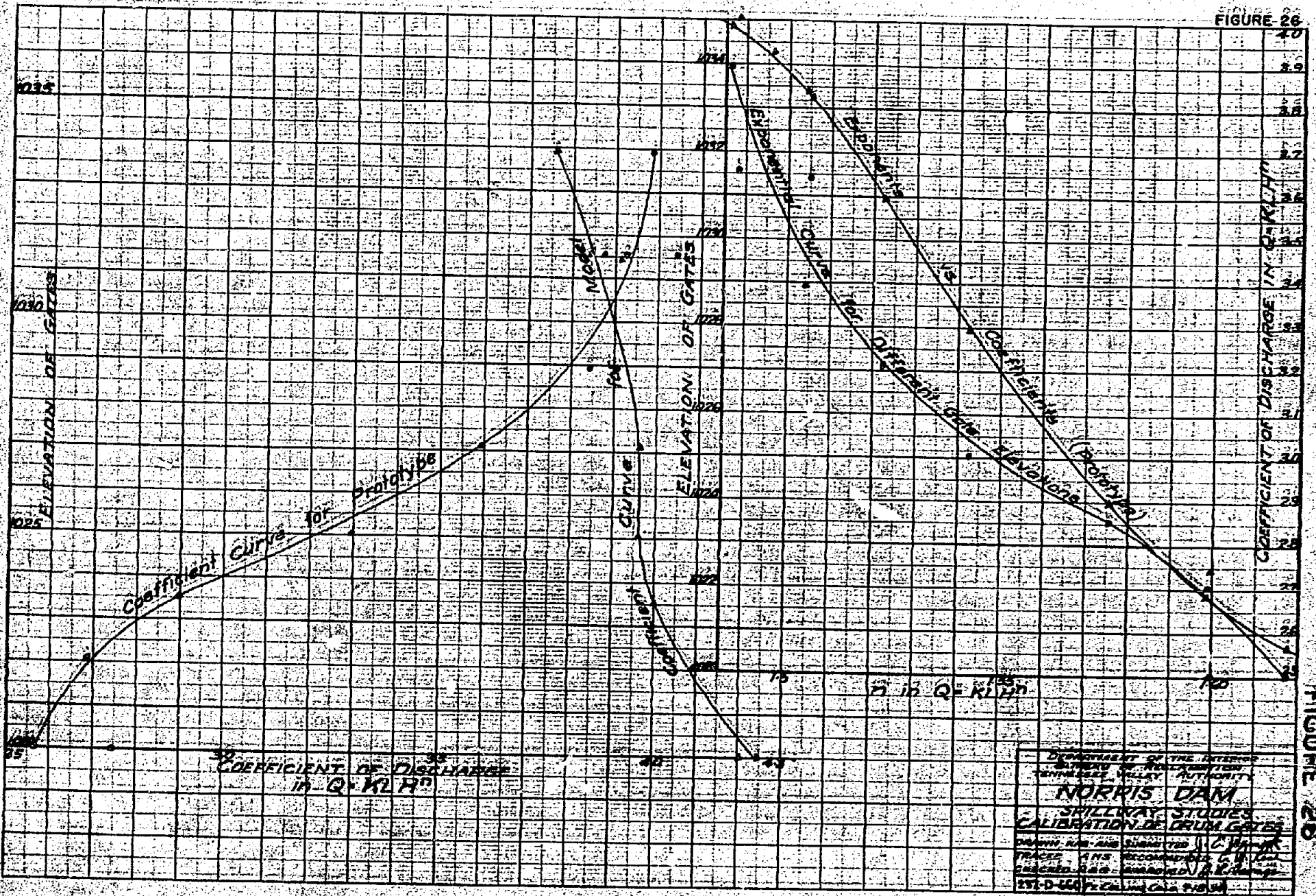


FIGURE 26

Values of K_p and n were read from these curves for each foot of gate elevation. By substituting these values in the equation $Q = K_pLH^n$, and using different values of H, a discharge table (Table 5) showing quantities was prepared. Column 1 gives the head over the gates in feet. The successive columns show discharge in cubic-feet per second and reservoir elevation respectively for the gate elevation given at the head of the columns.

This table was compiled to furnish material for the construction of discharge curves. Therefore, only a sufficient number of points were calculated to fix the curves. Additional points may be calculated from the values given at the head of the columns.

The results from this table were plotted and are shown on Figure 27. From inspection of these curves, it can be seen that for any given elevation of gates and given head over the gates the quantity passing can be read directly. This diagram is based on having all the gate crests at the same elevation, as it is expected that the gates will be so operated. However, should the gates be at somewhat different levels, one-third of the sum of the discharges taken from the diagram for the three gate elevations might be used. If the gate crests differed only slightly in elevation the mean crest elevation could be used. No study was made of the discharge with

TABLE NO. 5

NORRIS DAM SPILLWAY

Discharge Table Computed from the Formula $Q = KH^n$

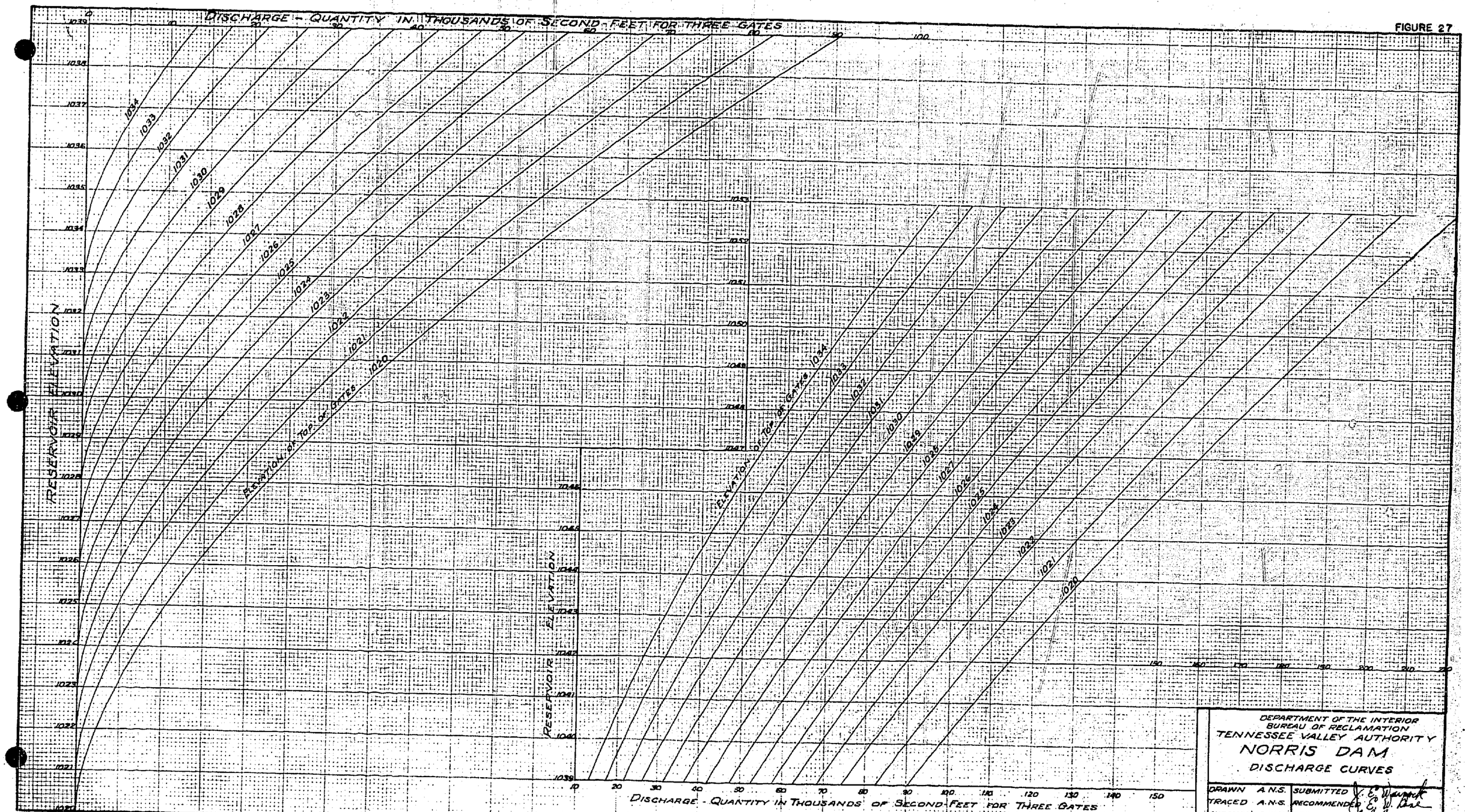
Discharge Q for all three Gates at the same Elevation.

Head over Gates in Feet	Gate El. 1020 ($n = 1.5173$, $K_p = 2.5690$, $K_L = 770.4$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1021 ($n = 1.6081$, $K_p = 2.6170$, $K_L = 785.1$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1022 ($n = 1.5972$, $K_p = 2.6960$, $K_L = 808.8$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1023 ($n = 1.5843$, $K_p = 2.8090$, $K_L = 842.7$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1024 ($n = 1.5660$, $K_p = 3.0030$, $K_L = 900.9$) Quantity in c.f.s.	El. Reservoir Water Surface
1	770	1021	785	1022	809	1023	843	1024	901	1025
2	2,544	1022	2,593	1023	2,447	1024	2,527	1025	2,667	1026
3	4,884	1023	4,594	1024	4,476	1025	4,804	1026	5,032	1027
4	7,252	1024	7,296	1025	7,404	1026	7,877	1027	7,885	1028
5	13,974	1025	14,005	1027	14,148	1028	14,404	1029	14,898	1030
8	22,248	1028	22,243	1029	22,400	1030	22,722	1031	23,376	1032
10	31,917	1030	31,944	1031	31,992	1032	32,367	1033	33,156	1034
12	42,662	1032	42,692	1033	42,806	1034	43,193	1035	44,102	1036
14	54,999	1034	54,704	1036	54,757	1036	55,143	1037	56,153	1038
16	68,256	1036	67,806	1037	67,776	1038	68,153	1039	69,213	1040
20	97,918	1040	97,076	1041	96,794	1042	97,027	1043	98,163	1044
24	131,307	1044	130,149	1046	129,512	1046	129,521	1047	130,596	1048
28	168,741	1048	166,764	1049	165,671	1050	165,362	1051	166,260	1052
32	209,410	1052	206,700	1053	205,053	1054	204,307	1055	-	-
36	253,346	1056	-	-	-	-	-	-	-	-

Head over Gates in Feet	Gate El. 1025 ($n = 1.5485$, $K_p = 3.2200$, $K_L = 966.0$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1026 ($n = 1.5360$, $K_p = 3.4100$, $K_L = 1023.0$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1027 ($n = 1.5240$, $K_p = 3.5740$, $K_L = 1072.2$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1028 ($n = 1.515$, $K_p = 3.704$, $K_L = 1111.2$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1029 ($n = 1.508$, $K_p = 3.796$, $K_L = 1138.8$) Quantity in c.f.s.	El. Reservoir Water Surface
1	964	1026	1,023	1027	1,072	1028	1,111	1029	1,139	1030
2	2,843	1027	2,966	1028	3,084	1029	3,176	1030	3,239	1031
3	5,294	1028	5,524	1029	5,720	1030	5,870	1031	5,970	1032
4	8,456	1029	8,591	1030	8,848	1031	9,078	1032	9,212	1033
5	15,486	1031	16,008	1032	16,450	1033	16,776	1034	16,978	1035
8	24,178	1032	24,895	1034	25,503	1035	25,940	1036	26,200	1037
10	34,157	1034	35,066	1036	35,832	1037	36,374	1038	36,682	1039
12	45,299	1037	46,489	1038	47,309	1039	47,946	1040	48,283	1041
14	57,812	1039	58,774	1040	59,838	1041	60,569	1042	60,927	1043
16	70,722	1041	72,144	1042	73,242	1043	74,137	1044	74,818	1045
20	99,913	1045	101,615	1046	103,060	1047	103,957	1048	104,328	1049
24	132,505	1049	134,480	1050	136,066	1051	137,029	1052	137,342	1053
28	168,230	1053	170,220	1054	172,079	1055	-	-	-	-

Head over Gates in Feet	Gate El. 1030 ($n = 1.502$, $K_p = 3.865$, $K_L = 1159.5$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1031 ($n = 1.4972$, $K_p = 3.918$, $K_L = 1176.4$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1032 ($n = 1.495$, $K_p = 3.951$, $K_L = 1185.3$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1033 ($n = 1.4895$, $K_p = 3.972$, $K_L = 1191.6$) Quantity in c.f.s.	El. Reservoir Water Surface	Gate El. 1034 ($n = 1.486$, $K_p = 3.985$, $K_L = 1195.5$) Quantity in c.f.s.	El. Reservoir Water Surface
1	1,140	1031	1,175	1032	1,185	1033	1,192	1034	1,196	1035
2	3,284	1032	3,318	1033	3,336	1034	3,345	1035	3,349	1036
3	6,036	1033	6,089	1034	6,112	1035	6,119	1036	6,117	1037
4	9,302	1034	9,347	1035	9,391	1036	9,392	1037	9,360	1038
6	17,102	1036	17,189	1037	17,203	1038	17,180	1039	17,135	1040
8	26,346	1038	26,448	1039	26,433	1040	26,370	1041	26,275	1042
10	36,836	1040	36,951	1041	36,883	1042	36,765	1043	36,606	1044
12	48,439	1042	48,528	1043	48,422	1044	48,234	1045	47,997	1046
14	61,060	1044	61,118	1045	60,954	1046	60,662	1047	60,353	1048
16	74,621	1046	74,644	1047	74,401	1048	74,033	1049	73,699	1050
20	104,232	1050	104,282	1051	103,817	1052	103,218	1053	102,537	1054
24	137,198	1054	136,975	1055	-	-	-	-	-	-

FIGURE 27



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

NORRIS DAM

DISCHARGE CURVES

DRAWN	A.N.S.	SUBMITTED	J. E. Warnock
TRACED	A.N.S.	RECOMMENDED	E. W. Lane
CHECKED	C.W.T.	APPROVED	L. L. Grange
232-D-661	FT. COLLINS, COLO. 7-13-34		

gates at widely differing elevations. It is probable that the first of the two methods described for gates at unequal elevations could be used with sufficient accuracy with any but extremely different gate settings.

The results of the coefficient tests show that the spillway will discharge the quantity anticipated in the preliminary calculations and have a small margin of safety in the form of additional discharge.

A discharge curve for the prototype spillway was the major requirement of the coefficient experiments, but in order to show the results in a more conventional form the values of C in the formula $Q = CLH^{3/2}$ as shown in Table 2 were plotted against reservoir elevations. In order to obtain a smooth curve through these observed points on cross-section paper, especially when there were only four or five observations, values of C for each curve were calculated. For each observed gate elevation, values of the head covering the observed range were assumed and values of C calculated by use of the formula $C = K_p H_p^{n-3/2}$ which is derived as follows:

$$Q_p = C L_p H_p^{3/2} \quad (1)$$

and

$$Q_p = K_p L_p H_p^{n} \quad (2)$$

gates at widely differing elevations. It is probable that the first of the two methods described for gates at unequal elevations could be used with sufficient accuracy with any but extremely different gate settings.

The results of the coefficient tests show that the spillway will discharge the quantity anticipated in the preliminary calculations and have a small margin of safety in the form of additional discharge.

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$$Q_p = C L_p H_p^{3/2} \quad (1)$$

and

$$Q_p = K_p L_p H_p^{n} \quad (2)$$

Equating (1) and (2)

$$C\,L_p\,H_p^{3/2} = K_p\,L_p\,H_p^{n}$$

Whence, dividing by L_p

$$C\,H_p^{3/2} = K_p\,H_p^{n} \quad (3)$$

$$\text{or} \quad C = K_p\,H_p^{n-3/2} \quad (4)$$

The values of K_p and n were taken from the curves on Figure 26. Curves were then drawn through these calculated points. As can be seen on Figure 28 these are smooth curves and pass through the observed points very consistently but there is no way of interpolating results for gate elevations between those observed directly.

By the above method, curves showing C plotted against reservoir elevation for gate elevations of 1020, 1022, etc., were plotted and are shown on Figure 29. These curves were constructed to make possible a comparison between the results of these tests and tests on other models that have been calculated for C in $Q = CLH^{3/2}$.

The fact that the curves cross each other can be explained by a study of the changing crest section as the gates are raised. The angle between the slope on the upstream face of the crest section and the chord of the gate section increases very rapidly as the gates are raised thus changing the conditions of flow radically.

All results were very carefully checked and the plotted curves show the final analysis of the data.

FIGURE 28

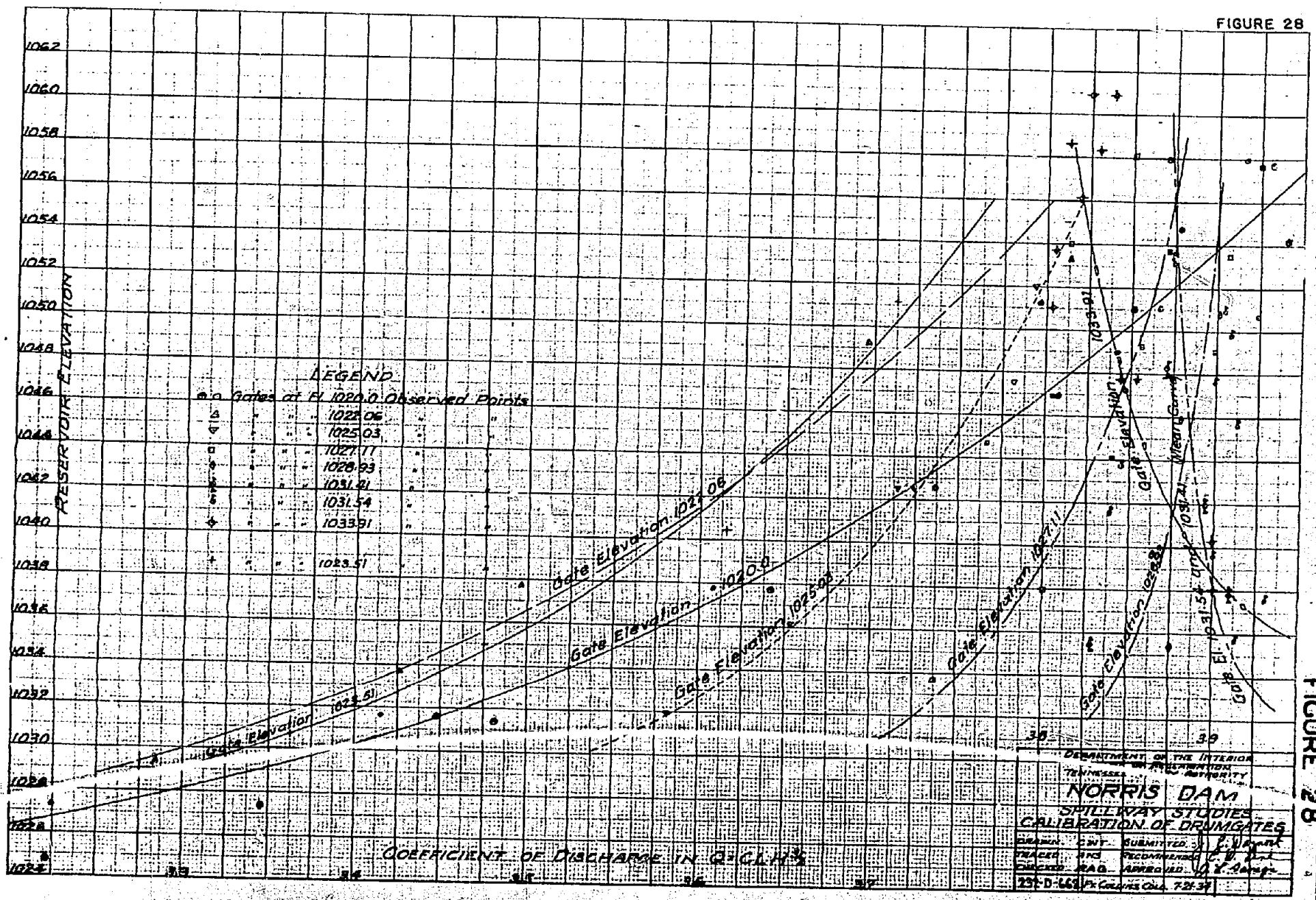


FIGURE 28

FIGURE 29

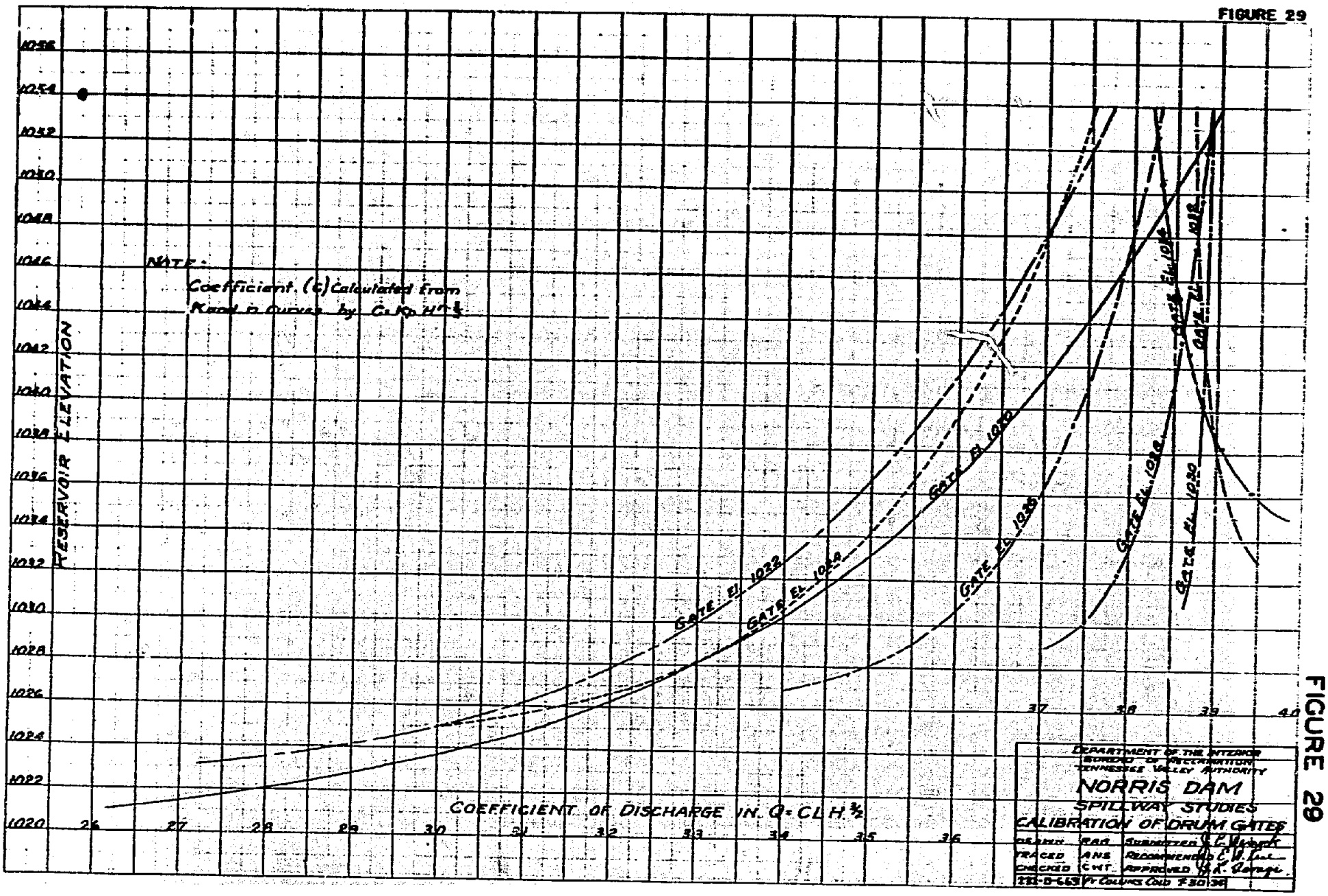


FIGURE 29

The values of C in $Q = CLH^{3/2}$ are the same for model and prototype as can be seen by analysis of the formula:

$$Q_p = Q_m N^{5/2} \quad (1)$$

But

$$Q_p = C_p L_p H_p^{3/2} \quad (2)$$

And

$$Q_m = C_m L_m H_m^{3/2} \quad (3)$$

Substituting Q_m from (3) in (1)

$$Q_p = C_m L_m H_m^{3/2} N^{5/2} \quad (4)$$

Equating (2) and (4)

$$C_m L_m H_m^{3/2} N^{5/2} = C_p L_p H_p^{3/2}$$

Substituting

$$L_p = L_m N$$

$$H_p = H_m N$$

$$C_m L_m H_m^{3/2} N^{5/2} = C_p L_m N (H_m N)^{3/2}$$

Dividing through by $L_m H_m^{3/2} N^{5/2}$

$$C_m = C_p \quad (5)$$

3. Pressures on Drum Gates

The pressures on the drum gates were measured while the gates were set for the coefficient runs. This explains why the measured pressures were taken with the gates at elevations other than even feet.

The pressures measured in feet of water on the model were converted to prototype by multiplying model pressures by 72 (the scale ratio). As no pressures were found to be substantially less than atmospheric, the results could be transferred by this simple means.

Plottings of the results are shown on Figures 30, 31 and 32. These curves show the position of the gate and the total pressure as measured by the piezometers converted to prototype dimensions. The curves shown are representative of the data obtained. More data, taken at the same gate elevations and at practically the same heads over the gates, were plotted and showed very consistent results.

It will be noted on Figure 30, Run 53 that the pressure is below atmospheric for all the piezometers, but the head for this run was approximately 18 percent more than the designed head. At the designed head (Run 51), the pressure is below atmospheric on only the three downstream piezometers and then only a small amount. On Figure 32 the Q for Run 11 was approximately 120,000 second-feet and for Run 12 approximately 160,000 second-feet.

FIGURE 30

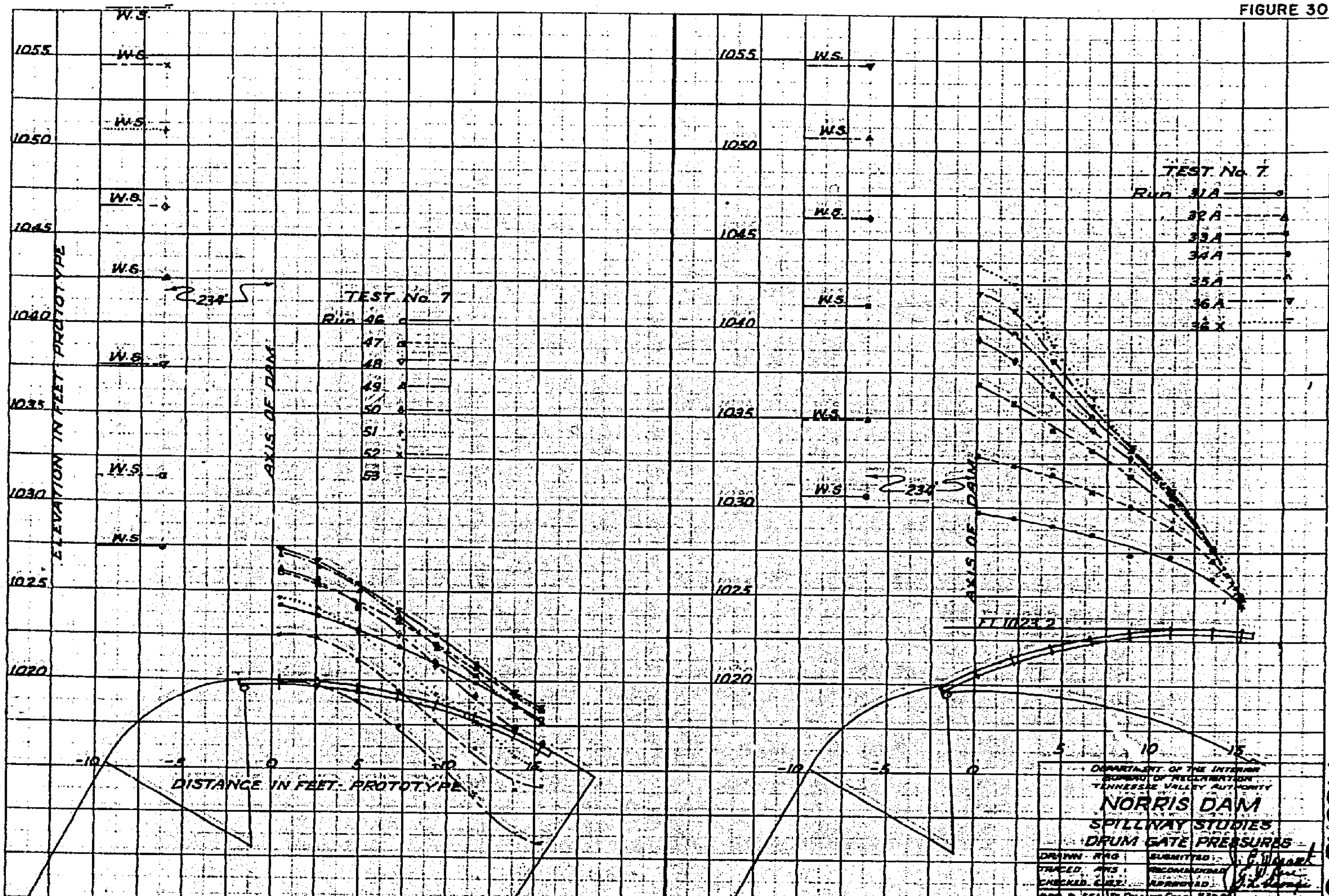
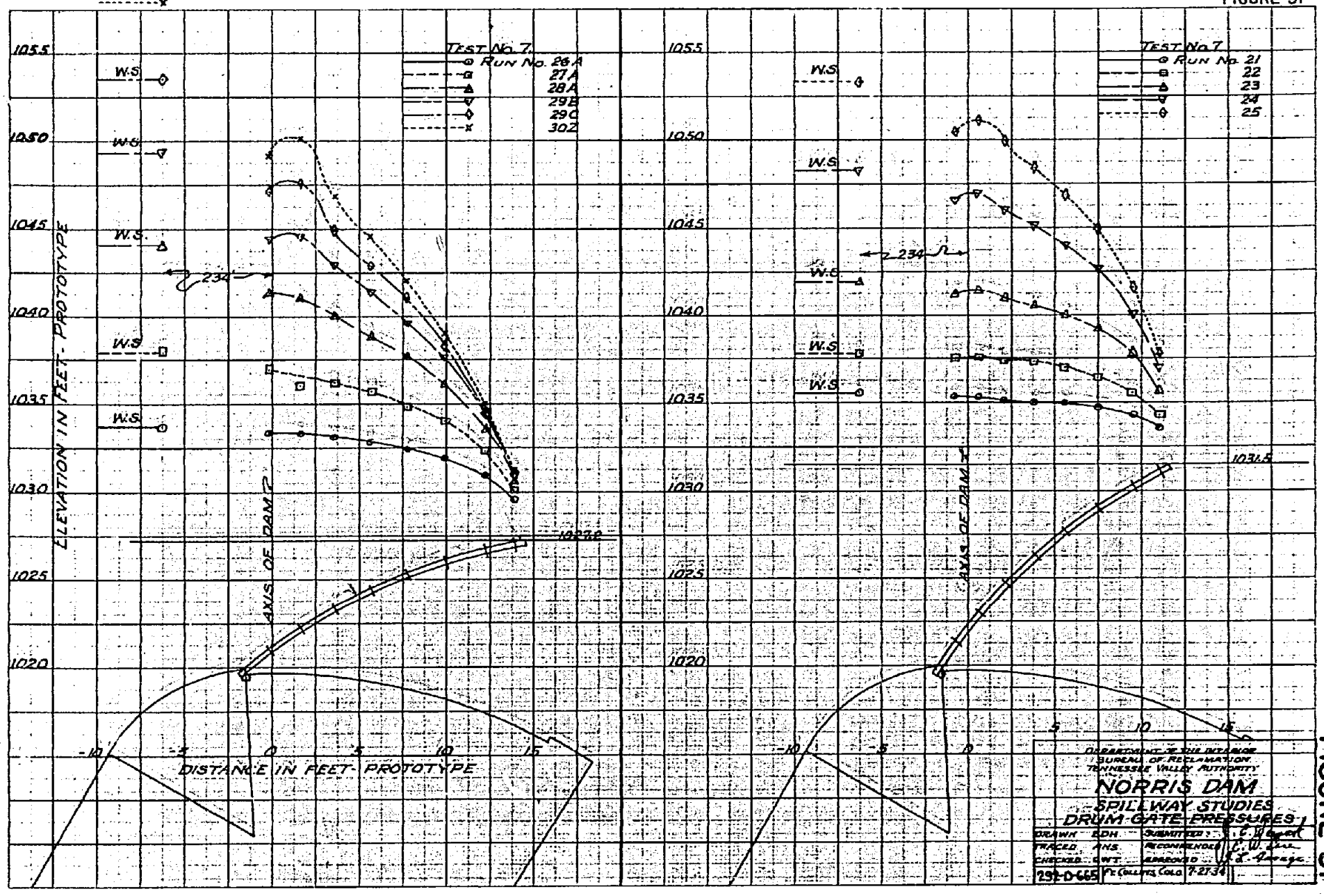


FIGURE 30

FIGURE 31



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TENNESSEE VALLEY AUTHORITY

NORRIS DAM
SPILLWAY STUDIES
DRUM GATE PRESSURES

DRAWN RDH. SUBMITTED: C. W. Lane
TRACED ANS. RECOMMENDED: C. W. Lane
CHECKED CWT. APPROVED: H. K. Gamage
292-D-665 Ft. Collins, Colo. 7-27-34

FIGURE 31

FIGURE 32

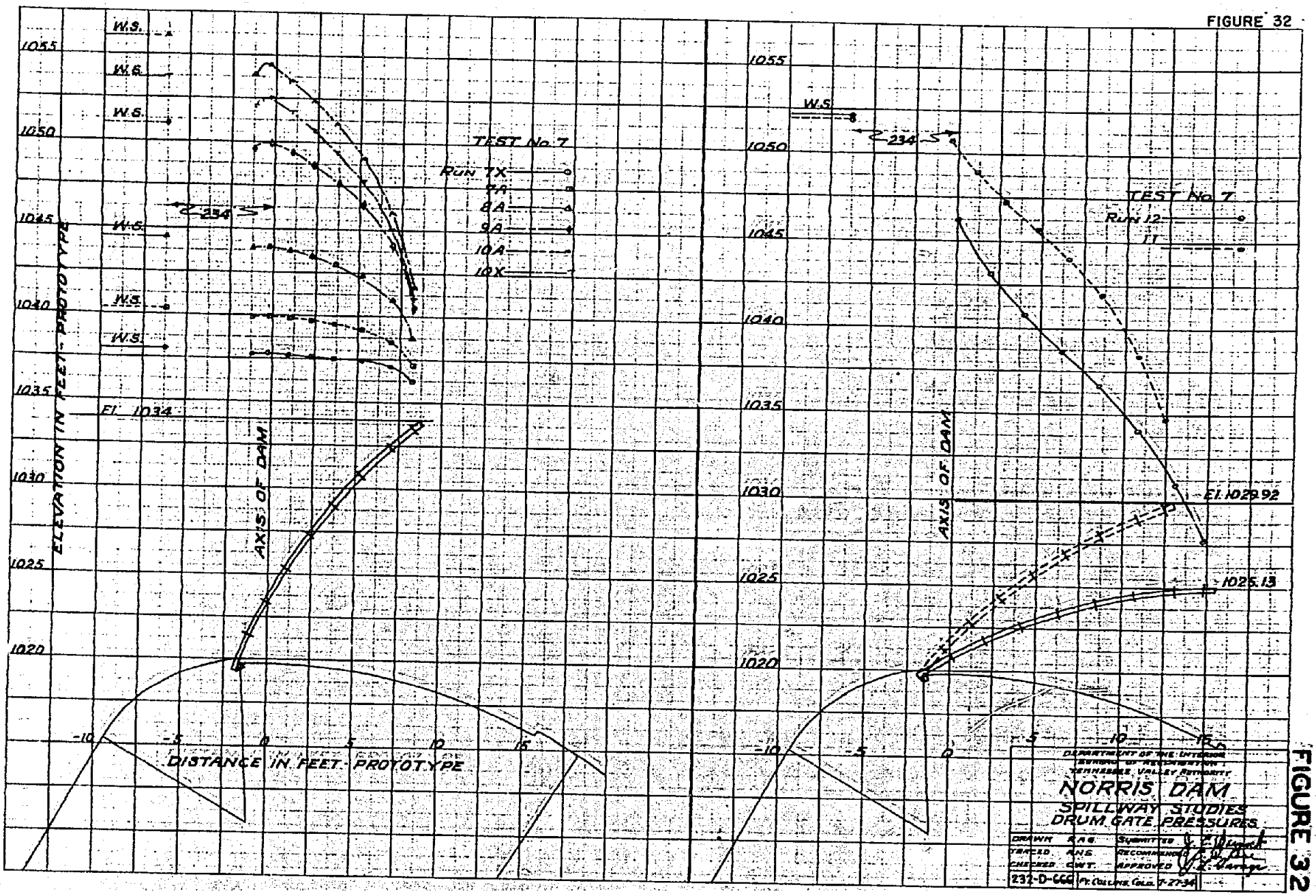


FIGURE 32

The results of these pressure tests were forwarded, upon completion, to the design department to assist in the design of the drum gates.

4. Tests on Training Walls

The results of the measurements on the upper training walls of the spillway section are shown on Figure 33. The elevations given are exact as determined from the model and converted to prototype. The slope of the top of the training wall was determined from the model.

The results of the test show that the upper corner of the training wall could be reduced to a curve of approximately 35-foot radius and result in a saving of concrete. The point of intersection of the sloping training wall with the horizontal wall could be moved closer to the downstream face of the dam, thus making a steeper slope than that given and would result in a further saving.

The results of the pressure tests on the training wall between the spillway and tailrace were plotted as shown on Figures 34, 35 and 36. The locations of the piezometer openings as they were installed is shown on Figure 36.

FIGURE 33

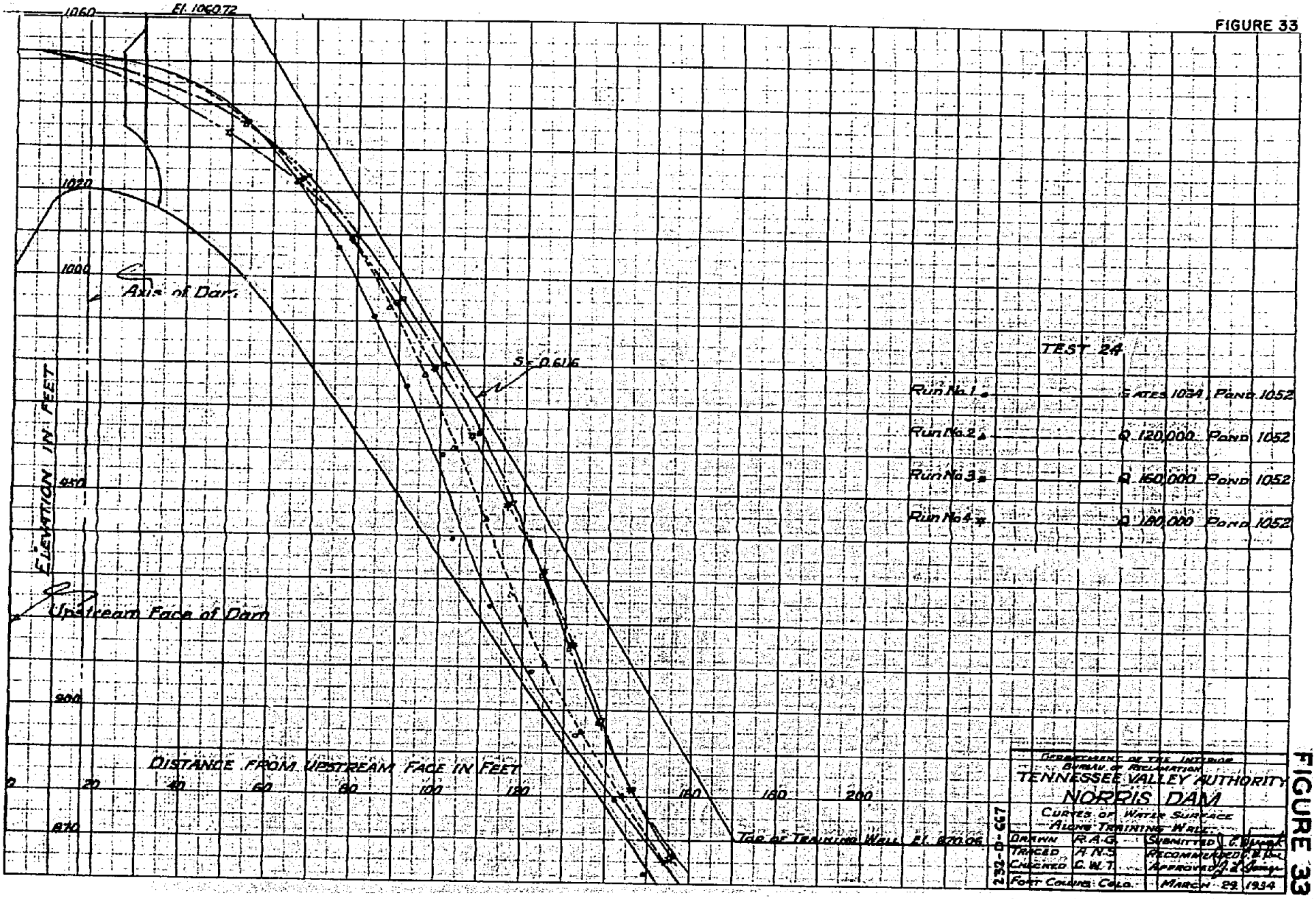


FIGURE 33

FIGURE 34

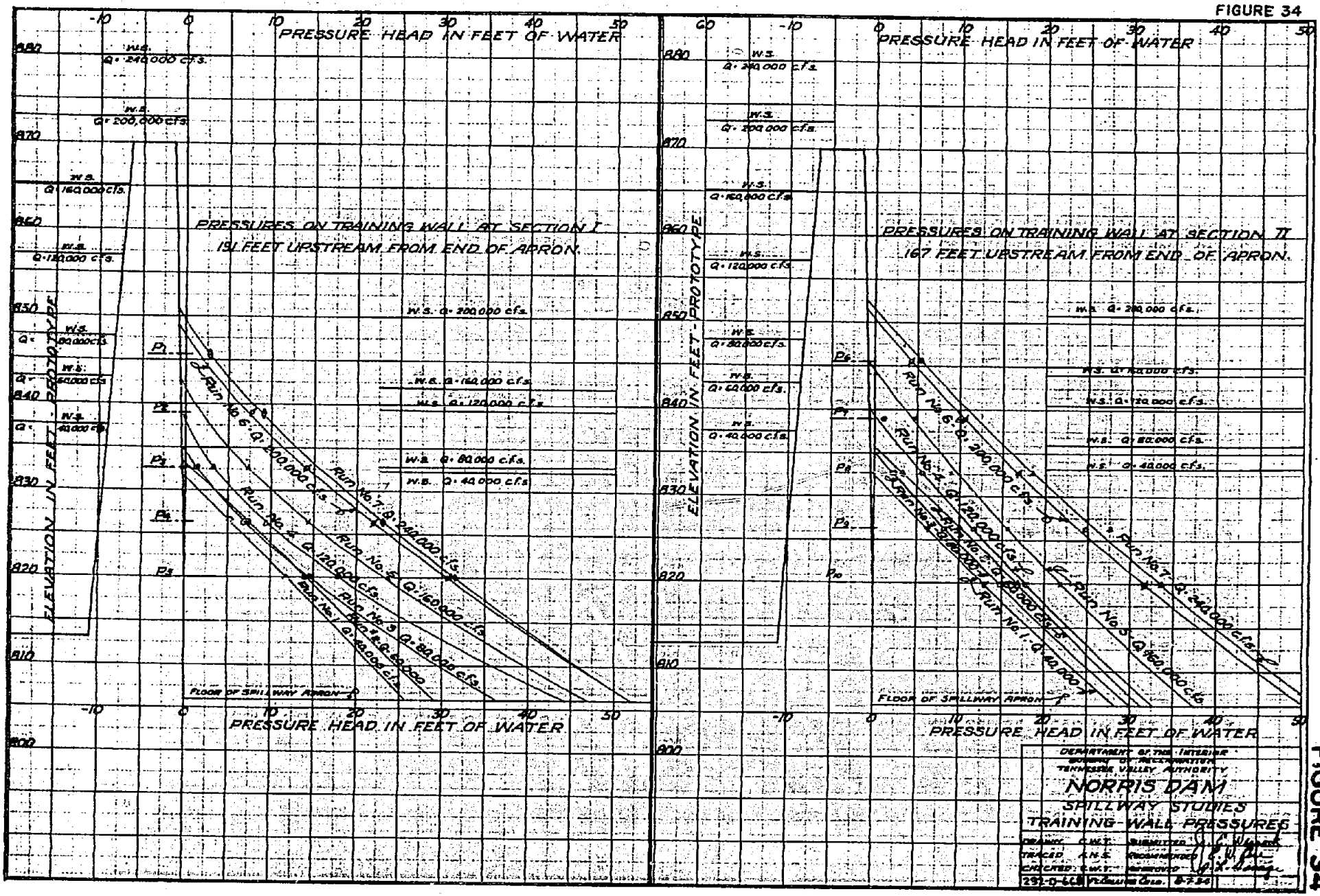


FIGURE 34

FIGURE 35

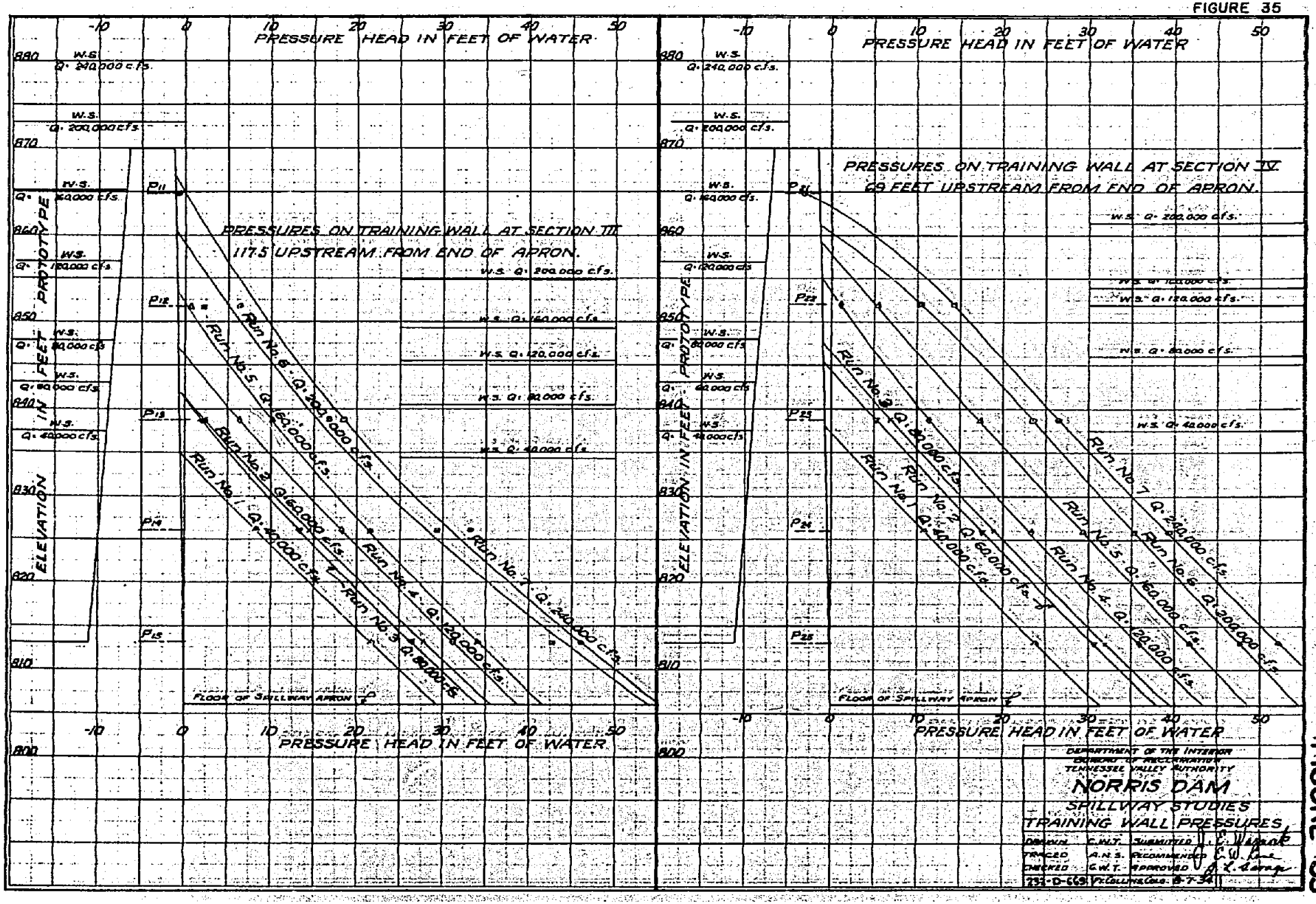


FIGURE 35

FIGURE 36

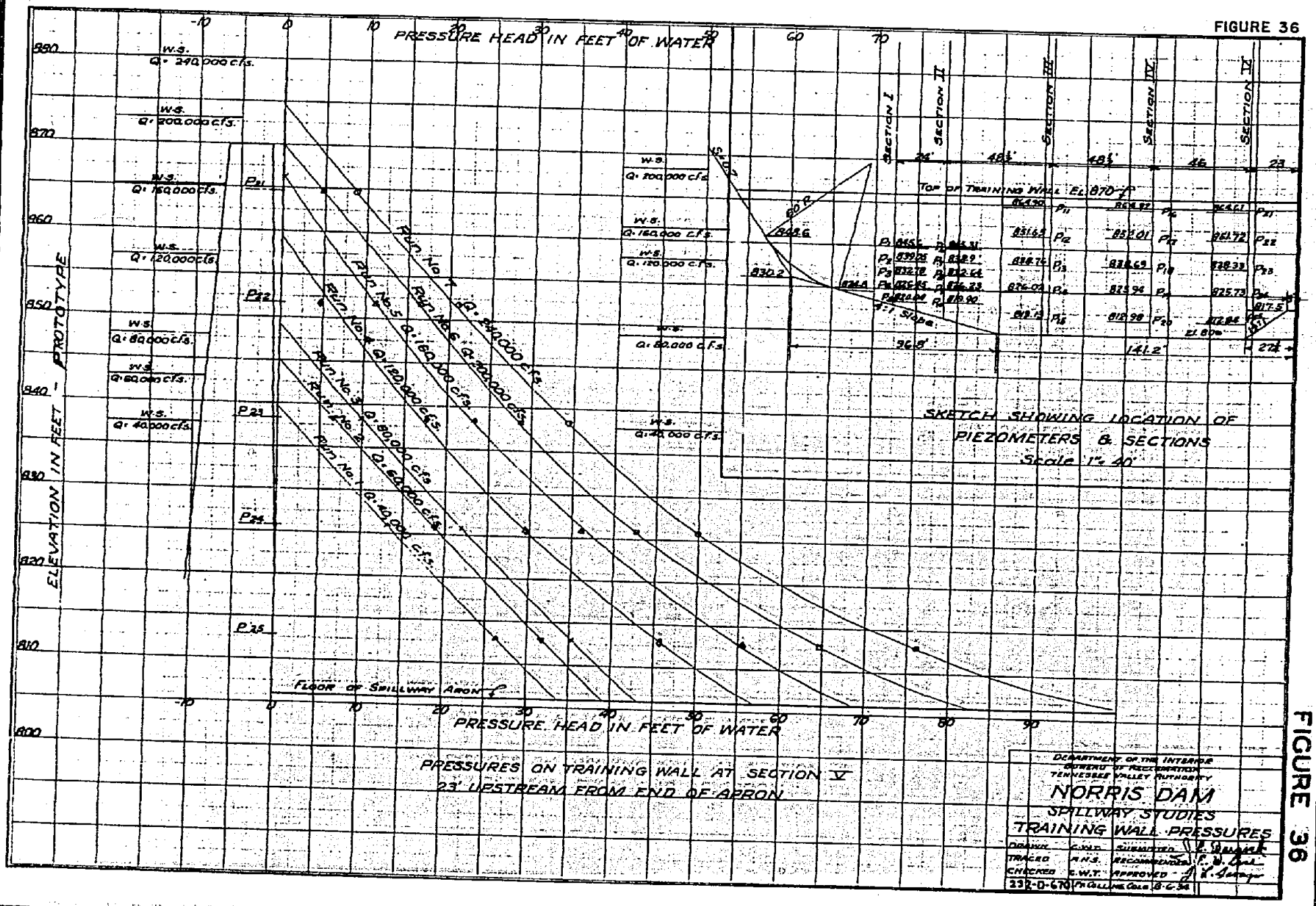


FIGURE 36

The total pressures in feet of water are plotted horizontally from the zero line at the right hand edge of the training wall. The curves are drawn in so that the pressure can be read at any elevation on the training wall.

At high discharges, the section of wall from the front face of the powerhouse extending downstream is, in effect, an overflow dam. For all discharges above 185,000 second-feet water flows over the top of the wall from the powerhouse tail-race into the stilling pool and at 240,000 second-feet reaches a depth of ten feet. This action is shown to be a maximum at Section IV, Figure 35, and there is a slight negative pressure near the top of the wall. The water surface in the stilling pool is high enough at Section V to make the effect negligible. There is, however, a flow over the wall and into the stilling pool from the front edge of the powerhouse, to the end of the apron.

The results of these tests were sent to the design department immediately upon completion of the tests for their guidance in the design of the wall.

5. Velocity Studies

Measurements of the velocities on the apron of the model were made for comparison with the computed velocities on

the prototype to determine the magnitude of the error which results from the fact that the model might not have relatively the same coefficient of roughness as the prototype. The results of the comparison of the velocities measured on the model with those computed from the prototype showed only a small difference between observed and computed values. For discharges of 40,000, 120,000 and 200,000 second-feet the observed losses to a point 12 feet downstream from the beginning of the horizontal apron were 102, 41.4 and 20.6 feet respectively as compared with 93, 38.5 and 22.5 feet respectively for the computed values.

The difference in the observed and computed loss was less than might be expected. In the case of the 200,000 second-feet discharge the observed loss was less than that computed although it might have been expected to be greater. Since neither the laws of flow over the model or prototype are definitely known it is probably fruitless at this time to inquire into the cause of the discrepancy, especiall since the variation of velocity due to friction is negligible. The results show, however, that the difference between model and prototype velocities is not sufficient to cause doubt as to the substantial accuracy of the model tests.

Table 6 gives a summation of results of the velocity studies. Column 1 gives the prototype quantity in second-feet.

TABLE NO. 6

NORRIS DAM SPILLWAY

COMPARISON OF RESULTS.

Velocities Measured on Model and Converted to Prototype and

Velocities Computed from Prototype.

Quantity in c.f.s	Theoretical		Friction Loss in Velocity Head Feet of Water		Net				Percent Difference Calculated and Measured Velocities
					Velocity		Velocity Head		
	Velocity	Velocity Head	Computed	Measured	Computed	Measured	Computed	Measured	
1	2	3	4	5	6	7	8	9	10
40,000	122.6	233.9	92.9	102.2	95.2	91.6	141.0	130.6	3.74
120,000	124.8	242.0	38.5	41.4	114.4	112.6	203.5	197.1	1.61
200,000	124.4	241.0	22.5	20.6	118.4	117.7	218.5	215.5	0.56

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Column 2, the theoretical or spouting velocity in feet per second; Column 3, the velocity head in feet of water corresponding to the velocity in Column 2; Column 4, the friction loss in head (feet of water) as computed from the prototype using Kutter's formula with n = 0.013; Column 5, the measured friction loss in head (feet of water) from the model and converted to prototype. Column 6 gives the net velocity in feet per second or difference between spouting velocity and loss of velocity due to friction as computed from the prototype; Column 7, the net velocity as measured by the pitot tube; Column 8, the difference between Columns 3 and 4 in feet of water; Column 9, the measured velocity head in feet of water corresponding to the velocity given in Column 7. These values are not the difference between values given in Columns 3 and 5 because the actual water surface as measured on the model was used, instead of the calculated water surface, in computing the theoretical velocity head. This difference is less than 0.5 percent of the values given so does not affect the results materially.

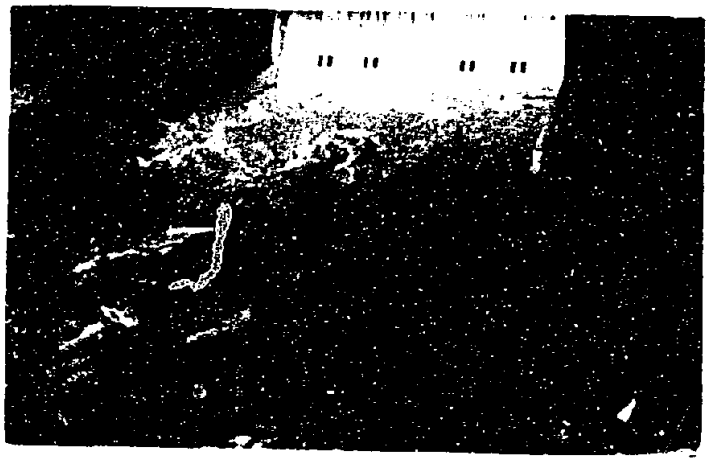
6. Gate Operating Program

The results of the testing to determine a gate operating program show that when the gates are at widely different elevations the conditions in the stilling pool are not satis-

factory. The jets from the gates at widely different elevations combine on the rollway and set up a disturbance before the water enters the pool. For instance, with the center gate up and the two outside gates down, the jets from the outside gates spread in under the jet from the center gate because it strikes farther down on the rollway. This disturbance is carried down through the pool.

In testing for an operating program, the possibility of some irregularity that might cause one or more of the gates to become inactive was considered. For instance on Plate 42 A and B is shown the action with Gates 1 and 2 at Elev. 1034 and Gate 3 at 1020 (Reservoir Elev. 1052) which would simulate either Gate 3 dropping from some cause, or Gates 1 and 2 becoming stuck in an elevated position. The results show bad conditions in the pool but not serious enough to cause alarm over the safety of the structure. Plate 42 C and D show results with Gates 1 and 3 at Elev. 1034 and Gate 2 at Elev. 1030. Plates 42 to 47, inclusive, show the results with various combinations of gate openings when the reservoir is at Elev. 1034 and also at Elev. 1052.

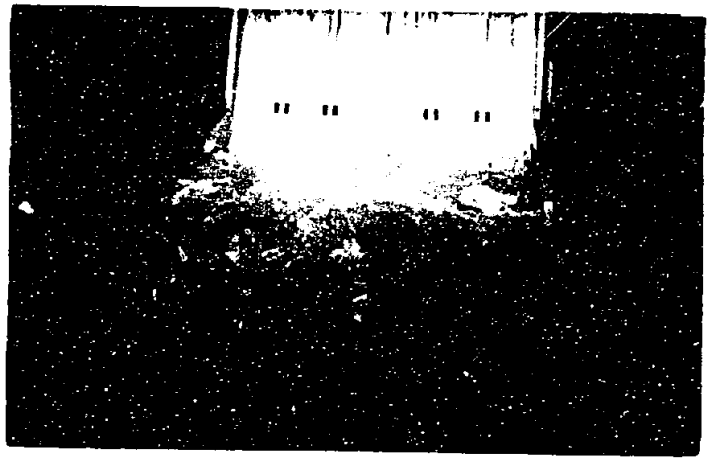
The most undesirable condition resulted when the tailwater was at a minimum, the reservoir at Elev. 1034, all gates at Elev. 1034 and one of them was suddenly dropped to Elev. 1020.



A. RESERVOIR ELEVATION 1052.

B. RESERVOIR ELEVATION 1052.

GATES 1 AND 2 ELEV. 1034 - GATE NO. 3 ELEV. 1020.

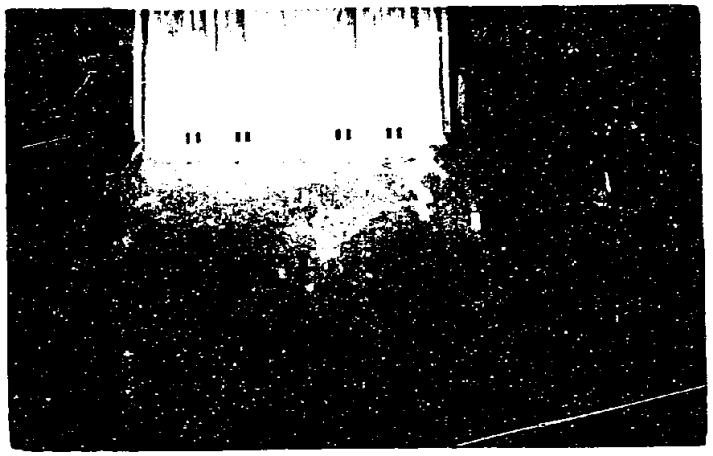


C. RESERVOIR ELEVATION 1052.

D. RESERVOIR ELEVATION 1052.

GATES 1 AND 3 ELEV. 1034 - GATE NO. 2 ELEV. 1020.

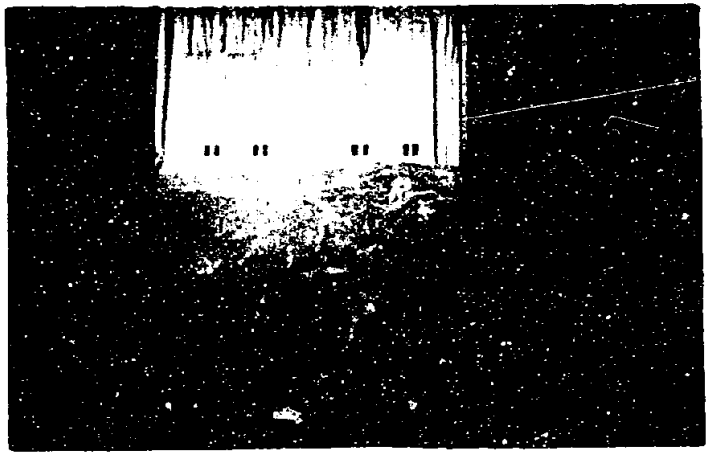
COMBINATIONS OF GATE OPENINGS.



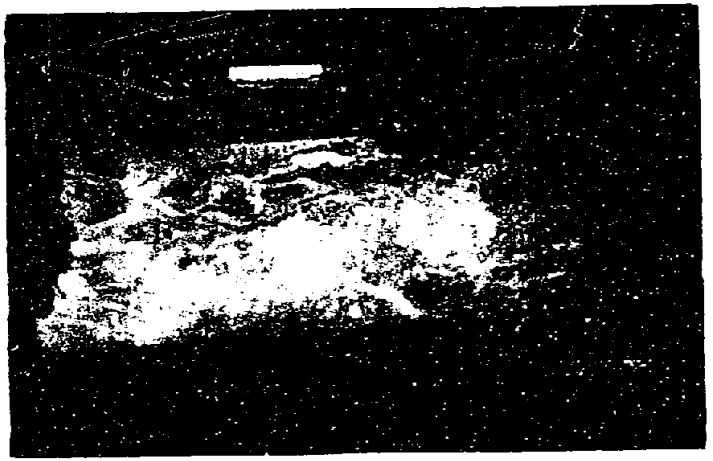
A. RESERVOIR ELEVATION 1052.

B. RESERVOIR ELEVATION 1052.

GATES 2 AND 3 ELEV. 1034 - GATE NO. 1 ELEV. 1020.



C. RESERVOIR ELEVATION 1052.

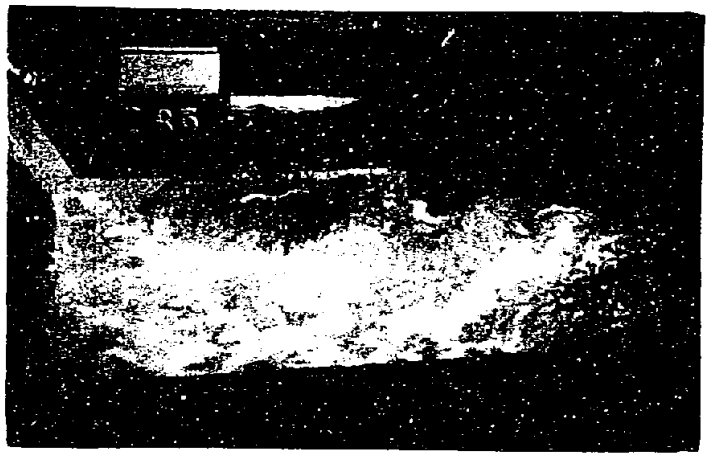


D. RESERVOIR ELEVATION 1052.

GATE NO. 3 ELEV. 1034 - GATES 1 AND 2 ELEV. 1020.

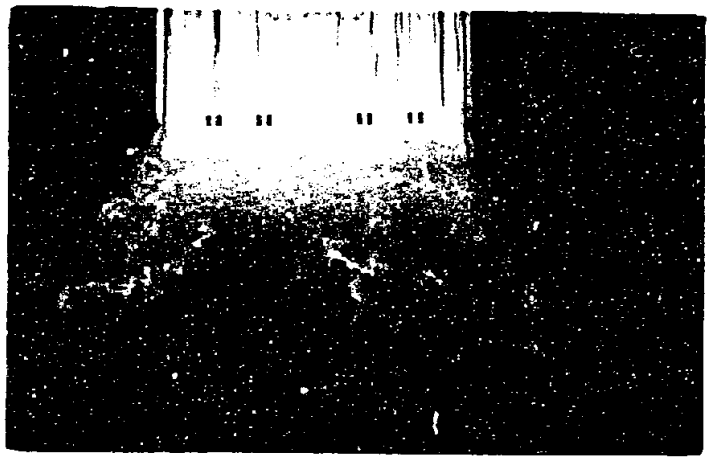
COMBINATIONS OF GATE OPENINGS.

A. RESERVOIR ELEVATION 1052.

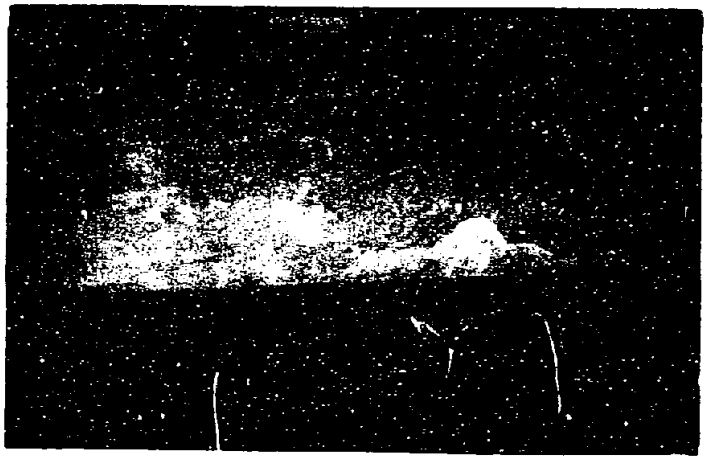


B. RESERVOIR ELEVATION 1052.

GATE NO. 1 ELEV. 1034 - GATES 2 AND 3 ELEV. 1020.



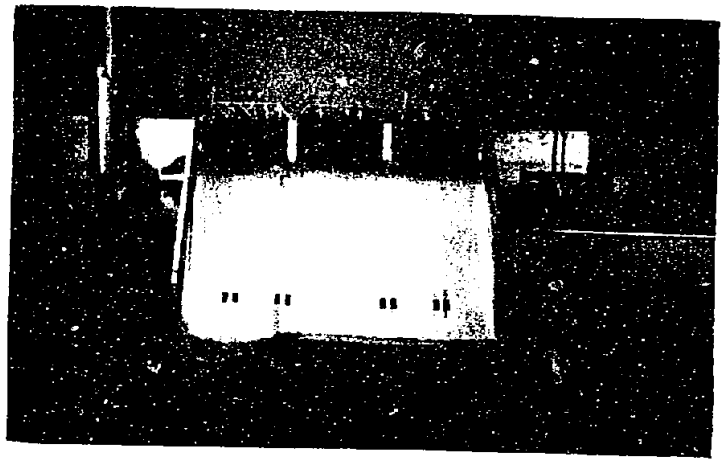
C. RESERVOIR ELEVATION 1052.



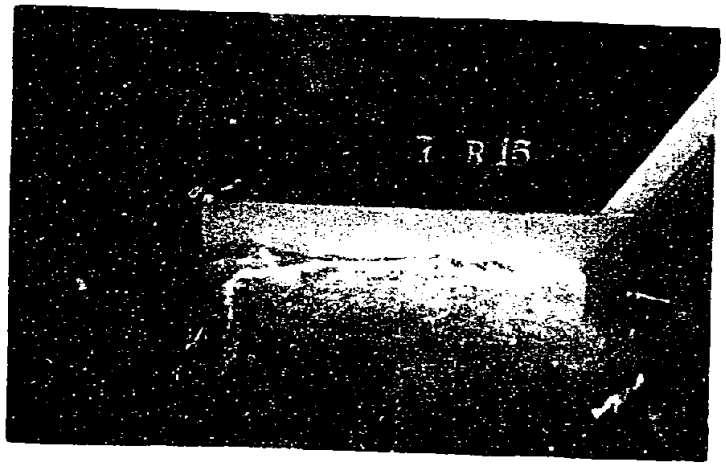
D. RESERVOIR ELEVATION 1052.

GATE NO. 2 ELEV. 1034 - GATES 1 AND 3 ELEV. 1020.

COMBINATIONS OF GATE OPENINGS.

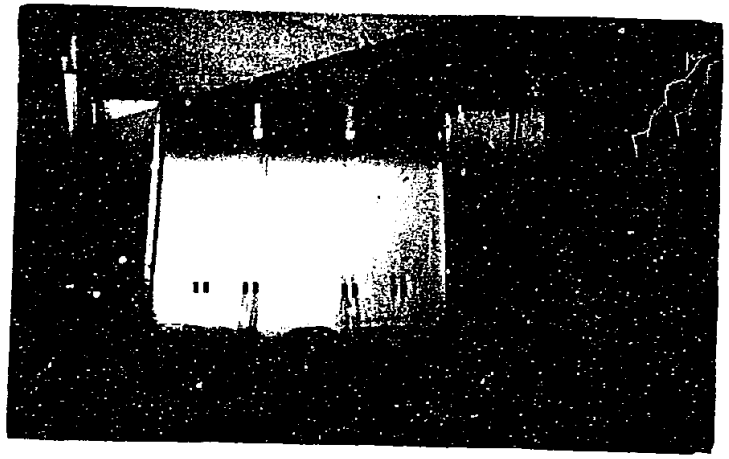


A. RESERVOIR ELEVATION 1034.

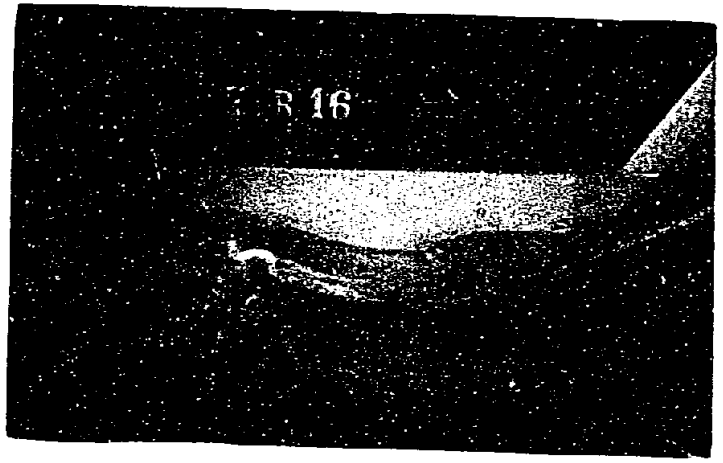


B. RESERVOIR ELEVATION 1034.

GATES 1 AND 2 ELEV. 1034 - GATE NO. 3 ELEV. 1020.



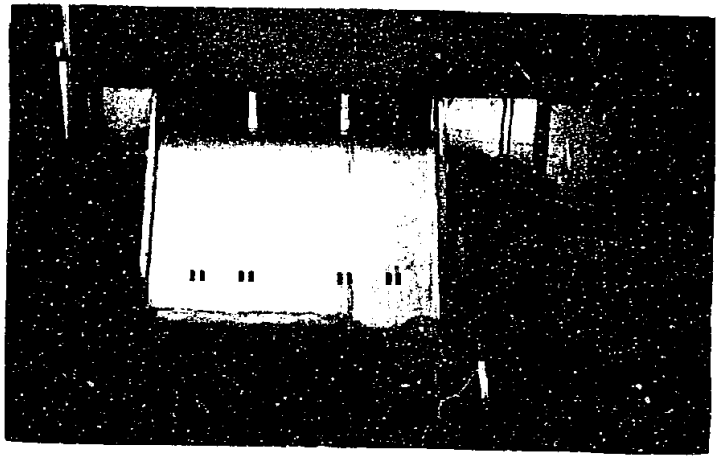
C. RESERVOIR ELEVATION 1034.



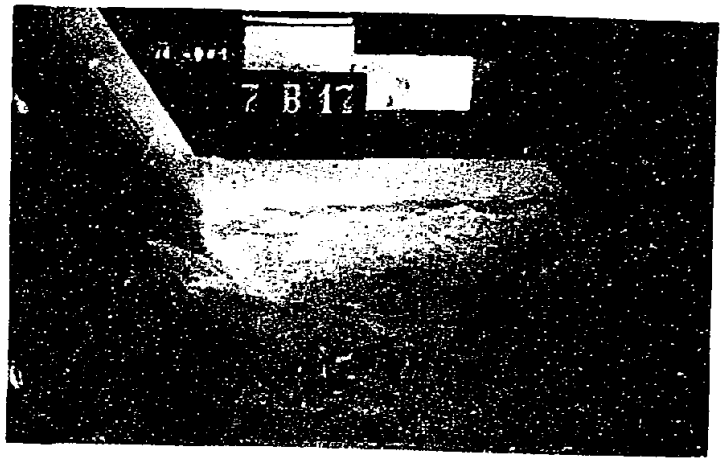
D. RESERVOIR ELEVATION 1034.

GATES 1 AND 3 ELEV. 1034 - GATE NO. 2 ELEV. 1020.

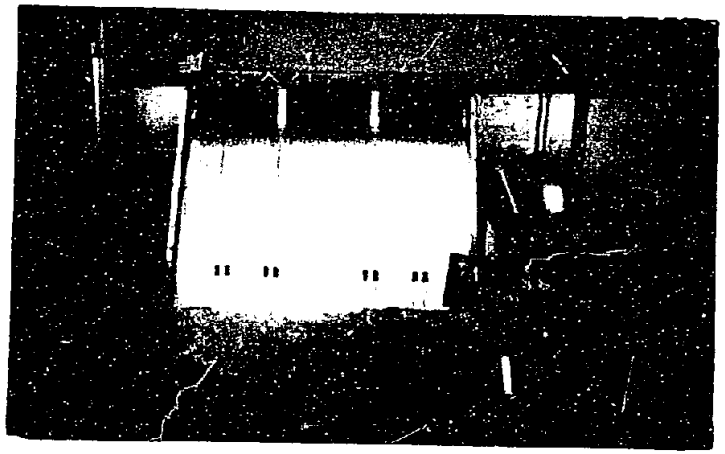
COMBINATIONS OF GATE OPENINGS.



A. RESERVOIR ELEVATION 1034.
GATES 2 AND 3 ELEV. 1034 - GATE NO. 1 ELEV. 1020.



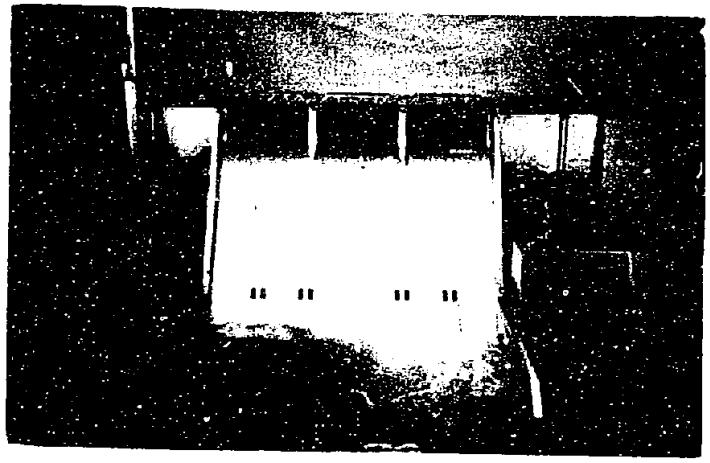
B. RESERVOIR ELEVATION 1034.



C. RESERVOIR ELEVATION 1034.
GATE NO. 1 ELEV. 1034 - GATES 2 AND 3 ELEV. 1020.

D. RESERVOIR ELEVATION 1034.

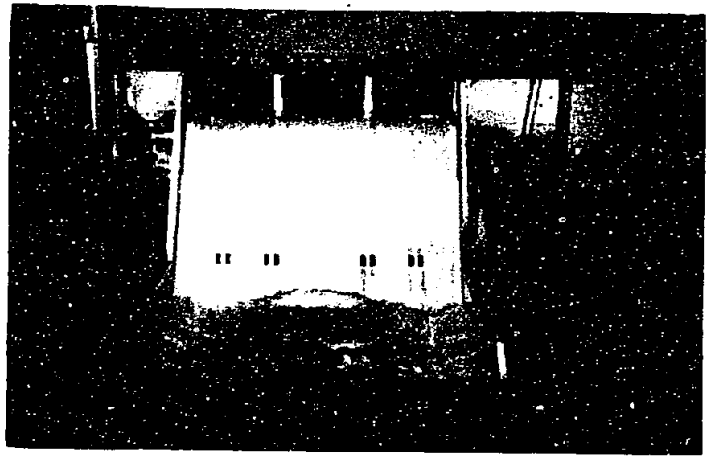
COMBINATIONS OF GATE OPENINGS.



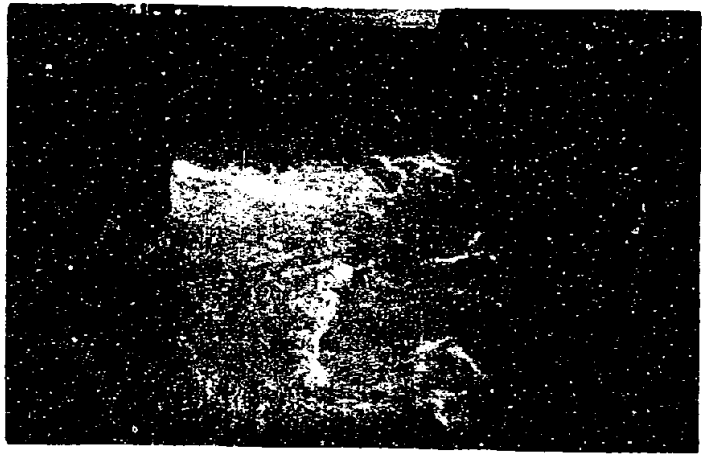
A. RESERVOIR ELEVATION 1034.

B. RESERVOIR ELEVATION 1034.

GATE NO. 3 ELEV. 1034 - GATES 1 AND 2 ELEV. 1020.



C. RESERVOIR ELEVATION 1034.



D. RESERVOIR ELEVATION 1034.

GATE NO. 2 ELEV. 1034 - GATES 1 AND 3 ELEV. 1020.

COMBINATIONS OF GATE OPENINGS.

This allowed approximately 18,300 second-feet of water to rush into the pool. Although the tailwater was swept off the sloping apron and onto the flat part of the apron, whirls soon developed which prevented the tailwater from being swept past the sill. The tailwater very quickly built up sufficiently to control the jump.

As can be seen from Plates 42 to 47, inclusive, the more undesirable conditions were caused when the gate farthest from the powerhouse was completely down and the center gate and gate nearest the powerhouse were completely raised. This condition results because there is more area in the whirl when it turns toward the powerhouse rather than away from it. The larger whirl is of more severe proportions. When the gate nearest the powerhouse is completely down and the other two completely raised the whirl beyond the end of the apron can turn both into the tailrace and toward the right bank and is somewhat dissipated in this dividing process. This causes less undesirable conditions.

c. Auxiliary Passages

1. Sluices

With the sluices level through the dam and center line at Elev. 865 the jets from the sluice openings fell into

the pool near the toe of the slope and caused serious disturbances. The jets from the pairs of sluices combined, sweeping back the tailwater and forming whirls. The impact from the jets falling into comparatively shallow tailwater and striking the apron would have caused very serious erosion at those points. The floors of the sluices at the openings were also subjected to very severe impact and consequently a high probability of being eroded by the action of the water flowing over the spillway. This condition was partially remedied by placing deflectors above the openings of the sluices on the downstream face of the dam.

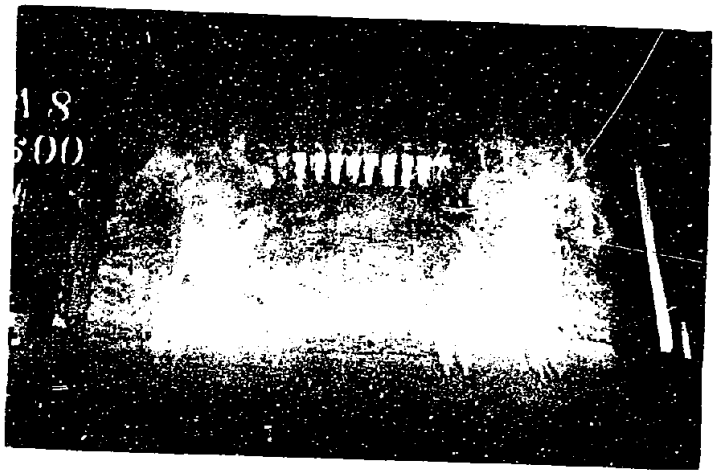
The openings set up disturbances in the flow conditions when the spillway was operating alone. This action can be seen on Plate 11 C. The set-up was generally very unsatisfactory even with the deflectors above the openings.

By changing the inverts of the sluices to follow a parabola from the valves to the face of the dam, the exits were lowered thus allowing the jets to slide into the pool rather than fall onto the surface of the water. Conditions were generally improved but the velocities crossing the sill were still very unevenly distributed as can be seen from Plate 48 A. A good jump was formed where the jets entered the pool,

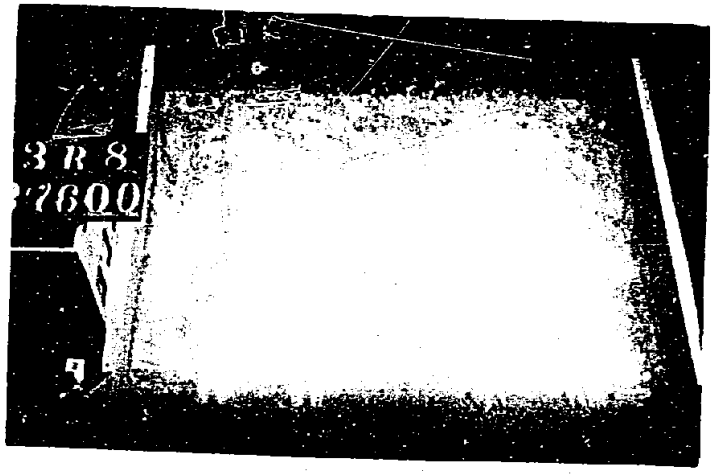
A. NO AUXILIARY SILL.

B. AUXILIARY DIFFUSER SILL, FRONT EDGE 2¾' DOWNSTREAM FROM END OF 4:1 SLOPE.

1¾:1 SLOPING SILL TO ELEV. 819 AT END OF APRON.



C. NO SILL AT END OF APRON. DIFFUSER SILL, FRONT EDGE 30' FROM END OF 4:1 SLOPE.



D. CURVED FACE SILL, TOP ELEV. 819, AT END OF APRON.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806. SLUICES PARALLEL THROUGH DAM.
SLUICES ONLY DISCHARGING 37,600 SECOND-FEET.

but unstable flow conditions were caused by the water entering the jump from the sides.

The flow conditions with the spillway operating alone was improved and the possibility of erosion on the floors of the sluices at the openings was decreased materially. The impact on the bottom of the sluices was further reduced by placing deflectors on the face of the dam above the sluice openings. These deflectors, when made the same width as the sluices, increased rather than decreased the impact on the sluice floors. By testing several widths of these deflectors, it was found that a deflector extending over two sluices and half the width of one sluice beyond the sluice opening on each end, the impact could be reduced (Figure 10, Half Plan and Plate 25). The results of the tests on this type of deflector showed very little possibility of erosion on the floor of the sluices at the openings, and very little disturbance in the flow conditions with the spillway operating alone. It was noted, however, that a pressure less than atmospheric was set up in the sluice passage at some flows. This was measured as will be later explained.

The chief objection to this set-up was that the jets from the sluiceways combined and resulted in two main jets

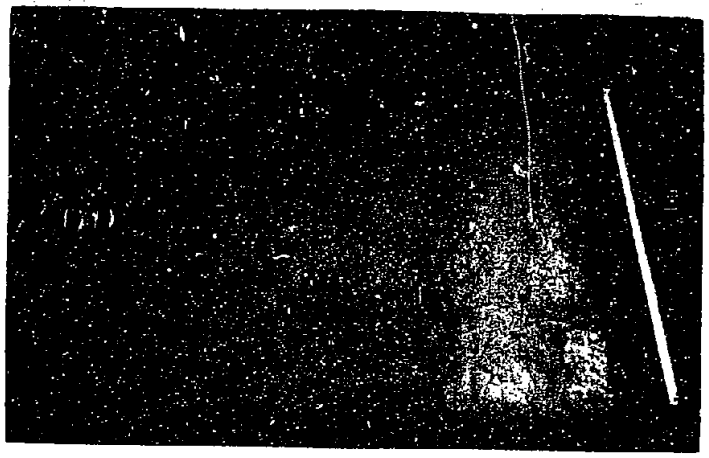
crossing the sill (Plate 48 A). These forward velocity jets were separated, as can be seen from the plate, by an area of back flow. There were also back flow areas next to each training wall.

By placing the low diffuser sill, shown on Figure 7, Test 18, at the toe of the 4:1 slope it was anticipated that the velocities could be more evenly distributed across the width of the pool. The results are shown on Plate 48 B. As can be seen, the two high velocity jets still persisted. Very little added improvement was accomplished in flow conditions with the spillway operating alone.

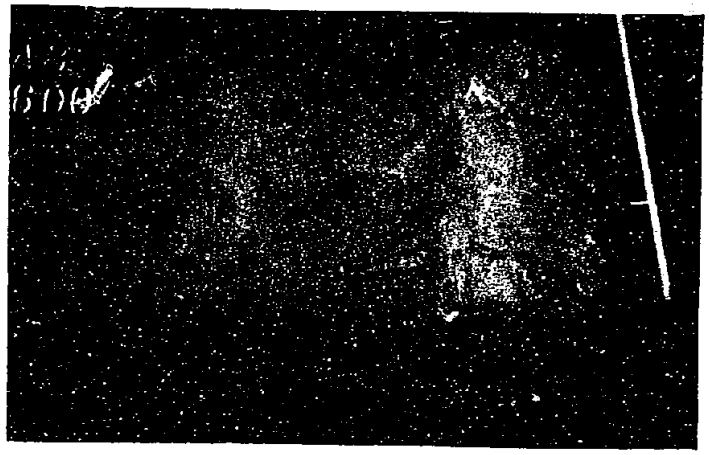
This sill was next tested at a position $32\frac{1}{4}$ feet downstream from the toe of the 4:1 slope, without the main sill at the end of the apron. The conditions with the sill at this position were worse as can be seen on Plate 48 C. Considering that such a structure would be exceedingly expensive, it was abandoned.

The results with the sluices discharging into the pool with the curved upstream face sill at the end of apron are shown on Plate 48 D. They were substantially the same as those with the $1\frac{1}{2}$:1 slope sill as shown on Plate 48 A.

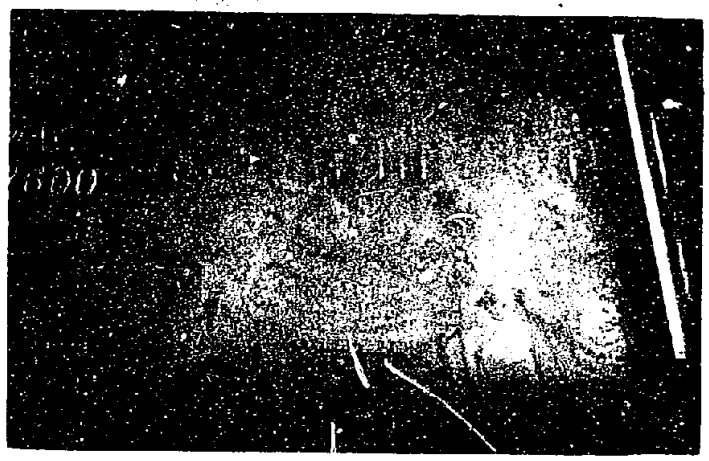
Plate 49 A shows the results with the sluices operating and the high diffuser sill with overhanging nose placed at the end of the apron. Very little improvement in conditions



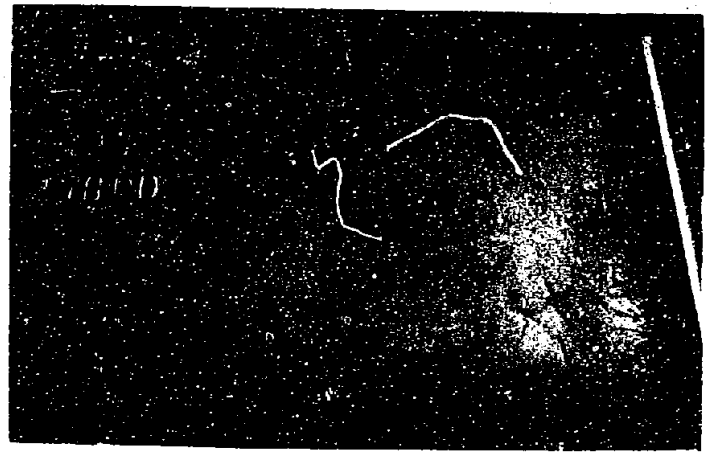
A. AT END OF APRON.



B. 30' UPSTREAM FROM END OF APRON.



C. 60' UPSTREAM FROM END OF APRON.



D. 90' UPSTREAM FROM END OF APRON.

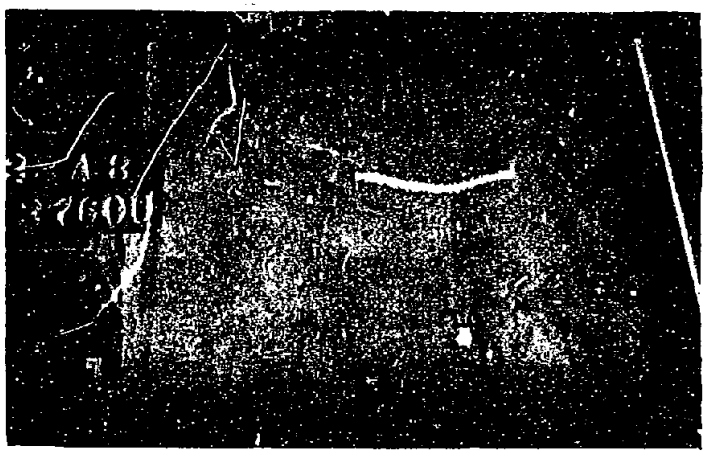
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - SLUICES PARALLEL THROUGH DAM.
LARGE DIFFUSER SILL WITH OVERHANG. SLUICES ONLY DISCHARGING 50,000 CUBIC-FT.

PLATE 49

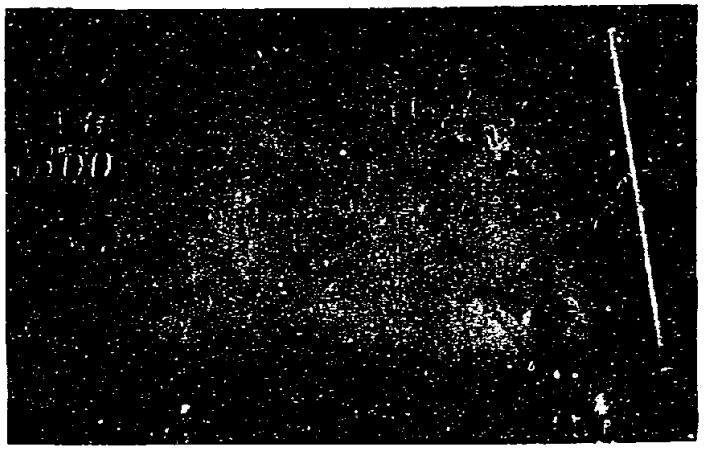
was effected with the sill in this position. Plate 49 B shows the results with the same sill moved 30 feet upstream. Conditions were slightly improved at this position. Plate 49 C shows the results with the same sill moved 60 feet upstream. Flow conditions were further improved with this set-up. The best results obtained by the use of this sill are shown on plate 49 D when the sill was moved 90 feet upstream from the end of apron. Although some whirls existed on the apron upstream from the sill they were not serious and after the water passed the sill the velocities were well broken up and very evenly distributed. This sill would have offered a solution to the problem, but it was hoped that an even better condition might be obtained by simpler means.

Plate 50 shows the results when the above sill with the over-hang removed was used at the end of the apron and 30, 60 and 90 feet upstream from the end of the apron. These results showed no noticeable change from the previous results.

Plate 51 A shows the openings of the sluices with no discharge after one sluice of each pair had been diverged from the other. The results of this change were very satisfactory. The jets from the sluices were very evenly distributed across the apron as shown on Plate 51 B, with the sluices discharging

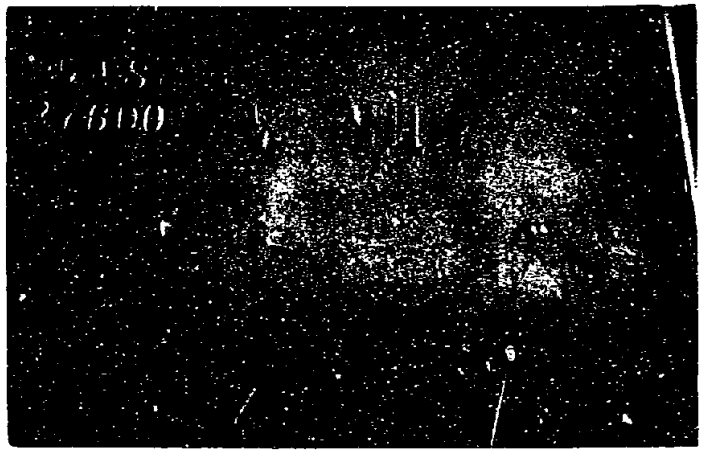


A. AT END OF APRON.



B. 30' UPSTREAM FROM END OF APRON.

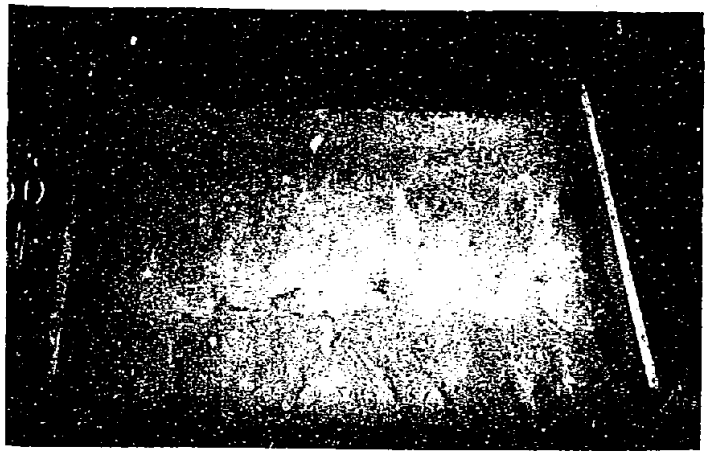
C. 60' UPSTREAM FROM END OF APRON.



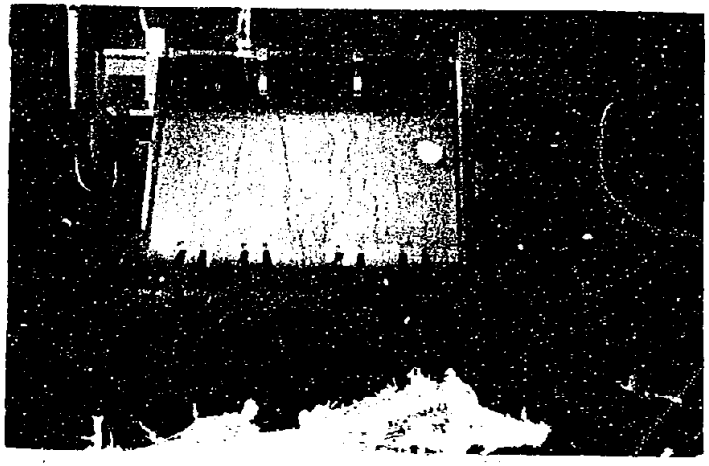
D. 90' UPSTREAM FROM END OF APRON.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - SLUICES PARALLEL THROUGH DAM.
LARGE DIFFUSER SILL, OVERHANG REMOVED. SLUICES ONLY DISCHARGING 37,600 SEC-FT.

A. NO FLOW.



B. DISCHARGE 37,600 SEC-FT.
TAILWATER LOWERED TO SHOW SPREADING OF JETS.



D. DISCHARGE 37,600 SEC-FT. NORMAL TAILWATER.

4:1 SLOPING APRON TO
1½:1 SLOPE SILL TO

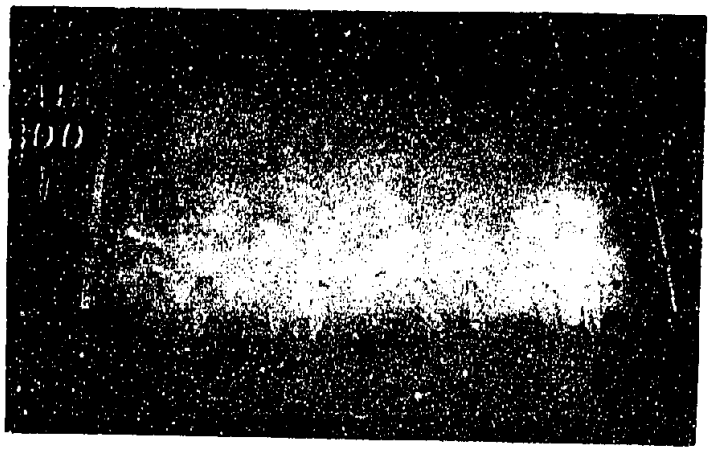
DIVERGING SLUICES.
N.

into the pool when the tailwater was lowered as much as possible. The flow conditions with normal tailwater are shown on Plate 51 C and D.

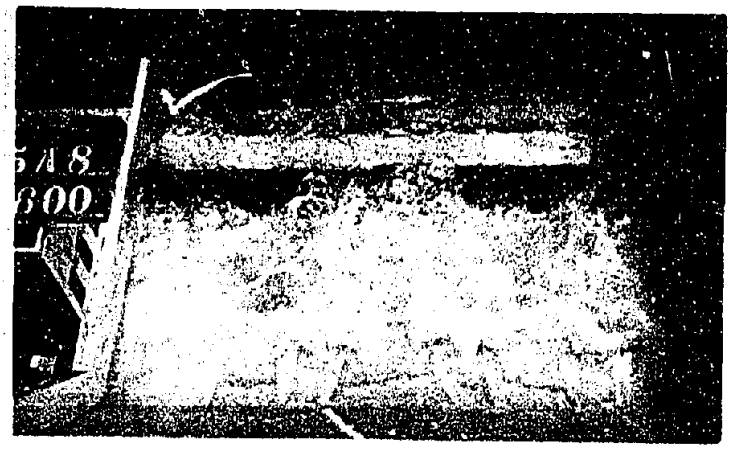
From the standpoint of sluices discharging alone, this set-up was very satisfactory, but when the spillway was operating with the sluices closed, the water flowing over the spillway struck the bottom of the sluices. In the diverging sluice, this water striking the bottom was turned off at an angle because the sluice opening was not parallel to the line of flow. The result was a fin omitting from the sluice openings causing disturbances in the stilling pool.

This condition was partly corrected by placing deflectors above the sluice openings (Plate 60 C). The action of the water flowing over the spillway with the sluices closed is shown on Plate 33 A. Some disturbance caused by the deflectors was still noticeable when water flowed over the spillway. A better solution to the problem was necessary.

Plate 52 A shows the flow conditions with the same set-up so far as the sluiceways were concerned, but with the Wheeler type of diffuser sill at the end of apron. The results are generally the same as in the above test.



A. AT END OF APRON.



B. 60' UPSTREAM FROM END OF APRON.

C. 90' UPSTREAM FROM END OF APRON.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
LOW DIFFUSER SILL(WHEELER TYPE). SLUICES ONLY DISCHARGING 37,600 SECOND-FEET.

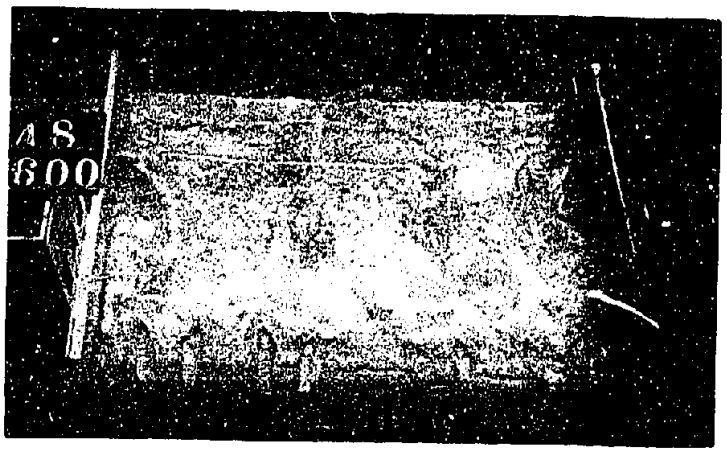
Plate 52 B shows the flow conditions with the same diffuser sill 60 feet upstream from the end of the apron. The results showed some improvement in that the velocities were more evenly distributed.

The flow conditions with the same diffuser sill 90 feet upstream from the end of the apron are shown on Plate 52 C. These results are very good with sluices discharging alone, but when they were closed and the spillway allowed to discharge the conditions were not sufficiently improved to warrant the expense of the sill.

Plate 53 A shows the flow conditions with the variable section sill at the end of apron. This sill had very little or no effect on the flow from the sluiceways.

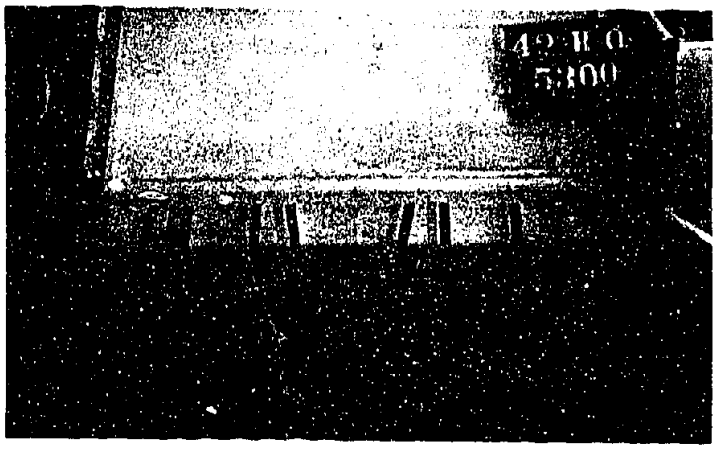
The various sloped upstream face sills tested at the end of the spillway apron had little or no effect on the discharge from the sluices when operating alone.

The parabolic hump built over the sluice openings to reduce the disturbance set up by them when the spillway was operating alone and to reduce the negative pressure in the sluice passageways showed very little effectiveness. Plate 54 A and B shows the appearance of the openings with no flow through sluices or over spillway. It can be seen that the length of the opening through the dam was materially increased by this change. This



A. DISCHARGE 37,600 SECOND-FEET.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
VARIABLE SECTION TRAPEZOIDAL SILL AT END OF APRON.



A. FRONT VIEW.

B. VIEW SHOWING PROFILE OF HUMP.

PARABOLIC HUMP OVER SLUICE OUTLETS.
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON.

caused the water flowing over the spillway to catch in the openings more than with previous set-ups. This action caused excessive disturbance in the pool and increased the possibility of erosion in the sluice openings (Plate 39 A&B). The negative pressure in the sluice passages was increased rather than decreased. The change showed worse rather than better conditions.

Previous mention has been made of pressures in the sluice passages. Measurements were made of these pressures to determine their magnitude and whether negative or positive. This information was desired by the design department to assist in the design of the control gates in the sluiceways.

The results of these pressure measurements made in connection with Tests 37, 38 and 42 are given in Table 7.

Column 1 gives the prototype quantity flowing over the spillway in second-feet. Columns 2, 6 and 10 show the pressures in feet of water as determined by the piezometer in the straight sluices and Columns 4, 8 and 12 show these values converted to prototype for the three set-ups. Columns 3, 7 and 11 show the pressures in feet of water as determined by the piezometer in the diverging sluices and Columns 5, 9 and 13

TABLE NO. 7

NORRIS DAM SPILLWAY

SUMMARY OF RESULTS.

Pressures in Sluice Passages

Prototype Quantity in c.f.s.	60' Radius Deflectors Each Deflector Covering 2 Sluices				No Deflectors over Sluice Openings				Parabolic Hump over Sluice Openings			
	Model Pressure From Water Column		Prototype Pressure Calculated		Model Pressure From Water Column		Prototype Pressure Calculated		Model Pressure From Water Column		Prototype Pressure Calculated	
	Straight Sluice	Diverging Sluice	Straight Sluice	Diverging Sluice	Straight Sluice	Diverging Sluice	Straight Sluice	Diverging Sluice	Straight Sluice	Diverging Sluice	Straight Sluice	Diverging Sluice
1	2	3	4	5	6	7	8	9	10	11	12	13
20,000	-0.0063	-0.0059	-0.454	-0.426	-0.0698	-0.0888	-5.026	-6.394	-0.0286	-0.0170	-2.059	-1.224
40,000	-0.0240	-0.0246	-1.728	-1.771	-0.0697	-0.1512	-5.018	-10.886	-0.0421	-0.0544	-3.031	-3.917
80,000	-0.1378	-0.1439	-9.922	-10.361	+0.0374	-0.1058	+2.693	- 7.618	-0.0568	-0.2656	-4.090	-19.123
120,000	-0.1101	-0.1169	-7.927	- 8.417	+0.1134	-0.0481	+8.165	- 3.463	-0.1031	-0.2826	-7.423	-20.347
160,000	-0.0681	-0.0664	-4.903	- 4.781	+0.1869	+0.0739	+13.457	+ 5.321	-0.0621	-0.2441	-4.471	-17.575
200,000	+0.0119	-0.0024	+0.857	- 0.173	+0.3099	+0.2169	+22.313	+15.617	+0.0549	-0.0796	+3.953	- 5.731

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show these values converted to prototype.

These results show a comparatively wide range of pressures in the sluice passages as the quantity over the spillway is increased from very low flows to maximum discharge. The negative pressure might be useful in increasing the discharge through the sluiceways when they were operating in conjunction with the spillway, but at the same time if the sluice control gates were closed and water was flowing over the spillway, these gates would have to be stronger to withstand the pressure. As the pressure varies for different flows the effect could not be relied upon to assist in the flow through the passages.

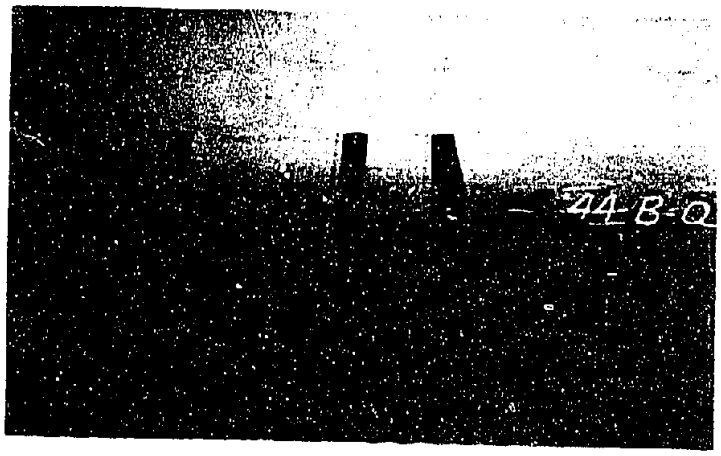
From a review of the results obtained from the sluice tests and a study of the results of the pressure tests, it was decided to change the operating program for the model. This change was to allow the sluices to discharge their maximum capacity as the flood approached and remain open after the spillway came into action. At the maximum discharge of 240,000 second-feet, the sluiceways would be discharging 40,000 second-feet and the spillway 200,000 second-feet. This procedure would greatly reduce the probability of erosion on the floors of the sluiceways at the openings and would eliminate the necessity of

deflectors of any type over the sluice openings.

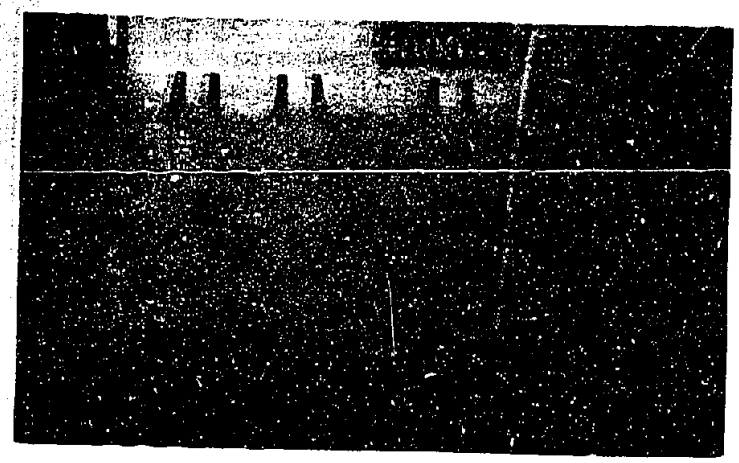
The results of the tests showed this procedure of operation very desirable, except for some disturbances set up at the openings of the diverging sluices. To eliminate this the openings of the diverging sluices were diverged to spread the jets from the sluices more and cause less disturbance in the flow conditions of the spillway water. The appearance of the opengings after changing are shown on Plate 55 A and the flow conditions with the sluices operating alone on Plate 55 B. The flow conditions past the openings of the sluices were greatly improved and the fin caused by the spillway water striking the water from the diverging sluice was almost completely removed.

To improve the appearance of the sluice openings, the openings of the straight sluices were diverged to the same bottom widths as the diverging sluices. The openings then appeared as on Plate 56 A. The flow conditions were not changed perceptibly as can be seen from Plate 56 D. This design has been accepted as final.

Because half of the sluices come through the dam perpendicular to the axis and the other four are at an angle to the axis, different conditions existed in the pool when different sluiceways were operated singly or in combination with others. This necessitated the determination of an operating program for

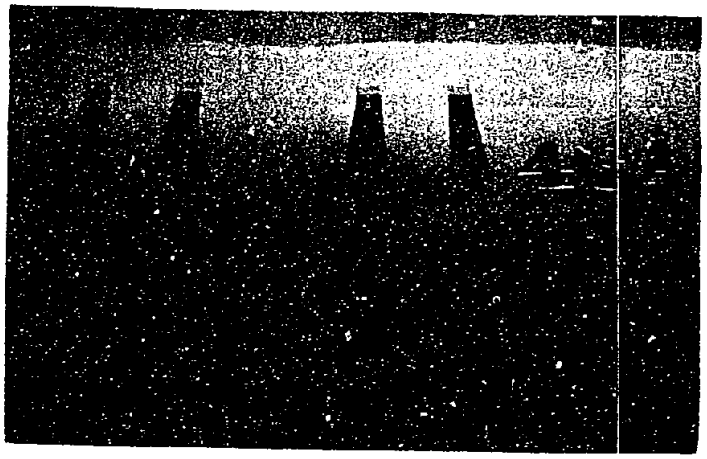


A. NO FLOW.

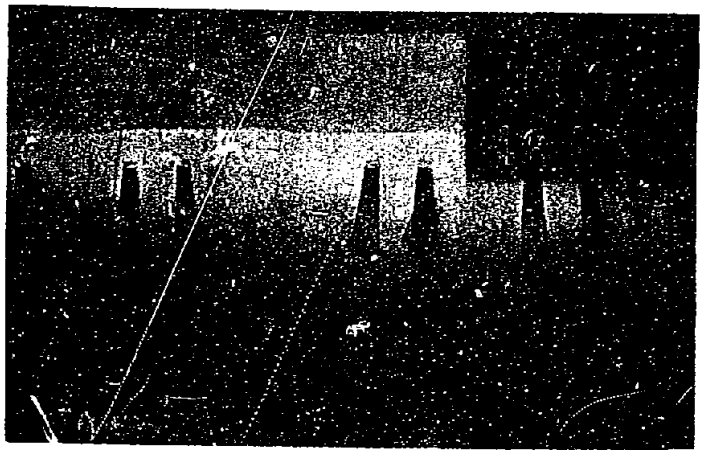


B. DISCHARGE 37,600 SECOND-FEET.

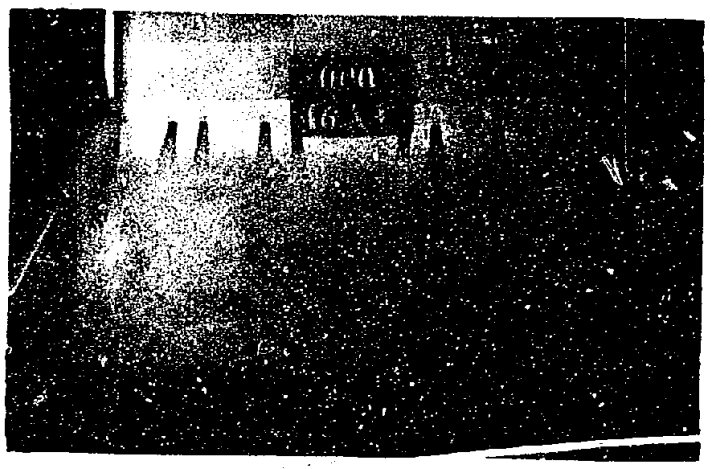
DIVERGING SLUICES WITH OUTLETS DIVERGED.
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON.



A. NO FLOW.



B. SLUICE NO. 3 DISCHARGING.



C. SLUICES 1, 3, 6 AND 8 DISCHARGING.

D. ALL SLUICES DISCHARGING.

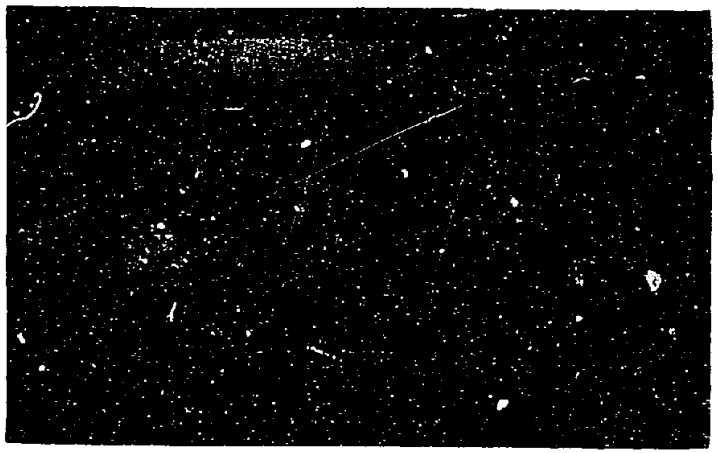
OPENINGS OF BOTH STRAIGHT AND DIVERGING SLUICES DIVERGED.
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
1¼:1 SLOPE SILL TO ELEV.817.5 AT END OF APRON.

the sluiceways. From a series of tests in which the sluices were operated singly, in combinations of twos, threes, fours, etc., the results showed that the following combinations gave the best flow conditions. The sluiceways are numbered from left to right, looking downstream. Open No. 3 Sluice, if only one sluice is needed; 3 and 6, if two sluices are needed; 3, 6 and 8, if three sluices are needed; 1, 3, 6 and 8, if four are needed; 1, 3, 4, 6 and 8, if five are needed; 1, 3, 4, 5, 6 and 8, if six are needed; 1, 2, 3, 4, 5, 6 and 8, if seven are needed and finally open all eight sluices. The flow conditions for this program are shown on Plates 57 and 58.

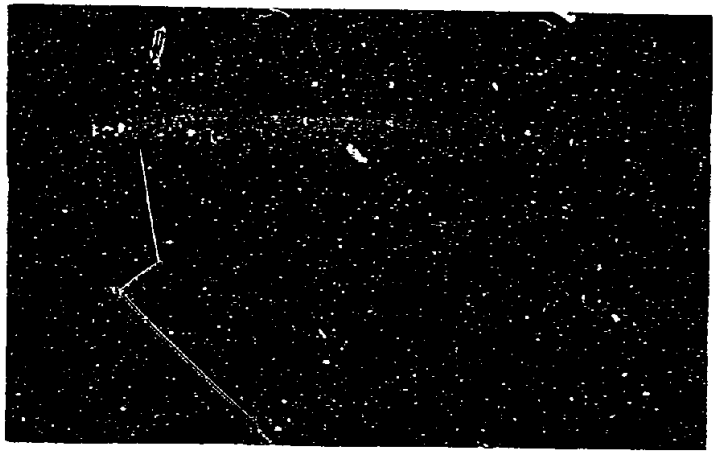
Plates 59, 60 and 61 show the appearance of the sluice openings as they were changed.

2. Needle Valves

The flow conditions with water discharging from the two 72-inch needle valves as originally designed are shown on Plate 62 A and B. These conditions, as have been mentioned before, were very unsatisfactory, because the jets struck the sloping portion of the tailrace and were deflected upward. This condition would not allow a jump to form, and the high velocities continued downstream, struck the bank of the tailrace excavation and turned back toward the powerhouse forming an eddy, which carried silt and gravel and deposited it in the tailrace in front of the draft tubes. Excessive scour would

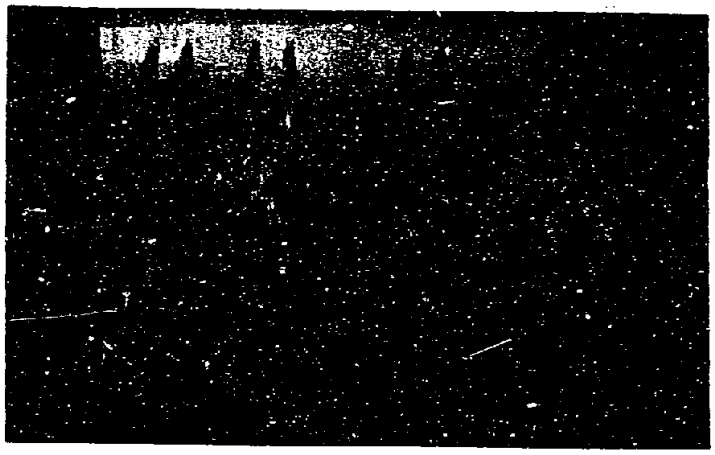


A. SLUICE NO. 3 DISCHARGING.



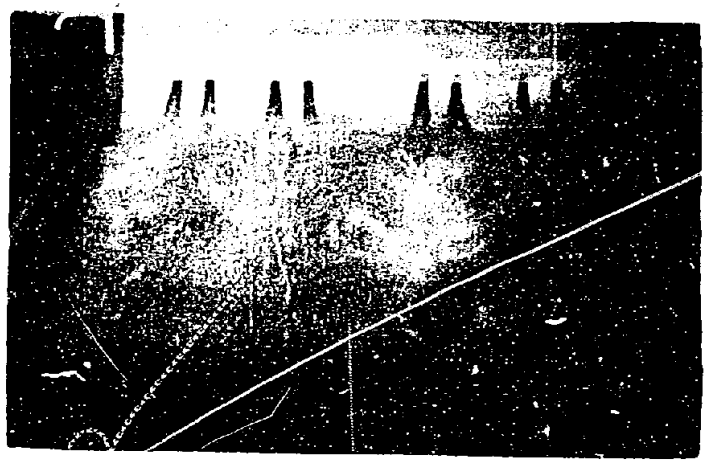
B. SLUICES 3 AND 6 DISCHARGING.

C. SLUICES 3, 6 AND 8 DISCHARGING.

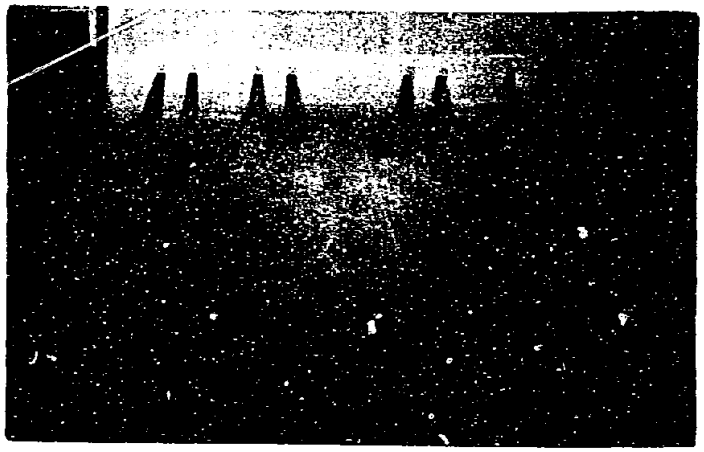


D. SLUICES 1, 3, 6 AND 8 DISCHARGING.

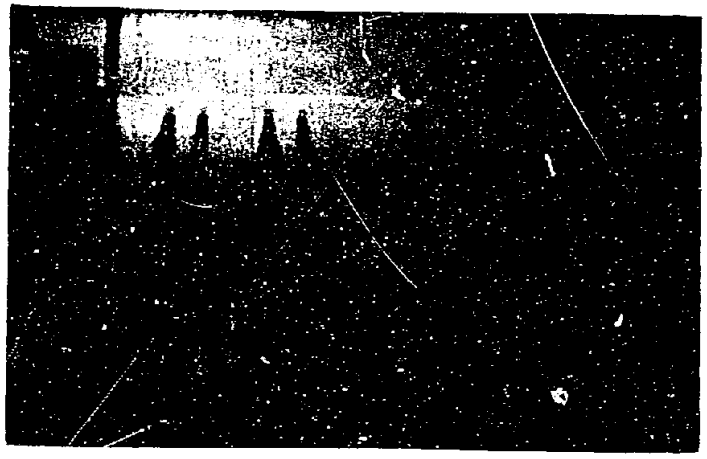
4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON. SLUICE OPERATING PROGRAM.



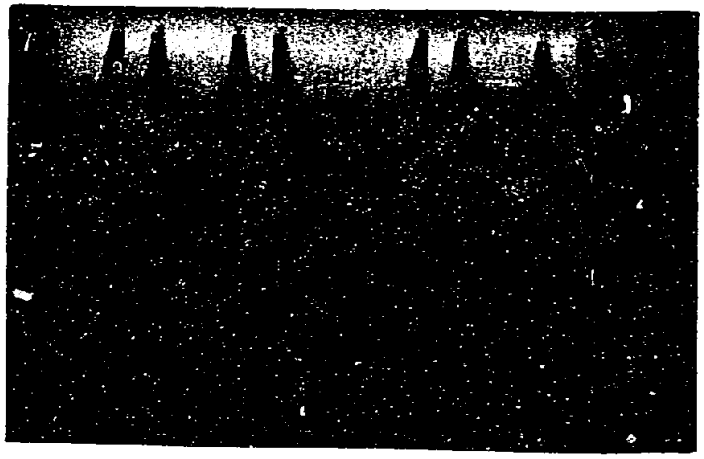
A. SLUICES 1, 3, 4, 6 AND 8 DISCHARGING.



B. SLUICES 1,3,4,5,6 AND 8 DISCHARGING.



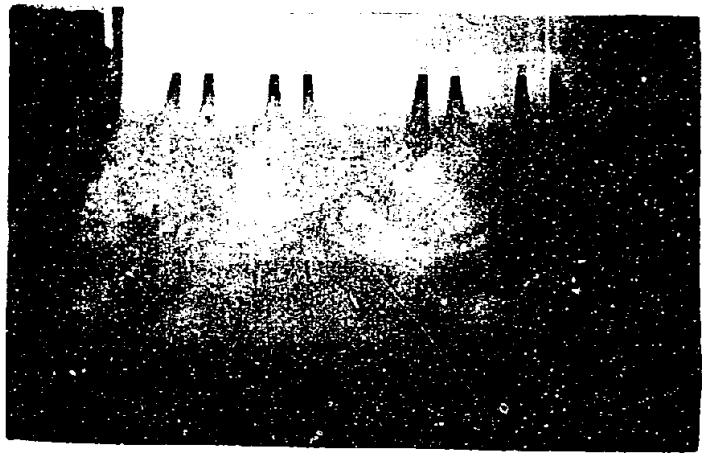
C. SLUICES 1,2,3,4,5,6 AND 8 DISCHARGING.



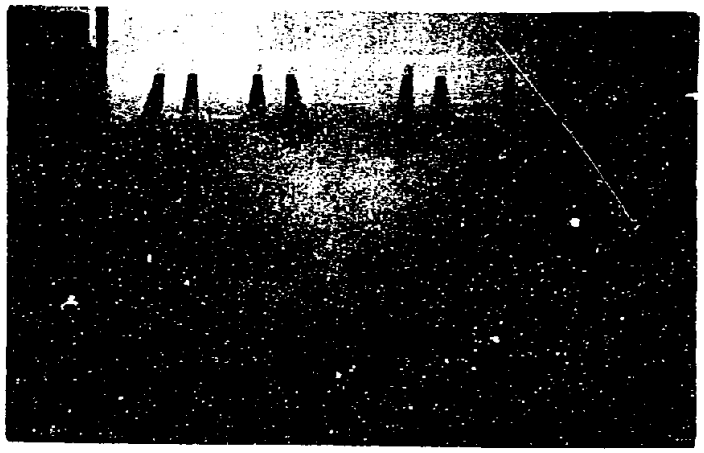
D. ALL SLUICES DISCHARGING.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON. SLUICE OPERATING PROGRAM.

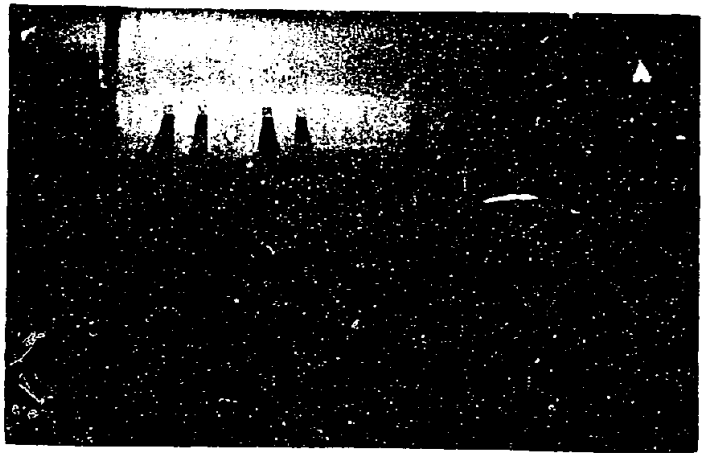
PLATE 68



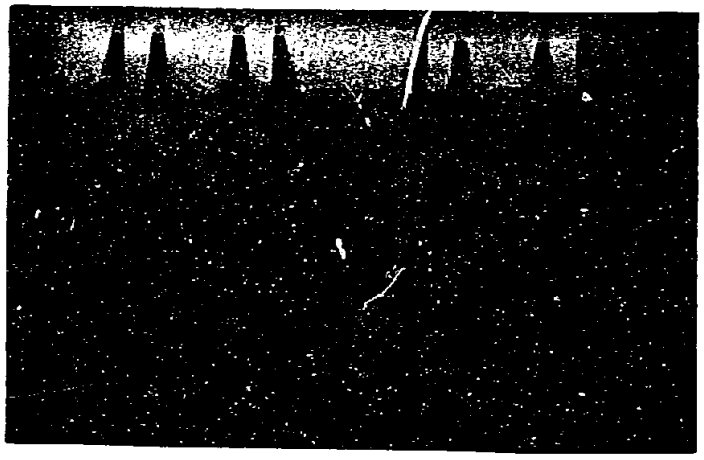
A. SLUICES 1, 3, 4, 6 AND 8 DISCHARGING.



B. SLUICES 1,3,4,5,6 AND 8 DISCHARGING.



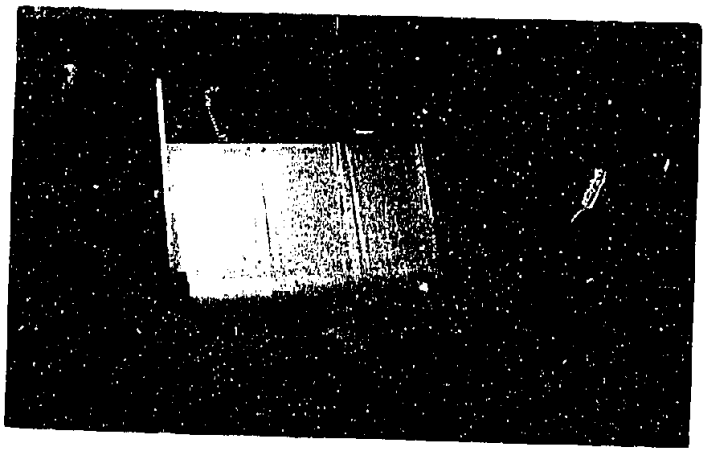
C. SLUICES 1,2,3,4,5,6 AND 8 DISCHARGING.



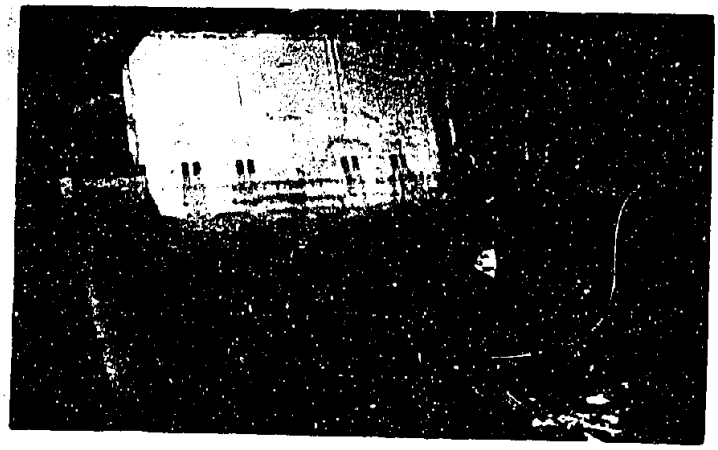
D. ALL SLUICES DISCHARGING.

4:1 SLOPING APRON TO LEVEL APRON ELEV. 806 - DIVERGING SLUICES.
1½:1 SLOPE SILL TO ELEV. 817.5 AT END OF APRON. SLUICE OPERATING PROGRAM.

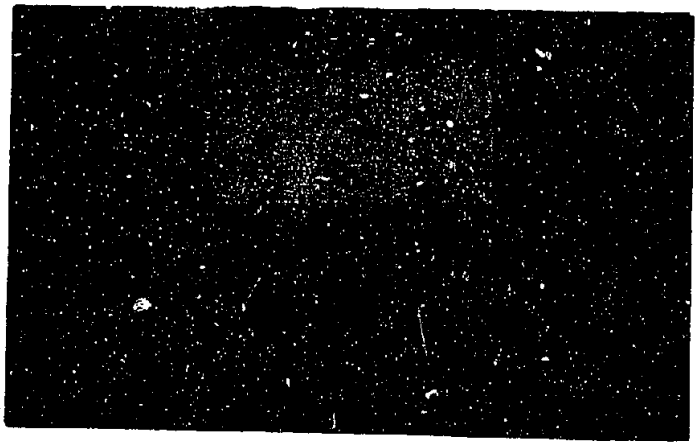
PLATE 68



A. NO SLUICES THROUGH DAM.

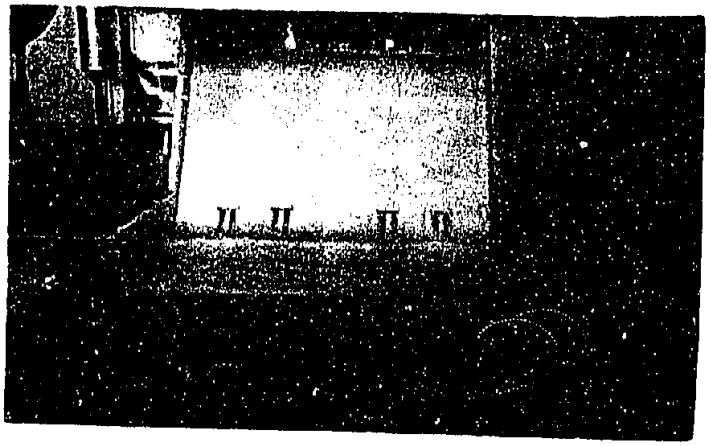


B. PARALLEL SLUICES. LEVEL INVERT ELEV. 860.

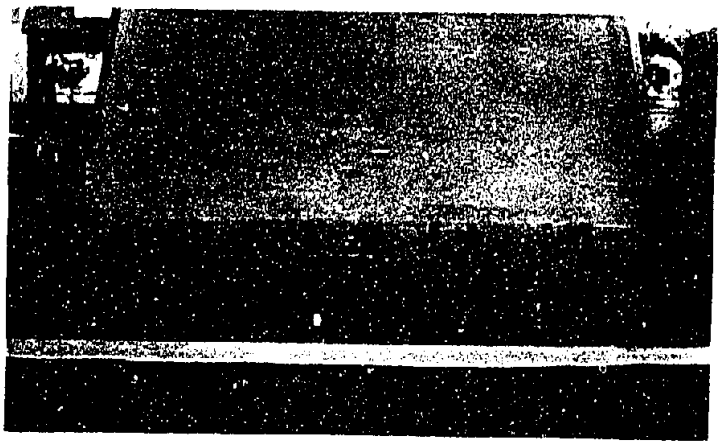


C. PARALLEL SLUICES - PARABOLIC INVERT.
NO DEFLECTORS ABOVE OPENINGS.

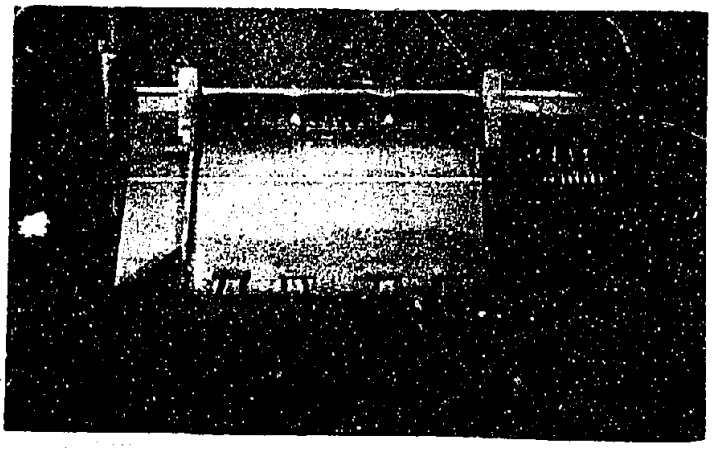
APPEARANCE OF SLUICE OPENINGS.



A. PARALLEL SLUICES - PARABOLIC INVERT.
DEFLECTORS ABOVE OPENINGS.

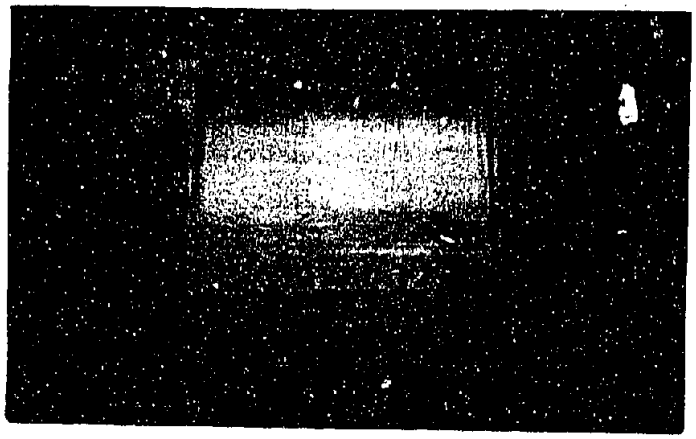


B. DIVERGING SLUICES - PARABOLIC INVERT.
NO DEFLECTORS ABOVE OPENINGS.

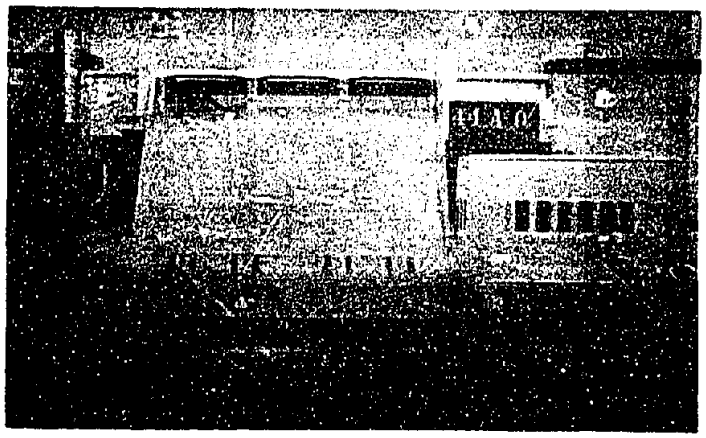


C. DIVERGING SLUICES - PARABOLIC INVERT.
DEFLECTORS ABOVE OPENINGS.

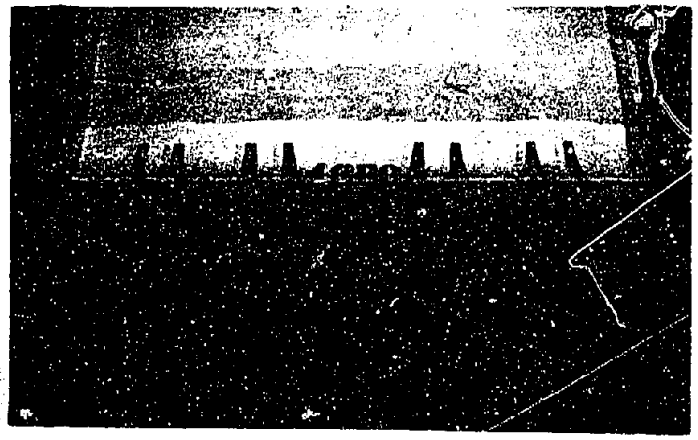
APPEARANCE OF SLUICE OPENINGS.



A. PARABOLIC HUMP IN DAM OVER SLUICE OPENINGS.



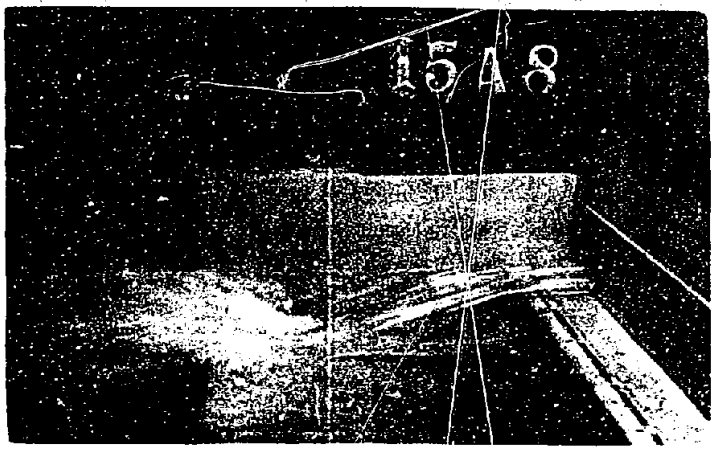
B. OPENINGS OF DIVERGING SLUICES DIVERGED.



C. OPENINGS OF BOTH STRAIGHT AND DIVERGING SLUICES DIVERGED.

DIVERGING SLUICES - PARABOLIC INVERT.

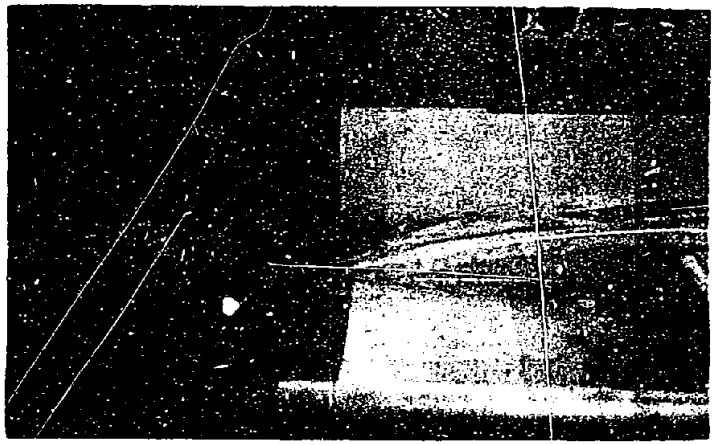
APPEARANCE OF SLUICE OPENINGS.



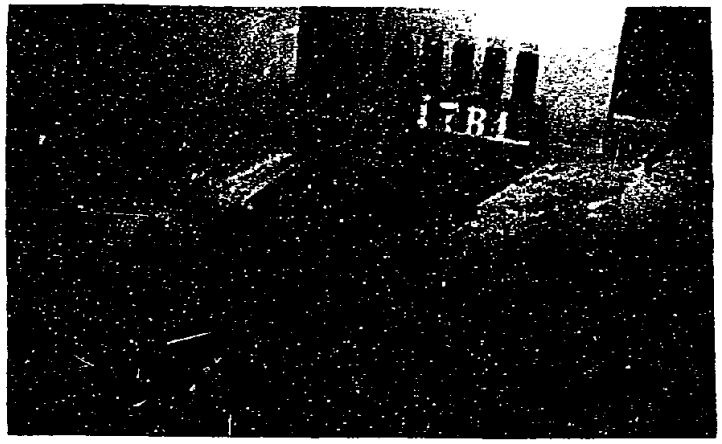
A. SIDE VIEW.

B. LOOKING UPSTREAM.

ORIGINAL DESIGN.



C. SIDE VIEW.



D. LOOKING UPSTREAM.

TRAJECTORY APRON FROM VALVES TO FLOOR OF TAILRACE.

NEEDLE VALVES DISCHARGING 5,200 SECOND-FEET.

PLATE 68

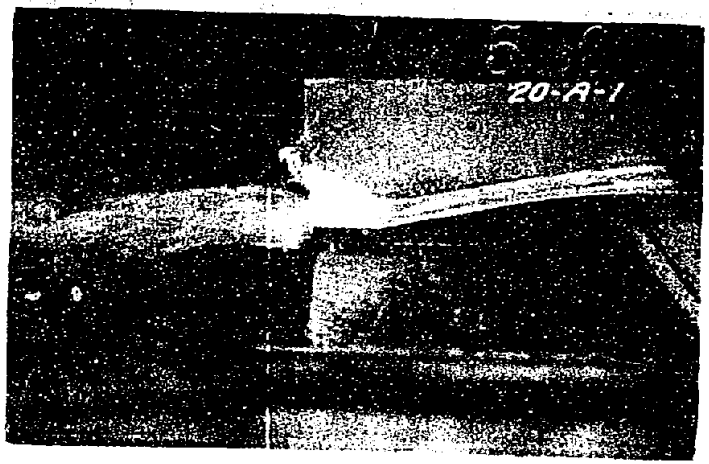
have occured near the end of the spillway training wall and would have affected the safety of that wall and the spillway apron.

The results of the first trajectory paving were not satisfactory and the apron was raised, giving the results shown on Plate 62 C and D. These conditions were more satisfactory, but a good jump was not formed and an eddy similar to the first condition was noticeable, causing debris to be deposited in front of the draft tubes.

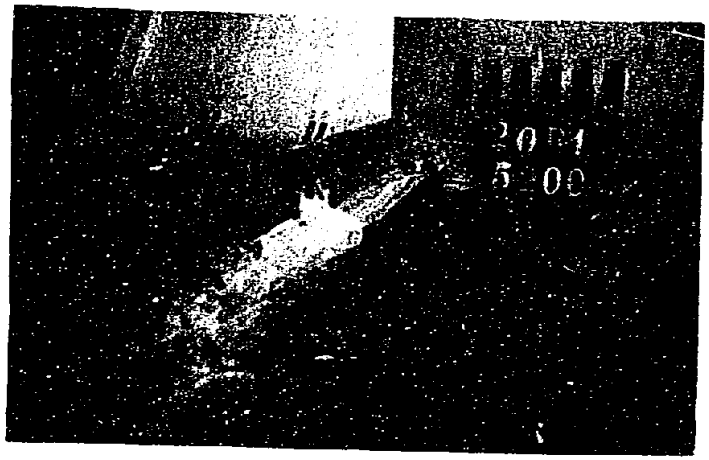
The flow conditions using the Ensign baffle on the apron are shown on Plate 63 A and B. The baffle had very little or no effect in slowing down the water.

The flow conditions with a 90-foot length of pool at the end of the apron showed very good results (Plate 63 C and D). The pool was originally built with the theoretical length and depth necessary to form the jump, neglecting friction on the apron. As was predicted, the jump formed very well and little trouble was encountered with the eddy carrying debris back into the tailrace. The energy was well dissipated in the jump, but the pool seemed longer than necessary and for this reason was shortened to the 90-foot length.

The results indicated that a shorter pool of less depth might be used to effect a satisfactory jump. The condi-



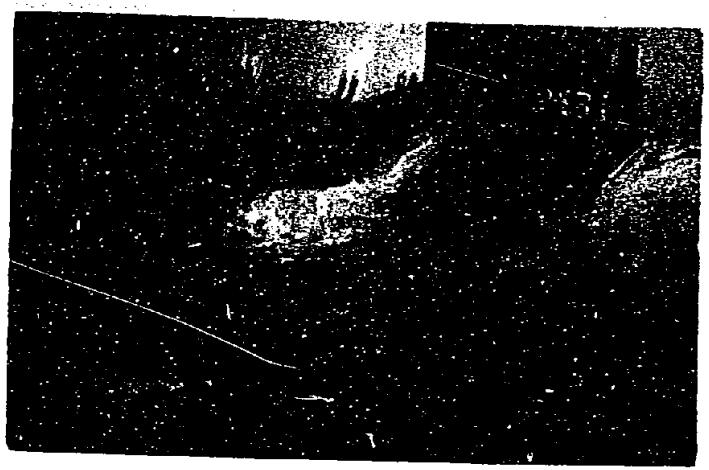
A. SIDE VIEW.



B. LOOKING UPSTREAM.

ENSIGN TYPE OF BAFFLE ON APRON.

C. SIDE VIEW.



D. LOOKING UPSTREAM.

STILLING POOL AT END OF APRON.

TRAJECTORY APRON FROM VALVES TO FLOOR OF TAILRACE.
NEEDLE VALVES DISCHARGING 5,200 SECOND-FEET.

tions using a 60-foot pool with a 1:1 slope at the downstream end were not as satisfactory as the results with the longer pool but when a diffuser sill was placed on this slope the conditions were very satisfactory (Plate 64). This solution was recommended with possible modifications but as later plans for the project eliminated the needle valves entirely, further study was unnecessary.

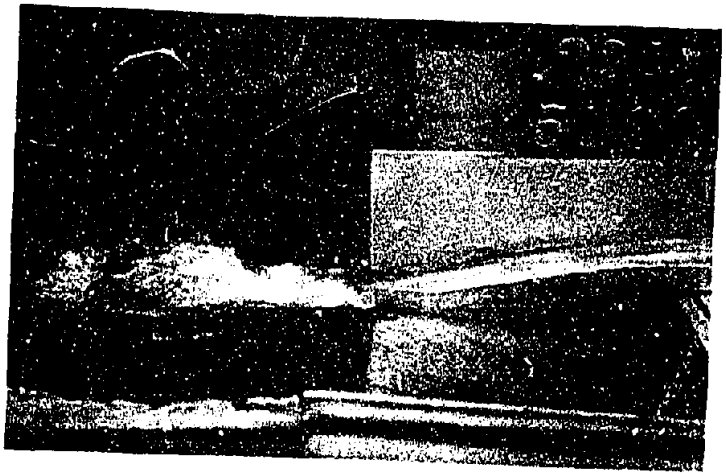
3. Powerhouse

No quantitative studies of the flow conditions from the powerhouse were made but no undesirable conditions were observed in the series of tests made on the spillway. Plate 65 A shows the flow conditions with the final design of the powerhouse operating alone at a flow of 5,300 second-feet. Plate 65 B shows the original design of the powerhouse with no flow. Plate 65 C and D show different views of the final design of the powerhouse.

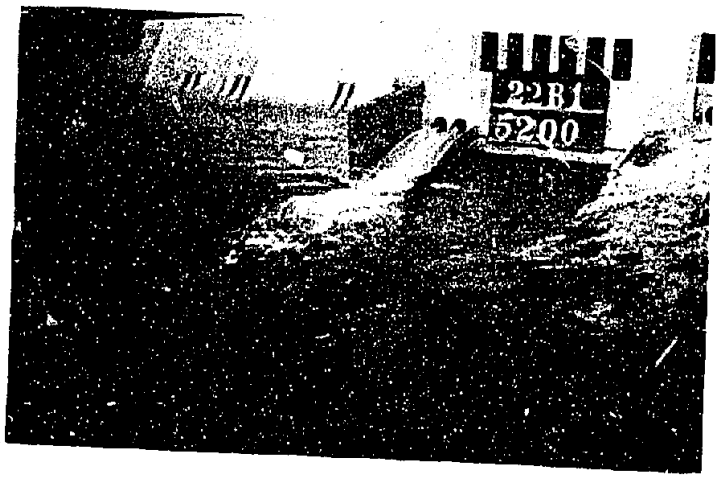
d. Architectural Studies

The results of the architectural studies in connection with the sluice openings have been shown. The appearance of the completed project was kept constantly in mind in conjunction with the changes.

Plate 66 A shows the general appearance of the model with the first bridge design. This bridge across the spillway

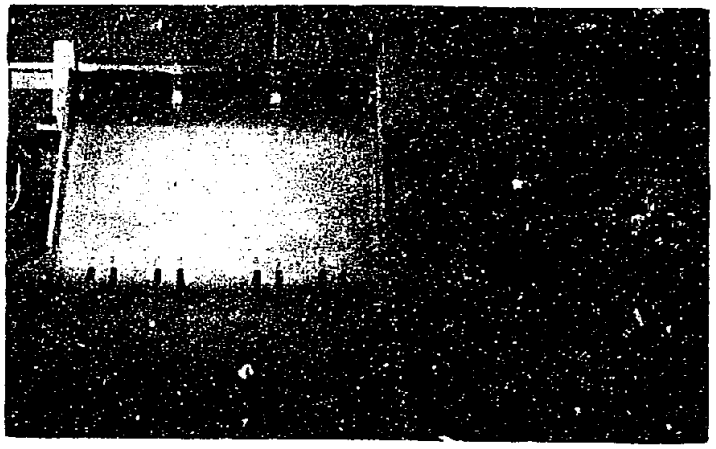


A. SIDE VIEW.

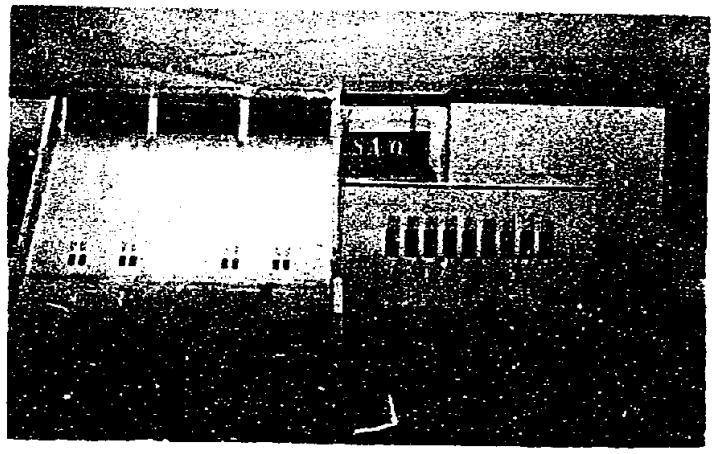


B. LOOKING UPSTREAM.

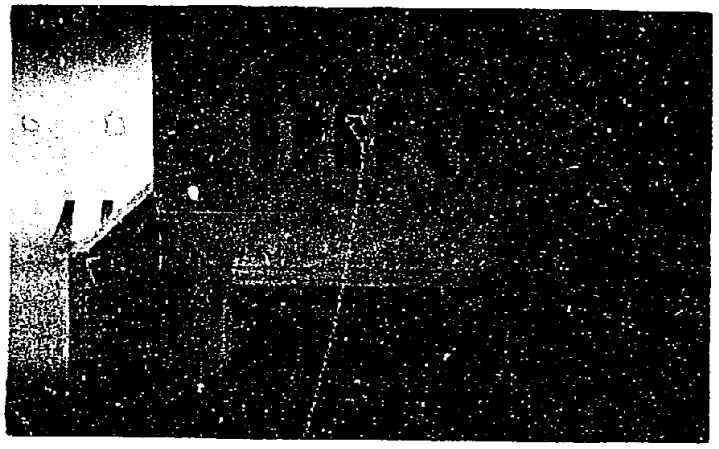
SHORT STILLING POOL AT END OF APRON. DIFFUSER TYPE SILL AT END OF POOL.
TRAJECTORY APRON FROM VALVES TO FLOOR OF TAILRACE.
NEEDLE VALVES DISCHARGING 5,200 SECOND-FEET.



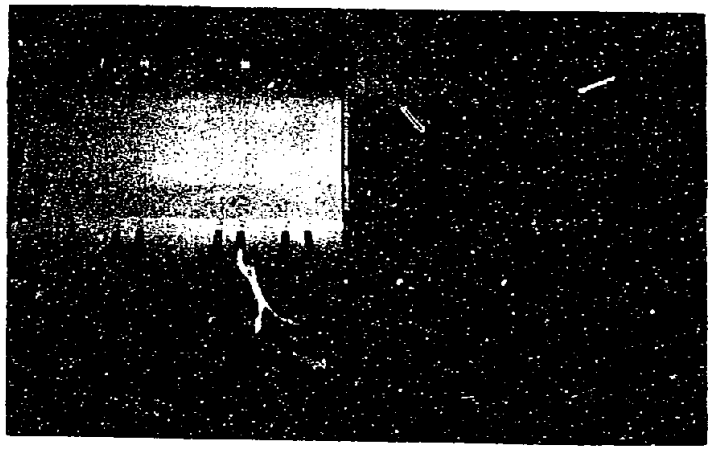
A. FINAL DESIGN - DISCHARGE 5,300 SECOND-FEET.



B. ORIGINAL DESIGN - NO FLOW.



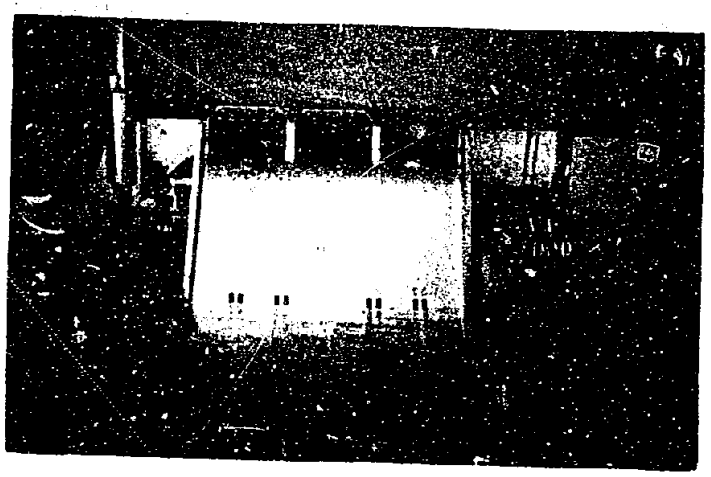
C. FINAL DESIGN.



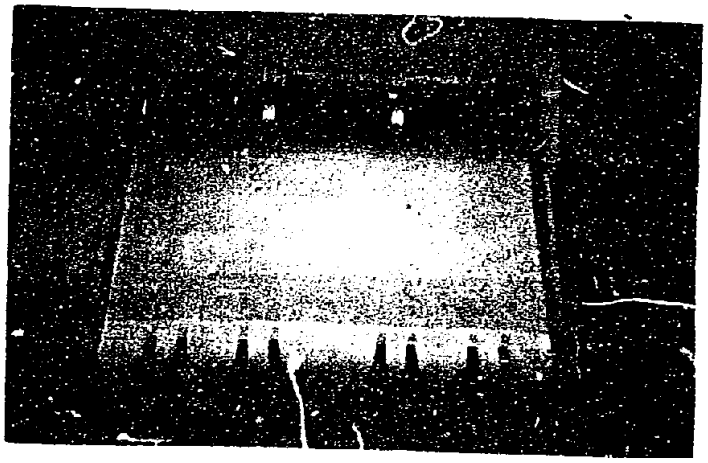
D. FINAL DESIGN.

POWER HOUSE STUDIES.

PLATE 69

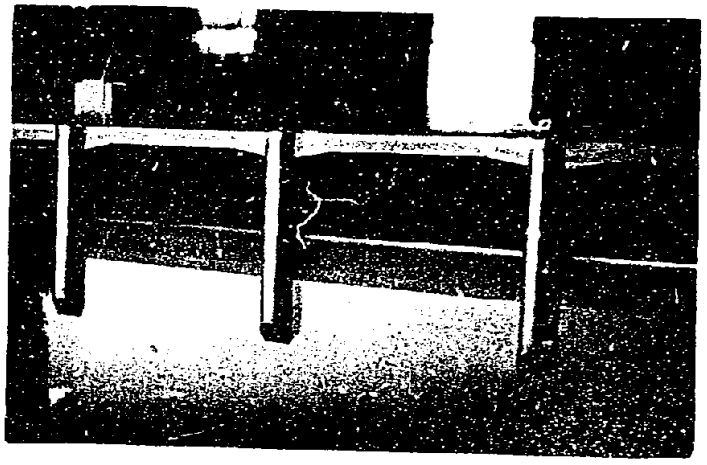


A. THROUGH TRUSS BRIDGE.



B. PLATE GIRDER BRIDGE - GENERAL VIEW UPSTREAM.

C. PLATE GIRDER BRIDGE - SHOWING ROADWAY.



D. PLATE GIRDER BRIDGE - LOOKING DOWNSTREAM.

BRIDGE ACROSS SPILLWAY.

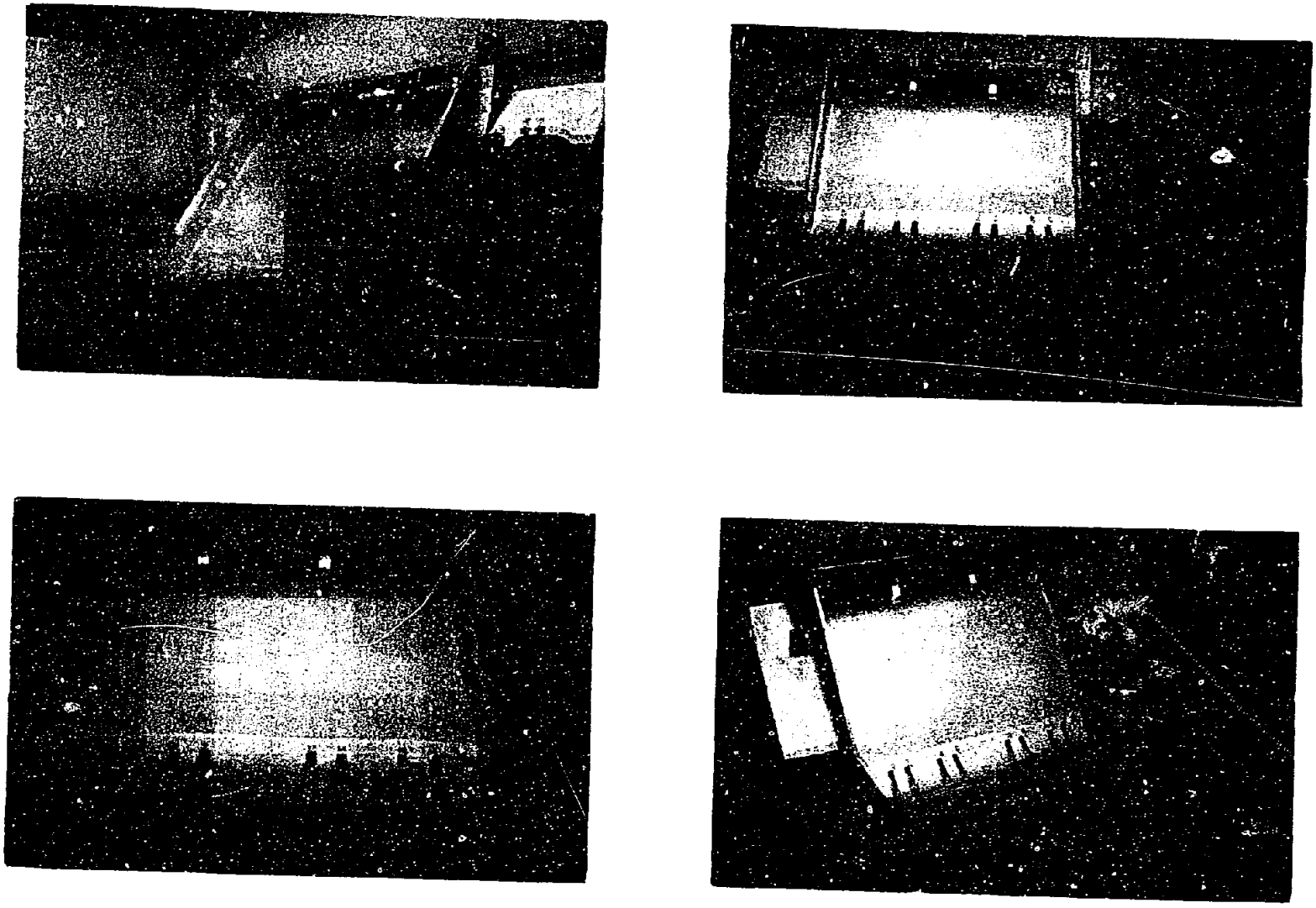
PLATE 1646

was built from an artist's conception of the completed project and appeared somewhat out of proportion when placed across the spillway. No pictures were obtained of the pony truss bridge but it gave the project a much better general appearance than the through truss bridge.

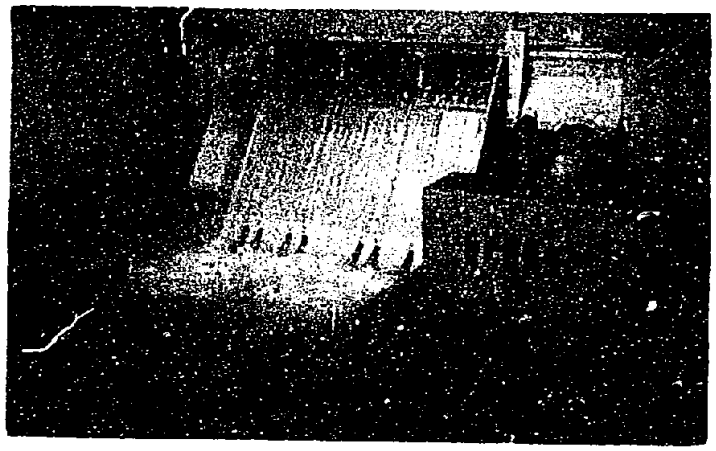
Plate 66 also shows the plate girder bridge across the spillway crest. This bridge is much more in keeping with the other features of the dam and adds greatly to the appearance of the structure. The change in design of the downstream face of the piers to support this bridge was beneficial to the aeration of the jets from the gates when they were near to and at their maximum elevation and was as good as previous designs at lower elevations.

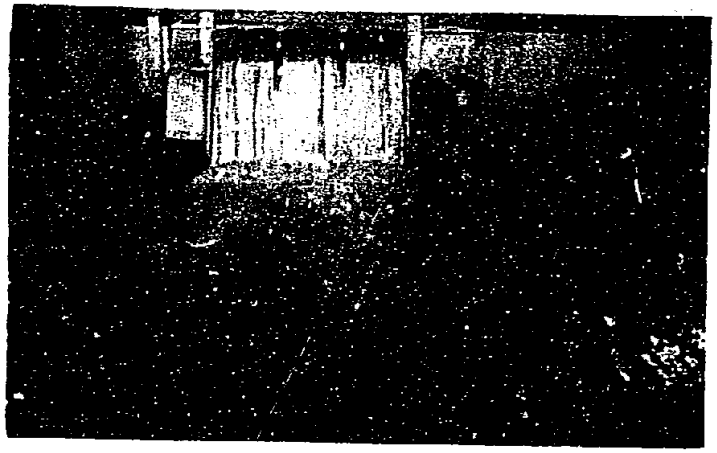
Plates 67 and 68 show the general appearance of the model in the final design and should give a very good conception of the completed project.

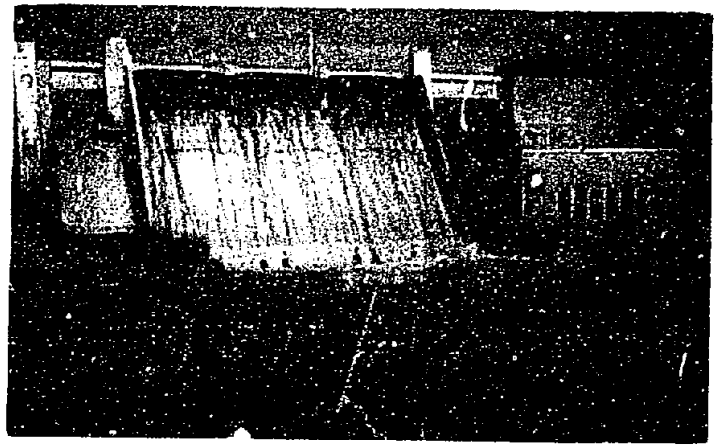
Plates 59, 60 and 61 show the evolution of the changes on the sluice outlets as regards to appearance. These changes were made primarily for improvement of flow conditions with appearance as a secondary consideration.



GENERAL VIEWS OF MODEL - FINAL DESIGN.







GENERAL VIEWS OF MODEL - FINAL DESIGN.

SUMMARY AND CONCLUSION

The stilling pool for the spillway is of ample dimensions and satisfactory design to secure the dissipation of the energy contained in the water flowing through the sluiceways and the range of flows up to and slightly above 200,000 second-feet over the spillway. No bad conditions of flow will result if the recommended operating program is followed and it is believed that no extremely serious condition will result, if, in the case of an emergency, this program cannot be followed. While the approved conditions produce a satisfactory condition of flow, a diffuser type sill produces worthwhile improvement and might result in some saving in pavement and training walls. The pool is amply secured against sweeping out even if possible retrogression of the stream bed lowers the tailwater somewhat below that predicted.

It is recommended that the smaller flows be discharged through the sluiceways by opening first, No. 3, then 6, 8, 1, 4, 5, 2 and finally 7 in the order named. (Numbers increasing from powerhouse). The sluiceways should be left open as the quantity that must pass the dam increases and if the flood rises to such proportions that the gates must be lowered, all three gates should be kept at the same elevation (if possible) until they are down in position as part of the permanent crest. This program insures no serious results being effected from the passage of a large flood.

The discharge curve prepared for the spillway can serve as a reliable source of information to the operating engineer as regards the amount of water passing the dam. The results of the calibration also show that the quantity for which the spillway was designed can be passed through the structure and a margin of safety allowed.

The design of the piers is such that the jets from the gates aerate naturally at all flows. This eliminates any necessity of air pipes under the gates to admit sufficient air for keeping the area under the jets from the gates at atmospheric pressure when the gates are raised and water is flowing over them.

The pressure studies on the drum gates showed a low pressure area existed along the lip of the gate at the designed head, but this did not become of consequent magnitude until the head raised somewhat above that for which the crest was designed. As this would be a condition of very remote probability, the crest design can be considered as efficient.

The results of the studies on the apron velocities show that close agreement of the model and prototype velocities substantiate the accuracy of the model tests and little doubt should remain but that the model velocities are acceptable as relative to the prototype.

The shear keys incorporated in the stilling pool apron for construction purposes will cause no harmful effect on the flow conditions. They were relatively so small that their effect on the flow conditions with the spillway operating was not perceptible. It was not possible to note any effect in the flow from the sluiceways except when No. 3 or No. 7 sluice was operating alone. The keys are directly in front of the openings of the sluices and a very slight wave was formed. This wave, being scarcely noticeable, would cause no bad conditions in the stilling pool and probably could not be detected on the prototype.

Studies made by means of motion pictures throughout the series of tests have borne out the results as set forth in the foregoing discussion and have brought to notice some conditions that otherwise might have been overlooked.

Although the experiments described in this report were primarily for the purpose of developing the best designs for the conditions existing at Norris Dam, many of the conclusions reached will be of value in the design of future structures.

The operating engineer of the project will find much valuable material in this report to be used as a guide in the operation of the project.

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)

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.

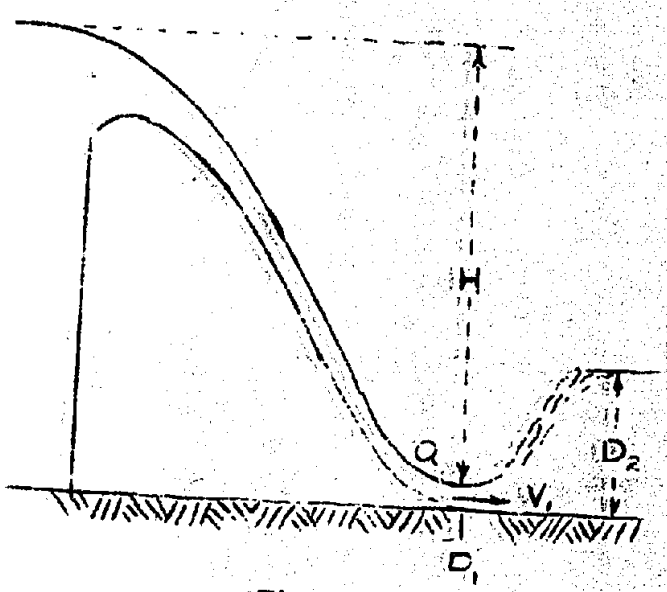


Figure 1.

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.

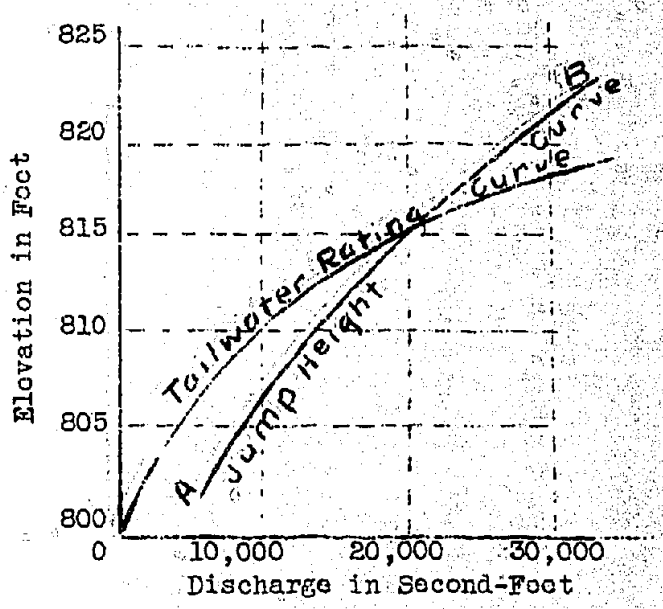


Figure 2.

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 Reco 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.

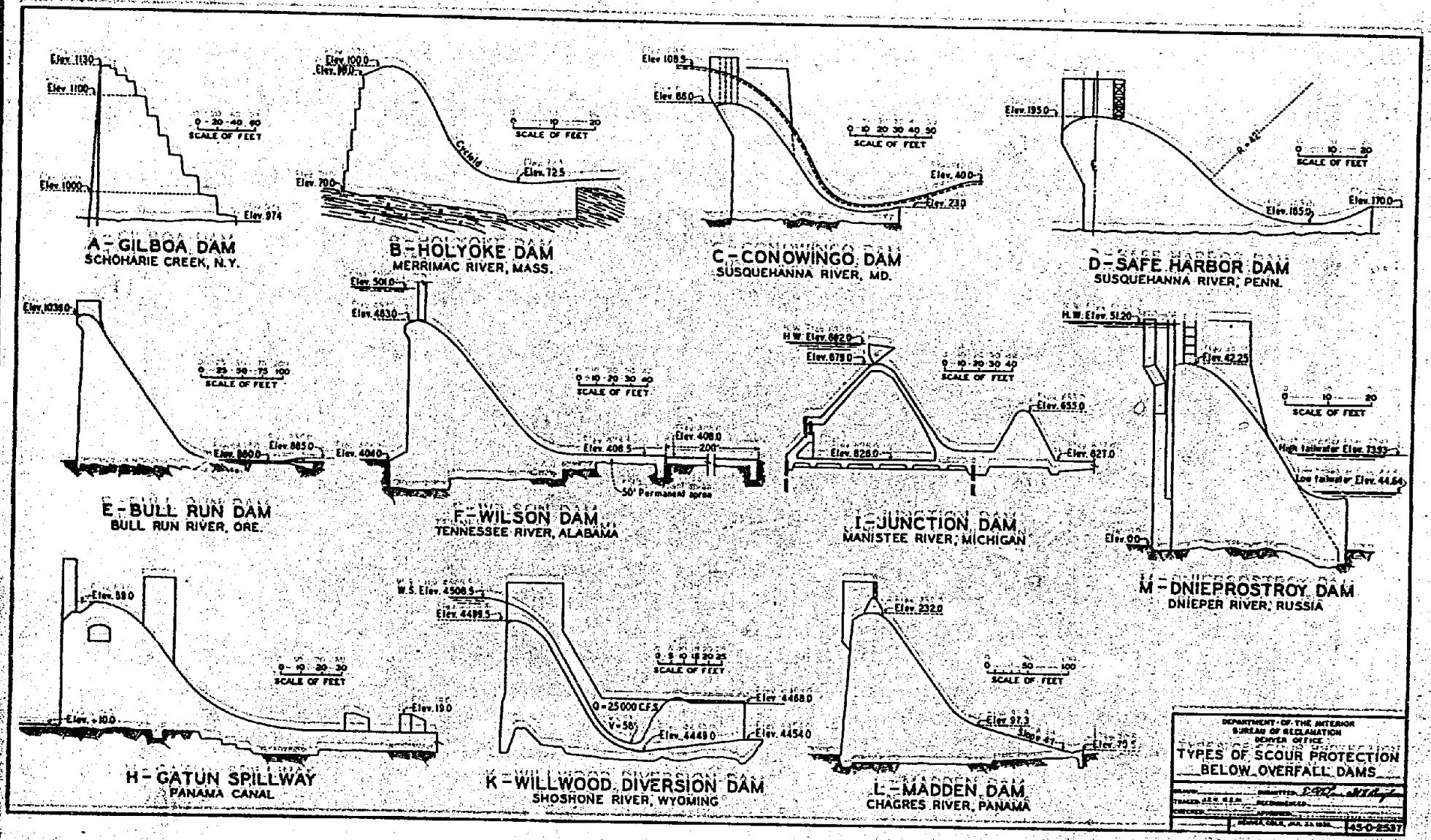
If the tailwater level is not high enough to form a perfect jump it may be raised by building a low secondary dam below the main dam with a sufficient height to cause the jump to form at the foot of the main dam for all conditions of discharge. This method has been extensively used for dams on earth 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.*

*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. 90.0
Elev. 70.0
Elev. 72.5
Crest
0 10 20
SCALE OF FEET
B - HOLYOKE DAM
MERRIMAC RIVER, MASS.
Elev. 109.5
Elev. 86.0
Elev. 40.0
Elev. 23.0
0 10 20 30 40 50
SCALE OF FEET
C - CONOWINGO DAM
SUSQUEHANNA RIVER, MD.
Elev. 195.0
R = 42'
Elev. 185.0
Elev. 170.0
0 10 20
SCALE OF FEET
D - SAFE HARBOR DAM
SUSQUEHANNA RIVER, PENN.
Elev. 1036.0
Elev. 860.0
Elev. 865.0
0 25 50 75 100
SCALE OF FEET
E - BULL RUN DAM
BULL RUN RIVER, ORE.
Elev. 501.0
Elev. 483.0
Elev. 404.0
Elev. 408.5
Elev. 408.0
200'
50' Permanent apron
0 10 20 30 40
SCALE OF FEET
F - WILSON DAM
TENNESSEE RIVER, ALABAMA
H.W. Elev. 682.0
Elev. 679.0
Elev. 655.0
Elev. 628.0
Elev. 627.0
0 10 20 30 40
SCALE OF FEET
I - JUNCTION DAM
MANISTEE RIVER, MICHIGAN
H.W. Elev. 51.20
Elev. 42.25
Elev. 00
High tailwater Elev. 7.35
Low tailwater Elev. 44.64
0 10 20
SCALE OF FEET
M - DNIEPROSTROY DAM
DNIEPER RIVER, RUSSIA
Elev. 58.0
Elev. +10.0
Elev. 19.0
0 10 20 30
SCALE OF FEET
H - GATUN SPILLWAY
PANAMA CANAL
W.S. Elev. 4508.5
Elev. 4499.5
Q = 25000 CFS
V = 58'
Elev. 4449.0
Elev. 4468.0
Elev. 4454.0
0 5 10 15 20 25
SCALE OF FEET
K - WILLWOOD DIVERSION DAM
SHOSHONE RIVER, WYOMING
Elev. 232.0
Elev. 97.3
Slope 4:1
Elev. 79.5
0 50 100
SCALE OF FEET
L - MADDEN DAM
CHAGRES RIVER, PANAMA
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
TYPES OF SCOUR PROTECTION
BELOW OVERFALL DAMS
45-0-2537

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 those 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

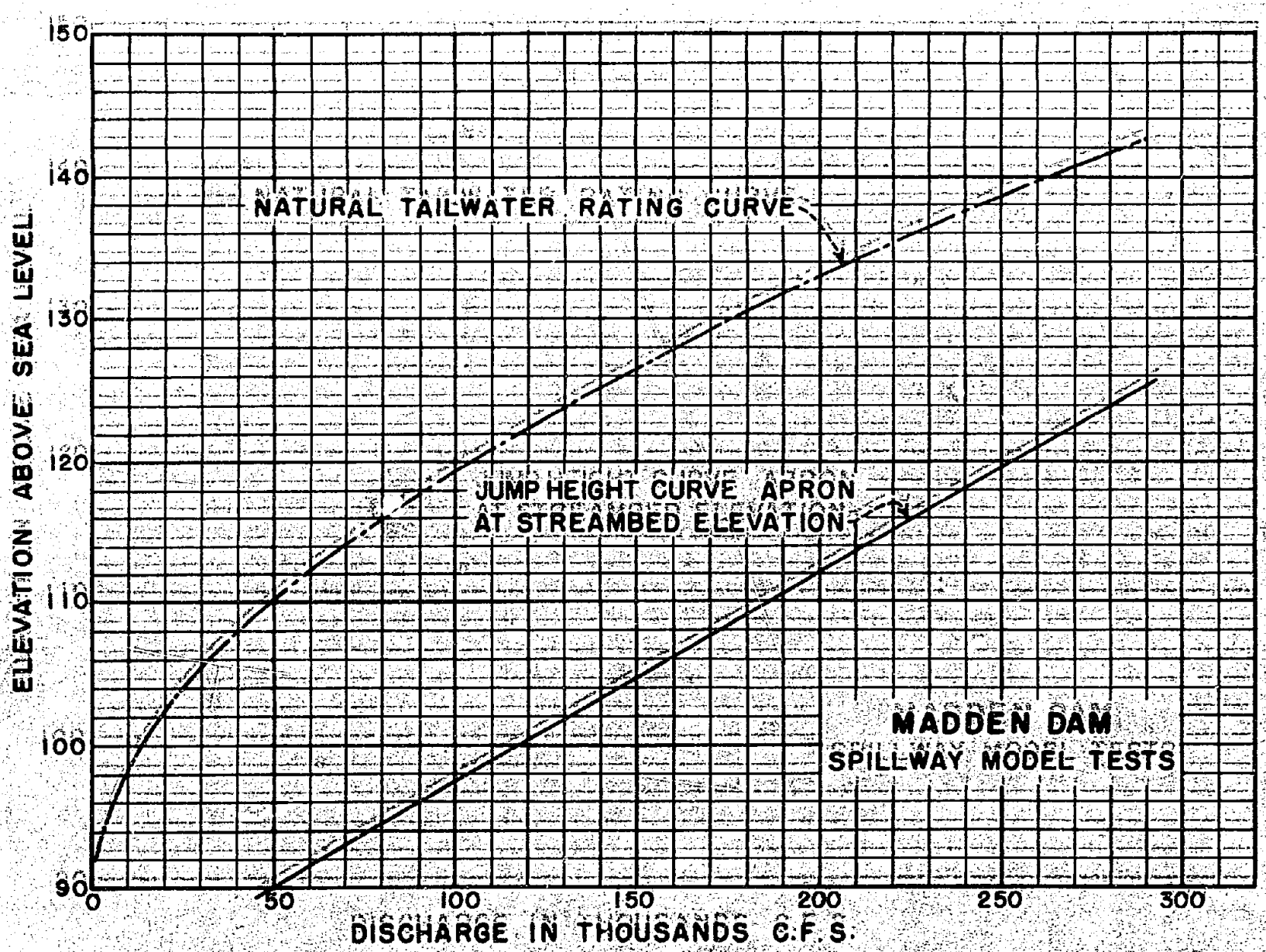

150
140
130
120
110
100
90
ELEVATION ABOVE SEA LEVEL
NATURAL TAILWATER RATING CURVE
JUMP HEIGHT CURVE APRON
AT STREAMBED ELEVATION
MADDEN DAM
SPILLWAY MODEL TESTS
0
50
100
150
200
250
300
DISCHARGE IN THOUSANDS C.F.S.

FIG. 3

X-D-1186

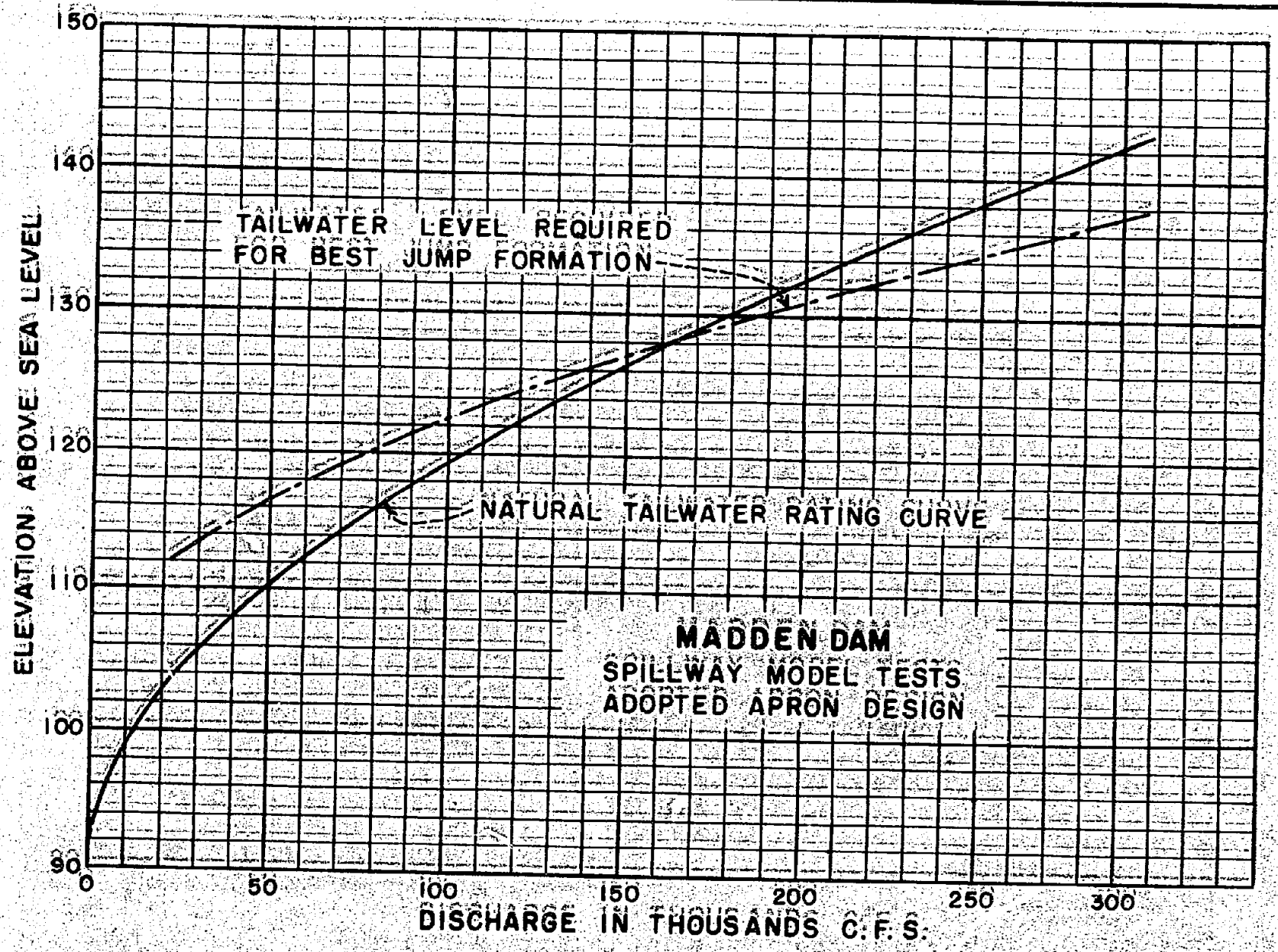

150
140
130
120
110
100
90
ELEVATION ABOVE SEA LEVEL
TAILWATER LEVEL REQUIRED
FOR BEST JUMP FORMATION
NATURAL TAILWATER RATING CURVE
MADDEN DAM
SPILLWAY MODEL TESTS
ADOPTED APRON DESIGN
0
50
100
150
200
250
300
DISCHARGE IN THOUSANDS C.F.S.

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 those 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

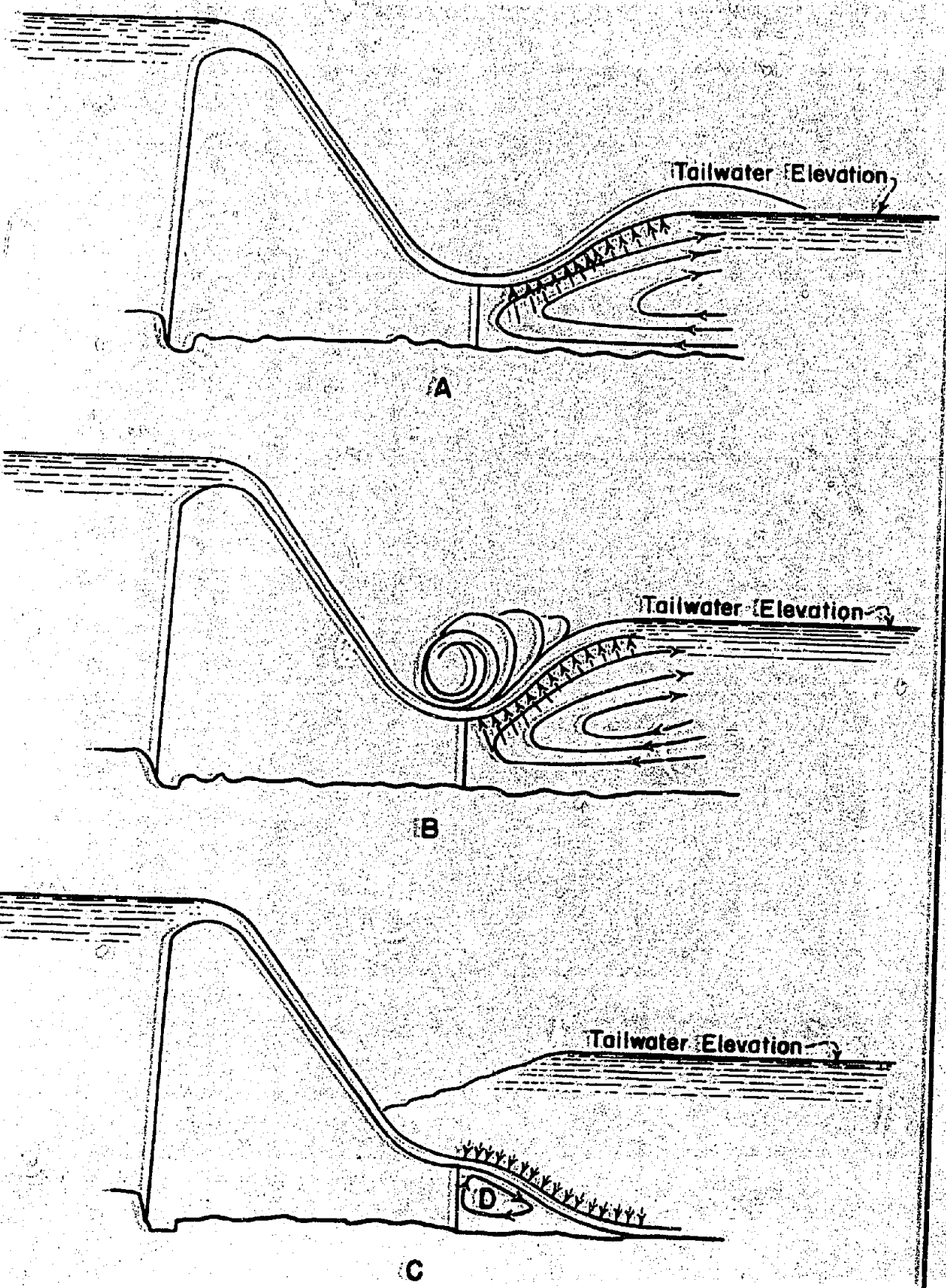


FIG. 5

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 5 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

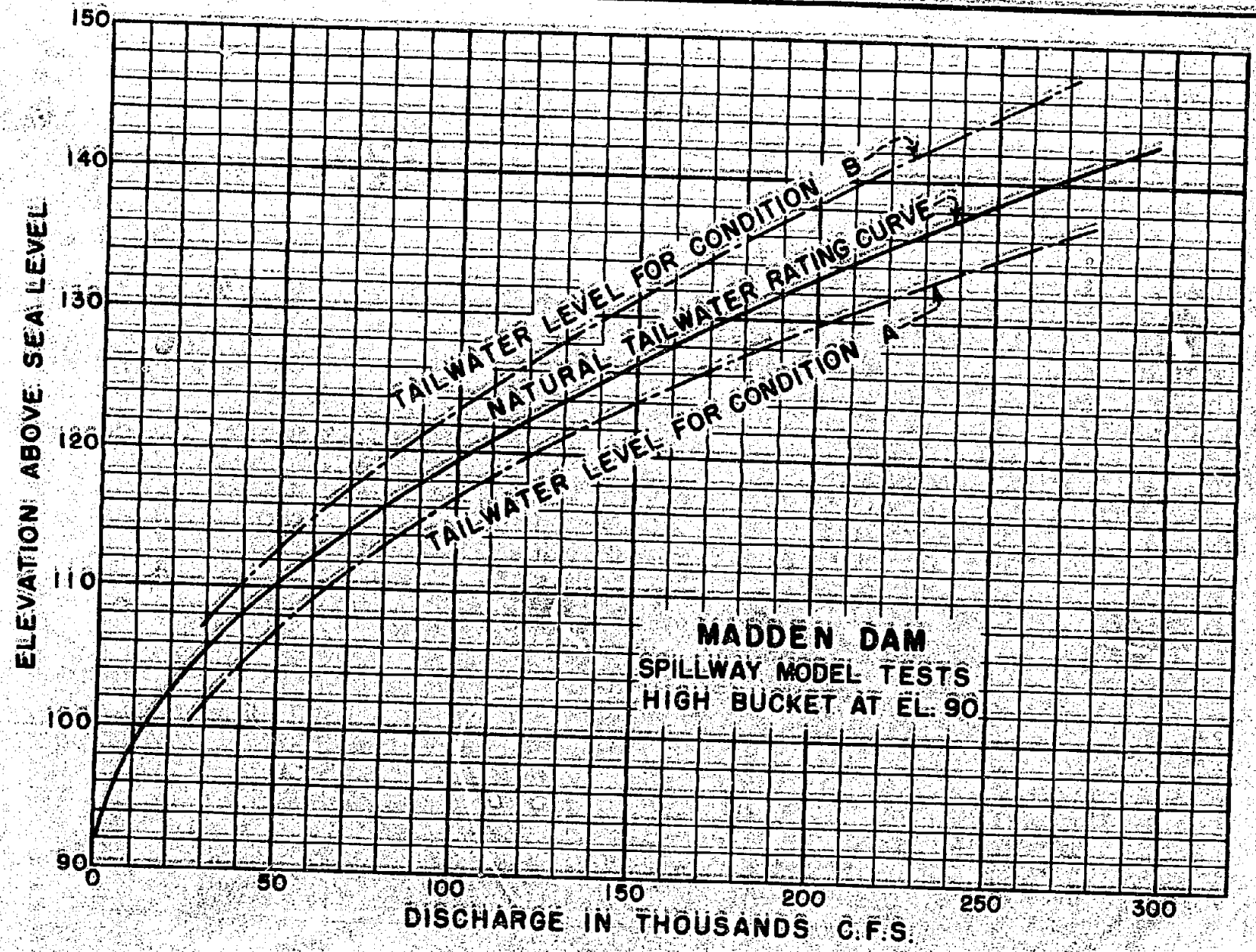

150
140
130
120
110
100
90
ELEVATION ABOVE SEA LEVEL
0
50
100
150
200
250
300
DISCHARGE IN THOUSANDS C.F.S.
TAILWATER LEVEL FOR CONDITION B
NATURAL TAILWATER RATING CURVE
TAILWATER LEVEL FOR CONDITION A
MADDEN DAM
SPILLWAY MODEL TESTS
HIGH BUCKET AT EL. 90

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

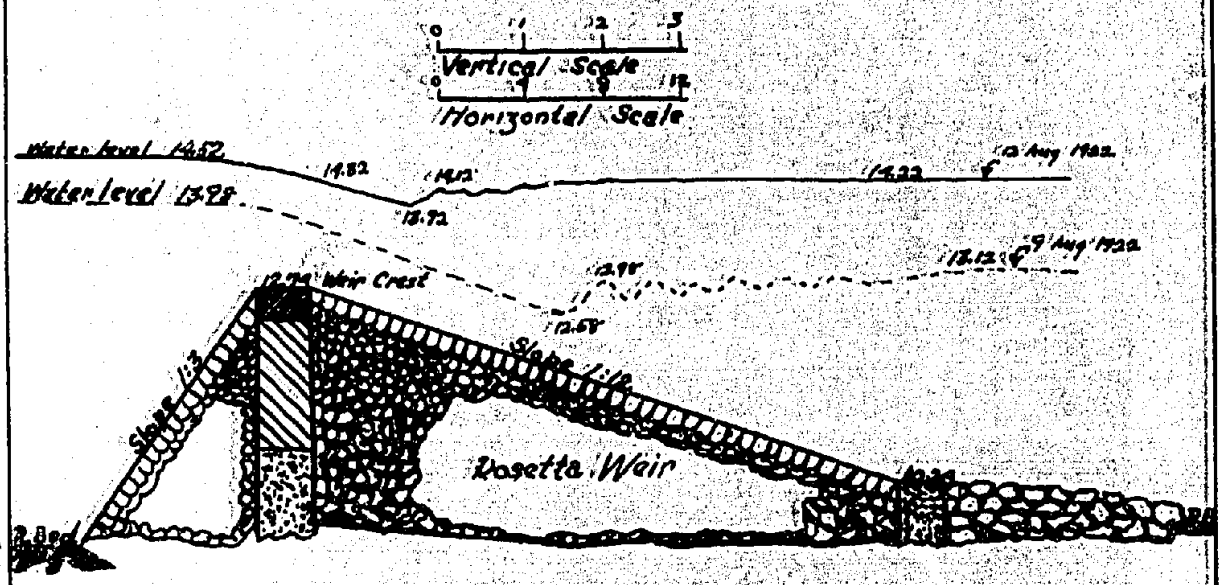
The foregoing analysis has also been simplified by assuming that the flow of the stream was uniformly distributed over the entire crest length of the dam. With control gates on the crest, however, the flow can be concentrated and at certain portions of the dam the flow will be greater than at others. 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



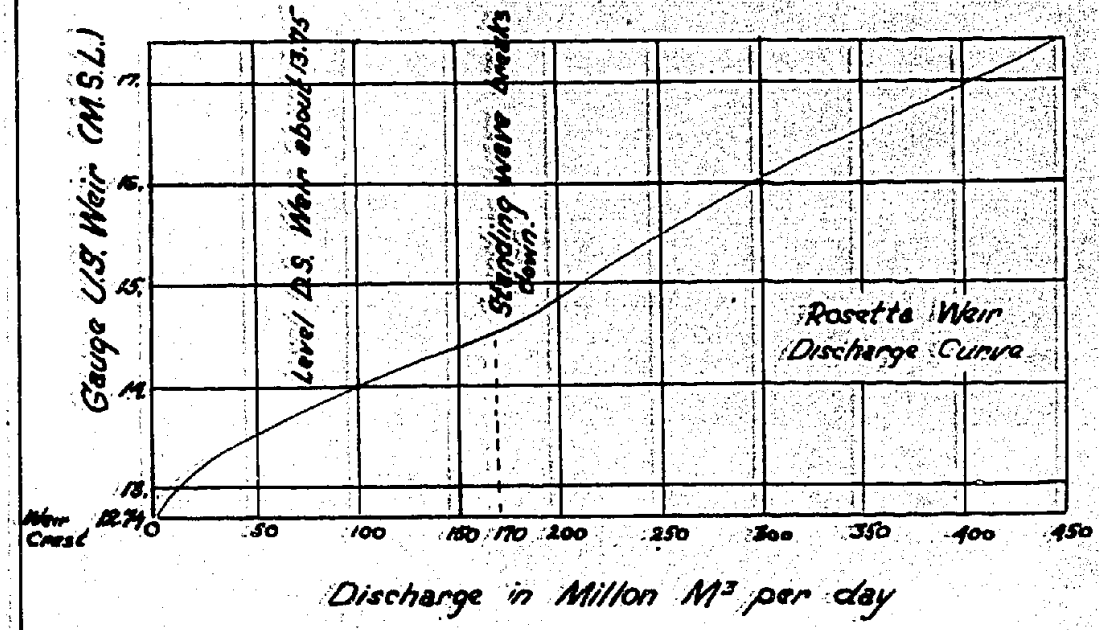


Fig. 7

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 con-

ditions 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

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

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.

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APPENDIX II.

THE LEAST SQUARE ADJUSTMENT OF EXPERIMENTAL DATA

by Carl Rohwer

THE LEAST SQUARE ADJUSTMENT OF EXPERIMENTAL DATA

Carl Rohwer

All experimental data are subject to error and it is the problem of the experimenter to find the most probable value of the experimental data. It is impossible to determine the exact value of any measured quantity but by repitition of the observations it is possible to reduce the error in the result to within the limits required by the experiment.

The most probable value of a series of observations may be found by the method of Least Squares, which was developed by the French mathematician Legendre in 1805. It takes its name from the fact that the most probable value is that value of a series of observations that makes the sum of the squares of the residuals or deviations a minimum. Legendre invented the method and worked out the procedure without apparently knowing why the method gave the most probable value. It is interesting to note in this connection that Napier invented logrithms without understanding the theory of exponents. A method of adjusting observations had previously, in 1774, been developed by Lapace, but, lacking a rational basis, his method was abandoned. He came very near the truth, however, because he assumed that the most probable value was the value that made the algebraic sum of the residuals zero, and their sum regardless of sign a minimum.

The method of least squares is based on the theory of errors and the laws of probability, and before taking up the practical application of the method it will be well to make a hasty resume of the theoretical principles involved.

The laws of probability resulted from the study of chance in gambling and the leading scientists of the time devoted their energies to the solution of the problem. James Bernoulli, uncle of the Swiss hydraulician Daniel Bernoulli, familiar to all engineers, stated the basic law as follows. "If a sufficiently large number of trials is made, the ratio of the favorable to the unfavorable events will not differ from their respective probabilities beyond a certain limit in excess or defect, and the probability of keeping within those limits, however small, can be made as near certainty as we please by taking a sufficiently large number of trials."

The mathematical probability of an event is the ratio of the number of ways in which an event may happen or fail to the total number of ways in which it may occur if each occurrence is equally probable. Thus, in flipping a coin, there are two possibilities; it may be either a head or a tail, but it can be a head

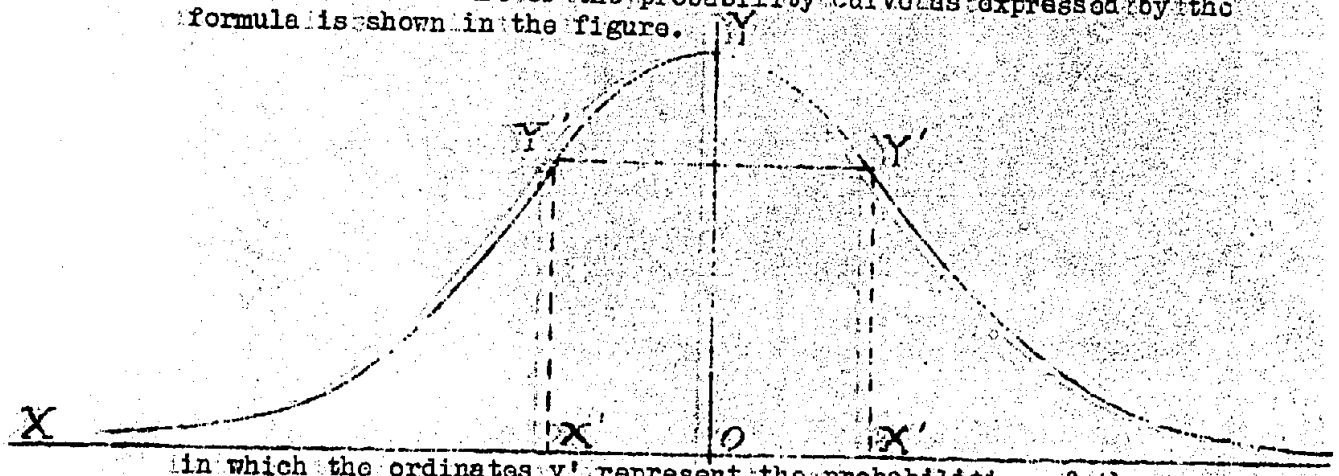
or a tail in only one way, so the probability of a head or a tail is in either case 1/2 or 1 in 2. If an event may happen in more ways than one, the probability of its occurrence is the sum of the individual probabilities. In compound events, such as throwing two aces with a single throw of two dice, the probability of the event is the product of the individual probabilities, in this case 1/6 x 1/6 or 1 in 36.

Accidental errors in observations follow the laws of chance, and the probability of an error is the ratio of the numbers of the errors of the given magnitude to the total number of errors. This law applies to accidental errors only; systematic or cumulative errors, such as instrumental errors and errors resulting from the personal equation of the observer, must first be eliminated.

The law of errors, as stated, is quite simple, but its application is far from simple. In order to use the law in the form given, it would be necessary to make an almost infinite number of observations before the ratio of the errors of the given magnitude to the total number of errors could be found with any degree of accuracy.

To overcome this difficulty, Gauss, Germany's most famous mathematician, worked out the mathematical distribution of experimental errors in 1809, and expressed the relation by the formula $y = k e^{-h^2x^2}$, where y is the probability of an error, x, k and h are constants that depend on the accuracy of the observations. Both increase as the precision of the measurements increases, and are constant only for a given set of experimental data.

The nature of the probability curve as expressed by the formula is shown in the figure.



in which the ordinates y' represent the probabilities of the errors, x'.

It will be observed that the curve is symmetrical about the y-axis, and when x is very large y is very small. Also when x is zero, y is a maximum. In other words, small errors are most common, large errors are relatively few, positive and negative errors are equally common, and the most probable value is that value in which the error is zero.

The fundamental equation of least square adjustments is

$$y = KK e^{-h^2x^2}$$

From it we can compute the probable error of a single observation in a series and the probable error of the most probable value of the series. From it Gauss proved mathematically that the method of least squares gave the most probable value of a series of observations.

The probable error has been defined as that error which is as likely to be exceeded as not. In other words, the probability of the error is 1 in 2 or 1/2. Referring to the curve, it is the value of x when y equals 1/2. It has been shown by calculus that, regardless of the nature of the experimental data, when y=1/2, hx = 0.4769. X is the probable error of the observation and is designated by r; hence, x = r = 0.4769/h. It has also been shown that when y is a maximum, or that the errors are the most probable ones, then

$$h = \sqrt{\frac{n}{2\Sigma x^2}}$$

Substituting the value of h in the equation r = 0.4769/h, we get

$$r = 0.4769\sqrt{2}\sqrt{\frac{\Sigma x^2}{n}} = 0.6745\sqrt{\frac{\Sigma x^2}{n}}$$

Here Σx^2 is the summation of the squares of the true errors and n is their number. But it is impossible to find the true errors, so we must use the residuals which we designate by V, instead. When V is substituted for x in the equation, the mathematicians have proved, to their own satisfaction at least, that r, the probable error of a single observation, equals $0.6745\sqrt{\frac{\Sigma v^2}{n-1}}$. By similar reasoning, they also show that the probable error of the mean, which is designated by r_o equals $\Sigma v^2/n(n-1)^{\frac{1}{2}} \times 0.6745$.

This is the simplest form of the probable error equation. It applies only to direct observations of equal weight on a single quantity. For other types of observations, different formulas result which will be given after describing the different types of observations.

All observations are either direct or indirect. In the first case, the reading is taken on the quantity whose value is to be determined. The measurement of a line with a tape, the weight of an object with a balance, and the temperature of a solution with a thermometer are all direct observations. It is to this type that the probable error equation given above applies. Indirect observations are not made on the quantity to be determined but are made on quantities that bear some relation to it. The determination of the area of a rectangle by measuring the length and breadth, the velocity of a current meter by measuring the time and distance, and the equation of a line by measuring the ordinates and abscissas of several points--are all indirect observations.

Observations are further divided into independent and conditioned observations. Independent observations are not limited by rigorous conditions but conditioned observations are. If we measure the three sides of a triangle each side bears no definite relation to the other, but if we measure the three angles of a triangle they must comply exactly with the mathematical law that the sum of the angles of a plane triangle is always equal to 180 degrees. Similarly, percentages in a complete analysis must equal 100 percent exactly.

Observations are frequently known to differ in precision and the numerical value assigned to their precision is known as the weight of the observation. Thus, an observation that has been repeated ten times when compared with one made once is given a weight of ten.

For independent observations indirectly observed, the probable error of an observation of weight unity is

$$r = 0.6745\sqrt{\frac{\Sigma Pv^2}{n - q}}$$

where ΣPv^2 is the summation of the weighted residuals, n the number of observations, and q the number of unknowns. To find the probable error of one of the q unknowns, divide r by the square root of the weight of the unknown.

For conditioned observations, the general equation of the probable error of an observation of weight unity is

$$r = 0.6745\sqrt{\frac{Pv^2}{n - q - n'}}$$

where the terms are the same as before except that n' is the number of conditions. The probable errors of the unknown are found as before.

For all types of observations the probable error, R, of the function $Z = f(Z_1 Z_2 Z_3)$ is given by the formula

$$R^2 = \frac{(dZ)^2}{(dZ_1)} r_1^2 + \frac{(dZ)^2}{(dZ_2)} r_2^2 + \frac{(dZ)^2}{(dZ_3)} r_3^2$$

It is the method used in finding the probable error in the area of a rectangle when the length and breadth have been measured. In this case the formula reduces to

$$R = \sqrt{r_2^2 Z_1^2 + r_1^2 Z_2^2}$$

The fundamental equation $y = K e^{-h^2x^2}$, as previously stated, is the basis of the proof of the method of least squares, or that the most probable value is the value that makes the sum of the squares of the residuals a minimum. The Egyptians knew that the arithmetic mean gave the most probable value of a series of direct measurements on a single quantity. By the method of least squares it can be shown that this assumption was correct.

The arithmetic mean is the simplest type of least square adjustment. The following example will show the method: On October 26, 1925, Mr. Parshall and I made a calibration of the optical evaporimeter used in our evaporation experiments. The results are given in column 1 of Table I and are the factor by which the optical scale readings in hundredths must be multiplied to find the evaporation in inches.

The mean value of the constants as determined by taking their sum and dividing by their number is 0.00002553, which, by the principle of least squares, is the most probable value of the constants. In column 2 of the table are given the residuals or differences between the individual values of the constant and the mean, and in column 3 are given the squares of the residuals and their sum. If any other value had been taken instead of the mean, the sum of the squares of the new residuals would have been greater than 10013.

From the sum of the squares of the residuals the probable error of any of the observations may be computed by substituting in the formula

$$r = 0.6745\sqrt{\frac{\Sigma v^2}{n-1}}$$

which gives $$r = 0.6745\sqrt{\frac{10013}{10-1}} = 22.6 \text{ or}$$

$$r = \pm\ 0.000000226$$

The probable error of the mean is obtained by substituting in the formula

$$r = 0.6745\sqrt{\frac{\Sigma v^2}{n(n-1)}} \text{ or } 0.6745\sqrt{\frac{10013}{10(10-1)}} = 7.2$$

This value when added to the mean or most probable value gives the usual form of expressing the result, viz.,

$$0.00002553 \pm 0.000000072$$

It will be observed that one of the residuals in column 2 of Table I is much larger than any of the others, and the question arises whether this observation should be discarded. From the fundamental equation $y = K e^{-h^2x^2}$ Chauvenet determined the ratio of the largest permissible residual to the probable error for different numbers of observations and fortunately prepared a table of the values. In this case the value for ten observations is 2.91 and since 2.91 x 22.6 = 65.8, which is less than the residual, the first observations may be discarded. For values other than 10 see Appendix.

As previously stated, the arithmetic mean is the simplest form of the least square adjustment, but unfortunately it can be used only in direct observations on a single quantity. If the observations are indirect, a different procedure must be followed. Take, for example, Rating #106 on a Lietz current meter made September 18, 1915 by Doke and Parshall. As shown in Table II, ten observations were made at velocities ranging from 0.423 to 6.506 feet per second. It is known that meters of this type have straight line equations of the form $V = aR + b$, where V is the velocity, R the revolutions per second, and a and b constants that must be determined for each meter. By substituting any two sets of corresponding values of V and R from Table II in the equation, we get two equations with two unknowns which can be solved by ordinary algebraic methods for a and b. Any other two sets of observations will also give values of a and b, which will differ from the first values obtained because the observations are not free from errors. To find the most probable values of a and b for the series of observations is the province of the method of least squares.

The procedure is as follows: Substitute all the observed values in the general equation $V = aR + b$. Equations 1 to 10 result. These equations are called the observation equations and are always equal in number to the number of observations. Since there are only two unknowns, the problem is to reduce the number of the observation equations to two. By the method of least squares it has been shown that this can be done as follows: Multiply each observation equation by the coefficient of the first unknown in the

TABLE I

Calibration of Optical Evaporimeter

by

Parshall and Rohwer

Oct. 26, 1925

Observed Constant	v	Residuals v^2
(1)	(2)	(3)
.00002632"	-79	6241
2584	-31	961
2562	- 9	81
2546	+7	49
2543	+10	100
2542	+11	121
2536	+17	289
2534	+19	361
2532	+21	441
2516	+37	1369
Mean .00002553"		Σv^2 10013

The probable error of an observation is

$$r=0.6745\sqrt{\frac{\Sigma v^2}{(n-1)}} = 0.6745\sqrt{\frac{10013}{10-1}} = 22.6 \text{ or } .000000226"$$

The probable error of the mean is

$$r_0=0.6745\sqrt{\frac{\Sigma v^2}{n(n-1)}} = 0.6745\sqrt{\frac{10013}{10(10-1)}} = 7.2 \text{ or } .000000072"$$

The mean is then

$$0.00002553 \pm .000000072$$

TABLE II
Rating #106

September 19, 1915 — Lietz #8374 — by Doke and Parshall

Observed			Least Square			Graphic		
	R	V	V	v	v^2	V	v	v^2
Point	Revs/sec	Ft/sec	Ft/sec	Residual	Resid.	Ft/sec.	Resid.	Resid.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	.146	.423	.403	+20	400	.415	+8	64
2	.254	.666	.655	+11	121	.667	-1	1
3	.411	1.011	1.024	-13	169	1.032	-21	441
4	.664	1.613	1.615	-2	4	1.621	-8	64
5	.751	1.818	1.818	0	0	1.824	-6	36
6	.984	2.360	2.364	-4	16	2.367	-7	49
7	1.454	3.455	3.463	-8	64	3.461	-6	36
8	1.490	3.537	3.548	-11	121	3.545	-8	64
9	1.912	4.522	4.535	-13	169	4.528	-6	36
10	2.744	6.506	6.482	-24	576	6.466	+40	1600
				Σv^2	1640		Σv^2	2391

$$r = 0.6745\sqrt{\frac{\Sigma v^2}{n-q}} = 0.6745\sqrt{\frac{1640}{10-2}} = 9.6 \text{ or } \pm 0.0096$$

Observation equation:

.423	=	.146 a + b	(1)	.062	=	.021 a +	.146 b	(11)
.666	=	.254 a + b	(2)	.169	=	.065 a +	.254 b	(12)
1.011	=	.411 a + b	(3)	.416	=	.169 a +	.411 b	(13)
1.613	=	.664 a + b	(4)	1.071	=	.441 a +	.664 b	(14)
1.818	=	.751 a + b	(5)	1.365	=	.564 a +	.751 b	(15)
2.360	=	.984 a + b	(6)	2.322	=	.968 a +	.984 b	(16)
3.455	=	1.454 a + b	(7)	5.024	=	2.114 a +	1.454 b	(17)
3.537	=	1.490 a + b	(8)	5.270	=	2.220 a +	1.490 b	(18)
4.522	=	1.912 a + b	(9)	8.646	=	3.656 a +	1.912 b	(19)
6.506	=	2.744 a + b	(10)	17.852	=	7.530 a +	2.744 b	(20)

25.911 = 10.810 a + 10 b (a)
Normal equation

42.197 = 17.748 a + 10.810 b (B)
Normal equation

Solving (A) and (B)
a = 2.340
b = 0.061 and V = 2.340 R + 0.061
Graphically V = 2.329 R + 0.075

equation. Equations 11 to 20 result. Then multiply each observation equation by the coefficient of the second unknown. Since the coefficient of the second unknown is 1 in each case, the new equations will be the same as equations 1 to 10, although in general they would be different. Each set of 10 equations is then added. Equations (A) and (B) result. These equations are called the normal equations and are always equal in number to the number of unknowns. The solution of these two equations gives the most probable values of a and b. In this case, a = 2.340 and b = 0.061 and the equation of the meter is then V = 2.340 R + 0.061.

In column 4, Table II, are shown the velocities computed by this formula. In column 3 are shown the corresponding observed values. The differences between the two sets of values are the residuals. These are recorded in column 5, and their squares in column 6. It is from the summation of these values that the probable error of a single observation is computed. In this case the probable error is given by the formula

$$r = 0.6745\sqrt{\frac{\Sigma v^2}{n - q}}$$

where n is the number of observations and q the number of unknowns. Substituting the values for n, q, and Σv^2 in this formula gives $\pm$ 0.0096 as the probable error in a single velocity determination.

As previously pointed out, the equation of the meter is known to be V = a R + b. This is evident when the observed points are plotted on rectangular coordinate paper and the usual practice is to determine the equation directly from the plot. By this method the equation of the meter is V = 2.329 R + 0.075. It differs but slightly from the least square equation. The velocities computed by this formula are given in column 7 of Table II. The residuals and their squares are given in columns 8 and 9. It will be noted that the summation of the residuals resulting from the use of the formula determined graphically is nearly 50 percent greater than for the least square formula. This is in accordance with the theory.

For practical purposes, however, the graphical solution gives a formula that gives more consistent results. As shown in Table II, column 5, the least square formula gives large residuals for the highest and lowest velocities, while the graphical formula gives a small residual for the smallest velocity and a large residual for the largest velocity. This means that the precent of error is large for small velocities and small for large velocities when the least square method is used, while when the graphical method is used the percent of error is kept more nearly constant for all velocities. This is necessary because large errors would otherwise result in measuring a stream, particularly when all the velocities are low.

The method of making a least square adjustment just de-

scribed applies only to linear functions. Transcendental and functions of higher degree must first be reduced to some linear form before the method can be used. In case it is impossible to reduce the function to the linear form, special methods must be used in making the least square adjustment.

As an example of the adjustment of non-linear functions, take the free flow discharge formula of the Improved Venturi Flume,

$$Q = 4\ w\ H_a^{1.522}\ W^{.026}$$

where Q is the discharge in cubic feet per second, W the throat width of the flume, and H_a the head producing the flow. The experimental data yielded the following equations for different sized flumes:

1 foot	$Q = 4\ W\ H_a^{1.527}$
2 "	$Q = 4\ W\ H_a^{1.552}$
3 "	$Q = 4\ W\ H_a^{1.565}$
4 "	$Q = 4\ W\ H_a^{1.574}$
6 "	$Q = 4\ W\ H_a^{1.592}$
8"	$Q = 4\ W\ H_a^{1.608}$

Plotting the exponents against the flume sizes on log paper shows that the general equation linking the exponents is of the form $n = a\ W^k$ where n is the exponent and a and K are constants.

The equation $n = aW^k$ is a non linear function, but it may readily be put in linear form by taking the logrithm of both sides of the equation. Then $\log n = \log a + k \log W$. By substituting the logs of the observed values in the equation we get the observation equations 1 to 6 in Table III in which log a and k are the unknowns. Since there are two unknowns and 6 equations, the 6 equations must be reduced to 2, and since these equations are linear the reduction may be made in the same way as in the last example, i.e., by multiplying each equation by the coefficient of the first unknown in the equation thus forming 6 new equations which are added together to form a normal equation and then by multiplying each observation equation by the coefficient of the second unknown in the equation and adding to form the second normal equation.

In this example the coefficient of log a is one in each case, so the observation equations 1 to 6 will not be changed by this operation. Multiplying by the coefficients of the second unknown gives the equations 7 to 12. The two normal equations formed

TABLE III

Determination of Free Flow Discharge Formula for Improved Venturi Flume

Observed				Least Square			Graphic		
W	log W	n	log n	n	v	v^2	n	v	v^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	.00000	1.527	.18384	1.525	-2	4	1.522	-5	25
2	.30203	1.552	.19089	1.553	+1	1	1.550	-2	4
3	.47712	1.565	.19451	1.566	+1	1	1.566	+1	1
4	.60406	1.574	.19700	1.577	+3	9	1.578	+4	16
5	.77815	1.592	.20194	1.593	+1	1	1.595	+3	9
8	.90309	1.608	.20629	1.604	-4	16	1.606	-2	4

$\Sigma v^2 = 32$ $\Sigma v^2 = 59$

The probable error of any exponent is

$$r = 0.6745\sqrt{\frac{\Sigma v^2}{n - q}} = 0.6745\sqrt{\frac{32}{6 - 2}} = \pm 1.9 \text{ or } \pm .0019$$

Observation equations:

$$\begin{aligned}
0.18384 &= \log a + k\,(.00000) && (1)\\
.19089 &= \log a + k\,(.30203) && (2)\\
.19451 &= \log a + k\,(.47712) && (3)\\
.19700 &= \log a + k\,(.60406) && (4)\\
.20194 &= \log a + k\,(.77815) && (5)\\
.20629 &= \log a + k\,(.90309) && (6)\\
\hline
1.17447 &= 6\log a + k\,(3.06445) && \text{(a) Normal equation}
\end{aligned}$$

$$\begin{aligned}
00000 &= 0 \log a + 0k && (7)\\
.05765 &= (.30203) \log a + k\,(.09122) && (8)\\
.09280 &= (.47712) \log a + k\,(.22764) && (9)\\
.11900 &= (.60406) \log a + k\,(.36489) && (10)\\
.15714 &= (.77815) \log a + k\,(.60552) && (11)\\
.18630 &= (.90309) \log a + k\,(.81557) && (12)\\
\hline
.61289 &= (3.06445) \log a + k\,(2.10484) && \text{(B) Normal equation.}
\end{aligned}$$

Solving (A) and (B)

$k = 0.0242$

$a = 1.525$

from their sums are designated by A and B. The solution of the normal equations gives $a = 1.525$ and $k = 0.0242$. These values when substituted in the equation $n = a\,W^k$ give $n = 1.525\,W^{0.0242}$ as the best value of the assumed form of equation for the exponents of the head H_a.

The solution of the equation results in the values given in column 5, Table III, which, when subtracted from the corresponding observed values, give the residuals shown in column 6. The residuals squared and their sum are given in column 7. The probable error of an observation is then obtained by substituting in the formula

$$r = 0.6745\sqrt{\frac{\Sigma v^2}{n - q}}$$ the values for Σv^2, n and q which gives

$$r = 0.6745\sqrt{\frac{32}{6 - 2}} = \pm 1.9$$

that is, the probable error in any exponent is ± 0.0019.

The formula $Q = 4\,W\,H_a^{1.522\,W^{0.026}}$ was determined graphically from the log plot of the data. It will be noted that it differs slightly from the least square equation. Exponents computed from the portion $1.522W^{0.026}$ are given in columns 8 of Table III, with their residuals and squares in columns 9 and 10. Comparison of the sums of the squares of the two sets of residuals shows that although the difference is small, the least square method gives the better fitting equation.

In the examples given, all the observations were of equal weight. In case the observations are of different weights, the procedure is the same except that the observation equations are each multiplied by the weight of the observation in addition to the coefficient of the unknown and in finding the probable error the summation of the weights by the residuals squared is used instead of the summation of the residuals squared.

So far in the discussion no attention has been given to conditioned observations, i.e., those that are limited by rigorous conditions such as, that the sum of the angles of a plane triangle must equal 180 degrees exactly. Adjustments of this type are made by solving each conditioned equation for an unknown in terms of the remaining unknowns and then substituting these values in the observation equations. The normal equations are then formed as before. By this method we get the solution of all the solutions exactly satisfying the conditioned equations that make the sums of the squares of the residuals a minimum.

APPENDIX

For Applying Chauvenet's Criterion

n	t
3	2.05
4	2.27
5	2.44
6	2.57
7	2.67
8	2.76
9	2.84
10	2.91
11	2.96
12	3.02
13	3.07
14	3.12
15	3.16
16	3.19
17	3.22
18	3.26
19	3.29
20	3.32
21	3.35
22	3.38
23	3.41
24	3.43
25	3.45
30	3.55
40	3.70
50	3.82
75	4.02
100	4.16
200	4.48
500	4.90