UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

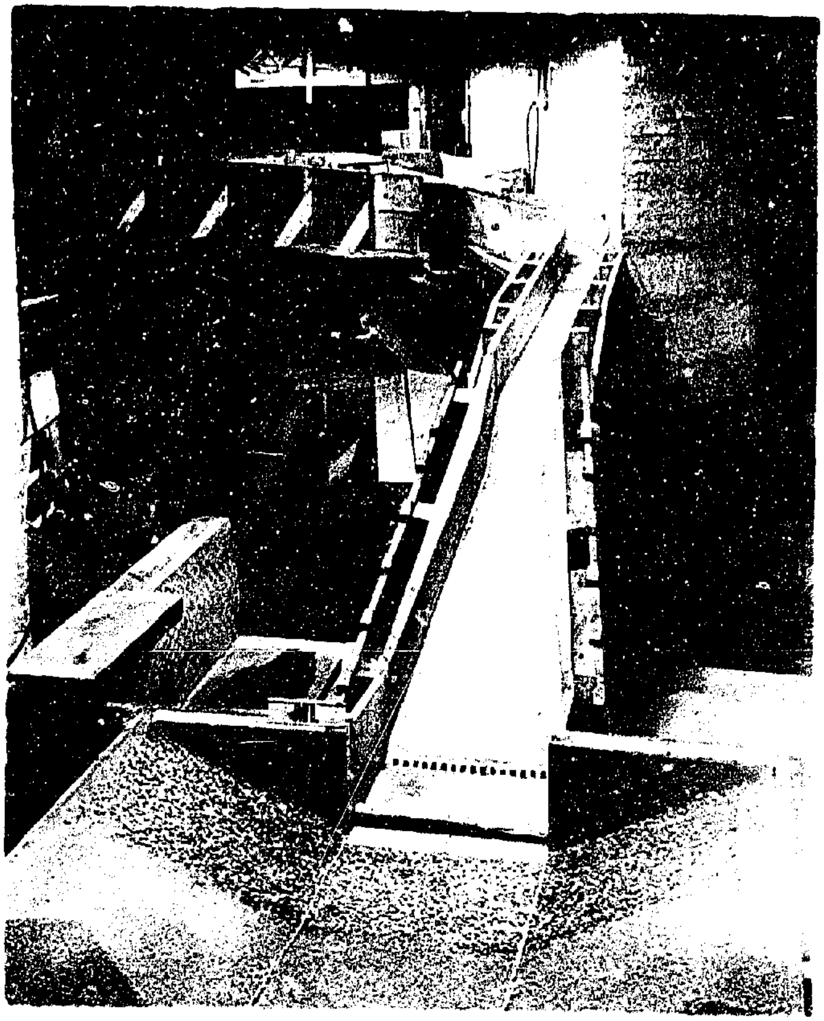
HYDRAULIC LABORATORY REPORT NO. 158

HYDRAULIC MODEL STUDIES FOR THE DESIGN OF THE SPILLWAY AND AUTOMATIC SPILLWAY GATES AT MOON LAKE DAM - MOON LAKE PROJECT

By

J. E. WARNOCK

Denver, Colorado
October 25, 1944



SPILLWAY FOR MOON LAKE DAM
MODEL OF FINAL DESIGN

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COMPENSATION FOR DISPROPORTIONATE FRICTIONAL RESISTANCE
IN OPEN-CHANNEL HYDRAULIC MODELS

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UNITED STATES
DEPARTMENT OF THE INTERIOR
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Branch of Design and Construction
Engineering and Geological Control
and Research
Denver, Colorado
October 25, 1944

Laboratory Report No. 158
Hydraulic Laboratory

Compiled by: J. E. Warnock

Subject: Hydraulic model studies for the design of the spillway and automatic spillway gates at Moon Lake Dam - Moon Lake Project.

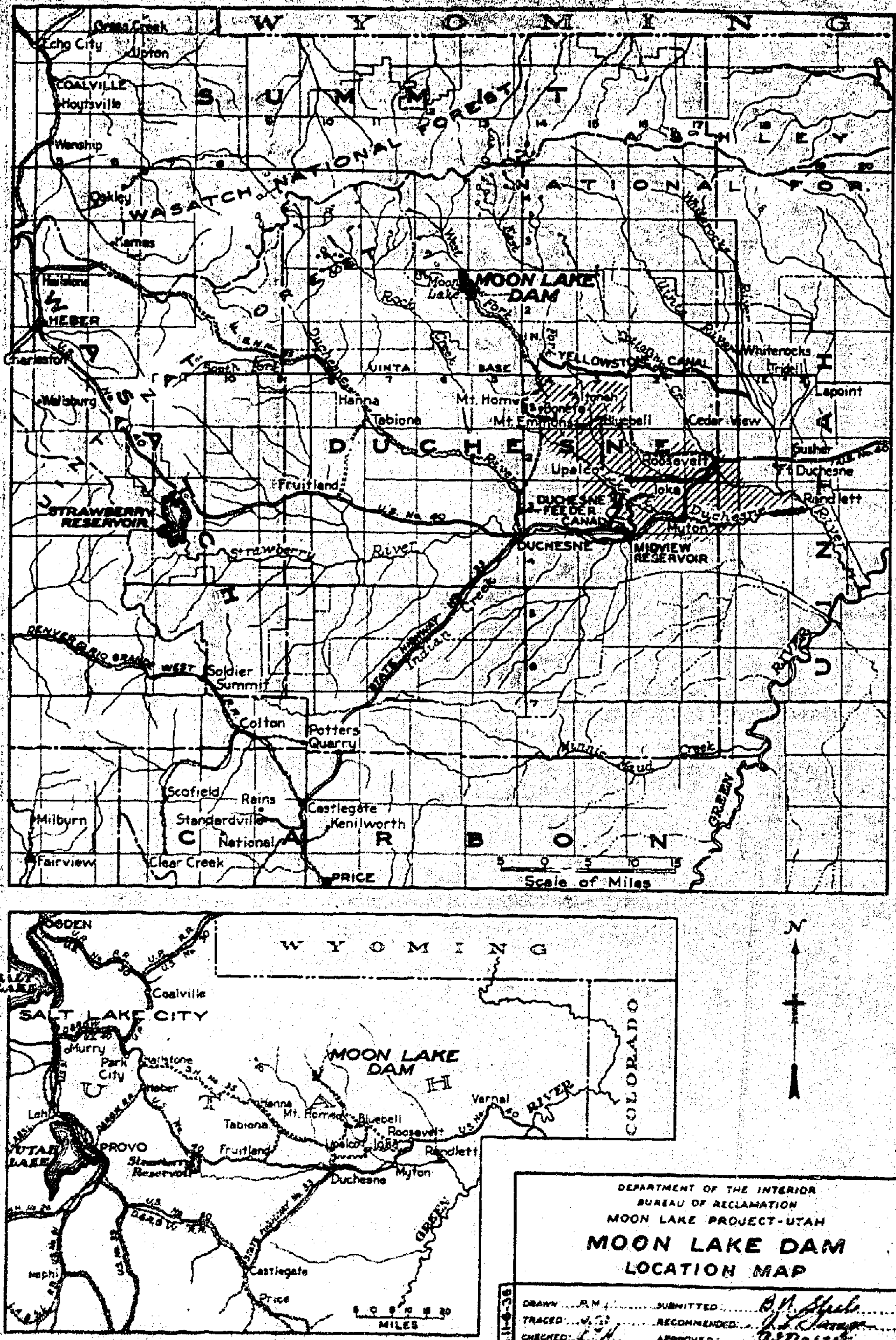
INTRODUCTION

1. Description of project. The Moon Lake Project is located on the West Fork of the Lake Fork of the Duchesne River in northeastern Utah, approximately 32 miles north of the town of Duchesne, Utah, and about 75 miles east and slightly south of Salt Lake City, Utah. The project (figure 1) was constructed to provide a reservoir for storage of the surplus water in the West Fork to be distributed to the irrigated farm lands downstream on the Lake Fork and on the Duchesne River.

The Moon Lake Dam, constructed at the lower end of the original Moon Lake, is an earth-fill dam. The maximum height of the dam above the river bed is about 90 feet. The crest length is approximately 1,120 feet. The increase in the storage capacity of Moon Lake is 30,000 acre-feet. The spillway for the passage of flood waters has an estimated maximum flow of 10,000 second-feet with the reservoir at elevation 8137.0.

2. Original side-channel spillway design. During the design of the hydraulic structures for the project, two different plans for the spillway were developed. The first design (figures 2 and 3) had a concrete, overflow crest 220 feet in length and a side-channel which spilled into a 800-foot length of concrete-lined tunnel. This tunnel discharged into the river channel, and the erosive energy of the floodwaters was minimized by a spray pool (figure 3D) at the downstream end of the tunnel. The model studies made to develop that particular design were described in technical memorandum No. 437, "Hydraulic Model Experiments for the Design of Moon Lake Spillway," by J. N. Bradley and J. B. Drisko.

3. Open-channel spillway design. When construction was started at the site, a very poor rock structure was found in the foundation for the side-channel spillway. This development made it necessary to



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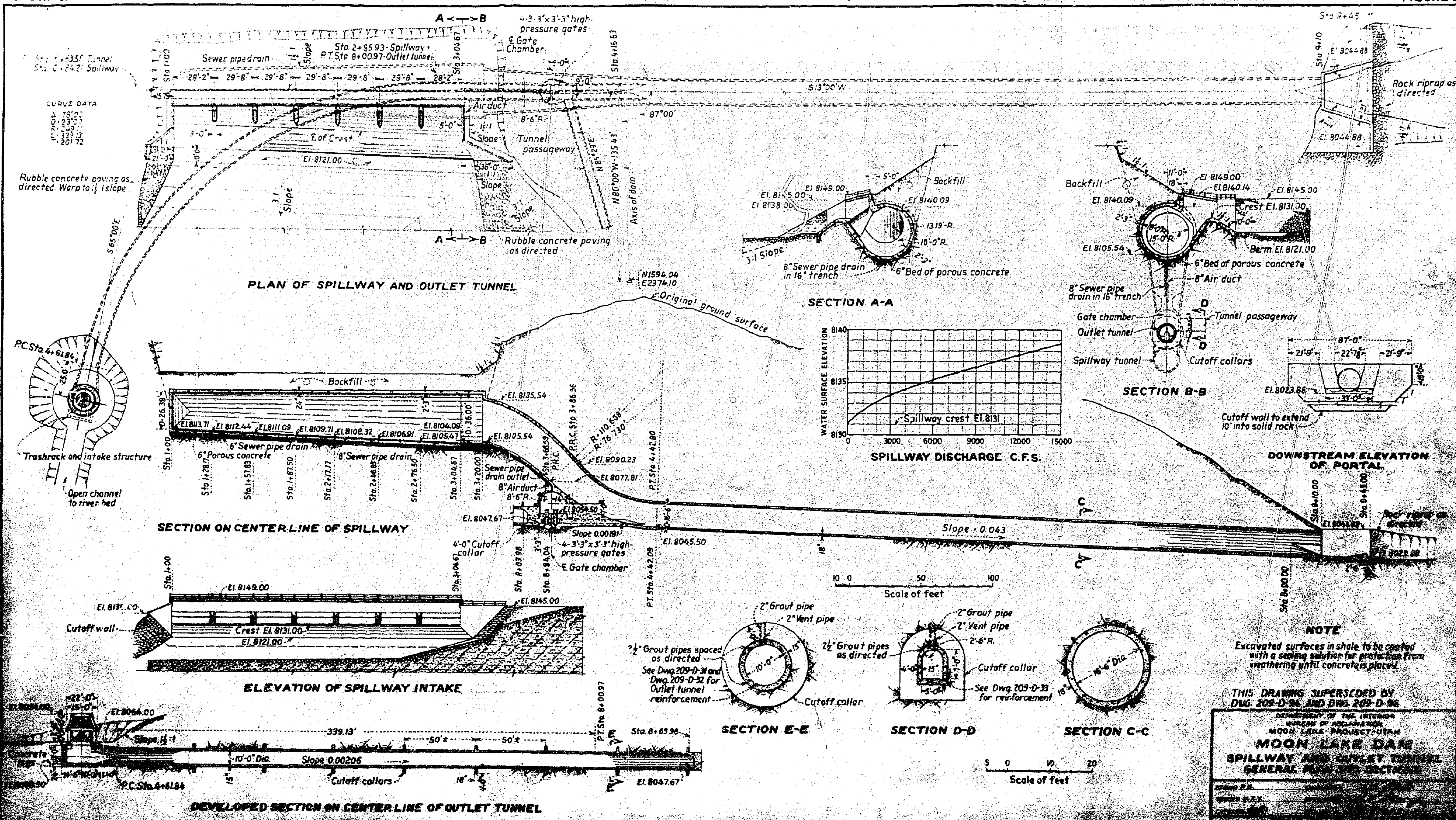
MOON LAKE PROJECT-UTAH

MOON LAKE DAM
LOCATION MAP

DRAWN R.M.	SUBMITTED B.N. Steele
TRACED	RECOMMENDED
CHECKED	APPROVED
25971	DENVER, COLO. DEC. 19, 1934

REV. 11-6-36

203-D-21


PLAN OF SPILLWAY AND OUTLET TUNNEL
SECTION A-A
SECTION B-B
SPILLWAY DISCHARGE C.F.S.
WATER SURFACE ELEVATION
Spillway crest El. 8131
DOWNSTREAM ELEVATION OF PORTAL
SECTION ON CENTER LINE OF SPILLWAY
ELEVATION OF SPILLWAY INTAKE
DEVELOPED SECTION ON CENTER LINE OF OUTLET TUNNEL
SECTION E-E
SECTION D-D
SECTION C-C
CURVE DATA
Trashrack and intake structure
Open channel to river bed
Rubble concrete paving as directed. Warp to 1½:1 slope
Rubble concrete paving as directed
Original ground surface
Backfill
Gate chamber
Outlet tunnel
Spillway tunnel
Tunnel passageway
Cutoff collars
Cutoff wall to extend 10' into solid rock
Rock riprap as directed
6" Bed of porous concrete
8" Sewer pipe drain in 16" trench
8" Air duct
4-3'-3"x3'-3" high-pressure gates
Crest El. 8131.00
El. 8121.00
El. 8149.00
El. 8145.00
Slope 0.043
Slope 0.00206
Scale of feet
NOTE
Excavated surfaces in shale to be coated with a sealing solution for protection from weathering until concrete is placed.
THIS DRAWING SUPERSEDED BY DWG. 209-D-94 AND DWG. 209-D-96
DEPARTMENT OF THE INTERIOR
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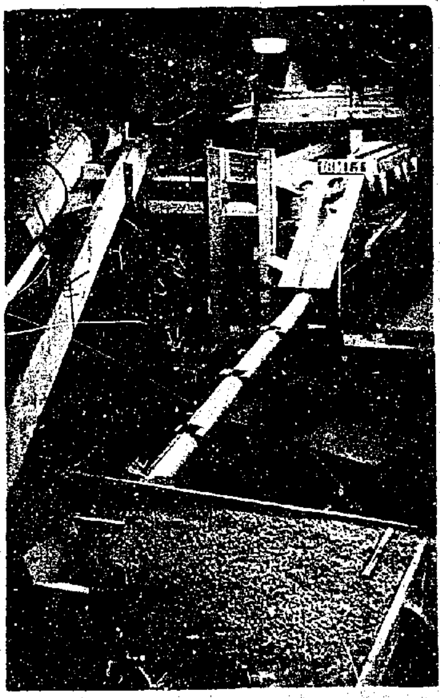
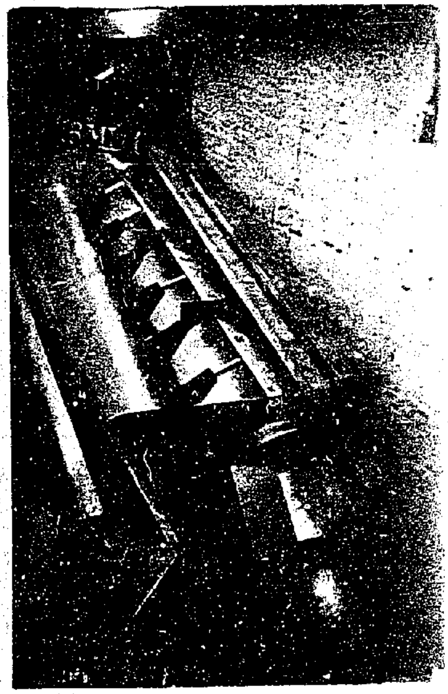
MOON LAKE PROJECT-UTAH
MOON LAKE DAM
SPILLWAY AND OUTLET TUNNEL
GENERAL PLAN AND SECTIONS

Figure 3.

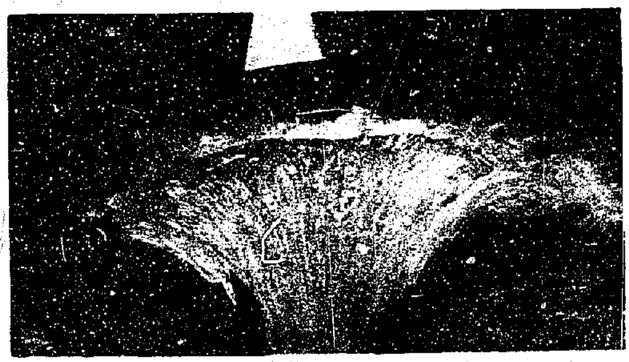


A. GENERAL VIEW OF MODEL



B. SPILLWAY - DISCHARGE 10,000 SECOND-FEET

C. VIEW LOOKING UPSTREAM THROUGH
PORTAL OF CHUTE



D. PORTAL SPRAY POOL - DISCHARGE 10,000 SECOND-FEET

RECOMMENDED DESIGN OF SPILLWAY
PLAN 1

prepare new plans. An open-channel spillway, controlled by two 24- by 16-foot automatic radial gates was designed to be located on the west end of the dam. The new design (figure 4) included a concrete, rectangular channel terminating in a chute with diverging walls and a stilling pool. The total difference in head between the reservoir surface of elevation 8137.0 and the tailwater is approximately 105 feet.

The results procured from the studies of the model of the open-channel spillway are presented in this memorandum.

4. Automatic spillway-gate design. The Moon Lake automatic spillway gates are of the radial type, 24 feet long and 16 feet high, designed to maintain by float control a practically constant reservoir water-surface elevation for all discharges through the gate. To determine the size of the float and the bleeders from the float wells, the problem was studied in an hydraulic model. The results are described in sections 22 to 27, inclusive.

5. Laboratory equipment and personnel. The studies on the model of the open-channel spillway were made in 1936 in the Denver hydraulic laboratory of the Bureau of Reclamation, at that time located in the basement of the Old Customhouse, 16th and Arapahoe streets, Denver, Colorado, by F. L. Panuzio, Junior Engineer, under the direct supervision of J. B. Drisko, Assistant Engineer, assisted by H. M. Martin, Assistant Engineer, in charge of construction. The studies were carried out under the direction of J. E. Warnock, Research Engineer.

The studies on the automatic spillway-gate model were made in the new Denver hydraulic laboratory of the Bureau of Reclamation in the basement of the Customhouse, 19th and Stout Streets, Denver, Colorado, in 1937, by V. L. Streeter, Assistant Engineer, under the direction of J. E. Warnock, Research Engineer.

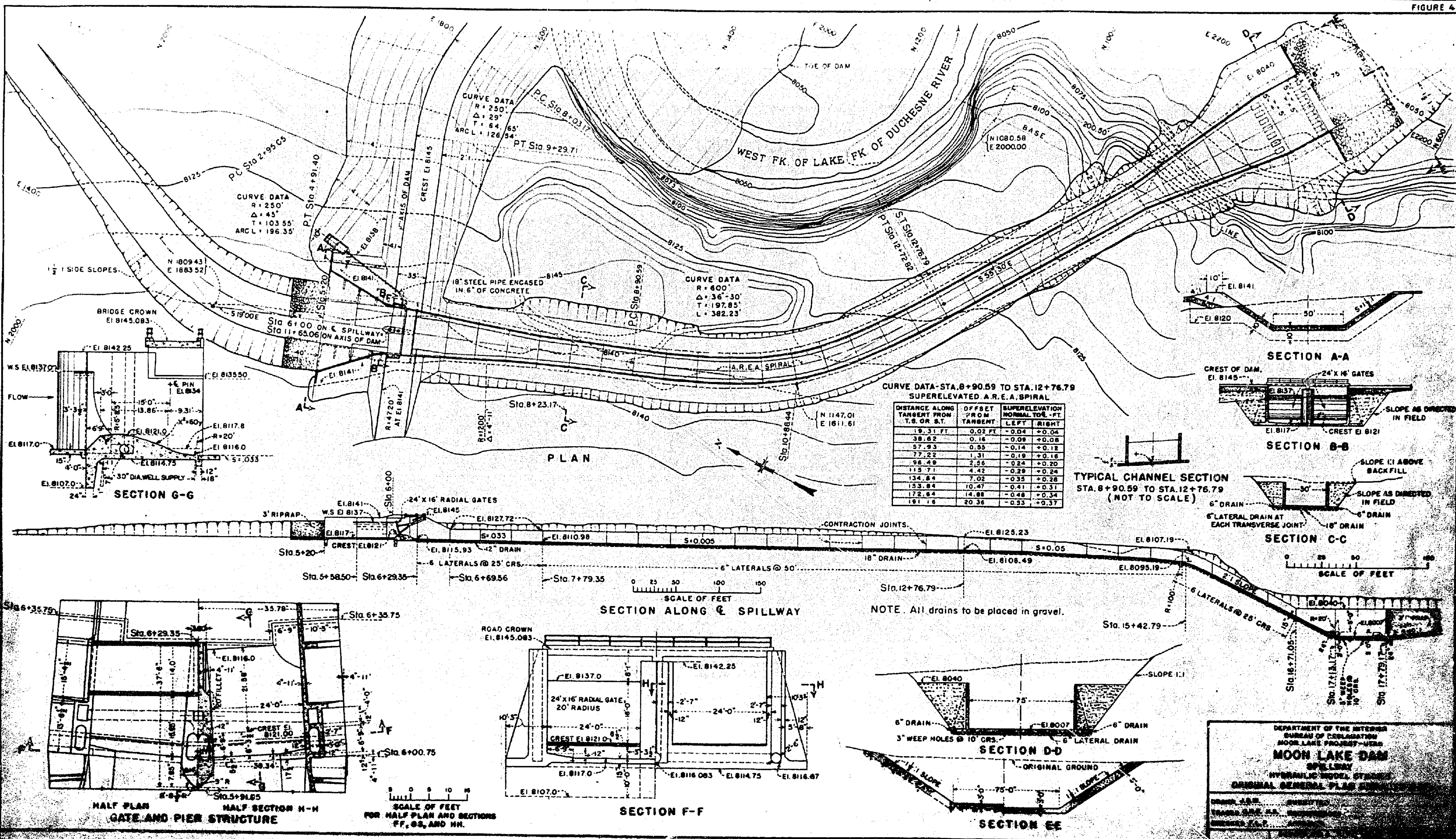
SUMMARY

6. Results and conclusions of open-channel spillway studies. The preliminary studies of the model of the open-channel spillway produced the following results:

(1) The flow in the approach and the crest section and through the superelevated spiral curve was satisfactory, but the flow immediately downstream from the radial gates was too near critical depth being such that a slight disturbance to the flow or a small increase in the discharge above the maximum would cause the formation of an hydraulic jump which would interfere with the full operation of radial-crest gates with the counterbalances.

(2) The pressures over the crest were positive for all discharges, and a coefficient of discharge of 3.27 was measured for the maximum discharge of 10,000 second-feet.

FIGURE 4



CURVE DATA-STA. 8+90.59 TO STA. 12+76.79
SUPERELEVATED A.R.E.A. SPIRAL

DISTANCE ALONG TANGENT FROM T.S. OR S.T.	OFFSET FROM TANGENT	SUPERELEVATION NORMAL TO ℄ - FT. LEFT	SUPERELEVATION NORMAL TO ℄ - FT. RIGHT
19.31 FT.	0.02 FT.	-0.04	+0.04
38.62	0.16	-0.09	+0.08
57.93	0.55	-0.14	+0.12
77.22	1.31	-0.19	+0.16
96.49	2.56	-0.24	+0.20
115.71	4.42	-0.29	+0.24
134.84	7.02	-0.35	+0.28
153.84	10.47	-0.41	+0.31
172.64	14.88	-0.48	+0.34
191.16	20.36	-0.53	+0.37

(3) Indications were that the angle of divergence of the 2:1 chute approaching the stilling pool could be increased.

(4) The stilling-pool floor, located at elevation 8007.0, was too high to allow a satisfactory dissipation of energy, such as to prevent excessive scour of the river bed, with a tailwater of less than elevation 8028.0 for the maximum designed discharge of 10,000 second-feet.

(5) The slope of the excavation in the stream bed downstream from the stilling pool was not stable with the maximum discharge and its corresponding tailwater.

In the final design studied on the model, the slope in the transition downstream from the radial gates was increased to make the flow more stable against the formation of an hydraulic jump in that section.

Different angles of divergence in the 2:1 chute were tested, and it was recommended that the angle be increased from 6° 27' to 8° 32' from the center line. This recommendation was not accepted, for structural reasons.

Since the probable range of tailwater was doubtful due to change of river control below the dam, two possible ranges were studied in conjunction with the maximum discharge of 10,000 second-feet, a range from elevation 8023.0 to 8028.0, and a range from elevation 8026.0 to 8028.0. In the first case, the stilling-pool floor was located at elevation 8002.0; in the latter case at elevation 8005.0. In either case a stepped apron with a height of 2.6 feet was placed at the toe of 2:1 slope, and a modified Rehbock sill was placed with its upstream edge 45 feet downstream from the toe of the 2:1 slope.

It is believed that the slopes of the sides of the excavated area, in the stream bed downstream from the stilling pool, should have been decreased from 1-1/2:1 to 2:1 to increase the stability with the maximum anticipated discharge of 10,000 second-feet.

7. <u>Results and conclusions of automatic spillway-gate studies.</u> A float-controlled gate will operate satisfactorily for upstream water-surface control. To design such a gate two methods are available: (1) use of a scaled model, and (2) solution of the two simultaneous differential equations given in section 23. Where an hydraulic laboratory is available, the model study is more satisfactory than an analytical study, and for the amount of information to be gained, much cheaper. The analytical method is very slow. It frequently takes one man two weeks, or more, to compute the action of the gate for one assumed flood. This same flood, or any other flood, may be observed in a short time when a model is available.

The following general conclusions may be drawn:

(1) Other factors remaining constant, the sensitivity of the gate varies:

(a) inversely as the float-well area,

(b) directly as the length of intake weir,

(c) inversely as the friction,

(d) directly as the area of bleeder opening, and

(e) inversely as the volume of the pipe between the intake structure and the weir.

(2) Wind waves will tend to change the controlled elevation of reservoir. This tendency can be effectively reduced by appropriate damping between the reservoir and the weir. A pipe with considerable volume (e.g. 100 feet of 18-inch pipe) will provide satisfactory damping due to inertia of water in the pipe.

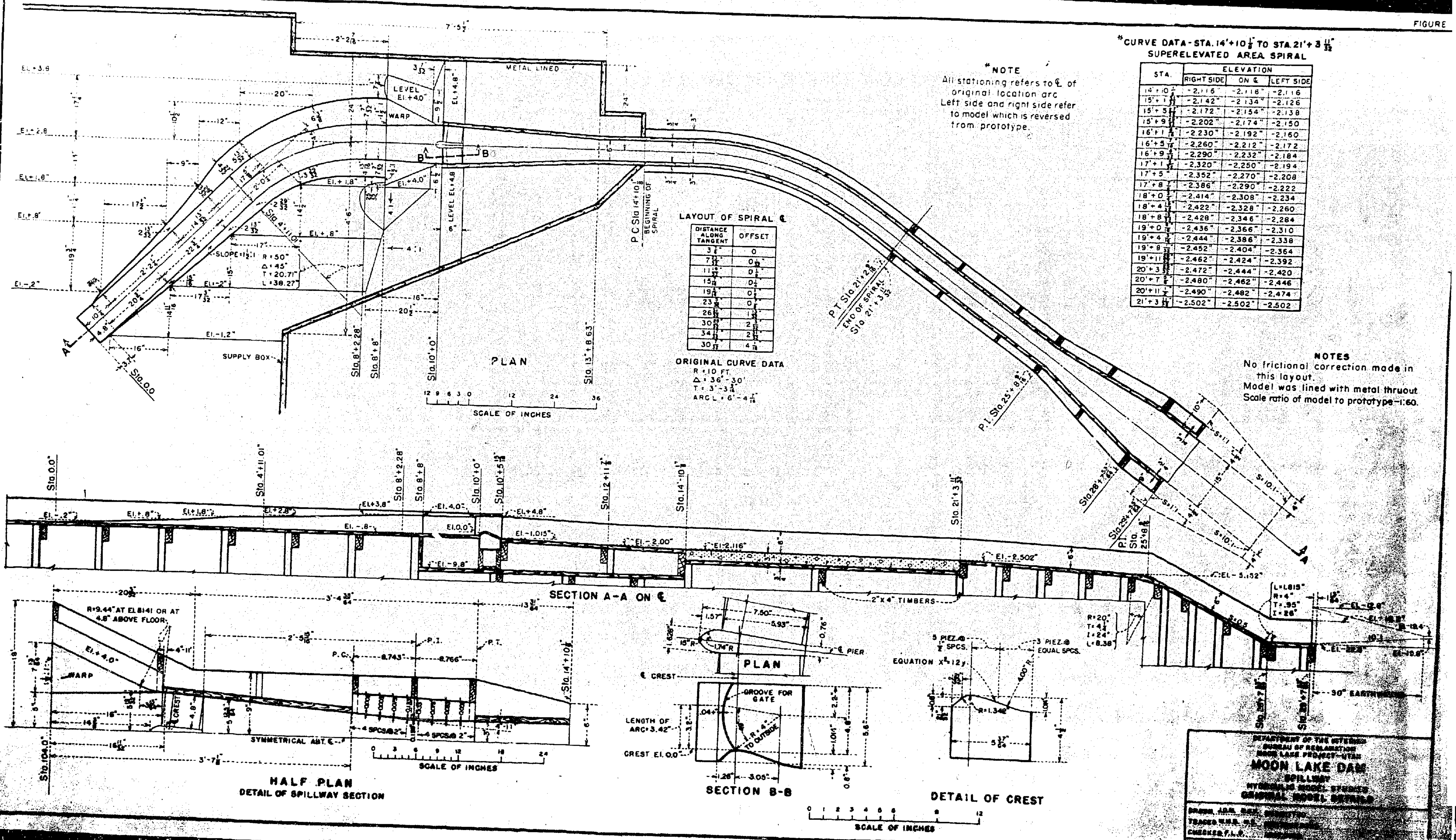
Sensitivity of a gate refers to its quickness of response to changes of reservoir elevation. If a gate is too sensitive it will hunt. If not sensitive enough the reservoir water surface will not be controlled within a narrow range.

FLOW CONDITIONS IN GATE CONTROL SECTION, CHANNEL, AND CHUTE

8. Design and construction of the spillway model. The model to a scale ratio of 1:60 (frontispiece and figure 5) was designed using the information contained in figure 4 as submitted by the design department. It was constructed of lumber and lined with galvanized iron for smoothness and watertightness.

As originally built, there was no allowance made in the model for the excess of frictional resistance over that in the prototype as assumed according to the laws of hydraulic similitude. Subsequent studies of the flow conditions in the transition below the gate section indicated the necessity of a correction for the excess frictional losses so as to obtain velocities in the model comparable to velocities at the same points in the prototype.

9. Correction for excess frictional resistance. When the model was first operated, an hydraulic jump formed in the transition below the radial gates, which was not indicated by the hydraulic computations in the prototype design. This jump in the model was stable at the maximum discharge of 10,000 second-feet and extended up to the downstream end of the gate pier. As the flow was decreased



*CURVE DATA - STA. 14'+10⅛" TO STA. 21'+3 11/32"
SUPERELEVATED AREA SPIRAL

STA.	ELEVATION RIGHT SIDE	ELEVATION ON ℄	ELEVATION LEFT SIDE
14'+10⅛"	-2.116"	-2.116"	-2.116
15'+1 11/32"	-2.142"	-2.134"	-2.126
15'+5 [illegible]"	-2.172"	-2.154"	-2.138
15'+9 [illegible]"	-2.202"	-2.174"	-2.150
16'+1 [illegible]"	-2.230"	-2.192"	-2.160
16'+5 7/16"	-2.260"	-2.212"	-2.172
16'+9 [illegible]"	-2.290"	-2.232"	-2.184
17'+1 [illegible]"	-2.320"	-2.250"	-2.194
17'+5"	-2.352"	-2.270"	-2.208
17'+8 [illegible]"	-2.386"	-2.290"	-2.222
18'+0 [illegible]"	-2.414"	-2.308"	-2.234
18'+4 [illegible]"	-2.422"	-2.328"	-2.260
18'+8 [illegible]"	-2.428"	-2.346"	-2.284
19'+0 [illegible]"	-2.436"	-2.366"	-2.310
19'+4 [illegible]"	-2.444"	-2.386"	-2.338
19'+8 [illegible]"	-2.452"	-2.404"	-2.364
19'+11 [illegible]"	-2.462"	-2.424"	-2.392
20'+3 [illegible]"	-2.472"	-2.444"	-2.420
20'+7 [illegible]"	-2.480"	-2.462"	-2.446
20'+11 [illegible]"	-2.490"	-2.482"	-2.474
21'+3 11/32"	-2.502"	-2.502"	-2.502

*NOTE
All stationing refers to ℄ of original location arc.
Left side and right side refer to model which is reversed from prototype.

LAYOUT OF SPIRAL ℄

DISTANCE ALONG TANGENT	OFFSET
3 [illegible]"	0
7 [illegible]"	0 1/32"
11 [illegible]"	0 [illegible]"
15 [illegible]"	0 [illegible]"
19 [illegible]"	0 [illegible]"
23 [illegible]"	0 [illegible]"
26 [illegible]"	1 [illegible]"
30 [illegible]"	2 [illegible]"
34 [illegible]"	2 [illegible]"
30 [illegible]"	4 [illegible]"

ORIGINAL CURVE DATA
R = 10 FT.
Δ = 36°-30'
T = 3'-3 [illegible]"
ARC L = 6'-4 7/16"

NOTES
No frictional correction made in this layout.
Model was lined with metal thruout.
Scale ratio of model to prototype - 1:60.

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ORIGINAL MODEL DETAILS

the jump moved down the channel and finally disappeared at a point 150 feet downstream from the pier, with a flow of 9,370 second-feet.

With this condition prevalent, it was considered advisable to correct the model for the excess of frictional losses to obtain the correct velocities. The theory and computation to determine the increase of the model slope over that in the prototype is discussed later.

It was concluded, from this as well as other similar studies, that in models of open channels where the flow conditions are determined by frictional losses, a correction should be made to allow for the excess of frictional resistance. This is especially the case in flat slopes, as in the transition in this model. As the slope increases, the effect of gravity increases, and it is believed that the correction is not as important. For ease in construction, the correction is usually applied to the points of change in the case of flat slopes. For the steeper slopes as in the 2:1 chute, the correction is applied by lowering the pool floor the necessary amount. That method makes it possible to preserve the angle of divergence in the chute. The length of chute is, of necessity, increased but the error is very small.

The slope of the spiral channel was corrected for excess frictional losses to make the flow therein dynamically similar, but no correction was made in the superelevation through the spiral curve because the dip of the water surface in that section is a function of the mean velocity, the radius, and the force of gravity.

10. Flow in transition below gates. With the model corrected for excess friction and dynamically similar velocity conditions established, tests were made to determine the discharge quantity at which the hydraulic jump would form in the transition below the gate. This critical point was found to be with a discharge equivalent to 11,130 second-feet in the prototype and the reservoir at elevation 8138.07. With that stage of flow, the jump would not occur of its own accord, but a slight disturbance would cause it to form. Once formed, it would stabilize at a point 5 to 10 feet downstream from the end of the pier, but could be disrupted by slight impetus with the hand. This condition is illustrated in figures 6 and 7.

With flows of greater magnitude, the jump was very stable with the front of the roller at the downstream tip of the gate, as in figure 7B. With a flow of 10,150 second-feet, a standing wave could be formed, as shown in figure 6C, while with flows of less than 10,000 second-feet, a jump could not be formed artificially (figure 6D).

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10. Flow in transition below gates. With the model corrected for excess friction and dynamically similar velocity conditions established, tests were made to determine the discharge quantity at which the hydraulic jump would form in the transition below the gate. This critical point was found to be with a discharge equivalent to 11,130 second-feet in the prototype and the reservoir at elevation 8138.07. With that stage of flow, the jump would not occur of its own accord, but a slight disturbance would cause it to form. Once formed, it would stabilize at a point 5 to 10 feet downstream from the end of the pier, but could be disrupted by slight impetus with the hand. This condition is illustrated in figures 6 and 7.

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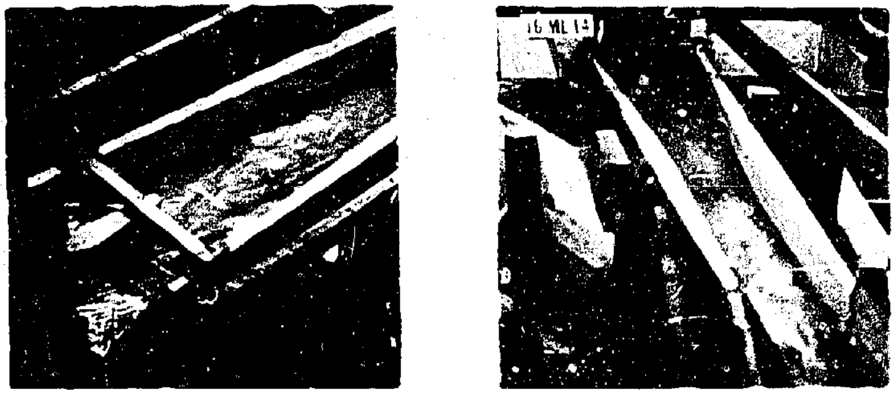
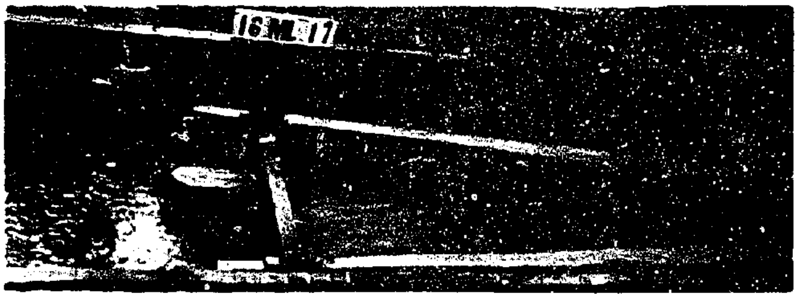
Figure 6.



A. DISCHARGE 11,130 SECOND-FEET
JUMP, IF FORMED, IS STABLE

B. DISCHARGE 11,300 SECOND-FEET
JUMP CAN NOT BE REMOVED

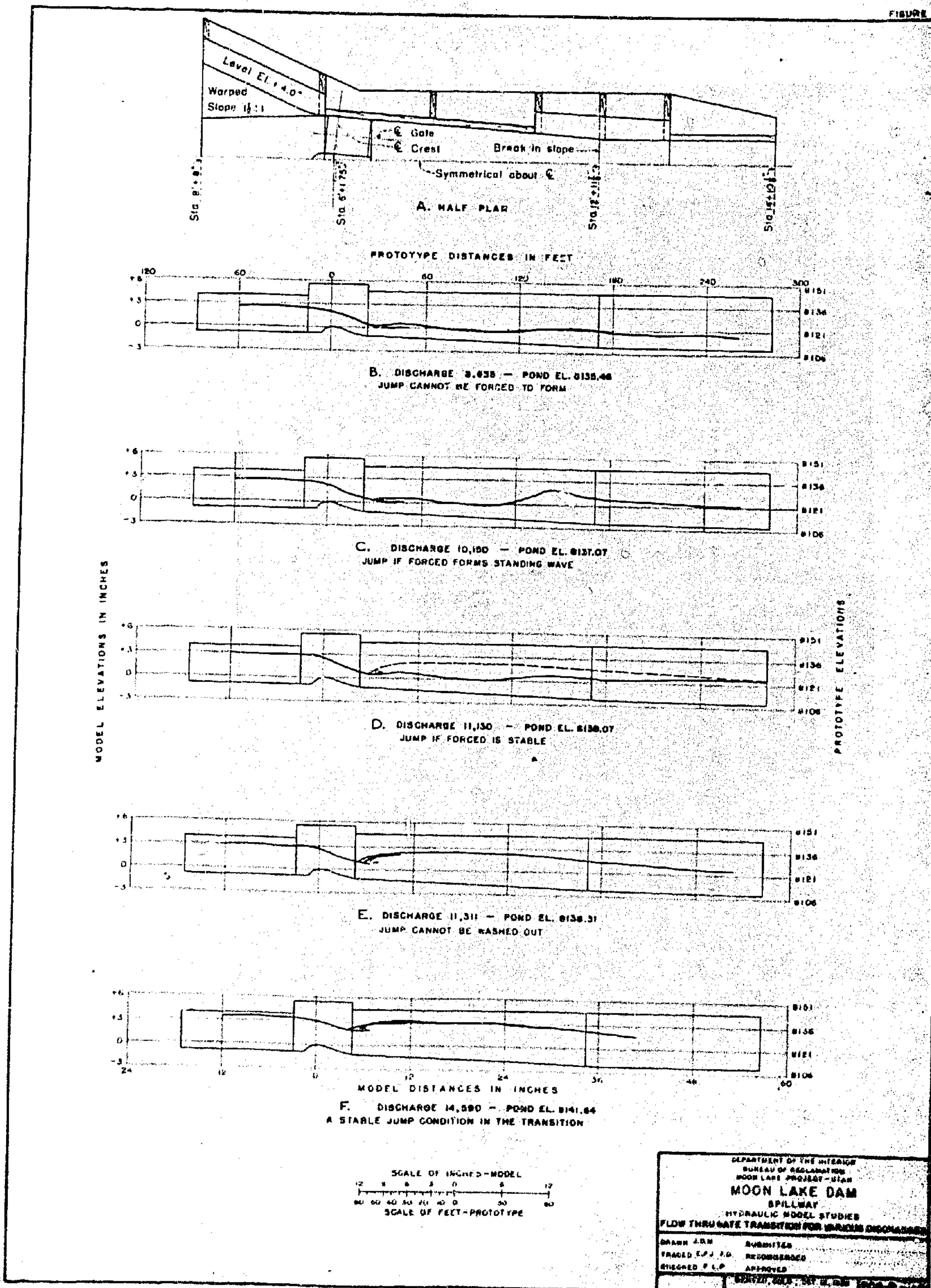


C. DISCHARGE 10,150 SECOND-FEET
STANDING WAVE CAN BE FORMED

D. DISCHARGE 8,630 SECOND-FEET
JUMP CAN NOT BE FORCED

FLOW THROUGH TRANSITION BELOW GATE

FIGURE 7


Level El. + 4.0"
Warped
Slope 1½:1
℄ Gate
℄ Crest
Break in slope
Symmetrical about ℄
A. HALF PLAN
PROTOTYPE DISTANCES IN FEET
B. DISCHARGE — POND EL. 8135.46
JUMP CANNOT BE FORCED TO FORM
C. DISCHARGE 10,180 — POND EL. 8137.07
JUMP IF FORCED FORMS STANDING WAVE
MODEL ELEVATIONS IN INCHES
PROTOTYPE ELEVATIONS
D. DISCHARGE 11,130 — POND EL. 8138.07
JUMP IF FORCED IS STABLE
E. DISCHARGE 11,311 — POND EL. 8138.31
JUMP CANNOT BE WASHED OUT
MODEL DISTANCES IN INCHES
F. DISCHARGE 14,590 — POND EL. 8141.64
A STABLE JUMP CONDITION IN THE TRANSITION
SCALE OF INCHES-MODEL
SCALE OF FEET-PROTOTYPE
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FLOW THRU GATE TRANSITION FOR VARIOUS DISCHARGES
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED

To insure against any possibility of an hydraulic jump forming in the transition, the radius of the vertical walls on each was increased from 1,200 to 1,600 feet and the 0.033 slope of the bottom was continued 60 feet farther downstream before intersecting the 0.005 slope through the spiraled section.

11. <u>Gate crest calibration and pressure measurement.</u> A calibration curve of the discharge capacity of the free crest was prepared by relating the flow as measured in the model by the 90-degree V-notch weir to the head measured on the crest by a point gage. These values were converted to prototype terms using the scale ratio N = 60, by the laws of similitude where,

$$H = Nh \text{ or } 60\ h$$

$$Q = N^{5/2}q \text{ or } 27{,}886\ q$$

$$L = Nl \text{ or } 60\ l$$

where Q and q are the prototype and model discharges, H and h, the prototype and model heads, and L and l, the prototype and model linear dimensions.

The relation between the head on the crest in feet (prototype) and the discharge in second-feet is shown on figure 8 for discharges as large as 17,400 second-feet.

The variation of the coefficient of discharge, C, in the equation $Q = CLH^{3/2}$ with the head on the crest is also shown in figure 8.

The dimensionless coefficient, c, in the equation $Q = cLH\sqrt{2gh}$ related to Froude's number, F, is shown in figure 8. The Froude number was obtained using the equation

$$F = \frac{V^2}{gR}$$

where V = nominal velocity

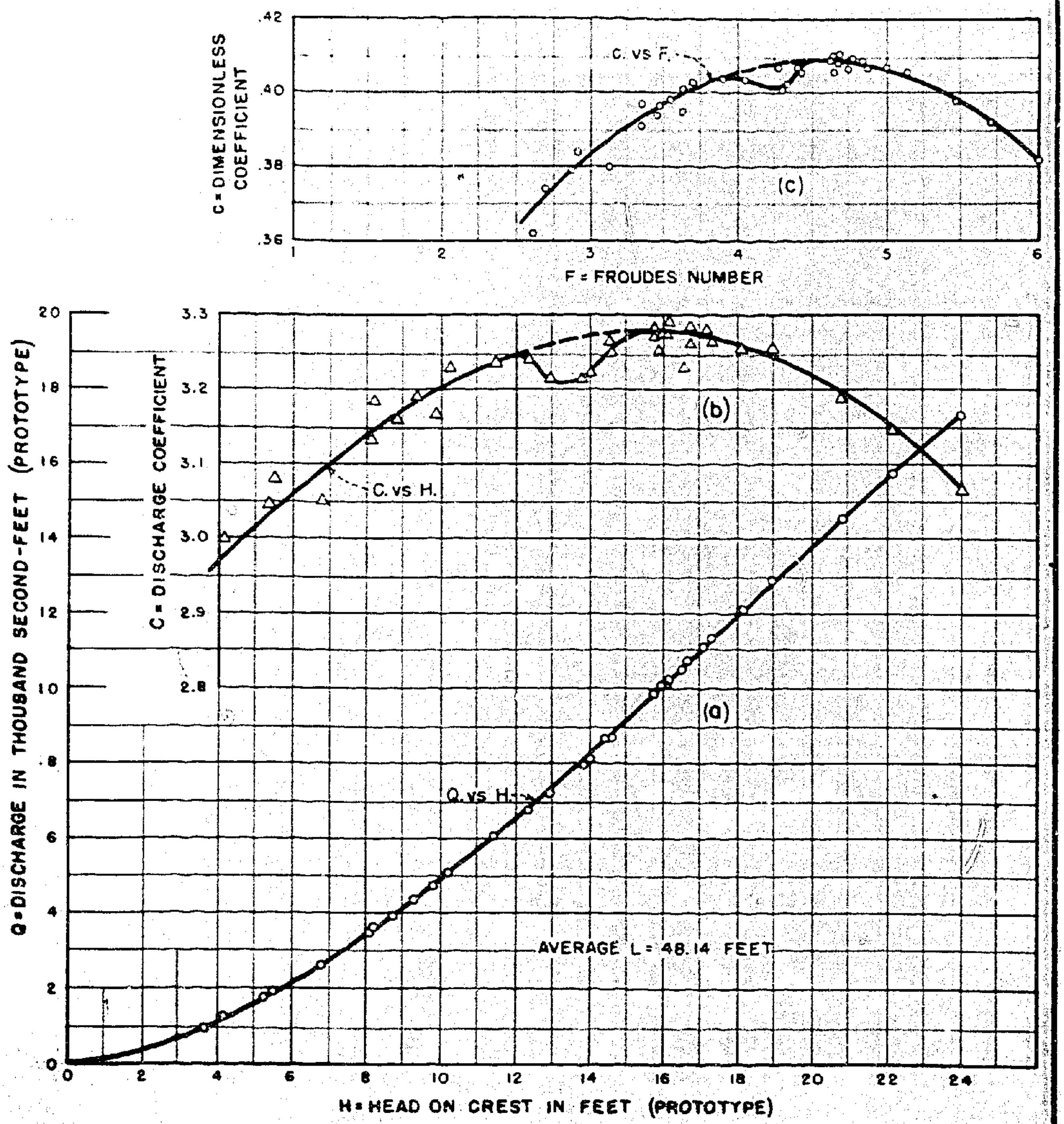
R = nominal hydraulic radius

$= \frac{HL}{4H + L}$ at the crest

g = acceleration due to gravity

The dimensionless coefficient in terms of the discharge coefficient may also be expressed

FIGURE 8



NOTE

$F = \frac{V^2}{gR} = \frac{8H}{L} + 2$

$Q = CLH^{3/2} = CLH\sqrt{2gH}$

WHERE

H = HEAD ON THE CREST

L = EFFECTIVE CREST LENGTH

R = NOMINAL HYD. RADIUS = $\frac{HL}{4H+L}$

V = THEORETICAL VELOCITY = $\sqrt{2gH}$

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MOON LAKE DAM

SPILLWAY

HYDRAULIC MODEL STUDIES

CREST CALIBRATION CURVES

DRAWN F.L.P.	SUBMITTED
TRACED H.M.S. R.F.	RECOMMENDED
CHECKED F.L.P.	APPROVED
DENVER, COLO., SEPT. 13, 1938	209-D-319

$$Q = cLH\sqrt{2gH} = CLH^{3/2}$$

and $$c = \frac{C}{\sqrt{2g}}$$

Piezometers were installed in the crest to measure the pressures with different discharges up to the maximum for the purpose of locating any undesirable negative pressures. The pressures measured are plotted on figure 9. All pressures were positive.

12. Angle of divergence in the chute. Although the flow conditions in the original design of the steep chute were satisfactory, studies were made to determine the effect of different angles of divergence, since an increase in the angle of divergence would allow the use of a shorter length of chute in which to permit the spreading of the jet. Visual observations and measurements of the water-surface profiles served as criteria in determining the effectiveness of each trial. The water-surface measurements are shown in figure 10. The greater depth of water on the left side as compared to that on the right is due to the spiraled channel of approach to the chute. The indications are that the length of the diverging chute could have been reduced to 30 inches or an angle of 8° 32' without materially affecting the conditions in the stilling pool.

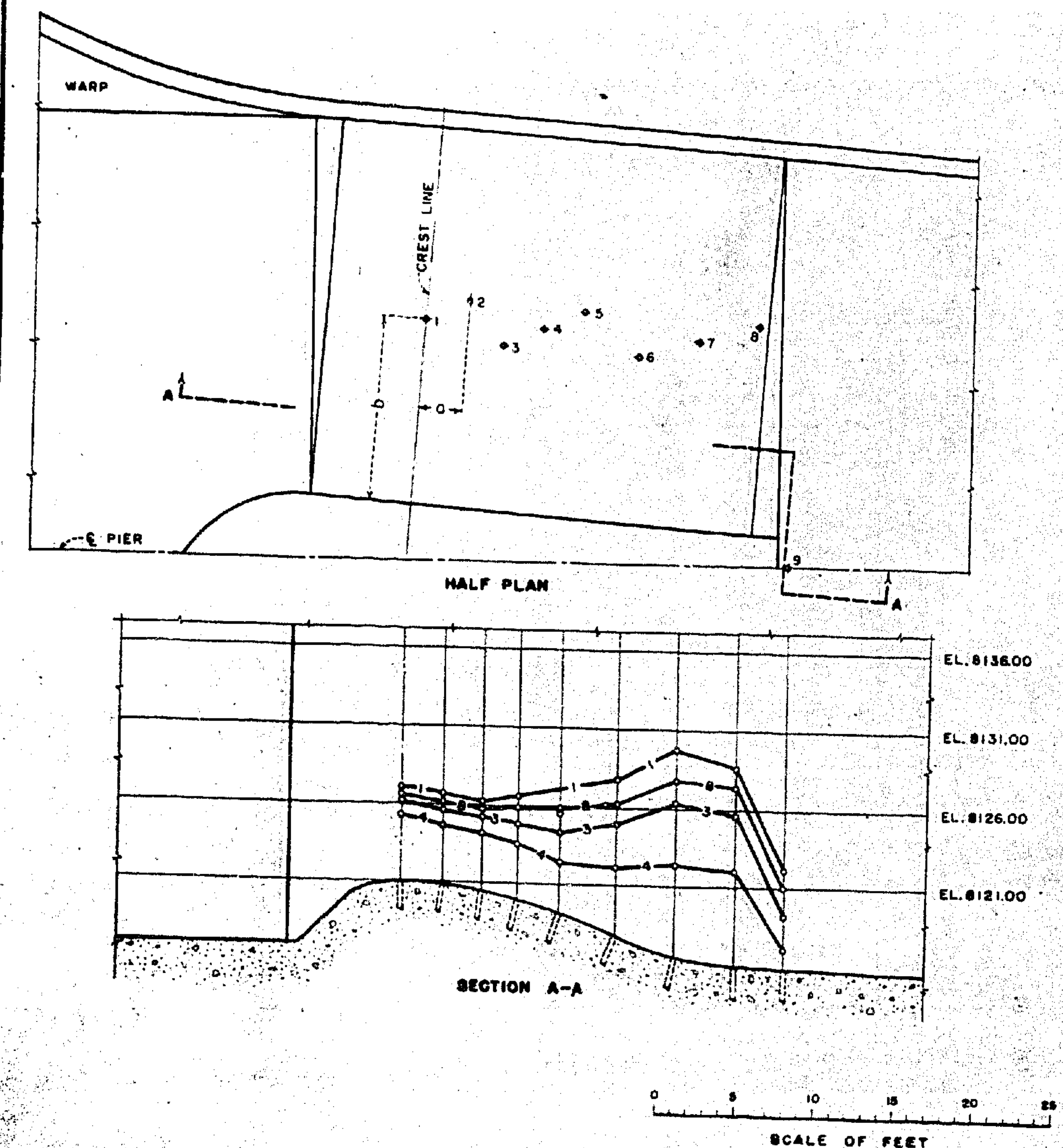
13. Water-surface profiles in channel. During the course of the test program, profiles and sections of the water surface were measured through the entire length of the channel, with flows of 5,885 and 10,050 second-feet, (figure 12).

FLOW CONDITIONS IN STILLING POOL AND STREAM BED

14. Original pool design and stream bed. The floor of the original design of the stilling pool was horizontal at elevation 8007 and was 66 feet in length from its junction with the slope of the chute. The top of the side walls was at elevation 8040 and the walls were parallel to the center line of the pool and 75 feet apart. The walls extended downstream to a point 6 feet upstream from the end of the floor. No sills or other appurtenances were included.

The original design (figure 12A) was duplicated in the model but with the riprap omitted. When the model was operated, it was found that the 1-1/2:1 slopes in the sand bed were not stable. The material in the slopes washed into the center of the pool in a

FIGURE 9



PIEZ. NO.	PROTO. DIMENSIONS	
	a	b
1	0	11.796'
2	2.650'	13.126'
3	4.996'	10.470'
4	7.802'	11.874'
5	9.924'	13.906'
6	13.886'	10.628'
7	17.340'	11.952'
8	21.[illegible]'	13.800'
9	END OF PIER	℄ PIER

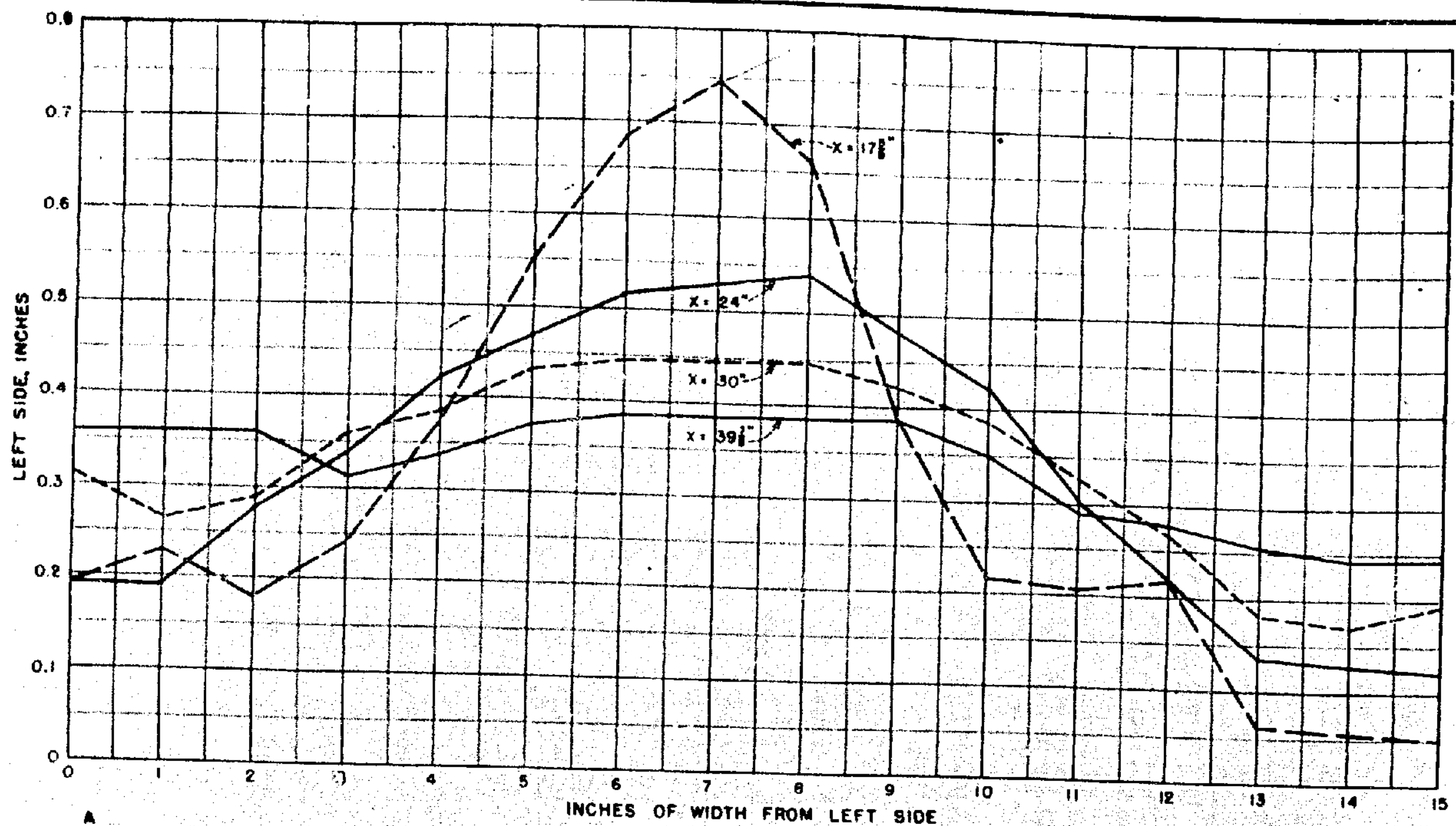
TEST	DISCHARGE	RESERVOIR ELEVATION
3-ML-1	10,058 CFS	8134.95
3-ML-2	7,975 CFS	8134.83
3-ML-3	6,037 CFS	8132.43
3-ML-4	3,589 CFS	8129.17

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT–UTAH

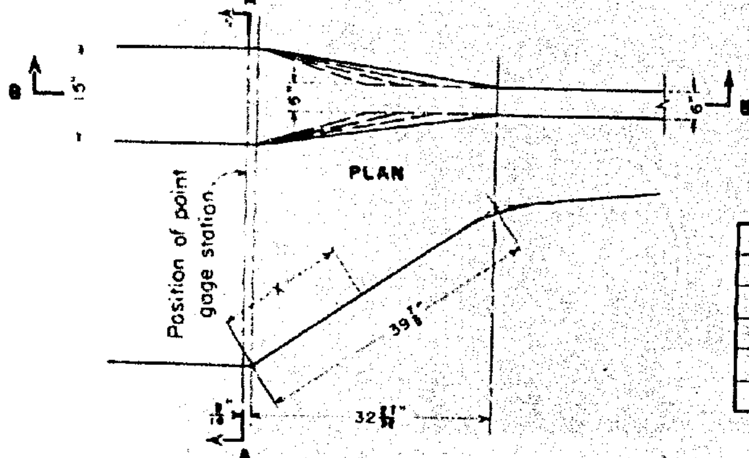
MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES
INDICATED PRESSURES OVER THE CREST

DRAWN J.D.M. ......SUBMITTED ....
TRACED C.F.J. M.H. ...RECOMMENDED ....
CHECKED F.L.P. ......APPROVED ....
DENVER, COLO., SEPT. 17, 1936

209-D-320



MOON LAKE CROSS-SECTION AT SECTION A-A
TO SHOW SPREAD OF JET



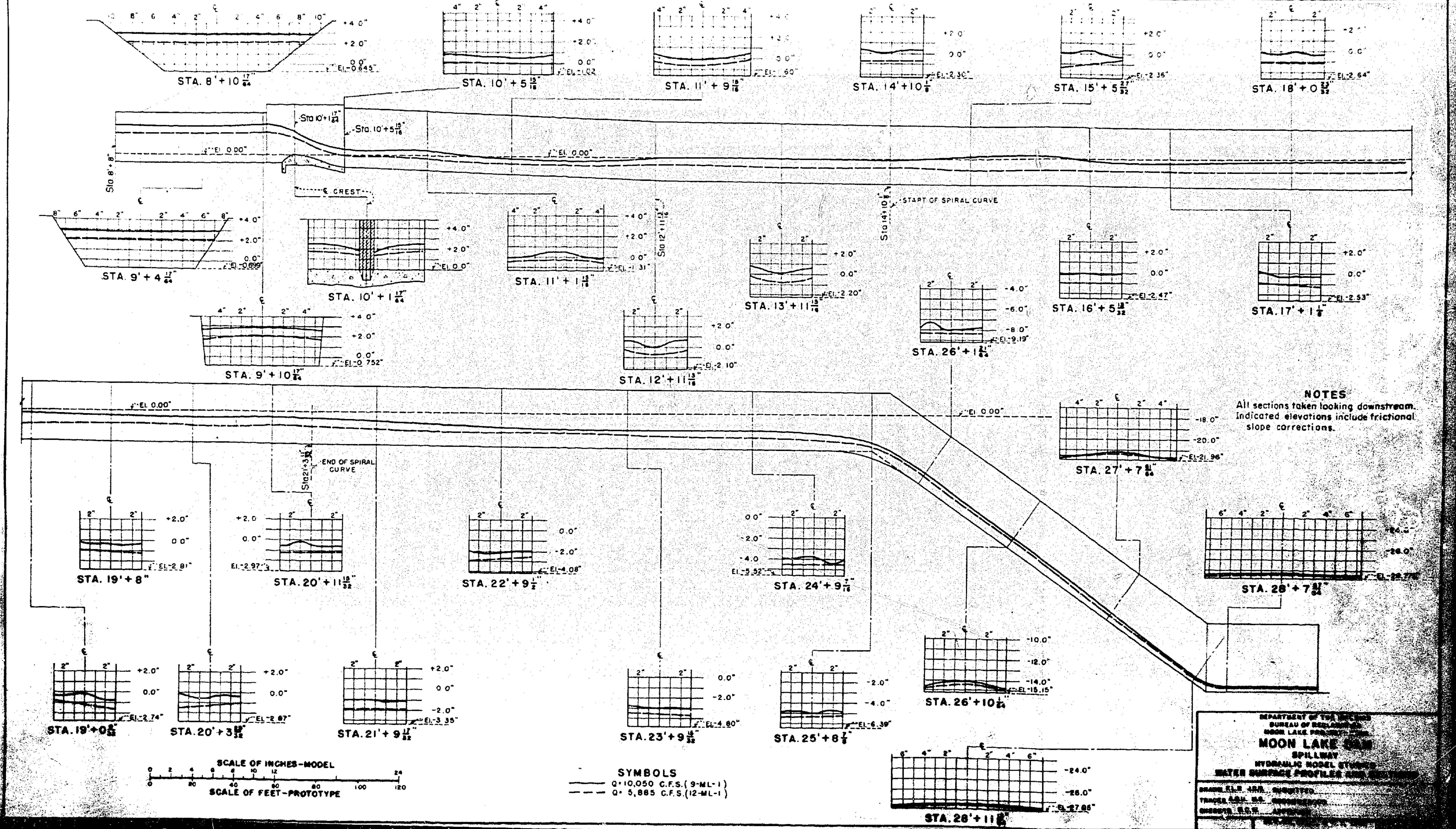
MODEL VALUES			PROTOTYPE VALUE	
X	Q CFS	POND LL	Q CFS	POND LL
39 7/8"	.3580	.2650'	9983	15.90'
30"	.3546	.2628'	9888	15.77'
24"	.3560	.2646'	9930	15.86'
17 5/8"	.3608	.2659'	10053	15.98'

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT-UTAH
MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES
EFFECT OF START OF FLARE ON SPREAD OF JET

DRAWN F.L.P. J.O.M. SUBMITTED
TRACED J.R.G. F.B. RECOMMENDED
CHECKED F.L.P. APPROVED
DENVER, COLO., OCT. 16, 1935 209-D-321

FIGURE 10

FIGURE 17



STA. 8'+10 17/64
STA. 10'+5 15/16
STA. 11'+9 15/16
STA. 14'+10 1/8
STA. 15'+5 27/32
STA. 18'+0 23/32
Sta 8'+8"
Sta 10'+1 17/64
Sta 10'+5 15/16
EL 0.00"
CREST
START OF SPIRAL CURVE
Sta 14'+10 1/8
STA. 9'+4 17/64
STA. 10'+1 17/64
STA. 11'+1 15/16
STA. 13'+11 15/16
STA. 16'+5 15/32
STA. 17'+1 1/8
STA. 26'+1 21/64
STA. 9'+10 17/64
STA. 12'+11 15/16
NOTES
All sections taken looking downstream.
Indicated elevations include frictional slope corrections.
STA. 27'+7 21/64
END OF SPIRAL CURVE
STA. 19'+8"
STA. 20'+11 13/32
STA. 22'+9 1/2
STA. 24'+9 7/16
STA. 28'+7 27/64
STA. 26'+10 1/64
STA. 19'+0 5/32
STA. 20'+3 13/32
STA. 21'+9 17/32
STA. 23'+9 15/32
STA. 25'+8 7/8
STA. 28'+11
SCALE OF INCHES-MODEL
SCALE OF FEET-PROTOTYPE
SYMBOLS
Q=10,050 C.F.S. (9-ML-1)
Q= 5,885 C.F.S. (12-ML-1)
MOON LAKE DAM
SPILLWAY

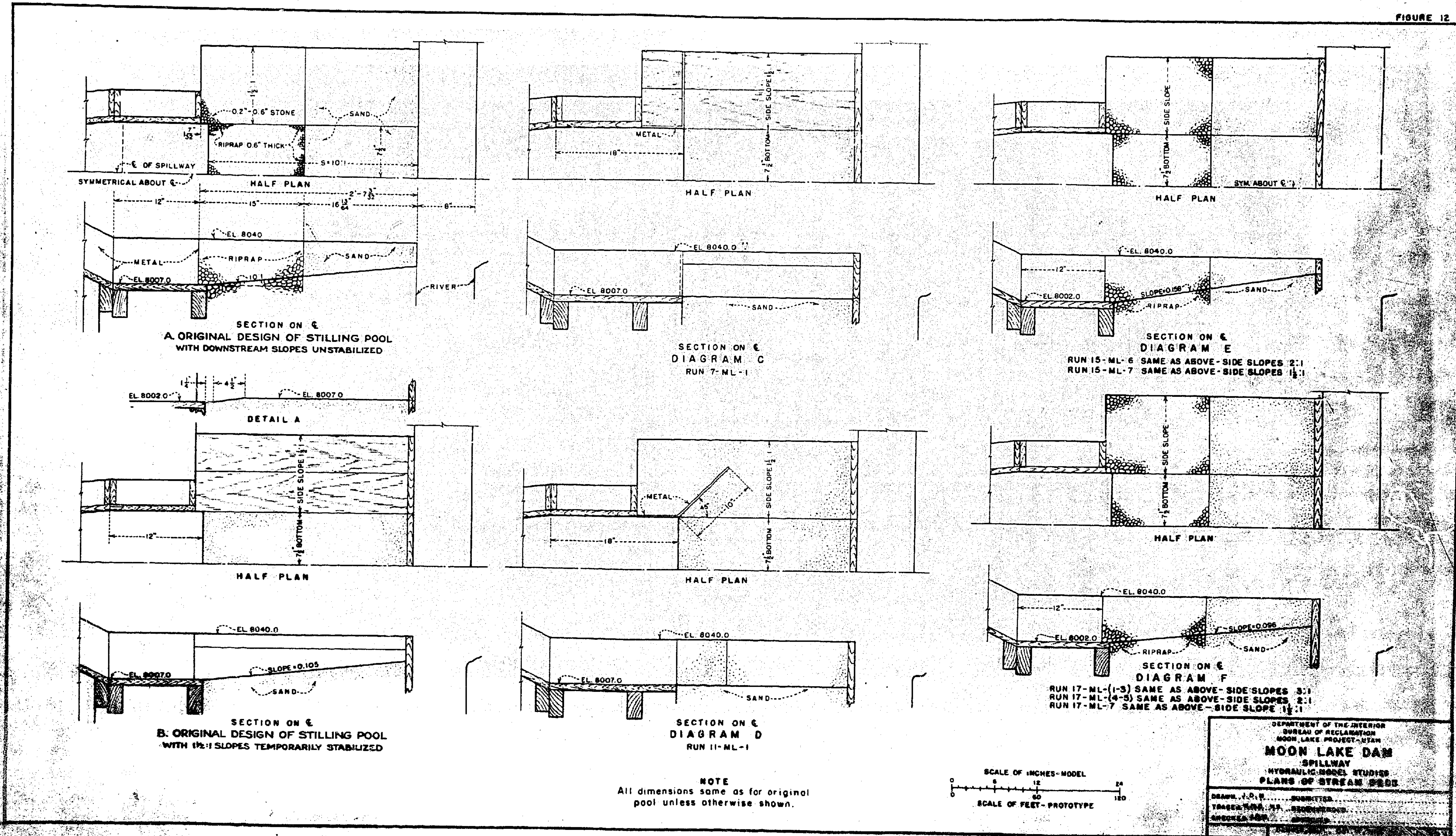


FIGURE 12
0.2"-0.6" STONE
SAND
RIPRAP 0.6" THICK
S=10:1
℄ OF SPILLWAY
SYMMETRICAL ABOUT ℄
HALF PLAN
12"
15"
2'-7 3/32"
16 13/64
6"
EL. 8040
METAL
RIPRAP
SAND
EL. 8007.0
10:1
RIVER
SECTION ON ℄
A. ORIGINAL DESIGN OF STILLING POOL
WITH DOWNSTREAM SLOPES UNSTABILIZED
EL. 8002.0
EL. 8007.0
DETAIL A
HALF PLAN
12"
7 1/2 BOTTOM
SIDE SLOPE 1 1/2:1
EL. 8040.0
EL. 8007.0
SLOPE=0.105
SAND
SECTION ON ℄
B. ORIGINAL DESIGN OF STILLING POOL
WITH 1½:1 SLOPES TEMPORARILY STABILIZED
METAL
18"
SIDE SLOPE 1½:1
7½ BOTTOM
HALF PLAN
EL. 8040.0
EL. 8007.0
SAND
SECTION ON ℄
DIAGRAM C
RUN 7-ML-1
METAL
10"
45°
18"
SIDE SLOPE 1½:1
7½ BOTTOM
HALF PLAN
EL. 8040.0
EL. 8007.0
SAND
SECTION ON ℄
DIAGRAM D
RUN 11-ML-1
NOTE
All dimensions same as for original pool unless otherwise shown.
SIDE SLOPE
7½ BOTTOM
SYM. ABOUT ℄
HALF PLAN
EL. 8040.0
12"
EL. 8002.0
SLOPE=0.156
RIPRAP
SAND
SECTION ON ℄
DIAGRAM E
RUN 15-ML-6 SAME AS ABOVE - SIDE SLOPES 2:1
RUN 15-ML-7 SAME AS ABOVE - SIDE SLOPES 1½:1
SIDE SLOPE
7½ BOTTOM
HALF PLAN
EL. 8040.0
12"
EL. 8002.0
SLOPE=0.096
RIPRAP
SAND
SECTION ON ℄
DIAGRAM F
RUN 17-ML-(1-3) SAME AS ABOVE - SIDE SLOPES 3:1
RUN 17-ML-(4-5) SAME AS ABOVE - SIDE SLOPES 2:1
RUN 17-ML-7 SAME AS ABOVE - SIDE SLOPE 1½:1
SCALE OF INCHES-MODEL
0
6
12
24
0
60
120
SCALE OF FEET-PROTOTYPE
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT-UTAH
MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES
PLANS OF STREAM BEDS
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED

very short time, making it difficult to obtain comparative results in the scour which might occur in the bottom of the pool immediately downstream from the apron. The 1-1/2:1 slopes were temporarily stabilized by the use of wooden slopes, as shown in figures 12B and 13.

Consecutive 15-minute runs were made to determine the total length of time necessary to obtain a stabilized condition. A period of 45 minutes was determined as satisfactory, and subsequent scour tests were made using that period of time.

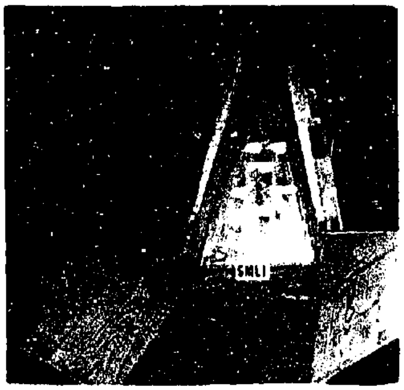
During the preliminary tests, the river bed in the model was held in place by a wooden template at the downstream end. The cross-section of the template was that of the river bed. It soon became evident that this was acting as a control, making it necessary to remove it. The minimum tailwater obtainable in the model with a flow of 10,000 second-feet was elevation 8032.0 whereas the expected tailwater in the prototype would be approximately elevation 8028.0 (figure 14).

In the preliminary tests, several combinations of dentated aprons and sills were tried. The results of two typical combinations which were fairly satisfactory are shown in figures 13E and G, and 15B and C. The details of the steps and sills are shown in figure 16.

These experiments indicated that a fair pool action was obtained with no step or sill on the apron, in which case the bed was only scoured an amount sufficient to allow the formation of a bottom roller. As had been determined in similar models, the dentated step at the bottom of the chute slope at the entrance to the pool was very effective, providing it was of the proper size.

The dentated step breaks the solid sheet of water into small jets and causes them to impinge on the surface roller of the hydraulic jump in the pool near the center of the mass. In addition, there is a certain amount of loss of energy by the impingement of the jets from one elevation against those of the lower elevation.

In this series of tests the best position for a sill was found to be 45 feet downstream from the junction of the chute and the pool. The least scour in the river bed and the least disturbance in the pool occurred when using either a Rehbock sill 2.8 feet high (D_1) or a triangular sill 2.5 feet high with the vertical face downstream (B_1). The Rehbock sill would have been preferable, because of its better action, for the lower range of tailwater.

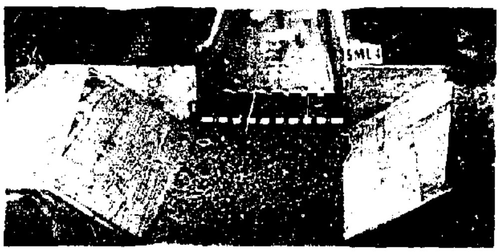


A. INITIAL BED SLOPING FROM
ELEVATION 8007.0 TO 8022.8

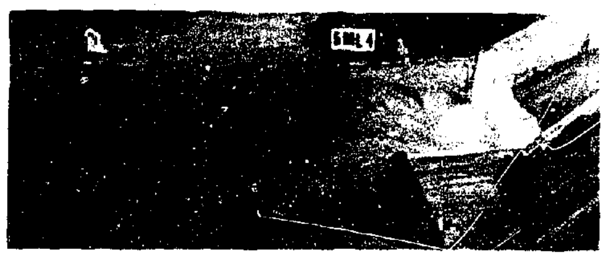
B. ORIGINAL APRON WITH
STEP C_1 AND SILL D_1

C. DISCHARGE 10,234 SECOND-FEET
TAILWATER ELEVATION 8032.0

D. SCOUR IN RIVER BED
AFTER 45 MINUTES



E. APRON WITH STEP C_1 AND
SILL D_1 AT 45-FOOT POINT

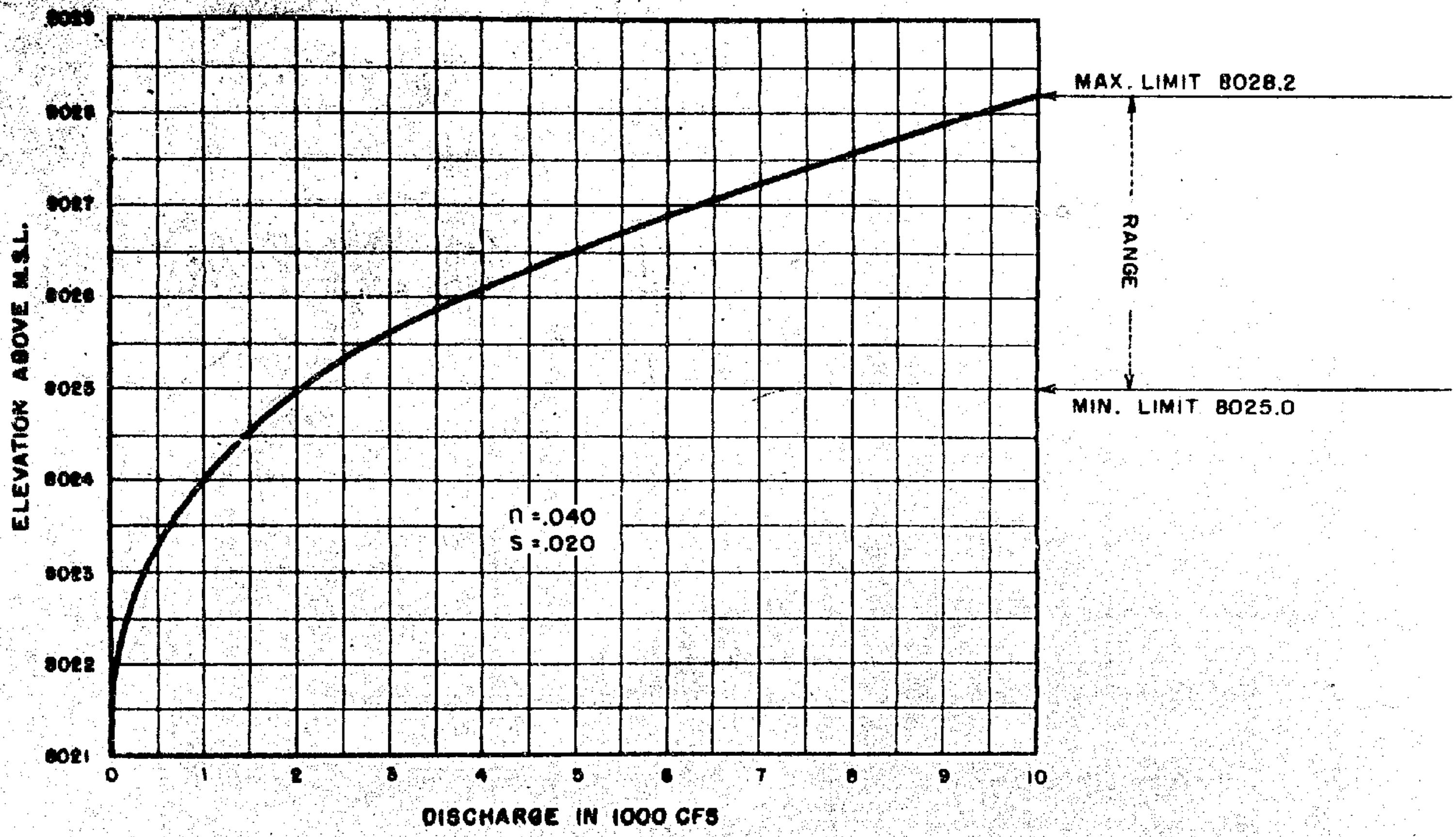


F. DISCHARGE 9,966 SECOND-FEET
TAILWATER ELEVATION 8032.0

G. APRON WITH STEP C_1 AND
SILL B_1 AT 45-FOOT POINT

H. DISCHARGE 10,076 SECOND-FEET
TAILWATER ELEVATION 8032.0

SCOUR AND ACTION OF POOL
ORIGINAL BED



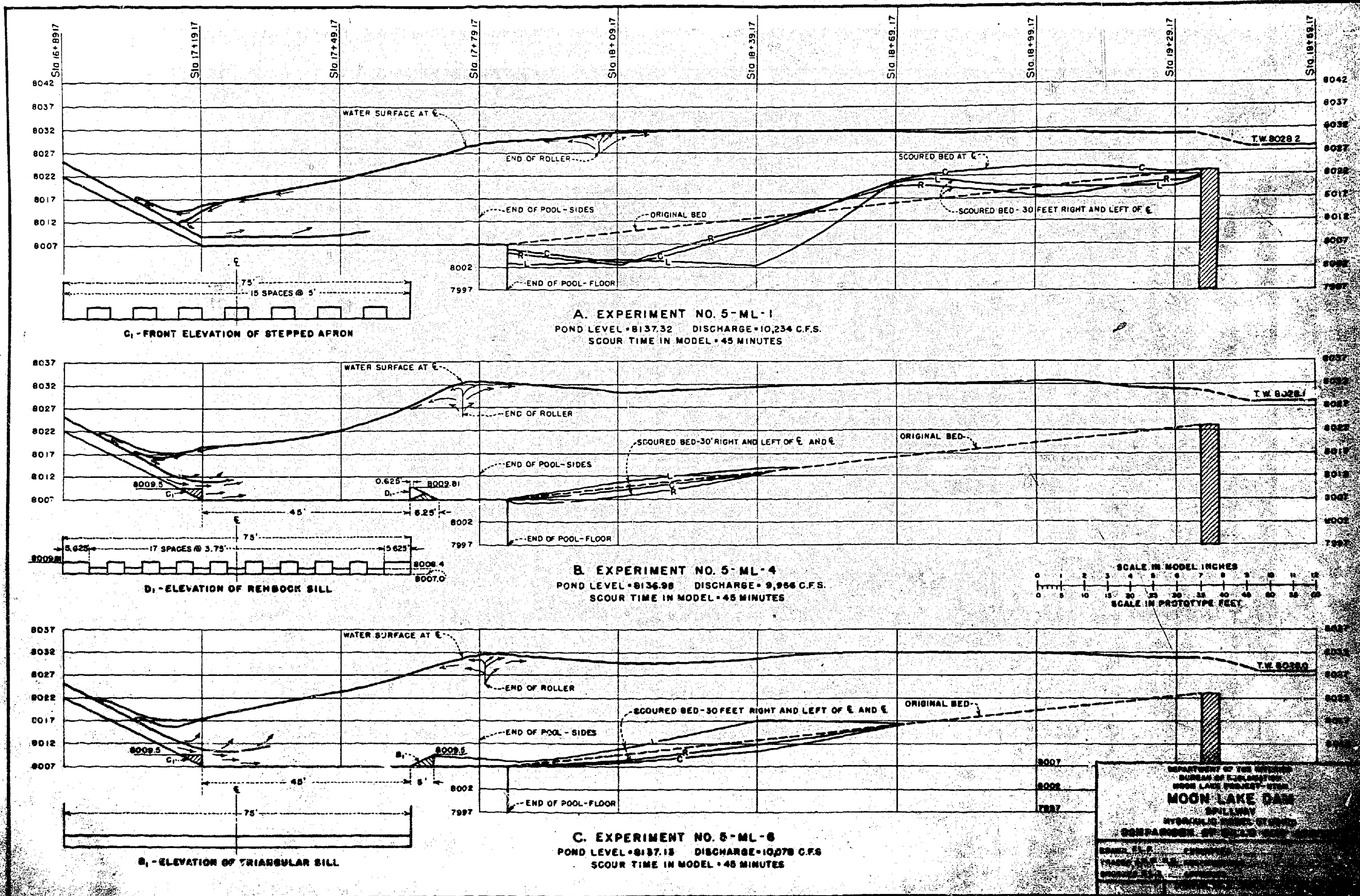
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT—UTAH

MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES
RATING CURVE AT SPILLWAY OUTLET

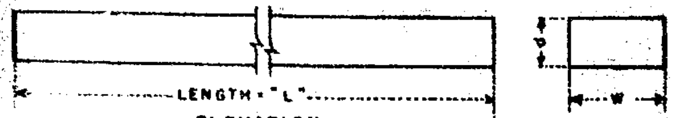
DRAWN F.L.P. SUBMITTED
TRACED W.M.S. H.H. RECOMMENDED
CHECKED F.L.P. APPROVED
DENVER, COLO., SEPT. 16, 1936 209-D-324

FIGURE 14

FIGURE 15



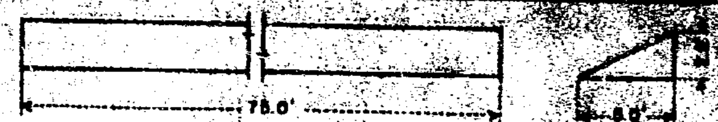
Sta 16+89.17
Sta 17+19.17
Sta 17+49.17
Sta 17+79.17
Sta 18+09.17
Sta 18+39.17
Sta 18+69.17
Sta 18+99.17
Sta 19+29.17
Sta 19+59.17
8042
8037
8032
8027
8022
8017
8012
8007
8002
7997
WATER SURFACE AT ℄
END OF ROLLER
SCOURED BED AT ℄
T.W. 8028.2
END OF POOL-SIDES
ORIGINAL BED
SCOURED BED - 30 FEET RIGHT AND LEFT OF ℄
END OF POOL-FLOOR
75'
15 SPACES @ 5'
C₁ - FRONT ELEVATION OF STEPPED APRON
A. EXPERIMENT NO. 5-ML-1
POND LEVEL = 8137.32 DISCHARGE = 10,234 C.F.S.
SCOUR TIME IN MODEL = 45 MINUTES
T.W. 8026.1
SCOURED BED-30' RIGHT AND LEFT OF ℄ AND ℄
8009.5
0.625'
8009.81
45'
6.25'
5.625'
17 SPACES @ 3.75'
8008.4
8007.0
D₁ - ELEVATION OF REHBOCK SILL
B. EXPERIMENT NO. 5-ML-4
POND LEVEL = 8136.98 DISCHARGE = 9,966 C.F.S.
SCOUR TIME IN MODEL = 45 MINUTES
SCALE IN MODEL INCHES
SCALE IN PROTOTYPE FEET
T.W. 8026.0
SCOURED BED-30 FEET RIGHT AND LEFT OF ℄ AND ℄
5'
B₁ - ELEVATION OF TRIANGULAR SILL
C. EXPERIMENT NO. 5-ML-6
POND LEVEL = 8137.13 DISCHARGE = 10,078 C.F.S.
SCOUR TIME IN MODEL = 45 MINUTES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE DAM
SPILLWAY



ELEVATION
TYPE A - RECTANGULAR SILLS

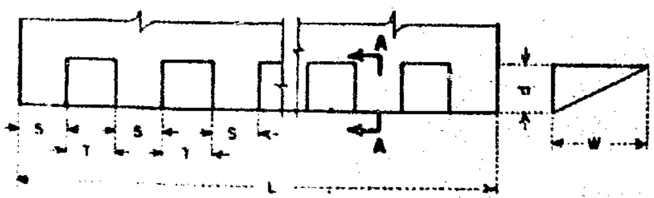
END VIEW

TYPE	NUMBER TEETH	DIMENSIONS IN FEET		
		d	W	L
A_1	1	2.5'	5.0'	75.0'
A_2	1	1.25'	4.00'	75.0'



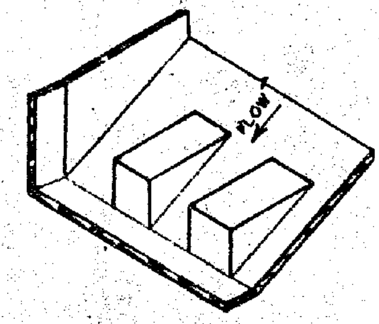
ELEVATION
TYPE B_1 - TRIANGULAR SILL

END VIEW

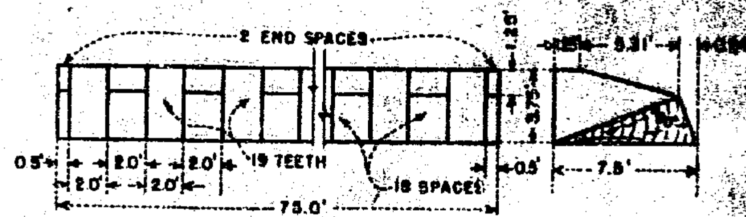


ELEVATION
TYPE C - DENTATED APRON

SECTION A-A



ISOMETRIC VIEW OF
TYPE C

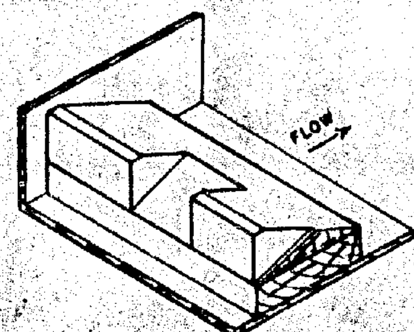


ELEVATION
TYPE D_6 - REHBOCK SILL

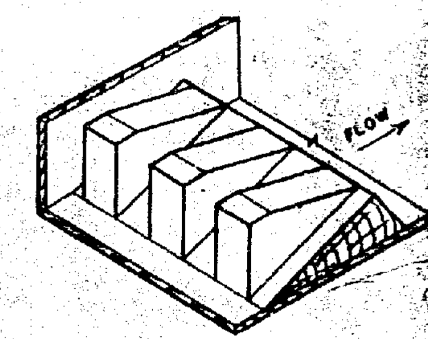
END VIEW

TYPE	NUMBER TEETH	DIMENSIONS IN FEET					
		d	L	W	T	S	S_E
C_1	7	2.5'	75.0'	5.0'	5.0'	5.0'	—
C_4	14	2.81'	75.0'	5.62'	2.5'	2.5'	3.75'
* C_5	14	1.87'	75.0'	3.74'	2.5'	2.5'	3.75'
C_6	14	3.75'	75.0'	7.50	2.5'	2.5'	3.75'
C_7	14	2.5'	75.0'	5.0'	2.5'	2.5'	3.75'
C_8	14	2.58'	75.0'	5.16'	2.5'	2.5'	3.75'
C_9	17	2.58'	75.0'	5.16	1.875'	1.875'	6.562'

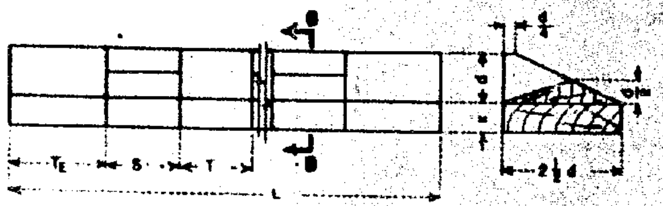
* SLOPE OF TOP FACE 1.7.25



ISOMETRIC VIEW OF
SILL D



ISOMETRIC VIEW OF
SILL D_6



ELEVATION
TYPE D - REHBOCK SILL

SECTION B-B

TYPE	DIMENSIONS IN FEET									NUMBER TEETH	NUMBER SPACES	NUMBER END TEETH
	x	d	d/2	d/4	$2\frac{1}{2}d$	T	S	T_E	L			
D_1	0	2.81'	1.405	.625'	6.25'	3.75'	3.75'	5.625'	75.0'	8	9	2
D_2	0	4.06'	2.03'	.78'	9.00'	3.75'	3.75'	5.625'	75.0'	8	9	2
D_3	0	5.0'	2.5'	1.25'	12.5'	5.0'	5.0'	5.0'	75.0'	6	7	2
D_4	0	7.5'	3.15'	1.87'	18.75'	5.0'	5.0'	5.0'	75.0'	6	7	2
D_{2c}	1.56'	4.06'	2.03'	.78'	9.00'	3.75'	3.75'	5.625'	75.0'	8	9	2
D_{3c}	2.50'	2.50'	2.5'	.625'	6.25'	3.75'	3.75'	5.625'	75.0'	8	9	2
D_{1c}	2.81'	2.81'	1.405'	.625'	6.25'	3.75'	3.75'	5.625'	75.0'	8	9	2

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT - UTAH

MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES
DENTATED STEPS AND SILLS

DRAWN J.B.H. .... SUBMITTED ....
TRACED J.B.H. .... RECOMMENDED ....
CHECKED F.L.P. .... APPROVED ....
DENVER, COLO., OCT [illegible], [illegible]

[illegible]-D-[illegible]

Since it was believed that the river bed, which had previously been assumed as a control, would not be permanent, the stream bed beyond the pool was lowered to elevation 8007.0. That condition was held constant throughout all subsequent tests.

15. Original design of stilling pool with revised stream bed. With the control removed, the pool arrangement which had seemed most suitable previously allowed the hydraulic jump to move downstream with the tailwater at elevation 8028.0 and a discharge of 10,000 second-feet. The change in the pool and the scour action is shown by 5-ML-4 and 6-ML-2 in figures 15 and 17.

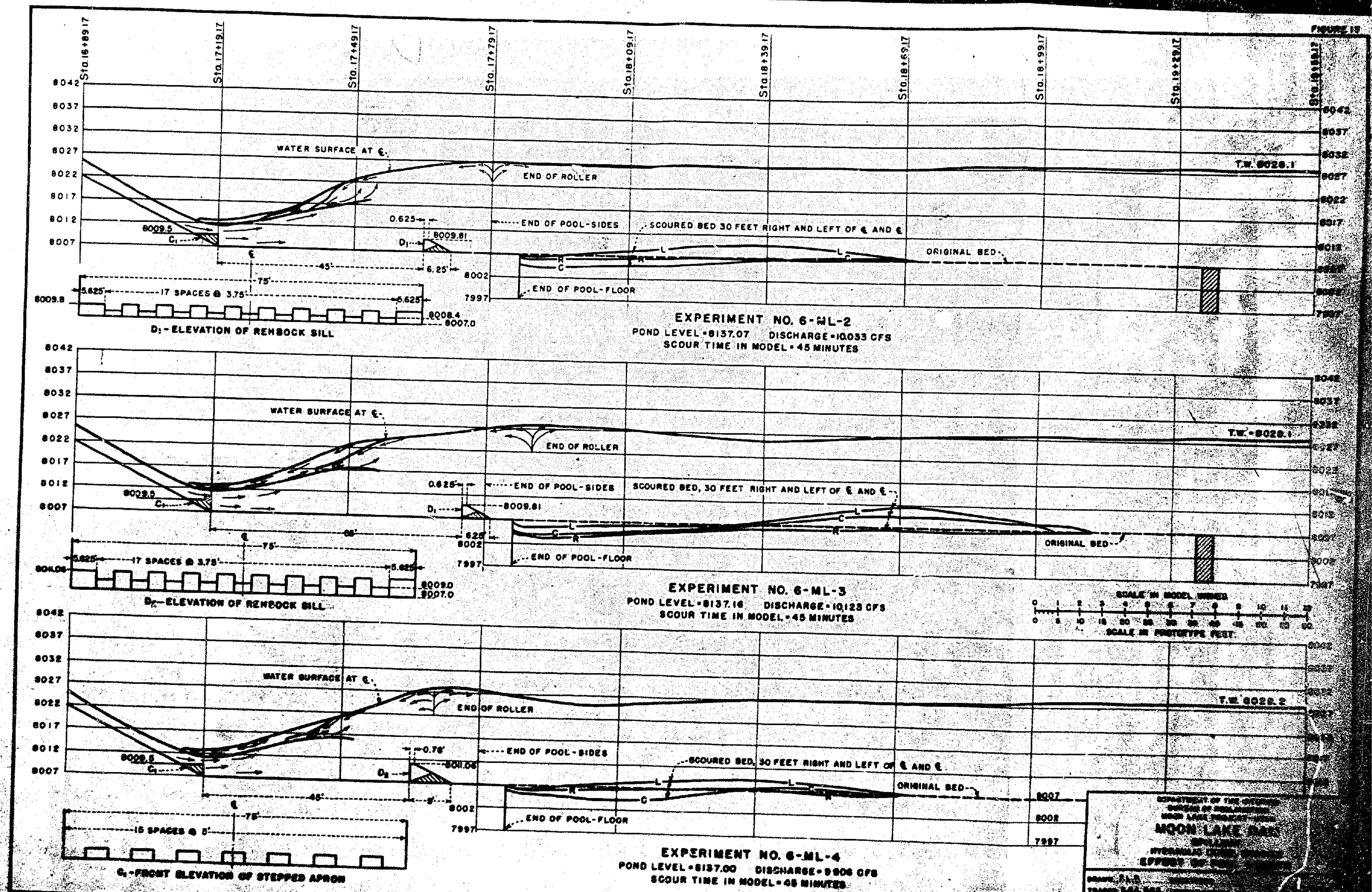
There was some uncertainty as to the reliability of the tailwater discharge relationship, as shown in figure 14. In fact, the necessity of designing the stilling pool to function with the tailwater at elevation 8025.0 at maximum discharge was imposed at this point.

With the arrangement as in 6-ML-2, the jump moved rapidly downstream as the tailwater was lowered until at elevation 8025.0 the jump ceased to form and the sheet of water was impinged directly on the sill, a condition which was unsatisfactory.

Additional tests with and without the dentated steps indicated the effectiveness of the step in maintaining the jump in the pool. It was also definitely determined that the best position of a sill with the pool floor at elevation 8007.0 was at a point 46 feet downstream from the junction of the chute and the pool. Other positions of the sill either decreased the effective range of tailwater, increased the disturbance in the pool, or increased the scour, as shown in 6-ML-2 and 3, figure 17. The results also indicated that with the pool floor at elevation 8007.0 and a length of 66 feet, the pool could not be depended upon to work satisfactorily at maximum discharge for any tailwater below elevation 8028.0.

16. Length of stilling pool increased. There were two possibilities of increasing the effectiveness of the stilling pool for a tailwater as low as elevation 8025.0 at the maximum discharge. The first method was to lengthen the stilling pool and maintain the floor at elevation 8007.0 and the second was to maintain the length of 66 feet and lower the pool floor and hence increase the depth in which to form the jump.

The length of the pool floor was increased from 66 feet to 90 feet and the length of the walls was increased from 60 feet to 90 feet. A series of tests was made similar to those previously described. The dentates at the upstream end of the pool were increased in height and the width of the dentates and the spacing was reduced by half. The resulting apron (C_4) with its narrower


Sta.16+89.17
Sta.17+19.17
Sta.17+49.17
Sta.17+79.17
Sta.18+09.17
Sta.18+39.17
Sta.18+69.17
Sta.18+99.17
Sta.19+29.17
8042
8037
8032
8027
8022
8017
8012
8007
8002
7997
WATER SURFACE AT ℄
END OF ROLLER
T.W. 8028.1
0.625
8009.81
END OF POOL-SIDES
SCOURED BED 30 FEET RIGHT AND LEFT OF ℄ AND ℄
ORIGINAL BED
8009.5
C₁
D₁
45'
6.25'
75'
5.625'
17 SPACES @ 3.75'
8009.8
8008.4
8007.0
END OF POOL-FLOOR
D₁ - ELEVATION OF REHBOCK SILL
EXPERIMENT NO. 6-ML-2
POND LEVEL - 8137.07 DISCHARGE - 10,033 CFS
SCOUR TIME IN MODEL - 45 MINUTES
WATER SURFACE AT ℄
END OF ROLLER
T.W. - 8028.1
0.625
END OF POOL-SIDES
SCOURED BED, 30 FEET RIGHT AND LEFT OF ℄ AND ℄
8009.5
C₁
D₁
8009.81
ORIGINAL BED
END OF POOL-FLOOR
17 SPACES @ 3.75'
5.625
8009.0
8007.0
D₂ - ELEVATION OF REHBOCK SILL
EXPERIMENT NO. 6-ML-3
POND LEVEL - 8137.16 DISCHARGE - 10,123 CFS
SCOUR TIME IN MODEL - 45 MINUTES
SCALE IN MODEL INCHES
SCALE IN PROTOTYPE FEET
WATER SURFACE AT ℄
END OF ROLLER
T.W. 8028.2
0.78'
END OF POOL - SIDES
SCOURED BED, 30 FEET RIGHT AND LEFT OF ℄ AND ℄
8009.5
C₁
D₂
8011.06
ORIGINAL BED
45'
75'
15 SPACES @ 5'
END OF POOL-FLOOR
C₁ - FRONT ELEVATION OF STEPPED APRON
EXPERIMENT NO. 6-ML-4
POND LEVEL - 8137.00 DISCHARGE - 9,906 CFS
SCOUR TIME IN MODEL - 45 MINUTES
MOON LAKE DAM

teeth was more effective in dissipating the stream of water flowing over it.

During the course of this series of tests, sills D_1, D_{1c}, D_2, D_{2c}, and D_3 were tried at points 45, 77.5, and 80 feet downstream from the junction of the slope with the floor of the pool.

With the tailwater at elevation 8028.0 and the sill at the 45-foot point, the best pool action and least scour was obtained with the 5.0 foot combination Rehbock sill (10-ML-11, figure 18) although the results using the 2.6-foot sill (10-ML-9, figure 18) were comparable and practically as good. With the sill at the same point and the tailwater lowered to elevation 8025.0, the best pool action was shown using the 5.6-foot combination Rehbock sill (10-ML-2, figure 19), while the least scour was obtained using either the D_{1c} or D_3 sill (10-ML-2 or 12), so that for all-round results the former was preferable. When that sill was moved out to the 80-foot point (10-ML-3), the scour was materially increased and the jump failed to form satisfactorily within the stilling basin.

Thus far in the tests a rather coarse material (curve 1, figure 20) has been used to represent the stream bed. This material had been used only as a basis of comparison with no attempt to duplicate the prototype material. A finer material would have been carried entirely away in the earlier tests. As the pool was improved, a refinement of the design was finally reached where little erosion was shown in the particular material in use. At that point a finer material (curve 2, figure 20) was substituted and additional experiments were made on the arrangement of apron and sill which previously had been found to be satisfactory. The results with different elevations (figure 21) of the tailwater indicated that with the pool floor at elevation 8007.0 satisfactory conditions could not be obtained with the tailwater at less than elevation 8028.0. The lengthening of the pool did not increase its effectiveness.

17. Depth of pool increased. The second possibility of increasing the effectiveness of the stilling pool was to lower the floor so as to increase the depth of tailwater in which the jump could form. The original rectangular pool was installed with the floor lowered to elevation 8007.0. The effectiveness of the dentated step was slightly increased by reducing the width of both the teeth and the spacing (C_9, figure 16). A combination sill, D_{3c}, was used similar to D_{1c}. Preliminary runs made using the wooden sides in the tailrace indicated minimum scour in the river bed immediately downstream from the apron. The finer material (curve 2, figure 20) and a variation of the tailwater from elevation 8025.0 to 8028.0 was used in these tests.

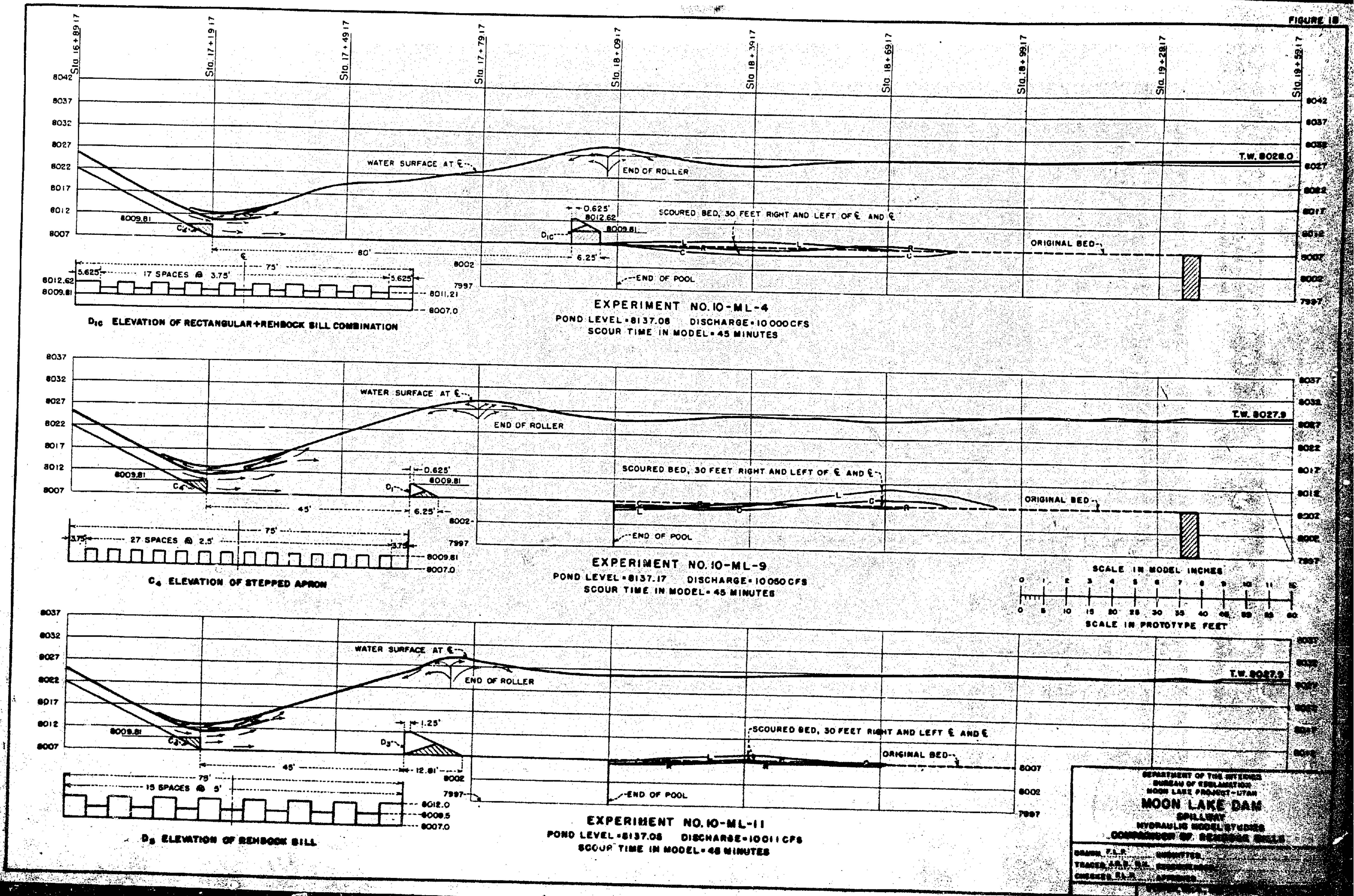


FIGURE 18
Sta. 16 + 89.17
Sta. 17 + 19.17
Sta. 17 + 49.17
Sta. 17 + 79.17
Sta. 18 + 09.17
Sta. 18 + 39.17
Sta. 18 + 69.17
Sta. 18 + 99.17
Sta. 19 + 29.17
Sta. 19 + 59.17
WATER SURFACE AT ℄
END OF ROLLER
T.W. 8028.0
SCOURED BED, 30 FEET RIGHT AND LEFT OF ℄ AND ℄
ORIGINAL BED
END OF POOL
D10 ELEVATION OF RECTANGULAR + REHBOCK SILL COMBINATION
17 SPACES @ 3.75'
EXPERIMENT NO. 10-ML-4
POND LEVEL = 8137.08 DISCHARGE = 10 000 CFS
SCOUR TIME IN MODEL = 45 MINUTES
T.W. 8027.9
C4 ELEVATION OF STEPPED APRON
27 SPACES @ 2.5'
EXPERIMENT NO. 10-ML-9
POND LEVEL = 8137.17 DISCHARGE = 10 050 CFS
SCOUR TIME IN MODEL = 45 MINUTES
SCALE IN MODEL INCHES
SCALE IN PROTOTYPE FEET
D3 ELEVATION OF REHBOCK SILL
15 SPACES @ 5'
EXPERIMENT NO. 10-ML-11
POND LEVEL = 8137.08 DISCHARGE = 10011 CFS
SCOUR TIME IN MODEL = 45 MINUTES
MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES

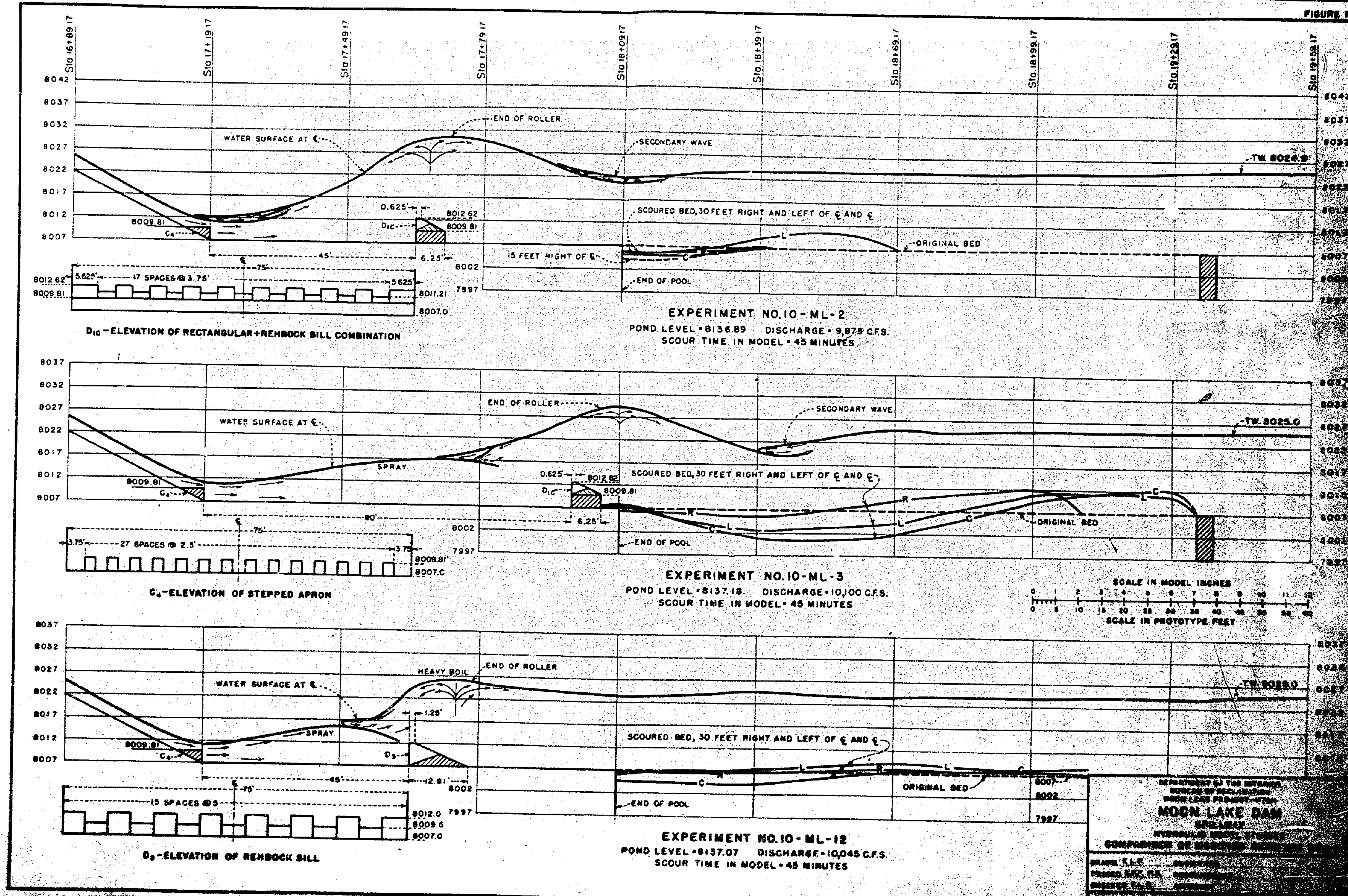
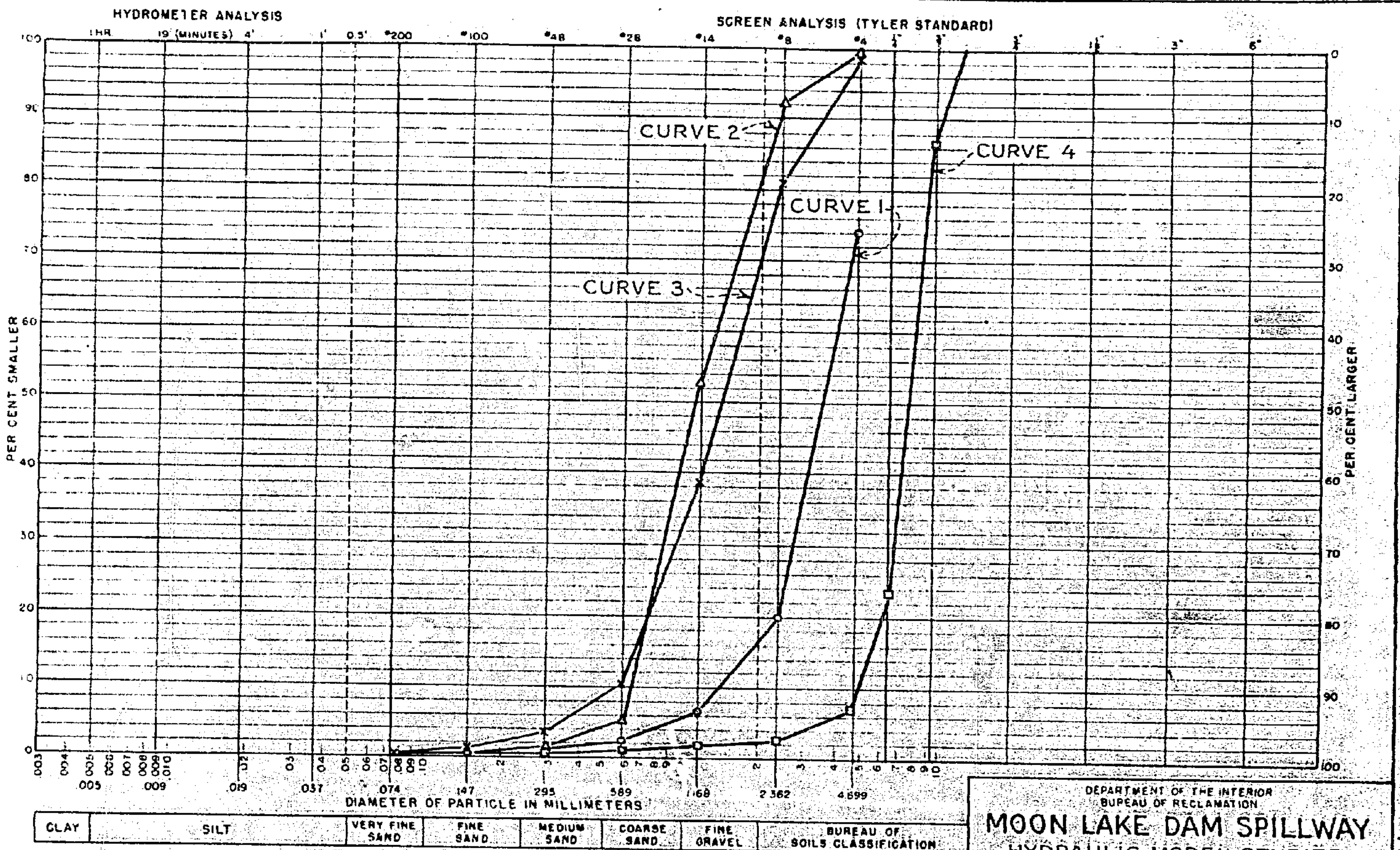


FIGURE 19
Sta 16+89.17
Sta 17+19.17
Sta 17+49.17
Sta 17+79.17
Sta 18+09.17
Sta 18+39.17
Sta 18+69.17
Sta 18+99.17
Sta 19+29.17
Sta 19+59.17
WATER SURFACE AT ℄
END OF ROLLER
SECONDARY WAVE
TW. 8024.9
SCOURED BED, 30 FEET RIGHT AND LEFT OF ℄ AND ℄
ORIGINAL BED
15 FEET RIGHT OF ℄
END OF POOL
8009.81
8012.62
0.625'
6.25'
45'
75'
5.625'
17 SPACES @ 3.75'
8011.21
8007.0
D1C - ELEVATION OF RECTANGULAR + REHBOCK SILL COMBINATION
EXPERIMENT NO. 10-ML-2
POND LEVEL = 8136.89 DISCHARGE = 9,875 C.F.S.
SCOUR TIME IN MODEL = 45 MINUTES
SPRAY
TW. 8025.0
80'
3.75'
27 SPACES @ 2.5'
C4 - ELEVATION OF STEPPED APRON
EXPERIMENT NO. 10-ML-3
POND LEVEL = 8137.18 DISCHARGE = 10,100 C.F.S.
SCOUR TIME IN MODEL = 45 MINUTES
SCALE IN MODEL INCHES
SCALE IN PROTOTYPE FEET
HEAVY BOIL
1.25'
12.81'
15 SPACES @ 5'
8012.0
8009.5
D3 - ELEVATION OF REHBOCK SILL
EXPERIMENT NO. 10-ML-12
POND LEVEL = 8137.07 DISCHARGE = 10,045 C.F.S.
SCOUR TIME IN MODEL = 45 MINUTES
MOON LAKE DAM

FIGURE 20



NOTES: Curve 1 Sand in runs prior to 10-ML-24.
Curve 2 " " " after " " ".
Curve 3 " " " 16-ML-1 through 17-ML-1.
Curve 4 Rip-rap used in all runs.

209-D-330

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

MOON LAKE DAM SPILLWAY
HYDRAULIC MODEL STUDIES
STREAM BED MATERIAL ANALYSIS

Pit Depth to
Lab. Sample No.

DRAWN CHECKED DATE

X-D-2424

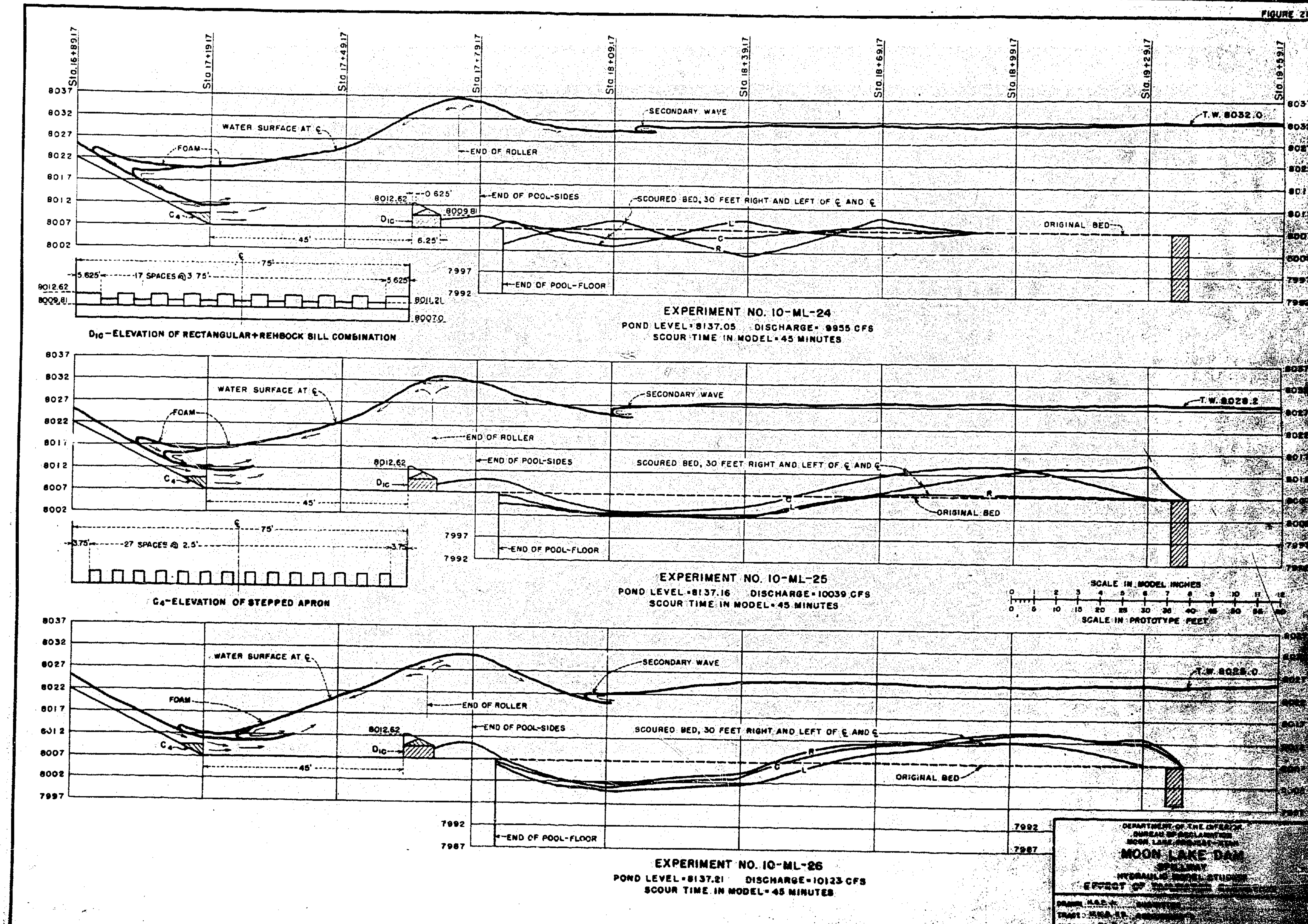


FIGURE 21
Sta 16+89.17
Sta 17+19.17
Sta 17+49.17
Sta 17+79.17
Sta 18+09.17
Sta 18+39.17
Sta 18+69.17
Sta 18+99.17
Sta 19+29.17
Sta 19+59.17
WATER SURFACE AT ℄
FOAM
SECONDARY WAVE
T.W. 8032.0
END OF ROLLER
END OF POOL-SIDES
SCOURED BED, 30 FEET RIGHT AND LEFT OF ℄ AND ℄
ORIGINAL BED
END OF POOL-FLOOR
8012.62
0.625'
8009.81
45'
6.25'
75'
5.625'
17 SPACES @ 3.75'
8011.21
8007.0
D10 – ELEVATION OF RECTANGULAR + REHBOCK SILL COMBINATION
EXPERIMENT NO. 10-ML-24
POND LEVEL = 8137.05 DISCHARGE = 9955 CFS
SCOUR TIME IN MODEL = 45 MINUTES
T.W. 8028.2
3.75'
27 SPACES @ 2.5'
C4 – ELEVATION OF STEPPED APRON
EXPERIMENT NO. 10-ML-25
POND LEVEL = 8137.16 DISCHARGE = 10039 CFS
SCOUR TIME IN MODEL = 45 MINUTES
SCALE IN MODEL INCHES
SCALE IN PROTOTYPE FEET
T.W. 8028.0
EXPERIMENT NO. 10-ML-26
POND LEVEL = 8137.21 DISCHARGE = 10123 CFS
SCOUR TIME IN MODEL = 45 MINUTES
MOON LAKE DAM

18. Stability of stream bed. Since a more or less absolute value had at this time been obtained in the scour in the river bed, the wooden sides were removed so that the stability of the side slopes might be studied during a flood. Tests were made using slopes of 1-1/2:1, 2:1, and 3:1 covered with riprap according to curve 4, figure 20.

With the 1-1/2:1 slope there was very little scour in the bottom of the river bed for ranges of tailwater from 8020.0 to 8025.0 but the side whirls at the end of the pool did erode pockets of material in the bank on the left side. By decreasing the side slopes to 2:1, the whirl was eliminated and nothing was gained in decreasing the slope to 3:1. The results of tests made using the 2:1 slopes are shown in figure 21.

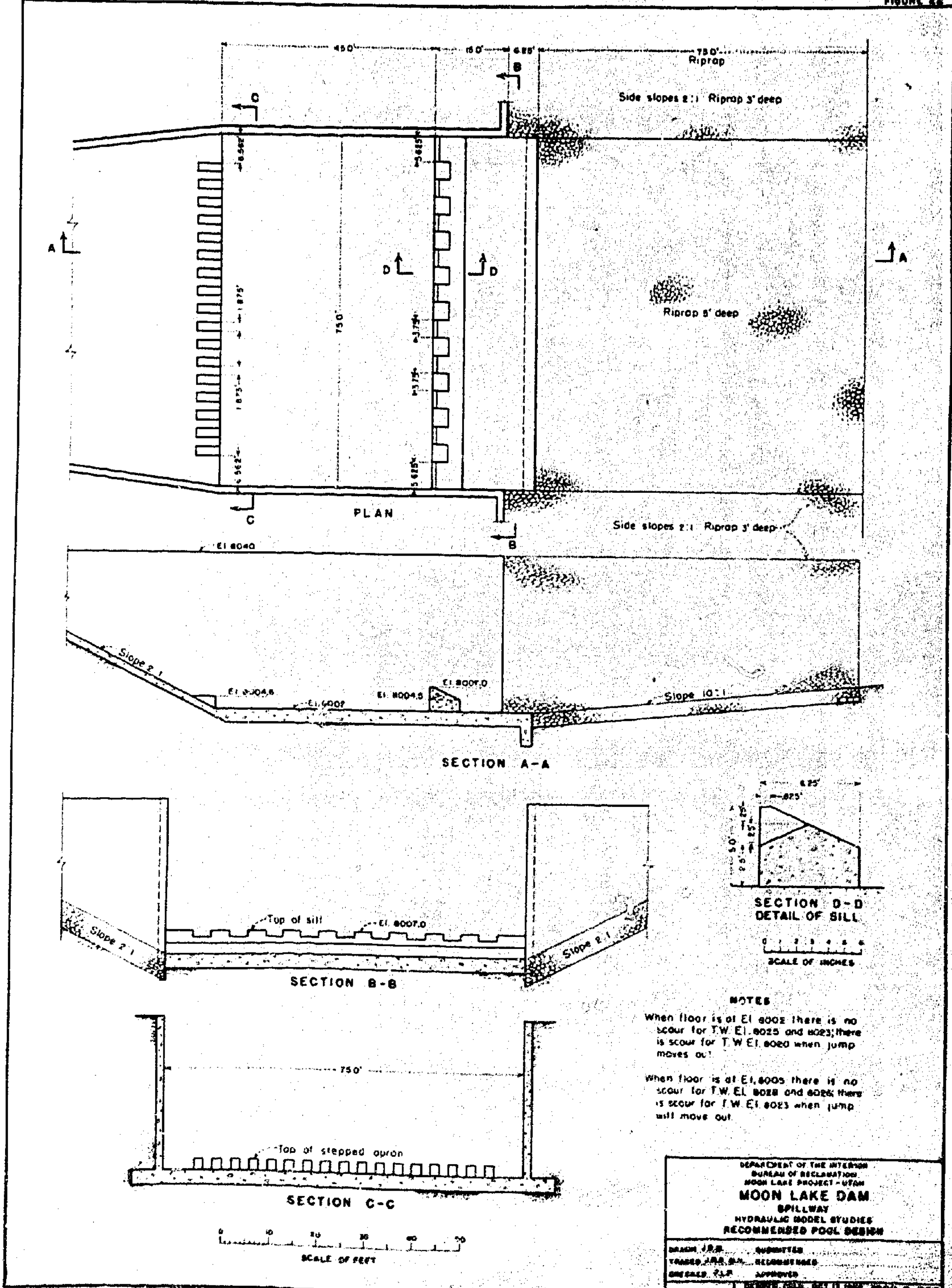
19. Final recommended design. The final recommendations for the pool design (figures 22 and 23) had the pool floor at elevation 8002.0, the dentated step C_9 at the toe of the sloping chute and the combination Rehbock sill D_{3c}, 45 feet downstream from the intersection of the sloping chute and the pool floor. The sides of the riprapped pool were placed on a 2:1 slope and the riprap extended 75 feet downstream from the end of the pool. The scour in the river bed, with this arrangement, is shown in figure 24. The minimum recommended tailwater is elevation 8023.0.

20. Final adopted design. In the design of the stilling pool which was finally adopted, the pool floor was placed at elevation 8005.0 instead of elevation 8002.0, as recommended. This change was made for structural reasons. With the pool floor at elevation 8005.0, the minimum satisfactory tailwater will be elevation 8026.0 or a 2-foot range below the expected tailwater at elevation 8028.0 for a maximum discharge of 10,000 second-feet.

In the final adopted design the slope in the transition below the gates was slightly increased (figure 25) so as to avoid any possibility of an hydraulic jump forming in the section which would interfere with the counterbalances on the radial gates. No change was made in the flaring chute, as suggested in the studies on that subject.

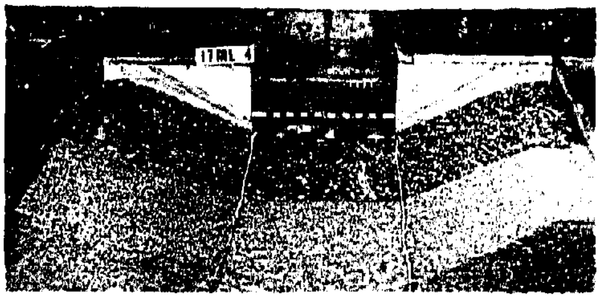
21. Studies of bucket pool. A series of tests was made to determine the desirability of using a pool of the bucket type, particularly to handle the possible large range of tailwater. Buckets with radii of 10, 15, and 20 feet were tried at elevations 7997.0, 7996.2, and 7995.3, respectively, (figure 26). The results are shown on figures 27 and 28.

FIGURE 22

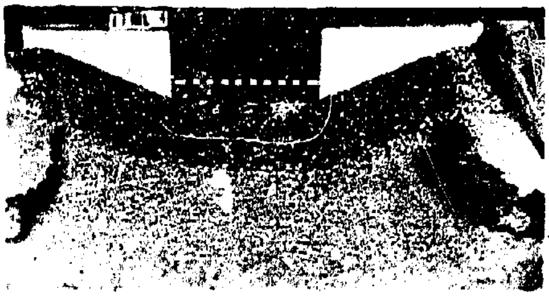


45.0'
15.0'
6.25'
75.0'
Riprap
Side slopes 2:1 Riprap 3' deep
Riprap 5' deep
75.0'
PLAN
Side slopes 2:1 Riprap 3' deep
El. 8040
Slope 2:1
El. 8004.6
El. 8002
El. 8004.5
El. 8007.0
Slope 10:1
SECTION A-A
Top of sill
El. 8007.0
Slope 2:1
SECTION B-B
6.25'
SECTION D-D
DETAIL OF SILL
SCALE OF INCHES
75.0'
Top of stepped apron
SECTION C-C
SCALE OF FEET
NOTES
When floor is at El 8002 there is no scour for T.W. El. 8025 and 8023; there is scour for T.W. El. 8020 when jump moves out.
When floor is at El. 8005 there is no scour for T.W. El. 8028 and 8026; there is scour for T.W. El. 8023 when jump will move out.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT - UTAH
MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES
RECOMMENDED POOL DESIGN

Figure 23.



A. STILLING POOL WITH STEP C_9 AND SILL D_{3C} PLACED 45 FEET DOWNSTREAM IN POOL



B. ACTION IN POOL

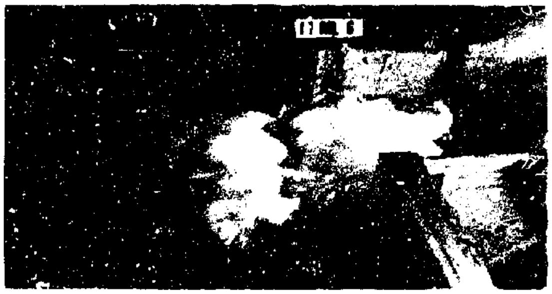
C. SCOUR PATTERN

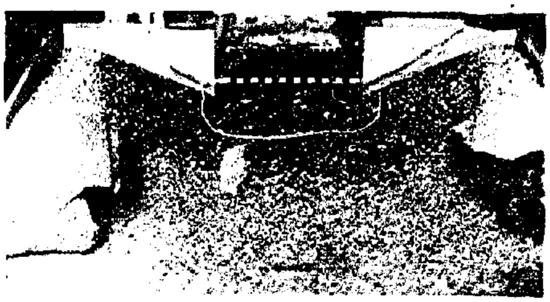
TAILWATER ELEVATION 8025.0

D. ACTION IN POOL

E. SCOUR PATTERN

TAILWATER ELEVATION 8023.0





F. ACTION IN POOL

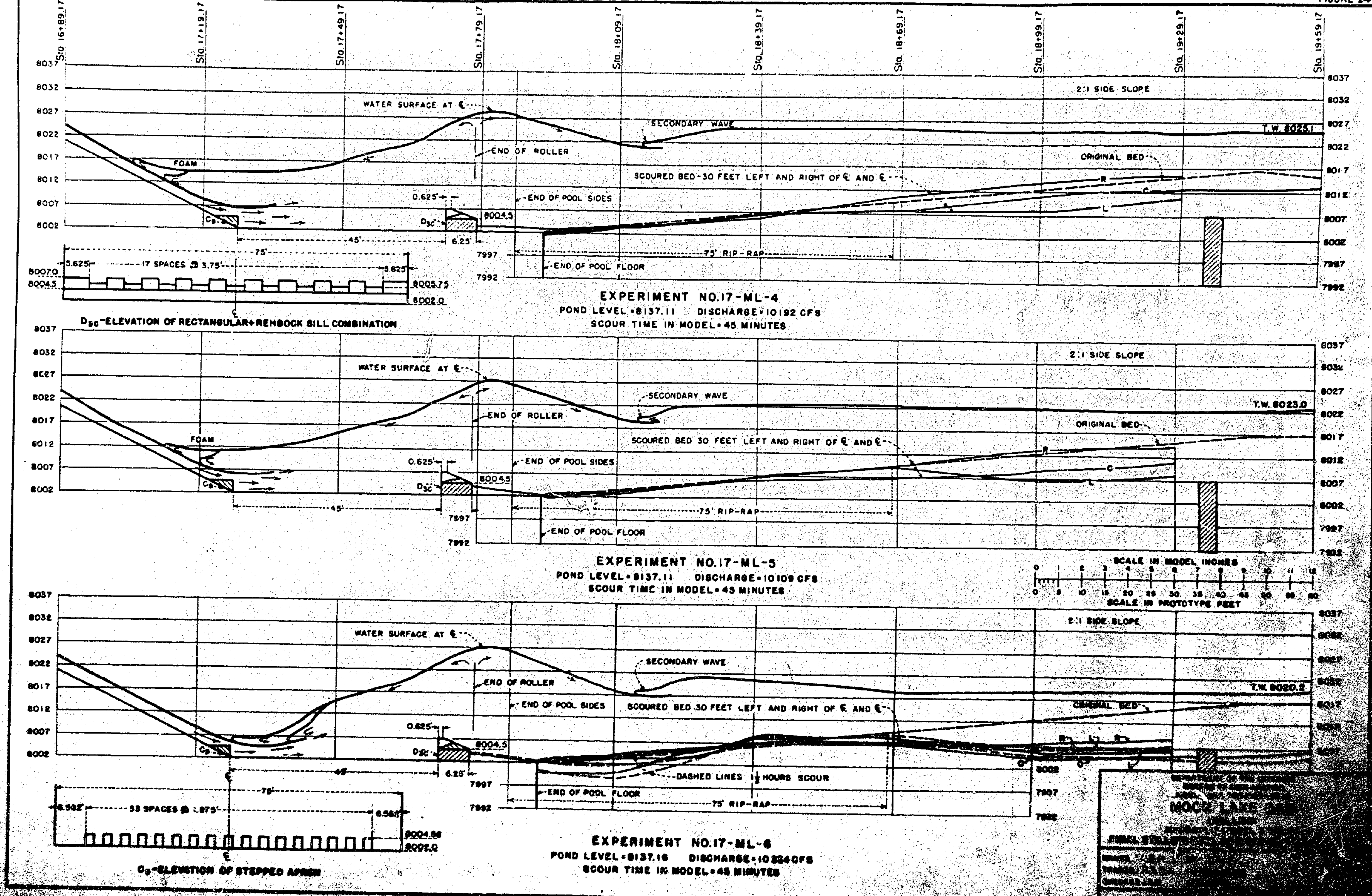
G. SCOUR PATTERN

TAILWATER ELEVATION 8020.0

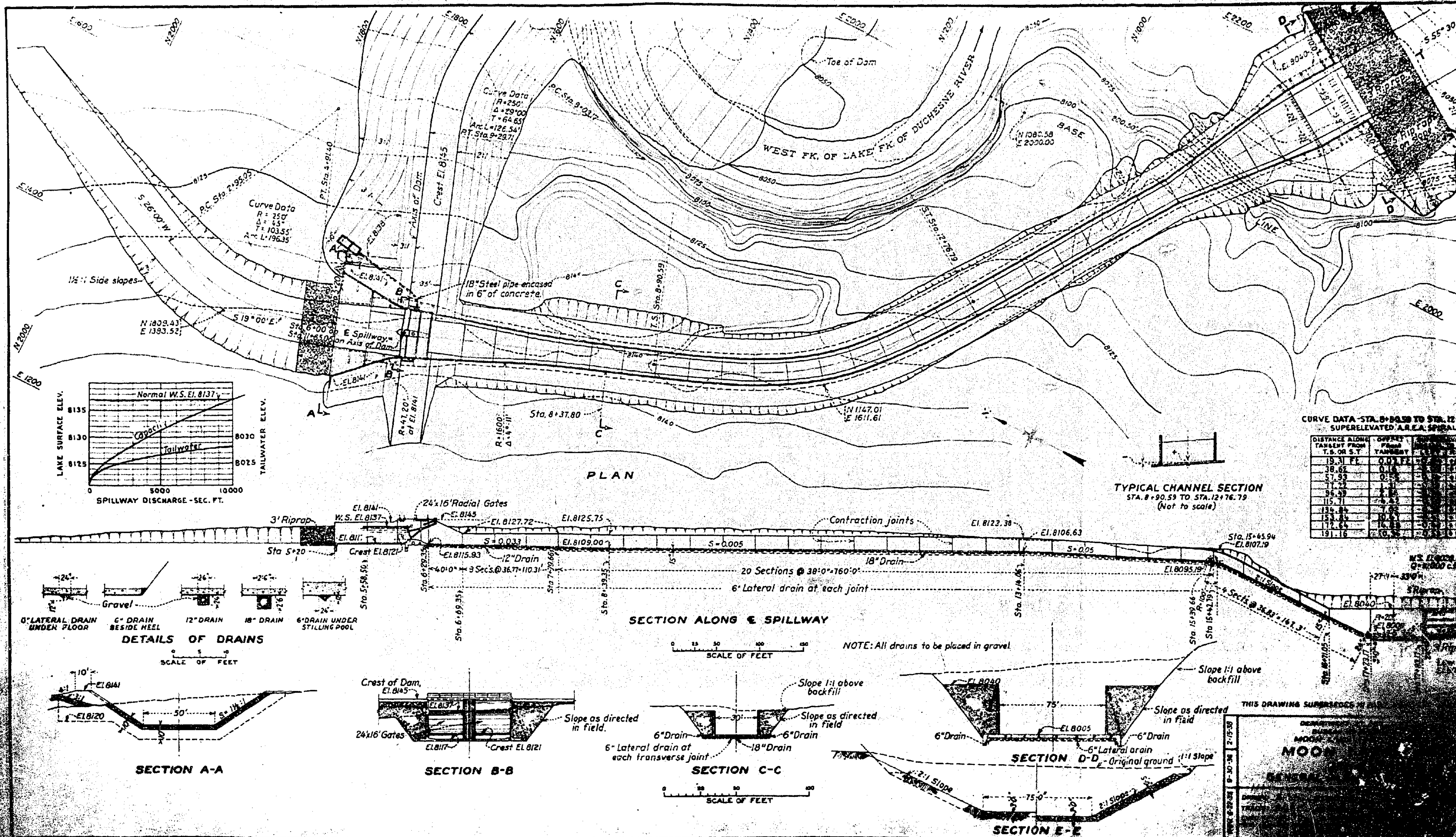
RECOMMENDED DESIGN OF STILLING POOL AND PROTECTION OF RIVER BED

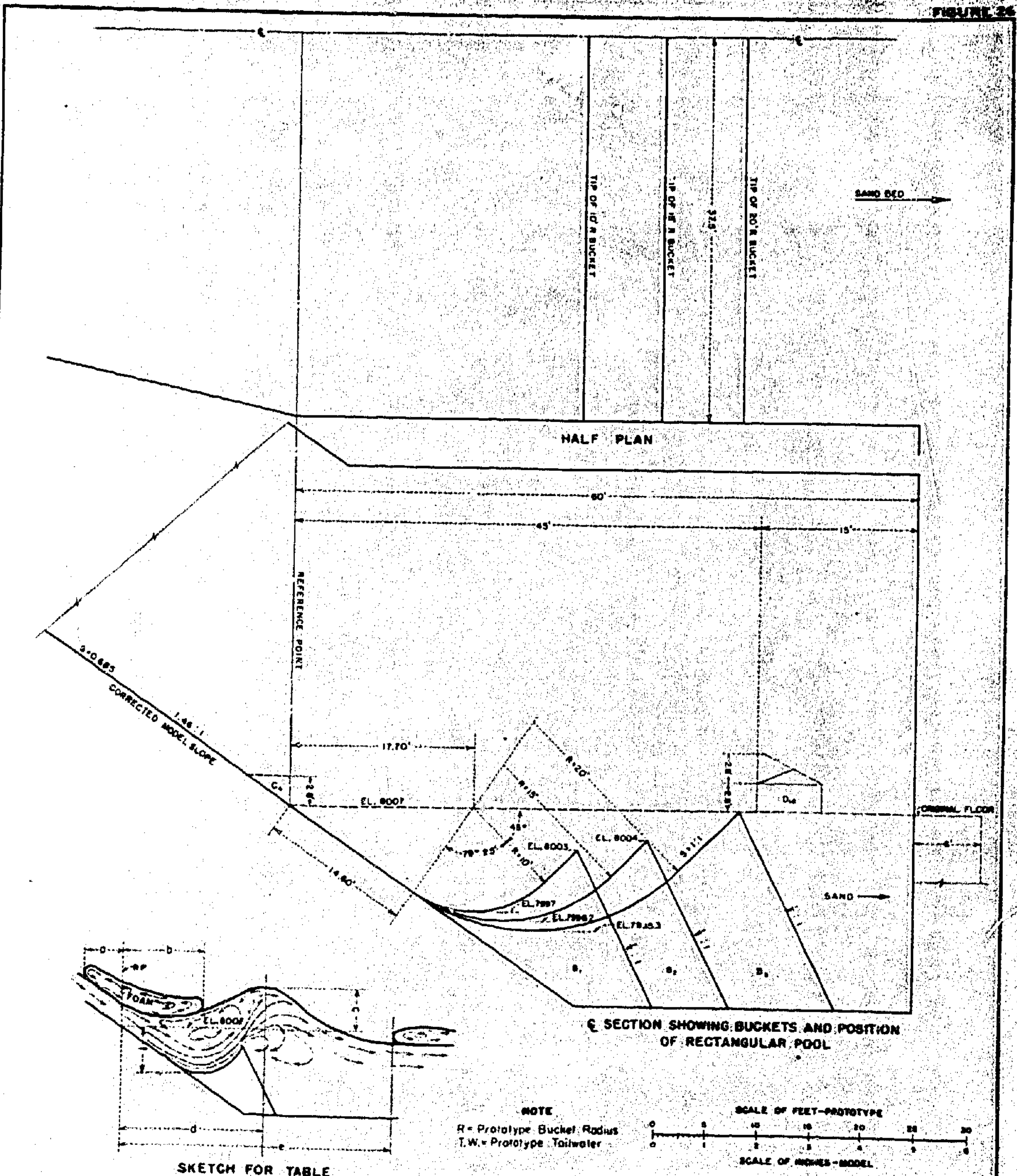
POOL 60 FEET LONG AT ELEVATION 8002.0. STREAM BED SLOPES FROM ELEVATION 8002.0 TO 8017.0. SIDE SLOPES 2:1. DISCHARGE 10,000 SECOND-FEET

FIGURE 24


Sta. 16+89.17
Sta. 17+19.17
Sta. 17+49.17
Sta. 17+79.17
Sta. 18+09.17
Sta. 18+39.17
Sta. 18+69.17
Sta. 18+99.17
Sta. 19+29.17
Sta. 19+59.17
2:1 SIDE SLOPE
WATER SURFACE AT ℄
SECONDARY WAVE
T.W. 8025.1
END OF ROLLER
ORIGINAL BED
FOAM
SCOURED BED-30 FEET LEFT AND RIGHT OF ℄ AND ℄
END OF POOL SIDES
0.625'
8004.5
45'
6.25'
75'
75' RIP-RAP
END OF POOL FLOOR
5.625
17 SPACES @ 3.75'
8005.75
8002.0
EXPERIMENT NO.17-ML-4
POND LEVEL-8137.11 DISCHARGE-10192 CFS
SCOUR TIME IN MODEL-45 MINUTES
D_{SC}-ELEVATION OF RECTANGULAR+REHBOCK SILL COMBINATION
T.W. 8023.0
EXPERIMENT NO.17-ML-5
POND LEVEL-8137.11 DISCHARGE-10109 CFS
SCOUR TIME IN MODEL-45 MINUTES
SCALE IN MODEL INCHES
SCALE IN PROTOTYPE FEET
T.W. 8020.2
DASHED LINES 1½ HOURS SCOUR
33 SPACES @ 1.875'
8004.88
8002.0
C_S-ELEVATION OF STEPPED APRON
EXPERIMENT NO.17-ML-6
POND LEVEL-8137.16 DISCHARGE-10234 CFS
SCOUR TIME IN MODEL-45 MINUTES

FIGURE


PLAN
WEST FK. OF LAKE FK. OF DUCHESNE RIVER
Toe of Dam
Curve Data R=250' Δ=29°00' T=64.65' Arc L=126.54'
P.C. Sta. 9+03.17
P.T. Sta. 9+29.71
Crest El. 8145
Axis of Dam
Curve Data R=250' Δ=45° T=103.55' Arc L=196.35'
P.C. Sta. 2+95.05
S 26°00' W
1½:1 Side slopes
S 19°00' E
N 1809.43 E 1383.53
Sta. 6+00 on ℄ Spillway = Sta. 11+65.50 on Axis of Dam
18" Steel pipe encased in 6" of concrete
R=41.20' at El. 8141
R=1600' Δ=4°11'
Sta. 8+37.80
T.S. Sta. 8+90.59
S.T. Sta. 12+76.79
N 1147.01 E 1611.61
N 1080.58 E 2000.00
BASE LINE
Riprap on slope
Normal W.S. El. 8137
Capacity
Tailwater
LAKE SURFACE ELEV.
TAILWATER ELEV.
SPILLWAY DISCHARGE - SEC. FT.
TYPICAL CHANNEL SECTION
STA. 8+90.59 TO STA. 12+76.79
(Not to scale)
CURVE DATA - STA. 8+90.59 TO STA. 12+76.79
SUPERELEVATED A.R.E.A. SPIRAL
3' Riprap
W.S. El. 8137
24'x16' Radial Gates
Crest El. 8121
Sta. 5+20
S = 0.033
S = 0.005
S = 0.05
12" Drain
18" Drain
Contraction joints
20 Sections @ 38'0" = 760'0"
6" Lateral drain at each joint
SECTION ALONG ℄ SPILLWAY
NOTE: All drains to be placed in gravel.
DETAILS OF DRAINS
6" LATERAL DRAIN UNDER FLOOR
6" DRAIN BESIDE HEEL
12" DRAIN
18" DRAIN
6" DRAIN UNDER STILLING POOL
Gravel
SECTION A-A
SECTION B-B
Crest of Dam, El. 8145
24'x16' Gates
Slope as directed in field.
SECTION C-C
Slope 1:1 above backfill
6" Drain
18" Drain
6" Lateral drain at each transverse joint
SECTION D-D
6" Lateral drain
Original ground
SECTION E-E
2:1 Slope
SCALE OF FEET
THIS DRAWING SUPERSEDES



RUN	a	b	c	d	e	f	T.W.	R	REMARKS
7	4"-6"	4½"	7"	12"	16½"	7993.5	8032	20'	Best
8	3"-4"	4"	6"	12"	16"	-	8028	20'	not so
9	2"-3½"	4"	6"	12"	16"	-	8025	20'	good
10	4"-6"	5"	7"	10½"	16"	[illegible]	8030	15'	Not
11	3"-4"	5"	7"	10½"	16½"	-	8028	15'	so
12	2"-3½"	4"	7"	11"	17"	-	8025	15'	good
13	4"-6"	6"	7"	11½"	19"	7998.5	8033	15'	Better
14	3"-4½"	6"	6"	11½"	19"	-	8028	15'	
15	2"-3½"	4½"	5"	11½"	18"	-	8025	15'	
16	4"-6"	6"	7"	11½"	19"	-	8031	10'	
17	3"-4"	6"	6"	11½"	[illegible]	8000	8028	10'	Best
18	2"-3"	4½"	5"	11½"	18½"	-	8025	10'	one of
19	0"-2"	5"	4½"	11½"	18½"	-	8025	10'	all
							8031	10'	shapes

DEPARTMENT OF THE INTERIOR
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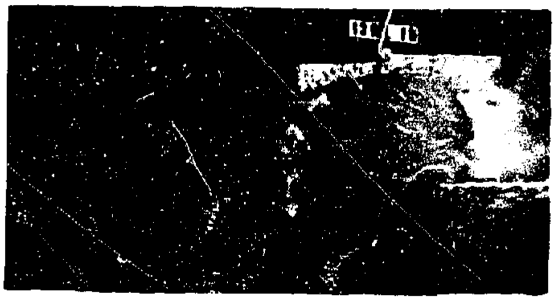
MOON LAKE PROJECT—UTAH

MOON LAKE DAM
SPILLWAY
HYDRAULIC MODEL STUDIES
BUCKET DESIGNS MODEL SCALE 1:20

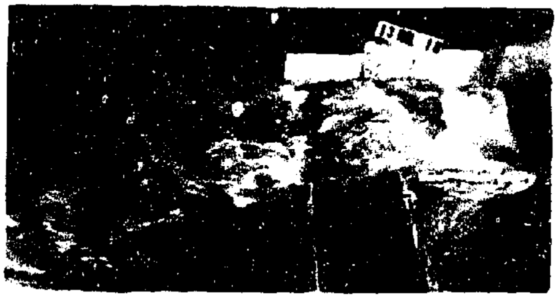
DRAWN A.B.M., H.H.R. SUBMITTED
TRACED H.H.R., G.H. RECOMMENDED
CHECKED F.L.P. APPROVED

DENVER, COLO., MARCH, 1935 [illegible]

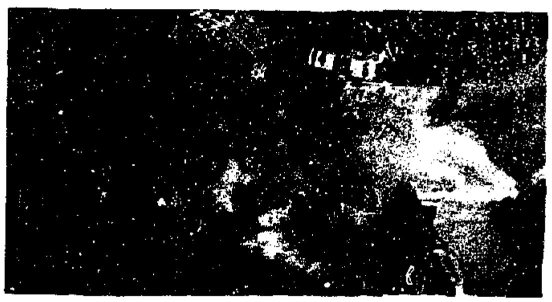
A. TAILWATER ELEVATION 8025.0 B. TAILWATER ELEVATION 8032.0
BUCKET RADIUS 20 FEET. INVERT ELEVATION 7993.60. POOL LENGTH 90 FEET



C. TAILWATER ELEVATION 8025.0 D. TAILWATER ELEVATION 8032.0
BUCKET RADIUS 15 FEET. INVERT ELEVATION 7988.25. POOL LENGTH 90 FEET

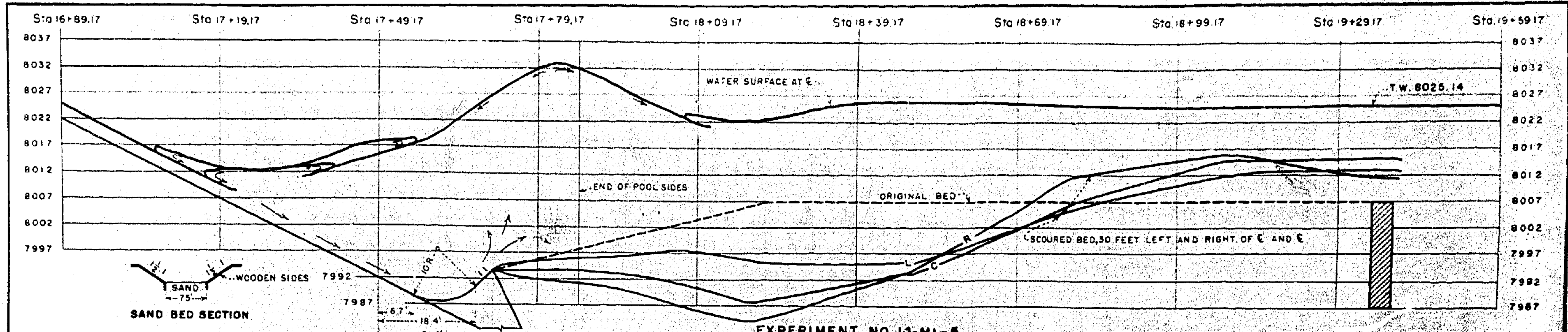


E. TAILWATER ELEVATION 8025.0 F. TAILWATER ELEVATION 8032.0
BUCKET RADIUS 10 FEET. INVERT ELEVATION 7988.00. POOL LENGTH 90 FEET

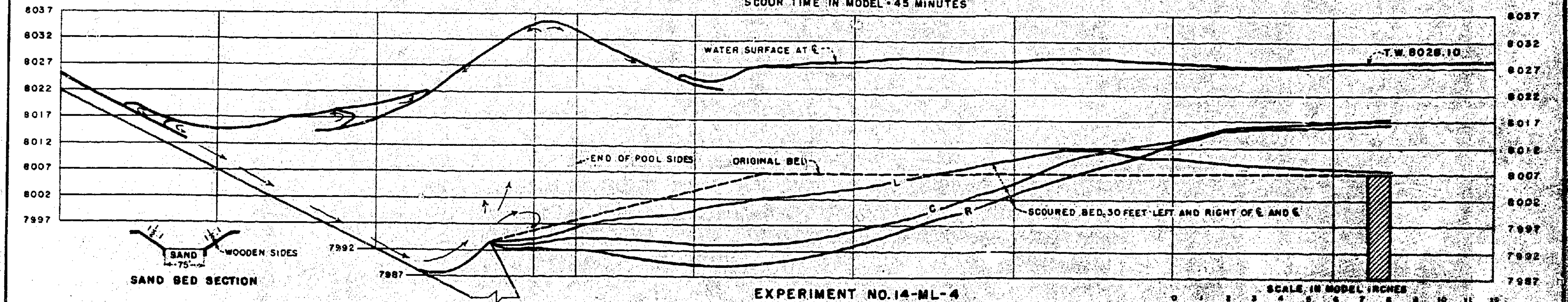


G. TAILWATER ELEVATION 8028.0 H. TAILWATER ELEVATION 8025.0
BUCKET RADIUS 10 FEET. INVERT ELEVATION 7987.50. POOL LENGTH 60 FEET

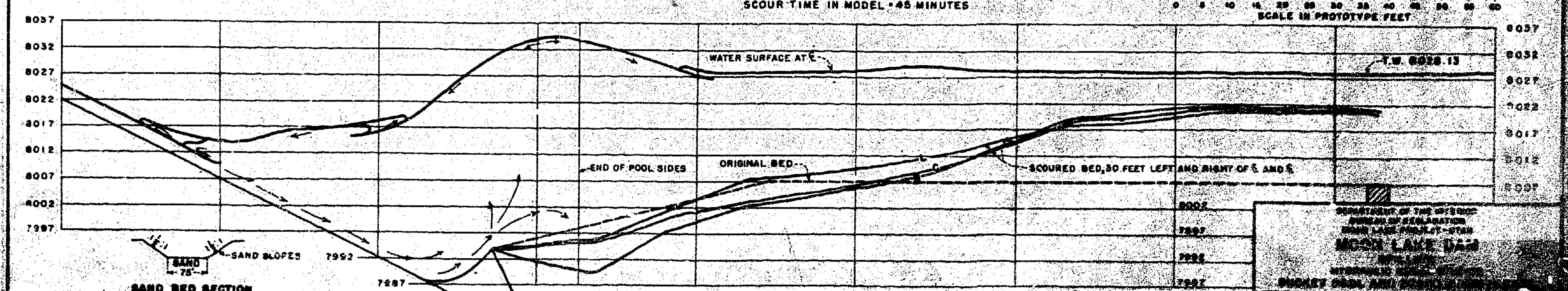
ACTION OF STILLING POOL USING CURVED BUCKETS



EXPERIMENT NO. 14-ML-5
POND LEVEL • 8137.13 DISCHARGE • 10,123 C.F.S.
SCOUR TIME IN MODEL • 45 MINUTES



EXPERIMENT NO. 14-ML-4
POND LEVEL • 8236.95 DISCHARGE • 9,947 C.F.S.
SCOUR TIME IN MODEL • 45 MINUTES



EXPERIMENT NO. 16-ML-2
POND LEVEL • 8137.0 DISCHARGE • 10,005 C.F.S.
SCOUR TIME IN MODEL • 45 MINUTES

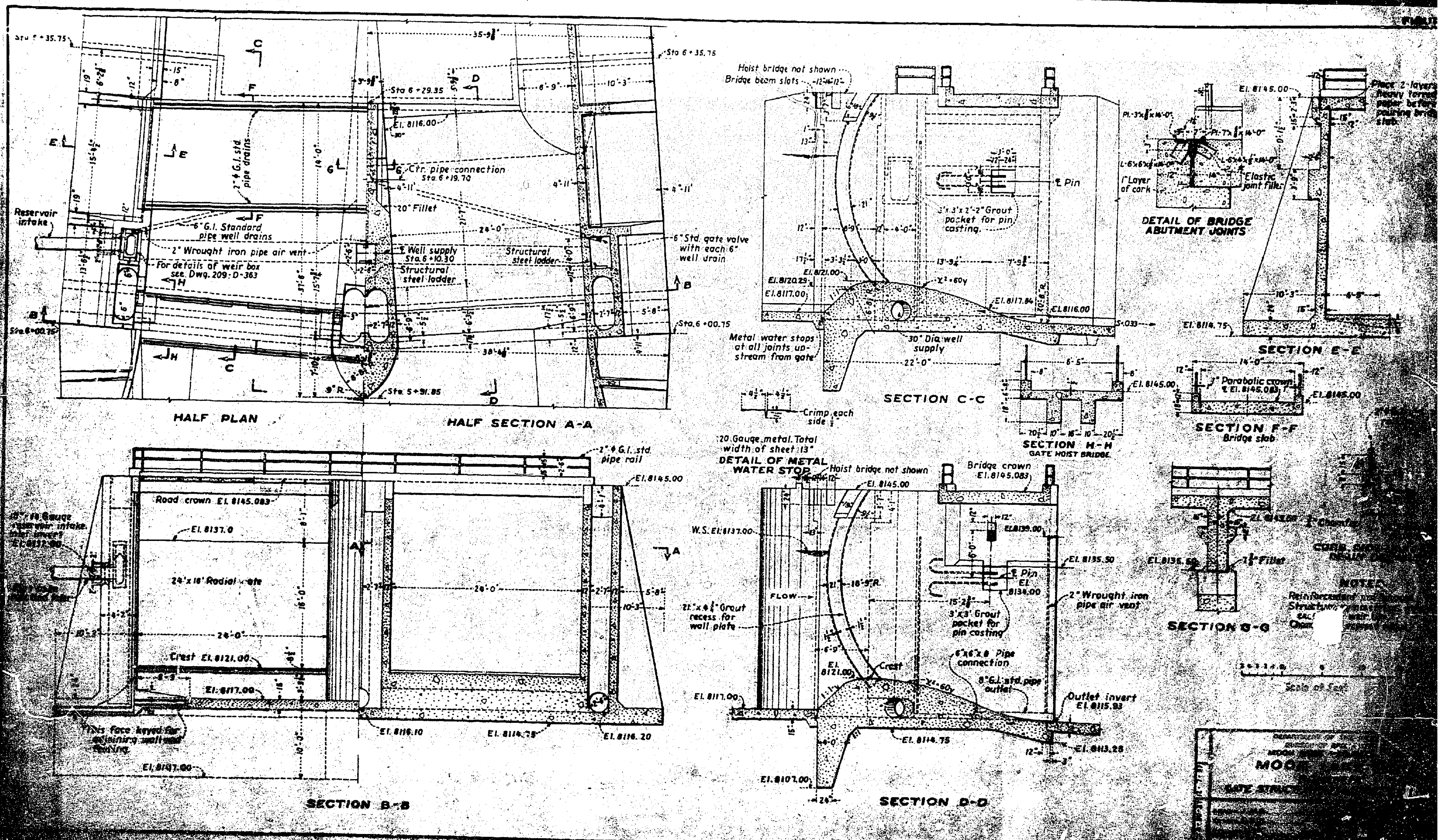
The bucket with the shorter radius was the most satisfactory not only because of the better action in the pool and less scour in the river bed but also because of the lesser construction cost. Further improvement in the action of the pool was obtained by extending the pool walls 20 feet downstream beyond the original length of 60 feet.

The lower the bucket was placed, the better the results and the larger the tailwater range. Very good results were obtained by placing the 10-foot radius bucket invert at elevation 7987.5. The material in the river bed was deposited against the downstream side of the bucket lip, and a slight hole was scoured further downstream, sufficient to allow the formation of the ground roller.

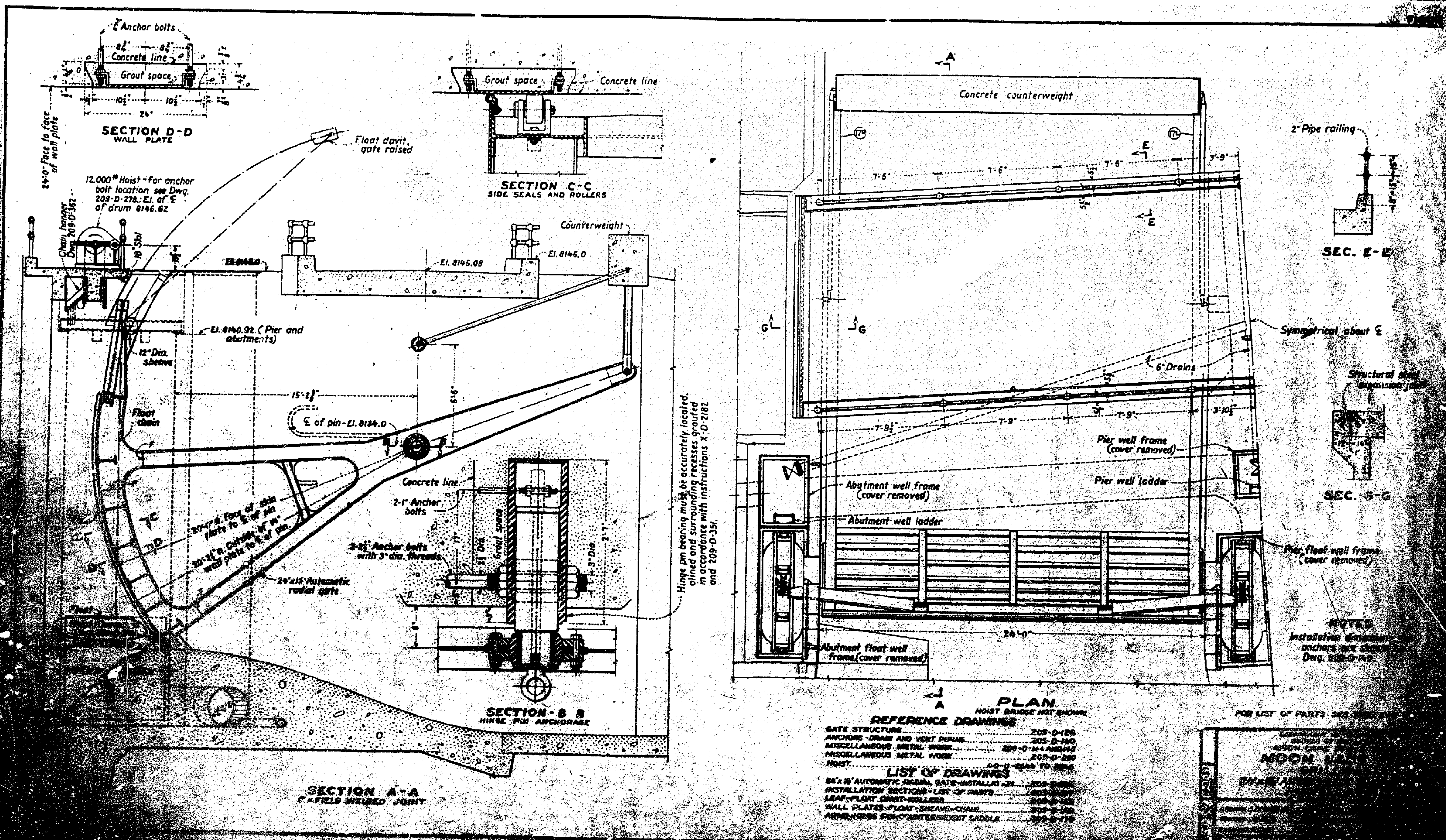
Although this type of pool would permit a wide range of tailwater and would give quite satisfactory protection against dangerous scour, it was not a satisfactory solution in this particular case because of the excessive excavation required to construct it. It would be satisfactory for a structure having an excess of tailwater over that needed to absorb the energy from the spillway jet by the hydraulic jump.

MODEL STUDIES OF AUTOMATIC SPILLWAY GATE

22. Principle of automatic gate operation. The Moon Lake automatic gates are of the radial type 24 feet long and 18 feet high. They are designed to maintain a practically constant reservoir water surface elevation for all discharges through the gate. The gate assembly consists of the gate, counterweight, floats, float chambers, bleeder system, and weir intake to the float chambers (figures 29, 30, and 31). The counterweight is supported on arms extending through the gate pivot downstream from the gate. Movement of the gate is effected by increasing or decreasing the amount of water in the float chambers. Floats are suspended in the float chambers by cables directly attached to the gate. These floats, there being two for each gate, are loaded, having a specific gravity somewhat greater than one. The gate is in an equilibrium position when the floats are about half submerged. In other words the moment of the counterweight equals the moment of the gate plus the moment of the floats when they are half submerged. When the water in the float chambers rises it exerts additional buoyant force on the floats and decreases their effective weight. The moment of the counterweight is then greater than that of gate and floats combined and the gate starts upward. When the water drops in the float chambers to a position below the midpoint on the float, the effective weight of the float is increased and the moment of gate and float of greater than that of the counterweight, hence the gate starts downward. As there is a certain amount of friction at the gate pin bearings, along the gate seals, and in flexing the cables, the



HALF PLAN
HALF SECTION A-A
Reservoir intake
2" Ф G.I. std pipe drains
6" G.I. Standard pipe well drains
2" Wrought iron pipe air vent
For details of weir box see Dwg. 209-D-363
Ctr. pipe connection Sta. 6+19.70
20" Fillet
Well supply Sta. 6+10.30
Structural steel ladder
6" Std. gate valve with each 6" well drain
Sta 6+35.75
Sta 6+29.35
Sta. 6+00.75
Sta. 5+91.85
El. 8116.00
SECTION C-C
Hoist bridge not shown
Bridge beam slots
3'x3'x2'-2" Grout pocket for pin casting
Pin
El. 8120.29
El. 8117.00
El. 8121.00
El. 8117.84
El. 8116.00
Metal water stops at all joints upstream from gate
30" Dia. well supply
Crimp each side
20 Gauge metal. Total width of sheet 13"
DETAIL OF METAL WATER STOP
DETAIL OF BRIDGE ABUTMENT JOINTS
1" Layer of cork
Elastic joint filler
SECTION E-E
El. 8145.00
El. 8114.75
SECTION H-H
GATE HOIST BRIDGE
SECTION F-F
Bridge slab
Parabolic crown
SECTION B-B
2" Ф G.I. std. pipe rail
El. 8145.00
Road crown El. 8145.083
El. 8137.0
24'x16' Radial gate
Crest El. 8121.00
El. 8117.00
El. 8116.10
El. 8114.75
El. 8116.20
El. 8107.00
This face keyed for adjoining wall and footing
SECTION D-D
Hoist bridge not shown
Bridge crown El. 8145.083
W.S. El. 8137.00
El. 8139.00
El. 8135.50
El. 8134.00
2" Wrought iron pipe air vent
3'x3' Grout pocket for pin casting
6"x6"x8" Pipe connection
8" G.I. std. pipe outlet
Outlet invert El. 8115.93
El. 8113.25
El. 8117.00
El. 8114.75
El. 8107.00
FLOW
Crest
SECTION G-G
Fillet
NOTES
Scale of Feet
MOON
GATE STRUC

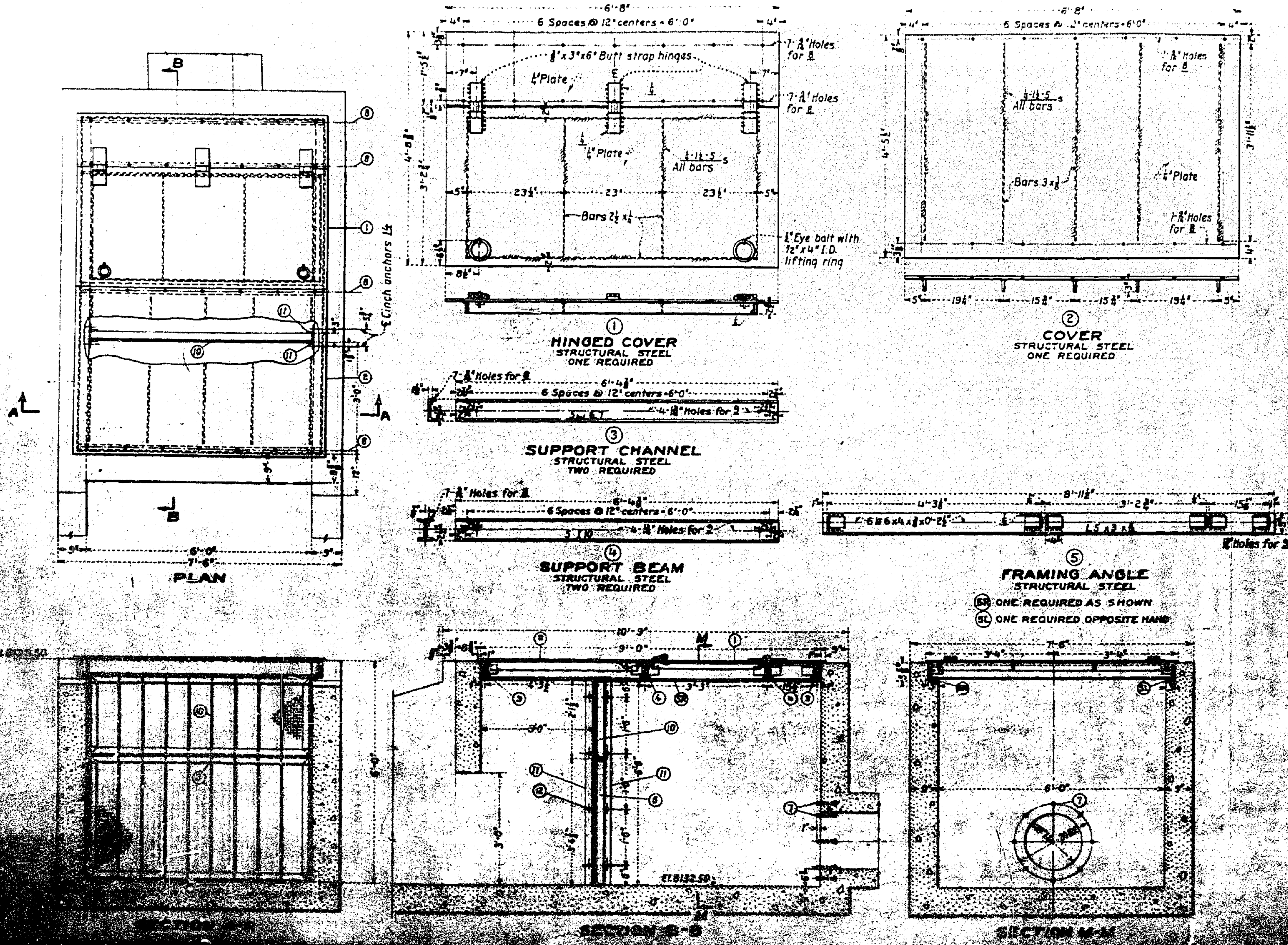


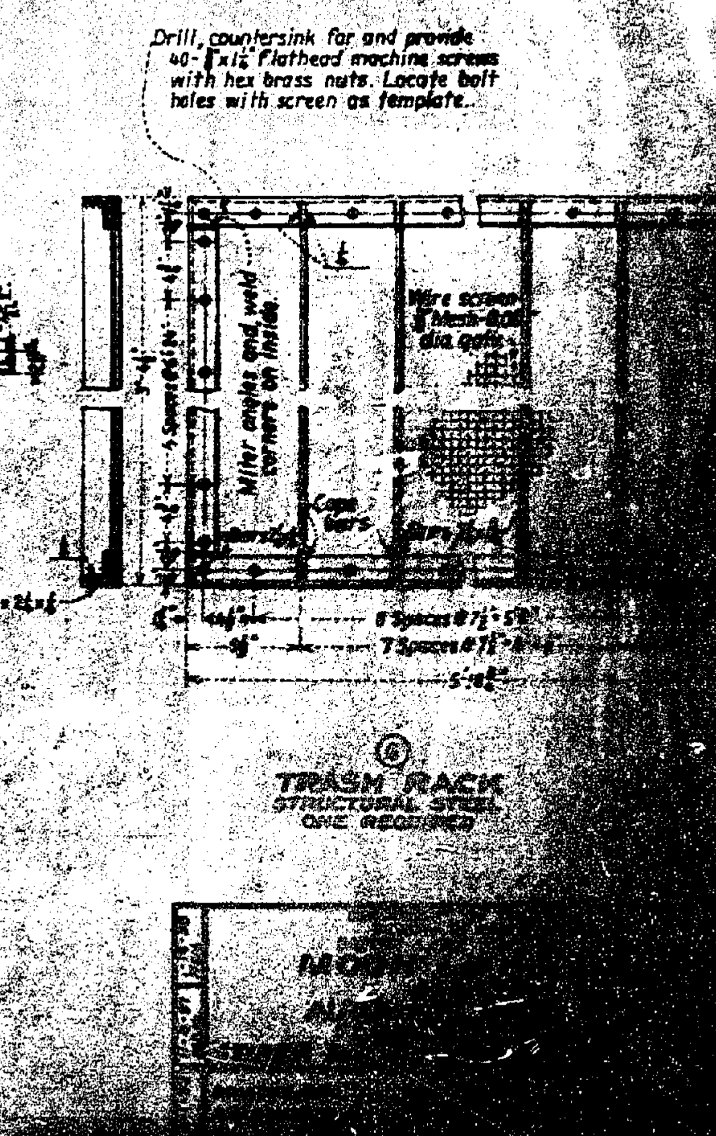
3/4" Anchor bolts
Concrete line
Grout space
SECTION D-D
WALL PLATE
24'-0" Face to face of wall plate
Float davit, gate raised
Grout space
Concrete line
SECTION C-C
SIDE SEALS AND ROLLERS
12,000# Hoist - For anchor bolt location see Dwg. 209-D-278. El. of ℄ of drum 8146.62
Chain hanger Dwg. 209-D-362
Counterweight
El. 8145.08
El. 8146.0
El. 8140.92 (Pier and abutments)
12" Dia. sheave
15'-2 3/8"
6'-6"
℄ of pin - El. 8134.0
Float chain
Concrete line
2-1" Anchor bolts
2-2 1/2" Anchor bolts with 3" dia. threads
Grout space
24'x15' Automatic radial gate
Hinge pin bearing must be accurately located, alined and surrounding recesses grouted in accordance with instructions X-D-2182 and 209-D-35I.
SECTION - B B
HINGE PIN ANCHORAGE
SECTION A-A
F-FIELD WELDED JOINT
Concrete counterweight
2" Pipe railing
SEC. E-E
Symmetrical about ℄
6" Drains
Structural steel expansion joint
Pier well frame (cover removed)
Pier well ladder
SEC. G-G
Abutment well frame (cover removed)
Abutment well ladder
Pier float well frame (cover removed)
24'-0"
Abutment float well frame (cover removed)
NOTES
PLAN
HOIST BRIDGE NOT SHOWN
REFERENCE DRAWINGS
GATE STRUCTURE 209-D-126
LIST OF DRAWINGS
MOON LAKE

LIST OF PARTS

PART NUMBER	DESCRIPTION	MATERIAL	NUMBER REQUIRED
1	Hinged cover	Structural steel	1
2	Cover	" "	1
3	Support channel	" "	2
4	Support beam	" "	2
5R	Framing angle	Structural steel	1
5L	Framing angle	" "	1
6	Trash rack	Structural steel	1
7	5/8"x 5" Sq. Hd. Mach. bolt & Sq. nut	Class "C" bolt steel	[illegible]
8	1/2"x1 1/4" Sq. Hd. Mach. bolt & Sq. nut	" " " "	[illegible]
9	3/4"x2" Sq. Hd. Mach. bolt & Sq. nut	" " " "	[illegible]
	For part numbers 10 to 14		
	inclusive, see dwg. 209-D-371.		

Notes:- Bolts called for are about 5% in excess of the number required.
All surfaces to be given one shop coat of bituminous primer.
All welding to be by electric arc process.
For welding symbols see Dwg. X-D-1357.





effective weight of the float, when half submerged, must be considerably decreased to cause movement of the gate upward, and likewise considerably increased to cause movement downward. This occurs when the water surface in the float chambers, respectively, rises or falls a quarter of the float height.

It may, therefore, be seen that motion of the gate depends on movement of water surface in the float chambers. Water is admitted to the float chambers from the reservoir which is being controlled. It must first flow over a weir and then into the chambers. Discharge into the float chambers increases with elevation of water surface in the reservoir, and is zero when elevation of the water is the same as, or less than, elevation of the weir.

Drains or bleeders in the bottom of the float chambers permit a continuous discharge from the chambers when they contain water. When the reservoir elevation is the same as the elevation of weir crest no water is admitted to the float chambers and the bleeders will drain the float chambers; hence the gate will close completely. When the elevation of reservoir is such that discharge over the weir equals bleeder discharge with float chambers filled, the gate will open completely. For reservoir levels between the above two conditions the gate will be maintained in any intermediate position holding the reservoir level constant.

23. Theory of gate control. It is possible to compute operation of the gate for any given flood, provided:

(1) Coefficients of gate discharge and bleeder system are known;

(2) Friction of the gate system can be closely estimated, and

(3) Backwater on the bleeder outlet is known for various gate openings.

The two differential equations controlling the system are:

(1) $Q_i \, dt = Q_g \, dt + A_r \, dy$

(2) $Q_w \, dt = Q_b \, dt + A_f \, dx$

wherein:

Q_i = discharge into reservoir = $F_1 (t)$

Q_g = discharge through gates = $F_2 (x, y)$

y = elevation of reservoir = $F_3 (t)$

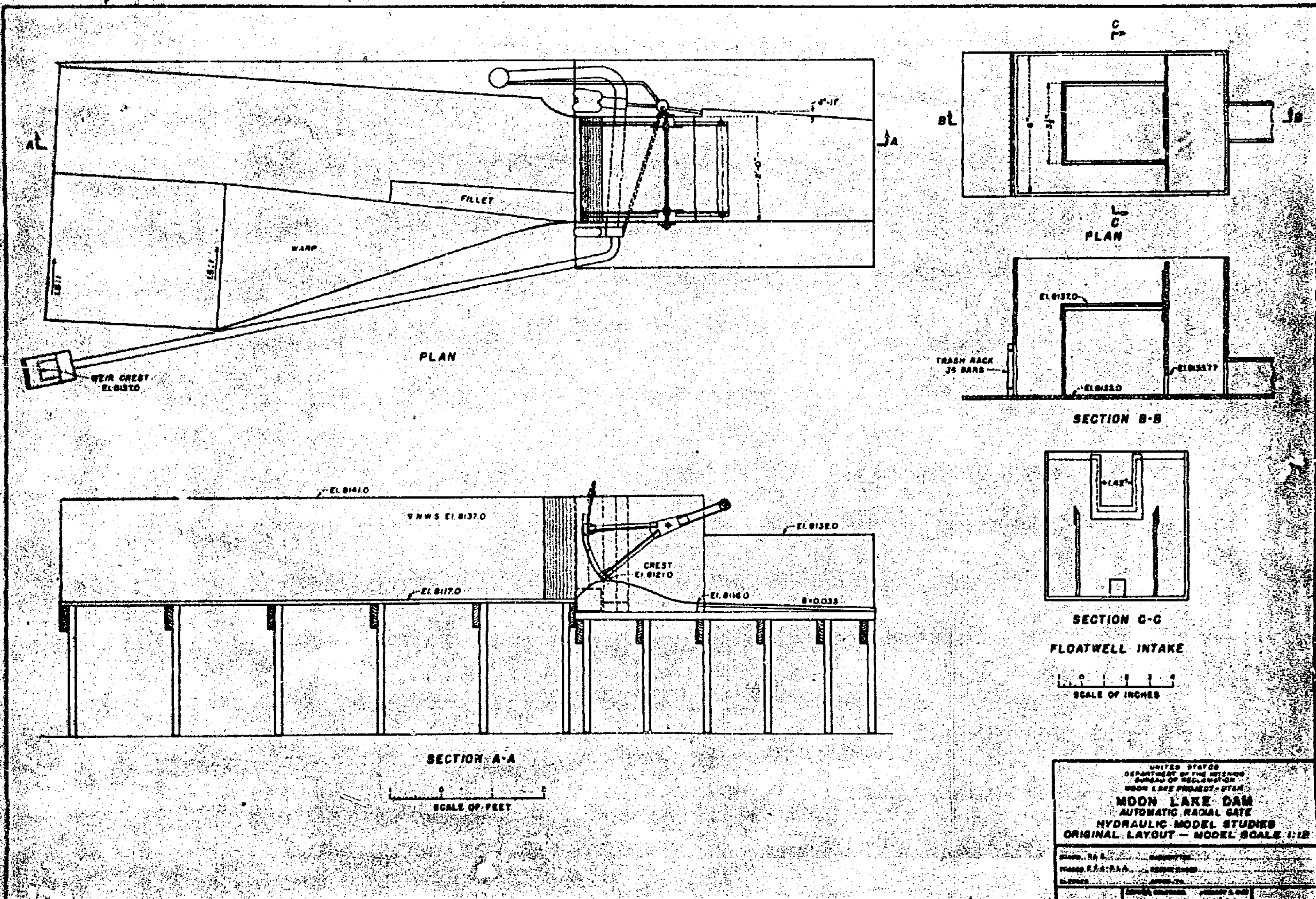
x	=	gate opening; also height of water in float chambers	$= F_4 (t)$
t	=	time	
A_r	=	area of reservoir	$= F_5 (y)$
Q_w	=	discharge over weir	$= F_6 (y)$
Q_b	=	discharge through bleeders	$= F_7 (x)$
A_f	=	area of float chambers	$= F_8 (x)$

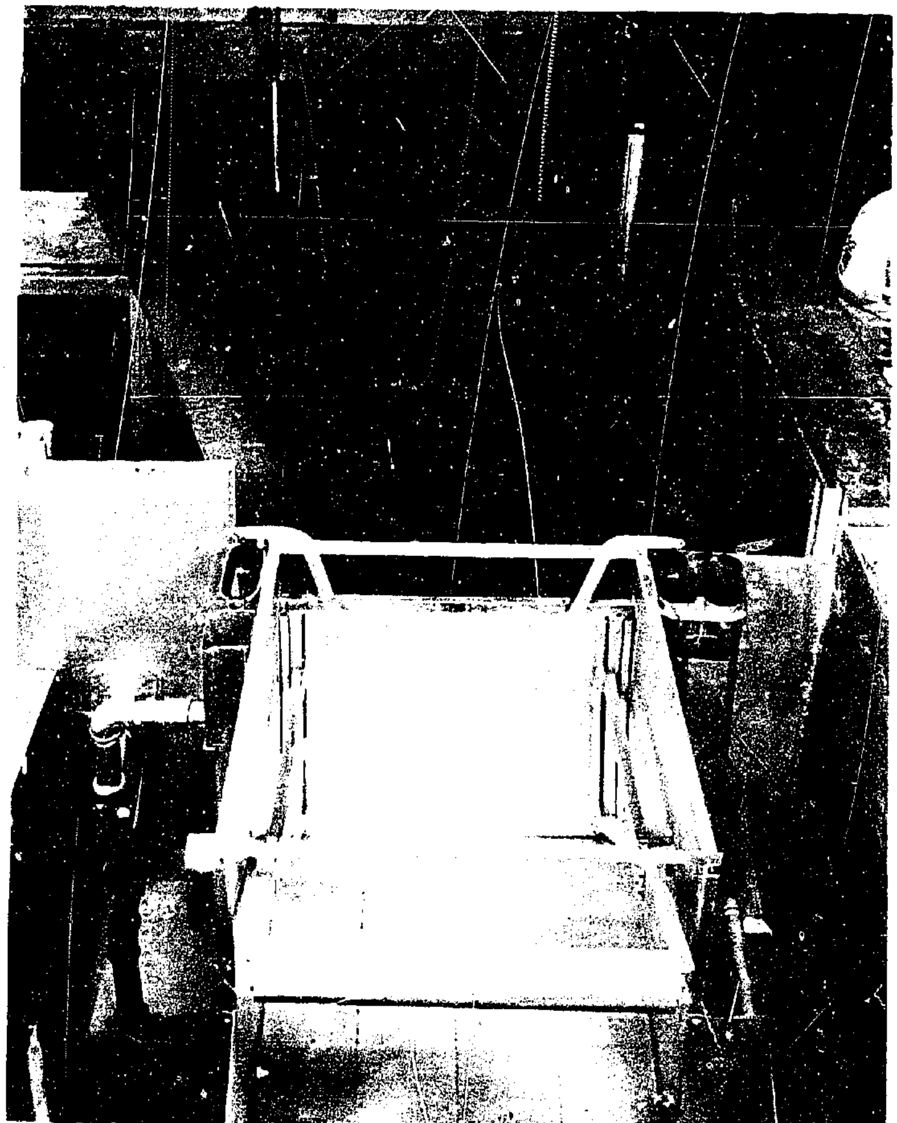
There is no general method of solving the two simultaneous differential equations. To compute gate operation $F_1 (t)$ is assumed; F_2, F_5, F_6, F_7, and F_8 are expressed in algebraic form; initial conditions are assumed; and F_3 and F_4 are computed. This computation can be made by approximate methods, or by numerical integration.

In general the process is tedious, and breaks in the process must be made for changes in direction of motion of the gate. Any desired accuracy may be obtained by the numerical solution, but the answer will be no better than the assumed coefficients and friction.

24. Model construction and operation. In the laboratory study of the gate, a scale of 1:12 was selected for a model. The maximum discharge through one of the prototype 24- by 16-foot gates is 5,000 second-feet. The selected scale reduced model discharge to 10 second-feet which was approximately the capacity of the 12-inch laboratory pump. The model, which comprised half of the actual gate section was made of galvanized iron and steel, with the crest of concrete and approaches of wood. The counterweight was a 2½-inch pipe closed on both ends, and loaded with mercury (figure 32). In the model the float chambers were installed for both gates. This permitted the intake box for the float chambers to be scaled down directly. The bleeder system was also installed for two gates (figures 32 and 33 show the gate in place).

During the first runs on the original design, the gate action was extremely poor. For small discharges, the action of the gate was intermittent. It would open slightly, close, then open again, somewhat more the second time and stay open a few seconds. It would then close and the procedure would be repeated again. Such action would not occur with the prototype, as the reservoir could not be lowered so rapidly.


PLAN
FILLET
WARP
WEIR CREST EL 8137.0
PLAN
SECTION B-B
TRASH RACK 35 BARS
EL 8137.0
EL 8133.0
SECTION C-C
FLOATWELL INTAKE
SCALE OF INCHES
EL 8141.0
N.W.S. EL 8137.0
EL 8117.0
CREST EL 8121.0
EL 8116.0
EL 8132.0
S=0.033
SECTION A-A
SCALE OF FEET
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT - UTAH
MOON LAKE DAM
AUTOMATIC RADIAL GATE
HYDRAULIC MODEL STUDIES
ORIGINAL LAYOUT — MODEL SCALE 1:12



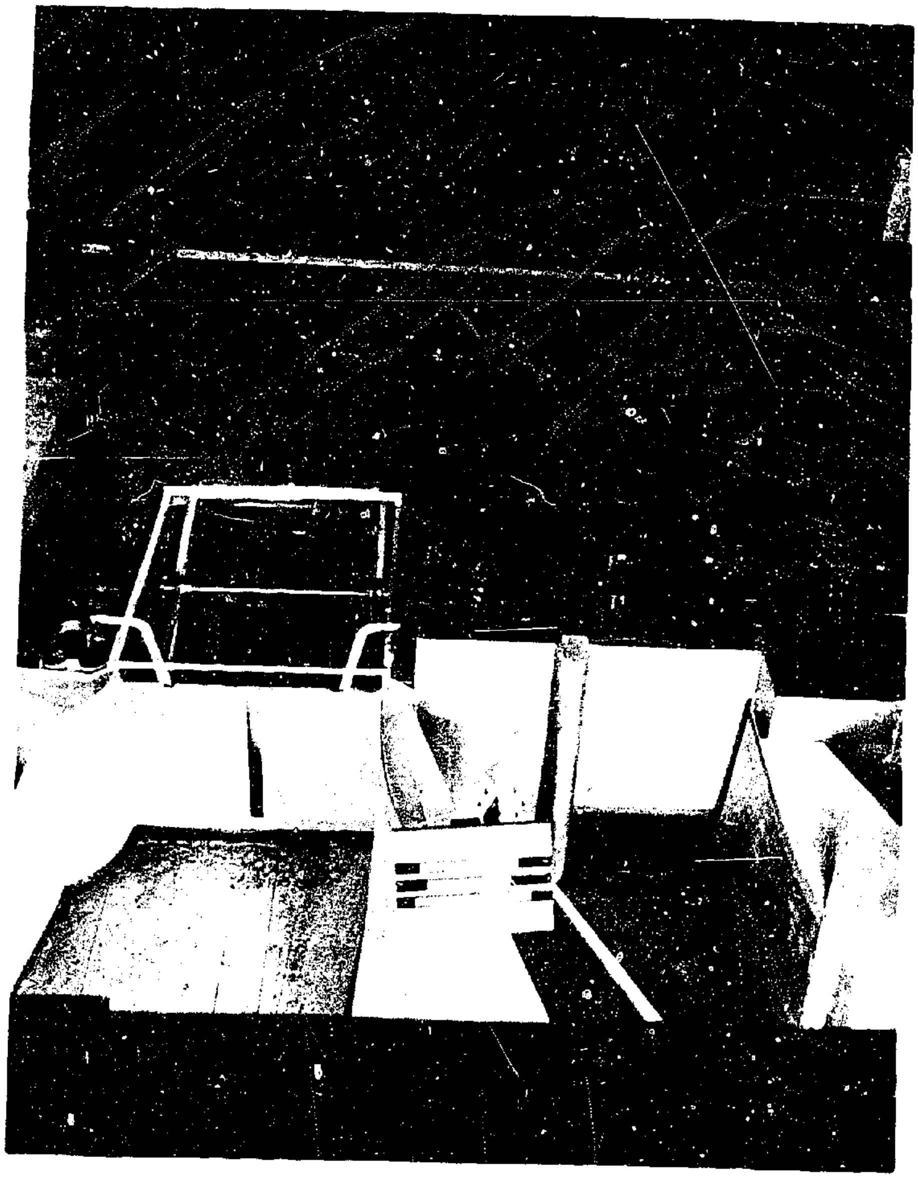
DOWNSTREAM VIEW OF GATE MODEL.

The forebay for the gate, which was the rating flume of the laboratory, was 8 feet wide and 140 feet long, or about 1,100 square feet in area. If the forebay were to scale, the surface area would have been 114,980 square feet. Hence it was about one percent of its scaled size. To improve this condition, such that the gate would operate more nearly like the prototype, a shield with a vertical slot in the center was installed immediately upstream from the gate as shown in figures 34 and 35. The slot extending from top to bottom of the gate was equal to 1/24th the width of the gate section. With this arrangement the effective forebay was 24 times larger, or about 1/4th the scaled size. This resulted in a scale ratio of 1:12 for the automatic parts of the gate, such as intake, weirs, floats, float chambers, and bleeder system. The forebay was, however, to a scale of 1:25. This alteration affected the action of the gate in changing the elevation of reservoir with respect to the weir crest, and in changing the amount of backwater over the bleeder outlet. Without the shield, the water surface in the forebay fluctuated greatly with gate movement. With the shield in place the drawdown around the intake box was the same order of magnitude as in the prototype. The lack of backwater over the bleeder resulted in somewhat greater bleeder discharges, but this could be remedied by throttling the valve on the end of the bleeder pipe.

Ideal operation of an automatic gate would require that gate discharge equal inflow into the reservoir at all times. This would maintain the reservoir at one elevation. In actual operation the gate opening is a function of reservoir elevation and time. The reservoir water surface must then rise above the weir crest to open the gate. Theoretically, the gate could be made to open wide for any arbitrary rise in reservoir by adjusting the size of intake weir and bleeder outlets. It would be impractical for the gate to operate on a very small change in reservoir elevation, as wind could pile water up in the downstream end of the reservoir and cause the gates to open, when actually the average elevation of the reservoir had not changed. In gate operation it is important that waves not open the gates.

In the model operation water was introduced into the upstream end of the forebay at a constant rate. Behavior of the gate was then observed as it adjusted itself to the inflow. The inflow was then changed and subsequent adjustments of the gate observed. With the shield in place the original design operated satisfactorily when there were no waves on the forebay. With waves on the forebay the discharge over the skimming weir in the intake box was much greater. Hence the gate overtravelled and the reservoir level was drawn down below the level at which it should have been controlled. It was therefore necessary to apply damping between the reservoir and the

Figure 34.



UPSTREAM VIEW OF GATE MODEL WITH SHIELD IN PLACE.

Figure 35.

DOWNSTREAM VIEW OF GATE MODEL IN OPERATION
WITH SHIELD IN PLACE.

weir intake, so that the water level on the upstream side of the weir would not fluctuate greatly. An orifice was unsatisfactory for damping purposes as

$$Q = Ca\sqrt{2gh} \quad , \; h = h_0 + h_1 \sin bt$$

$$\frac{b}{2\pi}\int_0^{\frac{2\pi}{b}} Q \, dt = \frac{b}{2\pi} \; C a \int_0^{\frac{2\pi}{b}} \sqrt{2g\,(h_0 + h_1 \sin bt)} \text{ for } dt < Ca\sqrt{2gh_0}$$

in which: h_0 = average head on orifice

h_1 = wave height

c = coefficient

a = area of orifice

Hence with waves, an orifice will discharge less when the water level is constant at the mean depth. A pipe provided satisfactory damping not because of friction, but due to inertia of the column of water in the pipe. To provide damping of this sort, the weir was removed from the intake box, and installed in the right-hand calming well. The intake structure was retained, with the exception of the weir. The trashrack was enlarged so that it would not act as an orifice. With this arrangement gate operation was not affected by waves on the forebay. The 100 feet of 18-inch pipe connecting the intake box to the skimming weir produced sufficient damping. A weir about 8 feet long was used in the calming well. As it requires more head than the weir, the pipe acts as a control even more so than the weir.

25. Alterations of models. The following changes were recommended in the final design:

(1) A weir, 8 feet long, installed in the right-hand calming well with crest at elevation 8156.60.

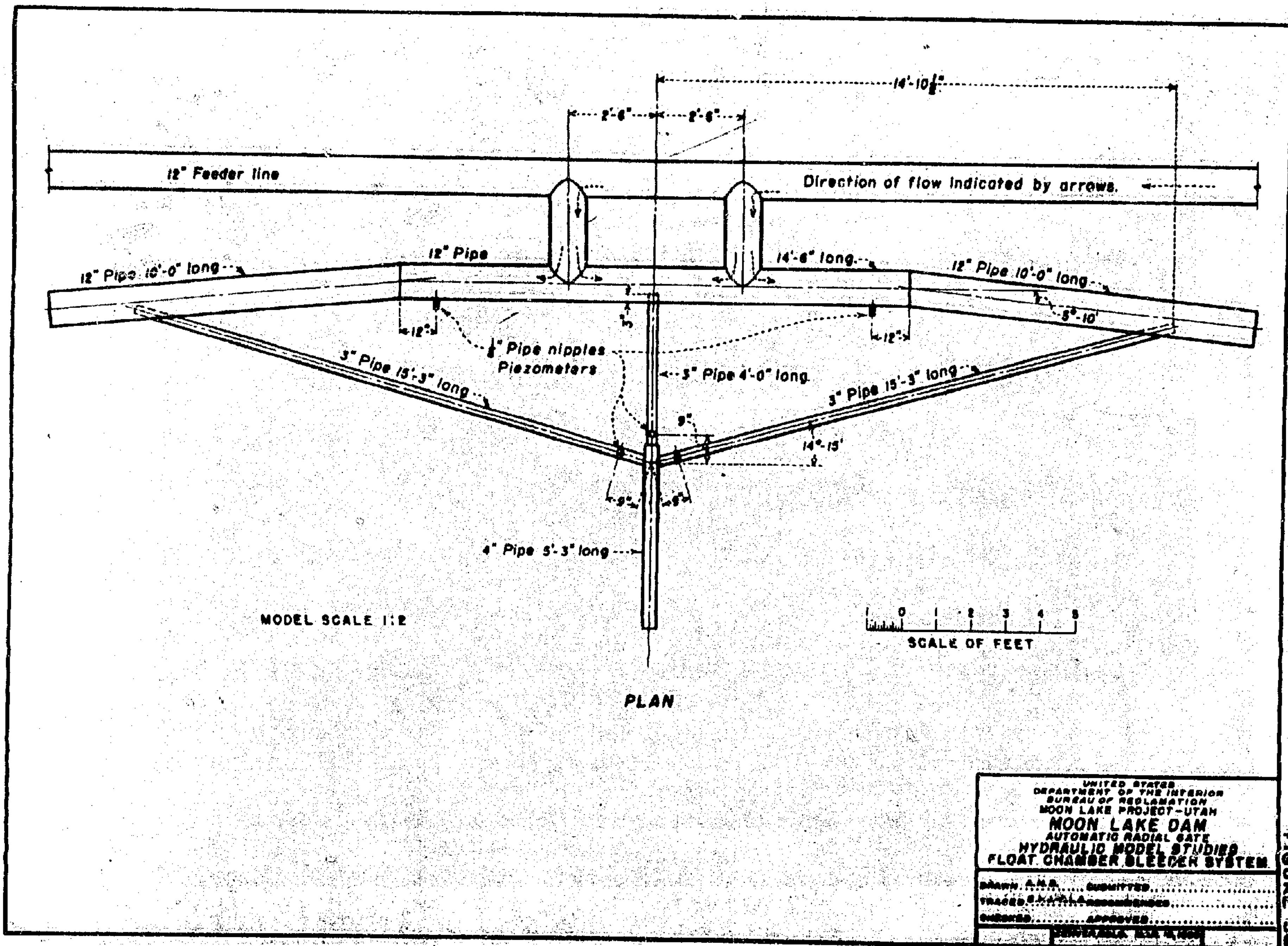
(2) Removal of weirs from intake structure and installation of an 18-inch pipe leading from intake structure to the upstream side of the weir in the calming well. Trashracks enlarged to extend over the entire upstream side of the intake structure.

(3) Bleeder valve in center calming well to remain closed during operation.

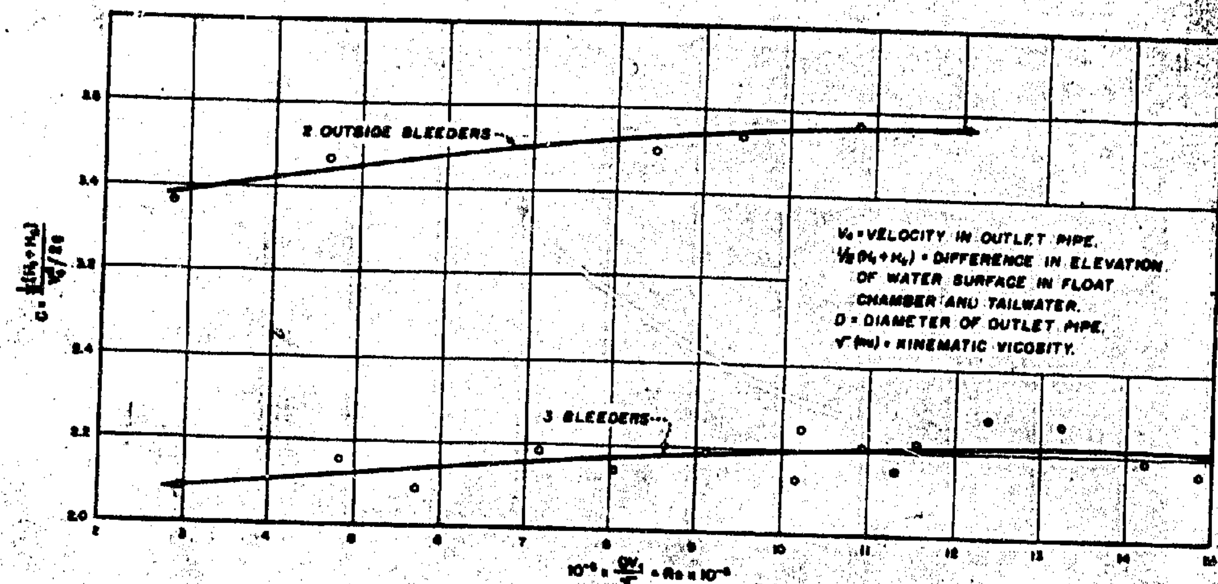
26. Crest calibration. Gate coefficients were obtained for partial openings on the 1:12 model of one of the two gates. These are plotted against gate opening (figure 37).

27. Model of float-chamber bleeder system. A model, to a scale of 1:2, (figure 36), was made of the bleeder system for the float chambers. Satisfactory information concerning head losses at

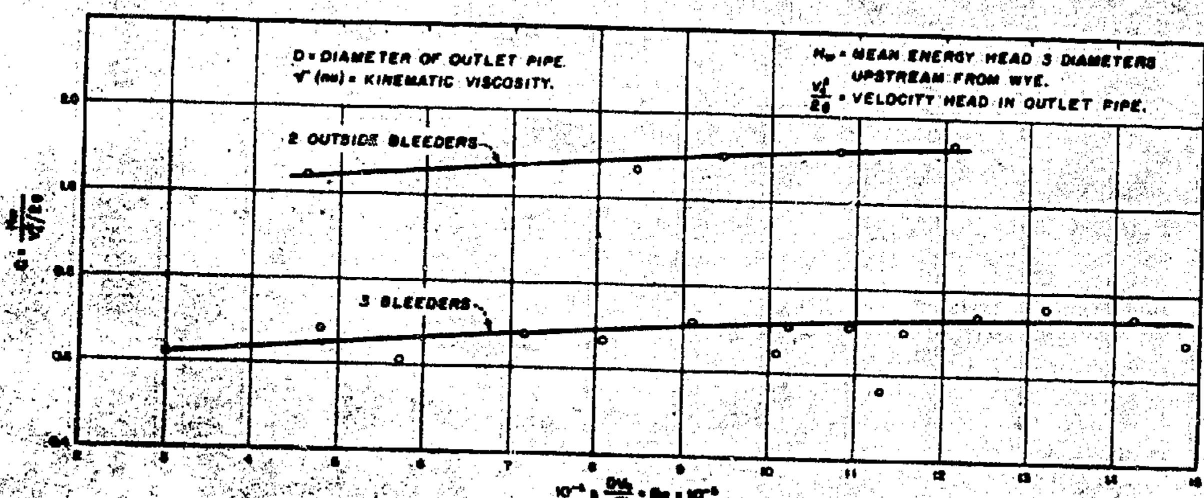
FIGURE 36



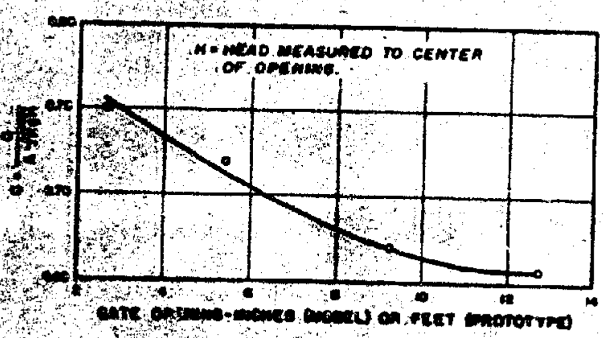
PLAN



TOTAL LOSS COEFFICIENTS OF BLEEDER SYSTEM



LOSS COEFFICIENTS AT WYE JUNCTION



FOR OPEN GATE C = 3.30
IN Q = CLH$^{3/2}$ FOR H = 17.47 FEET.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MOON LAKE PROJECT—UTAH

MOON LAKE DAM
AUTOMATIC RADIAL GATE
HYDRAULIC MODEL STUDIES
CALIBRATION CURVES

DRAWN..........K.L.B.......... SUBMITTED..........
TRACED..........R.F.A.-R.A.A.......... RECOMMENDED..........
CHECKED.......... APPROVED..........
DENVER, COLORADO

the junction of the three 6-inch bleeder pipes with the 8-inch outlet pipe was not known. A search of the literature on junction losses showed that it could be taken from $0.5 \frac{V_8^2}{2g}$ to $5.0 \frac{V_8^2}{2g}$, where V_8 represents the mean velocity in the outlet pipe. Coefficient curves obtained from the model are shown in dimensionless graphs (figure 37).

COMPENSATION FOR DISPORTIONATE FRICTIONAL RESISTANCE IN OPEN CHANNEL HYDRAULIC MODELS

28. Necessity for correction. Hydraulic models are constructed for the purpose of anticipating on a small scale the operating characteristics of proposed large scale installations. The information generally sought includes such data relative to the prototype as depth of water; location, size, and periodicity of waves; velocities; and pressures. The primary purpose of a model is to provide a means of predicting accurately those prototype properties.

In the usual spillway model, it is desired that the flow shall be as nearly geometrically similar to that of the prototype as possible. The flow lines should follow geometrically similar paths in model and prototype so that the depths of flow and the size and position of the hydraulic jump and other standing waves as well as the hydrostatic pressures and velocities of the prototype can be calculated. Such geometrical similarity implies simple scale relations according to which all significant dimensions of the prototype structure and flow should be anticipated in the model.

In practical laboratory model construction, it is relatively simple to obtain geometrical similarity of the structure unless wall roughness or wall texture is considered as a geometrical property. The forces arising from the effects of gravity - impact, pressure, centrifugal force - are consistent with the linear scale, in accordance with Froude relationship $\frac{V^2}{gd}$ = constant. However, the forces arising from the resistance of the viscous friction within the fluid or between the fluid and the boundaries are not reduced with the scale according to the same relationship. Consequently, unless some account is taken of this discrepancy, the flow patterns in the model cannot be said to anticipate truly those of the prototype.

In a spillway model, the viscous forces may be influenced by the air resistance at the surface, the wall resistance which is affected by the texture of the wall, and the viscosity of the fluid which is affected by the temperature and constituency.[1,2]

[1] Hydraulic Models, Manuals of Engineering Practice, No. 25, American Society of Civil Engineers, p. 9.
[2] Russell, George A., Hydraulics, 5th edition, 1941, pp. 91 to 99.

The resistance of the air, on the other hand, is thought to be very different in the model from that in the prototype. Indications are that because of the lower model velocities, air resistance in the models is considerably less than the appropriate values. This tends to produce higher velocities at the bottom of model chutes and spillways than would be in accord with consistent similitude relations. Directly opposed to this influence is that of wall resistance, which in the model is usually higher than appropriate, and which, therefore, tends to produce lower than consistent pool velocities.

Of the two factors, air resistance, and wall resistance, which do not vary properly with the geometric scale, only the wall resistance is subject to even approximate analysis in open channel work by present methods. Whether compensation for the one factor is advisable in view of the inadequate information concerning the effects of the others is a matter of much discussion.

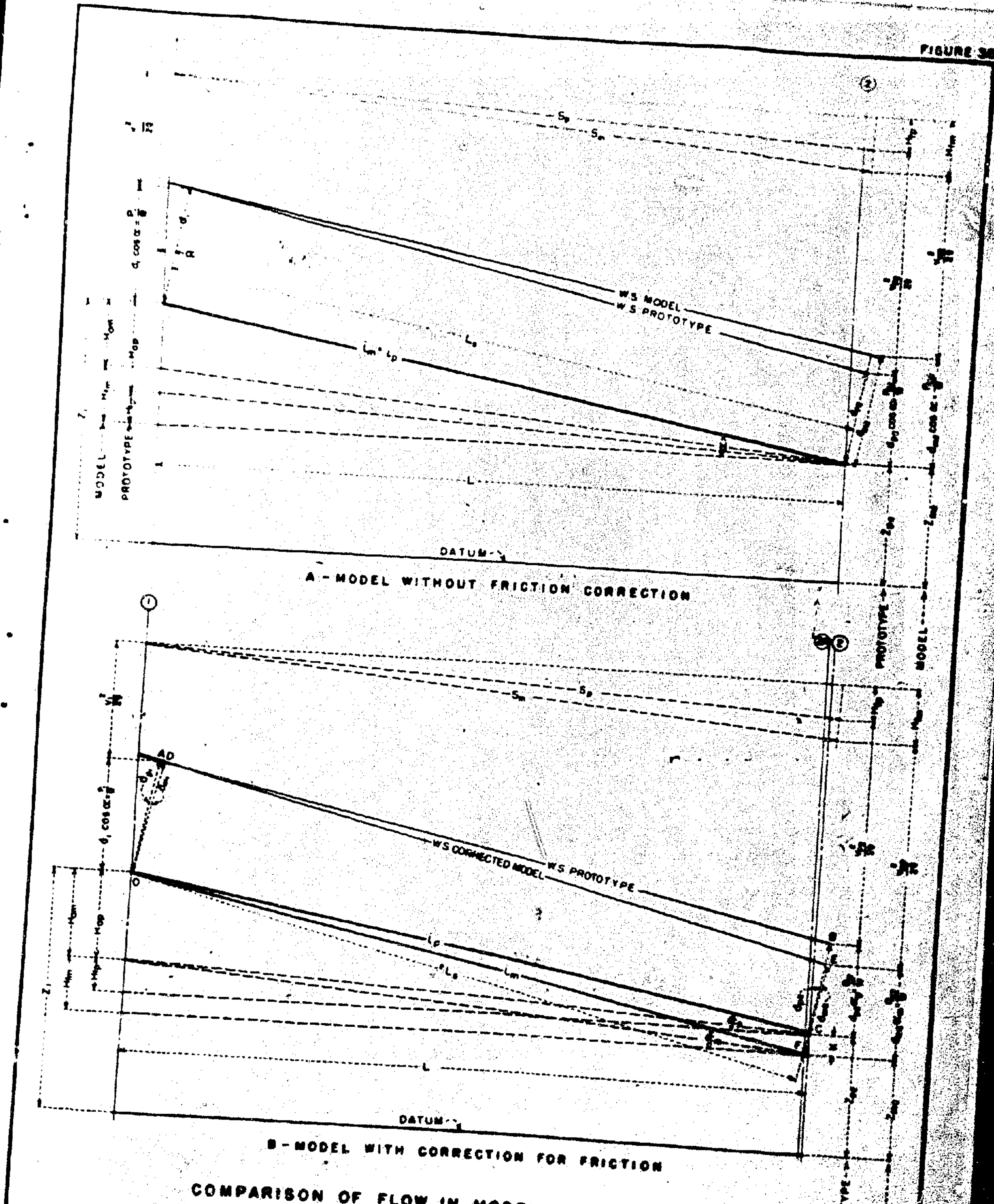
An examination of figure 38B will show that it is impossible to have geometrically similar flow in model and prototype even if a correction is applied by modifying the slope. Suppose the section of prototype channel OABC is rotated downward about the point O to the position ODEF. Then OABC = ODEF, and OA = OD, but when the two depths are converted into pressure head $\frac{P1p}{w} \neq \frac{P1m}{w}$. Likewise, the depths CB = FE at section 2 are equal, but the pressure heads which according to convention are represented vertically are different. The greater the original slope of the channel and the larger the correction slope in the model, the greater will be the deviation in the two pressure heads at section 2. Another error, which though small and usually neglected in mild slopes, is the slant length of reach L_s. It would appear that to be correct, section 2 should be moved upstream to section 2A for the model (figure 38B). Ordinarily, this refinement is not necessary.

In the case of steep slopes - 0.5 or greater - variables enter the problem which are not so readily understood. There is no proof, for instance, that the channel friction continues to be proportional to the square of the velocity. According to Harris,[3] "It is

[3] Harris, C. W., Hydraulics, John Wiley and Sons, p. 114

quite possible, in open channels, for the friction head to increase

FIGURE 38



COMPARISON OF FLOW IN MODEL AND PROTOTYPE [illegible] a fact that has been [illegible] incorporated in [illegible] greater [illegible] Chezy's equation [illegible] 11 feet per second. [illegible] Chezy's equation is [illegible] of actual depths on [illegible] work.

[illegible] velocities reach the [illegible] causes a mixing [illegible] volume of the [illegible] volume of water. It [illegible] are at work in the [illegible] and air resistance - [illegible] in the model. [illegible] the flow is considered [illegible] the water to a terminal [illegible]. Schoklitsch[4]

[illegible] Energy Dissipation, [illegible] Hydraulic Report [illegible]

[illegible] 15 feet per second, while [illegible] in a particular zone be-

[illegible] of the Flow of Water in [illegible] Hydraulic Report [illegible]

[illegible] per second, where a velocity [illegible] Manning's formula. [illegible] stream approaches a [illegible] sufficient to balance the [illegible] cannot. On [illegible] to attempt a slope cor- [illegible] and at the same [illegible] which is probably [illegible] steep chutes.

[illegible] available, a com- [illegible] correction would be as

[illegible] for all mild slopes of moderate [illegible] will usually be small but [illegible] and depths are to be duplicated

(2) Neglect the slope correction in steep chutes when the velocity approaches or is greater than 50 feet per second (prototype) as the model no longer simulates the prototype flow under this condition. With higher velocities, the air resistance provides a large factor of safety.

29. <u>Computation of flow in open channels.</u> The flow equation probably used most frequently by designers in computing the flow in open channels is

$$v = C\sqrt{RS} \qquad (1)$$

where C is a constant depending on the channel lining, v is the average velocity, R the hydraulic radius or hydraulic mean depth, and S is the slope of channel bottom expressed as the ratio of the vertical fall, h, to the length of channel l. This equation is known as the Chezy formula.

The value of C has been determined by a number of experimenters and those values are presented in tabular or diagram form in most standard textbooks [6,7] on hydraulics. The two derivations most

[6]Russell, George A., Hydraulics, 5th edition, 1941, pp.269-277.
[7]King, Horace W., Handbook of Hydraulics, 3rd edition, 1939, pp. 265-283.

frequently used are those of Kutter and Manning each of which depends upon a roughness factor n.

Ganguillet and Kutter in 1869, from then available open-channel experiments derived a formula for C which is dependent upon the slope S, the roughness n, and the hydraulic radius R, which is

$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \left(41.65 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}} \qquad (2)$$

This is a cumbersome equation to solve. However, Creager and Justin[8],

[8]Creager, W. P., and Justin, Joel D., Hydro-electric Handbook, 1927, p. 122.

Scobey[9], and others show diagrams which considerably simplify the procedure.

[9]Scobey, Fred C., Flow of Water in Irrigation and Similar Canals, U.S. Department of Agriculture, 1939, p. 66.

A much simpler equation for C is that of Manning which is

$$C = \frac{1.486\ R^{1/6}}{n} \qquad (3)$$

Russell[4] and King[5] compare the results from the two formulas.

Both formulas were derived from extensive studies of data obtained from canals and rivers in which velocities were much lower than those in steep chutes and spillways. There is a probability that neither formula yields results too closely approximating these high-velocity ranges.

30. Analysis of prototype spillway channel. The reservoir elevation, the maximum flow, and the height and type of crest are factors fixed by design considerations. As so fixed, they may not prove mutually consistent; any error in choosing the crest coefficient would result in either altered flow or altered reservoir elevation. With these quantities fixed, the problem is to determine whether the velocities and depths of water in the tentatively selected stilling pool are within permissible limits. The process is one of balancing losses, velocity heads, and depths against available energy head from reservoir to stilling pool by trial and error. If the reservoir elevation be taken as the initial energy head, then the summation of elevation, pressure head, velocity head, and accumulated loss in head due to friction at any station must equal the reservoir elevation.

Using the Moon Lake spillway as an example, the procedure of computing the hydraulic data for the prototype structure and then correcting for model analysis will be illustrated. With the reservoir elevation fixed, the elevation of the crest of the control section can be determined by applying the flow equation.

$$Q = C\ L\ H^{3/2} \qquad \text{or} \qquad H = \left(\frac{Q}{CL}\right)^{2/3} \qquad (4)$$

Using a value of C of 3.27 for the particular shape of crest and using 48 feet as the effective width of gate, and 10,000 second-feet as the maximum discharge,

$$H = \frac{10{,}000}{3.27 \times 48}^{2/3} = 15.88 \text{ feet.}$$

With the maximum reservoir surface at elevation 8137.0 and a head of 15.88 feet, or say 16 feet, the crest was located at elevation 8121.0. The section of the spillway crest indicates that the critical section will occur at or downstream from the crest.

Little is known of spillway entrance losses because of the complicated nature of the flow. It is customary therefore to start the calculation either with a rough estimate of the total loss down to some station just below the crest or with a trial and error estimate of the position and magnitude of the critical depth.

The position of this critical section can be determined by computing the value of the critical depth. The elements of the critical section for parallel flow are

$$d_c = \frac{v_c^2}{g} \qquad \text{or } v_c = (gd_c)^{\frac{1}{2}}$$

then

$$Q = b\,(g)^{\frac{1}{2}}\,d_c^{3/2} \tag{5}$$

Substituting and solving

$$d_c = \left(\frac{Q}{b\,(g)^{\frac{1}{2}}}\right)^{2/3} = \left(\frac{10,300}{45.0(32.2)^{\frac{1}{2}}}\right)^{2/3} = 11.05 \text{ feet}$$

From hydromechanics it is known that the distribution of pressure in a cross-section of a moving fluid will obey the hydrostatic law and will be affected only by gravity when flow occurs such that the fluid filaments have no acceleration components in the plane of the cross section. Movements of this type are known as parallel flow, the requirements of which are as follows:

(a) The stream lines should possess no substantial curvature.

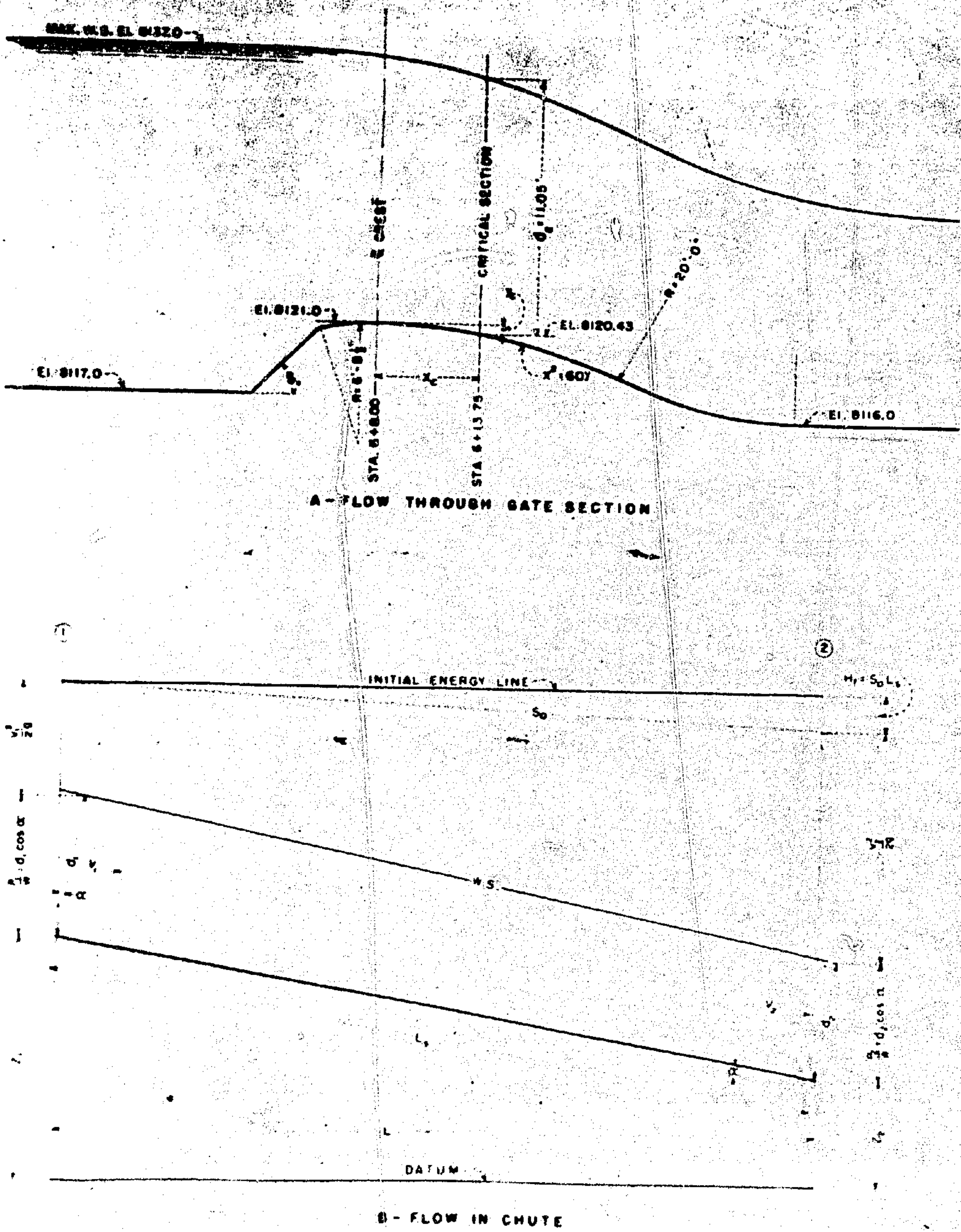
(b) The stream lines should not be substantially divergent.

The flow through the control section of the Mono Lake spillway (figure 39) does not satisfy the conditions of parallel flow inasmuch as it is modified by the curved channel. Since the force of gravity that acts in parallel flow is modified by the centrifugal force of the convex curvilinear surface of the invert of the channel, the minimum possible content of energy corresponding to a particular discharge is not equal to the minimum energy, 3/2 d_c, of parallel flow. Since this type of flow is difficult to evaluate and the inaccuracy of this source is limited to a short stretch of the surface curve, the critical section was determined by assuming parallel flow. With the additional assumption that the total head loss due to friction, h_f, down to the critical section is negligible, the elevation of the energy gradient line at the critical section will be

$$E_T = E_c = \frac{v_c^2}{2g} + d_c + Z_c \tag{6}$$

then $\qquad d_c = 11.05$ feet (assuming parallel flow)

and by equation 5 $v_c = (gd_c)^{\frac{1}{2}} = 18.83$ feet per second



FLOW IN THE PROTOTYPE

$$\frac{v_c^2}{2g} = 5.52 \text{ feet}$$

$$E_r = 5.52 + 11.05 + Z_c = 8137.0$$

$$Z = 8137.0 - 16.57 = 8120.43$$

$$y_c = 8121.0 - 8120.43 = 0.57 \text{ foot}$$

The horizontal distance from the crest to the critical section can be found from the equation of the crest $x^2 = -60y$

$$\text{or } x_c = 60 \times 0.57 = 5.85 \text{ feet}$$

which places the critical section at station 6 + 13.75.

From this section on down the chute, the calculation involves the successive solution of equation 6 as modified for the conditions encountered on the slope. At any station under consideration $v_c^2/2g$ becomes $v^2/2g$, d_c becomes $d \cos \alpha$; h_f is the accumulated head loss, E_c becomes E_b, the elevation of the bottom; and E_r remains the elevation of the reservoir water surface. Then

$$E_r = v^2/2g + d \cos \alpha + h_f - Z_b \qquad (7)$$

On steep slopes, the hydrostatic pressure head at any perpendicular depth, d, is d cos α, where α is the angle of the slope.

Two methods of solving equation 7 are available. The simpler method for long, continuous slopes is to assume a velocity increment between stations of some convenient amount, and solve directly for the length of the reach in which this increment occurs. According to King[5], accuracy in calculating the loss, h_f, requires that this velocity difference should not be greater than about two feet per second. If v is assumed, d becomes a known quantity from $d = q/v$, and h_f and Z_b may be expressed in terms of the reach L_s. Successive reaches are then calculated until the end of the chute is reached. The last reach must be determined by trial and error to correspond to the end of the chute.

The alternate method which is more generally applicable, particularly to channels with frequent breaks in slope, is to select an arbitrary reach based on convenient stations which fixes Z_b and L_s. A depth is assumed and v and h_f calculated. The authenticity of the assumption is checked by solving equation 7. This is a trial and error method throughout and is slower.

In order that equation 7 should be perfectly general, h_f would need to include all losses - air resistance, curvature, wall losses, aeration, etc. Little is known of the nature of any of these except wall losses, which are generally understood to be influenced by the roughness and irregularities of the wall and floor of any channel, and are in some measure proportional to the area of wetted surface and therefore proportional to the length of the reach. In consequence of this ignorance of the other factors, h_f is defined by

$$h_f = L_s S \text{ (figure 39B)} \tag{8}$$

where S is the head loss per unit length, or the energy gradient, and can be computed using the Chezy formula with either the Kutter or the Manning coefficient. The factor n, the roughness index, for a large number of engineering materials in various states of use have been defined[4]. The Moon Lake calculation was made using the coefficient C by the Kutter formula.

The remainder of the chute was analysed by applying the law of the conservation of energy (equation 7) to successive reaches. The closer the sections, the greater the accuracy. Commencing at the critical section at station 6+13.75, all values are known except the slope of the energy gradient S, and the value of the Chezy coefficient C. These are obtained by assuming a slope, S, for the energy gradient and obtaining a value of C. If the value of S as computed agrees with the assumed value, this figure is used as the slope of the energy gradient at section 1, figure 39B. If not, a new value of S is assumed, and the process repeated until an agreement is reached. When this is accomplished, then all the values are known at station 6 + 13.75 as shown in table 1.

Applying equation 7 between sections 1 and 2 (figure 39B)

$$Z_1 + \frac{P_1}{W} + \frac{V_1^2}{2g} = Z_2 + \frac{P_2}{W} + \frac{V_2^2}{2g} + SL$$

$$\left(Z_1 + \frac{P_1}{W} + \frac{V_1^2}{2g}\right) - \left(Z_2 + \frac{P_2}{W} + \frac{V_2^2}{2g}\right) = SL$$

or

$$\Delta\left(Z + \frac{P}{W} + \frac{V^2}{2g}\right) = SL \tag{9}$$

which is the condition to be satisfied.

The next section downstream (station 6+29.85, table 1) is then chosen (section 2 on figure 39B). A value of d is assumed

TABLE 1

HYDRAULIC COMPUTATIONS FOR PROTOTYPE

[illegible] LAKE DAM SPILLWAY

Station (1)	Elevation of Channel Floor (2)	Depth of Flow Ft. (3)	Pressure Head [illegible] Ft. (4)	Width of Section Ft. (b) (5)	Area of Section Sq. Ft. (a) (6)	Velocity Ft./sec. (V) (7)	Velocity Head $\frac{V^2}{2g}$ (8)	Hydraulic Radius Ft. (R) (9)	C in $V=C\sqrt{RS}$ (10)	Instantaneous Slope of Energy Gradient $S=\frac{V^2}{C^2R}$ (11)	Average Slope of Energy Gradient (S_a) (12)	Slant Length of Reach Ft. (L) (13)	Friction Loss $h_f=S_aL$ (14)	Eddy Loss h_e (15)	h_f+h_e (16)	Accumulation $\Sigma(h_f+h_e)$ (17)	Elevation of Energy Line $(Z+\frac{P}{w}+\frac{V^2}{2g})$ (18)	$\Delta(Z+\frac{P}{w}+\frac{V^2}{2g})$ (19)
0+00.00	[illegible]	Crest																
								n = 0.014, Slope of Channel Floor i = 0.[illegible]										
0+32.75	[illegible].40	11.[illegible]	11.[illegible]	60.[illegible]	[illegible]	16.[illegible]	6.[illegible]	0.70	137.[illegible]	0.[illegible]	-	-	-	-	-	-	[illegible]	-
0+60.84	6118.[illegible]	7.64	7.64	60.00	367.1	[illegible]	12.17	4.00	134.4	0.0084	0.0086	17.0	0.11	1.[illegible]	1.66	1.66	[illegible]	1.66
0+75.86	6114.[illegible]	7.66	7.[illegible]	48.66	344.[illegible]	[illegible]	13.00	5.17	134.1	0.0080	0.0081	80.0	0.4[illegible]	0.17	0.[illegible]	2.07	[illegible]	0.42
7+00.00	6112.[illegible]	[illegible]	[illegible]	57.[illegible]	[illegible]	[illegible]	13.[illegible]	4.56	135.0	0.0086	0.0077	80.0	0.35	-	0.[illegible]	2.46	[illegible]	0.57
7+70.25	6110.00	11.00	11.00	51.00	[illegible]	27.02	11.76	6.00	130.7	0.[illegible]	0.[illegible]	80.0	0.51	-	0.41	3.74	[illegible]	0.31
									n = 0.013	i = 0.[illegible]								
[illegible]	[illegible]	12.00	12.00	[illegible]	267.0	[illegible]	[illegible]	6.06	140.[illegible]	0.0042	0.0080	80.0	0.[illegible]	-	0.25	1.99	[illegible]	0.34
									n = 0.015	i = 0.[illegible]								
10+70.79	[illegible]	12.00	11.[illegible]	20.00	275.0	[illegible]	11.06	5.[illegible]	150.[illegible]	0.0[illegible]	0.0044	[illegible]	1.77	-	1.77	4.97	[illegible]	1.07
[illegible]	[illegible]	8.77	8.77	[illegible]	[illegible]	[illegible]	[illegible]	5.02	144.2	0.0090	0.0068	100.1	0.[illegible]	-	0.[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.46	147.1	0.0128	0.0108	100.1	1.0[illegible]	-	1.0[illegible]	6.73	[illegible]	1.0[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	144.[illegible]	0.[illegible]	0.0127	[illegible]	0.80	-	0.80	7.[illegible]	[illegible]	0.71
									n = 0.014	i = 0.[illegible]								
[illegible]	[illegible].13	[illegible]	[illegible]	[illegible]	170.[illegible]	[illegible]	[illegible]	[illegible]	126.[illegible]	0.0[illegible]	0.0678	[illegible]	2.11	-	2.11	[illegible]	[illegible]	[illegible]
[illegible]	[illegible].13	[illegible]	[illegible]	[illegible]	144.7	[illegible]	[illegible]	[illegible]	[illegible]	0.1297	0.0[illegible]	[illegible]	[illegible]	-	[illegible]	14.07	[illegible]	[illegible]
[illegible]	[illegible].13	1.[illegible]	1.78	[illegible]	121.7	[illegible]	[illegible]	1.[illegible]	119.[illegible]	0.2206	0.1780	[illegible]	[illegible]	-	[illegible]	[illegible]	[illegible]	[illegible]
17+[illegible]	[illegible]	1.70	1.[illegible]	71.00	127.[illegible]	[illegible]	[illegible]	1.[illegible]	137.[illegible]	0.[illegible]	0.[illegible]	[illegible]	7.[illegible]	-	7.[illegible]	[illegible]	[illegible]	7.[illegible]

TABLE 2

COMPUTATION OF LOSSES AND SLOPES FOR DYNAMICALLY SIMILAR MODEL

[illegible] LAKE DAM SPILLWAY

	Prototype						Model Values Expressed in Terms of Prototype							
Station (1)	Horizontal Length of Reach Ft. "L_h" (2)	Slant Length of Reach Ft. "L_s" (3)	Slope of Channel Floor "i_p" (4)	Drop in Channel Floor Ft. (5)	Average Slope of Energy Gradient "S_a" (6)	Friction Loss Between Sections Ft. "h_{fp}" (7)	$\frac{V_p}{V_m}$ (8)	S_m (9)	Friction Loss Between Sections in Model, Ft. $h_{fm}=S_mL_m$ (10)	Additional Friction Loss in Model, Ft. $h_{fm}-h_{fp}$ (11)	Accumulative Additional Friction Loss in Model, Ft. "Σl" (12)	Corrected Drop in Channel Floor for Model, Ft. Cols. 5 + 12 (13)	Drop in Channel Floor per Reach Ft. (14)	Corrected Slope for Channel Floor in Model "i_m" (15)
									$\frac{n_m}{n_p}=\frac{0.010}{0.014}$					
0+00.00	-	-	Crest											
[illegible]	-	-	0.[illegible]	0.[illegible]	-	-	-	-	-	-	-	-	-	-
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	0.11	2.00	0.[illegible]	0.[illegible]	0.11	0.11	[illegible]		-
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	0.[illegible]	2.00	0.0[illegible]	0.[illegible]	0.[illegible]	0.[illegible]	7.[illegible]	4.[illegible]	-
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.0077	0.[illegible]	2.00	0.0[illegible]	0.77	0.[illegible]	0.[illegible]	[illegible]	[illegible].10	0.[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	0.[illegible]	2.00	0.[illegible]	0.[illegible]	0.[illegible]	1.[illegible]	11.[illegible]	[illegible]	0.0[illegible]
									$\frac{n_m}{n_p}=\frac{0.010}{0.0[illegible]}$				1.[illegible]	0.[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.0[illegible]	0.[illegible]	[illegible]	0.01[illegible]	0.[illegible]	0.[illegible]	1.[illegible]	11.[illegible]	[illegible]	0.0[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	1.[illegible]	[illegible]	0.0[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.0[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	0.[illegible]	[illegible]	0.0[illegible]	1.[illegible]	0.[illegible]	[illegible]	[illegible]	[illegible]	0.0[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	1.[illegible]	[illegible]	0.[illegible]	[illegible]	1.[illegible]	[illegible]	[illegible]	[illegible]	0.0[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	0.[illegible]	[illegible]	0.[illegible]	[illegible].13	1.21	7.[illegible]	[illegible]	4.[illegible]	0.0[illegible]
									$\frac{n_m}{n_p}=\frac{0.010}{0.0[illegible]}$					
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.0678	[illegible]	[illegible]	0.[illegible]	[illegible]	[illegible]	[illegible]	[illegible].71	[illegible]	0.[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	[illegible]	[illegible]	0.1[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.1780	[illegible]	[illegible]	0.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.[illegible]
[illegible]	[illegible]	[illegible]	0.[illegible]	[illegible]	0.[illegible]	7.[illegible]	[illegible]	0.[illegible]	[illegible]	7.[illegible]	[illegible]	[illegible]	[illegible]	0.[illegible]

at this point and the corresponding values of Z, V, A, R, etc., are recorded on that line of the table. A slope for the hydraulic gradient, S, is then assumed for station 6+29.35. With this slope and the hydraulic radius, a value of C is obtained. This value of C is then substituted in the expression $S = \frac{V^2}{C^2R}$. If the assumed value of S agrees with the computed value, the slope is the one desired. If the two do not agree, a new value is assumed and the process repeated until an agreement is reached. This value of the instantaneous slope of the hydraulic gradient is then recorded in column 11 of table 1. The instantaneous gradient slopes for stations 6+13.75 and 6+29.35 are then averaged and this value is recorded in column 12 as S_a. The next step in the computations is to determine whether the assumed depth of flow at station 6+29.35 is correct.

Multiplying the average slope of the energy gradient, S_a, by the slant length of section, L_s, results in a value for the friction loss for this section of chute of 0.11 feet (column 14). This loss is shown diagrammatically in figure 39B. The eddy loss shown in column 15 was chosen arbitrarily as it is known that a loss exists in the section due to this disturbance. The elevation of the energy gradient at station 6+20.35 was obtained by adding up the energy components at this point $(Z + \frac{P}{W} + \frac{V^2}{2g})$ as recorded in column 18. If the drop in the energy gradient between stations 6+13.75 and 6+29.35 (column 19) agrees closely with the value of the total friction loss shown in column 16, this indicates that the assumed depth of flow for station 6+29.35 (column 3) is correct. Should these two values be appreciably different, it is then necessary to assume a new value of d (column 3) and repeat the former procedure until they do show agreement.

The hydraulic characteristics for the remaining downstream reaches of the chute are each computed individually in like manner, thus the depth and velocity of flow is determined for any point on the spillway.

31. Computation of losses and slopes for dynamically similar model. Since the frictional losses in the model are greater, in proportion to the scale ratio, than those in the prototype, it is necessary to increase the slope of the channel floor, i_m, in the model (figure 38) to obtain proportional velocities and depths in model and prototype.

Since the problem is one of correcting the slope of the bottom of the channel in the model to give hydraulic conditions similar to those in the prototype, equation 1 using the Manning coefficient (equation 3) can be written

$$S = \frac{v^2 n^2}{(1.486)^2 R^{4/3}} \qquad (10)$$

Using the subscript m to designate model terms and p to designate prototype terms, the ratio between the model energy gradient slope S_m, and that of the prototype S_p, is

$$\frac{S_m}{S_p} = \frac{v_m^2 n_m^2 (1.486)^2 R_p^{4/3}}{v_p^2 n_p^2 (1.486)^2 R_m^{4/3}} = \frac{v_m^2 n_m^2 R_p^{4/3}}{v_p^2 n_p^2 R_m^{4/3}} \qquad (11)$$

Since N is the scale ratio, and by the Froude law of [illegible]

$$\left(\frac{v_m}{v_p}\right)^2 = N^{-1} \quad \text{and} \quad \frac{R_p}{R_m} = N$$

equation 11 can be written

$$\frac{S_m}{S_p} = N^{-1} \left(\frac{n_m}{n_p}\right)^2 N^{4/3} = \left(\frac{n_m}{n_p}\right)^2 N^{1/3} \qquad [illegible]$$

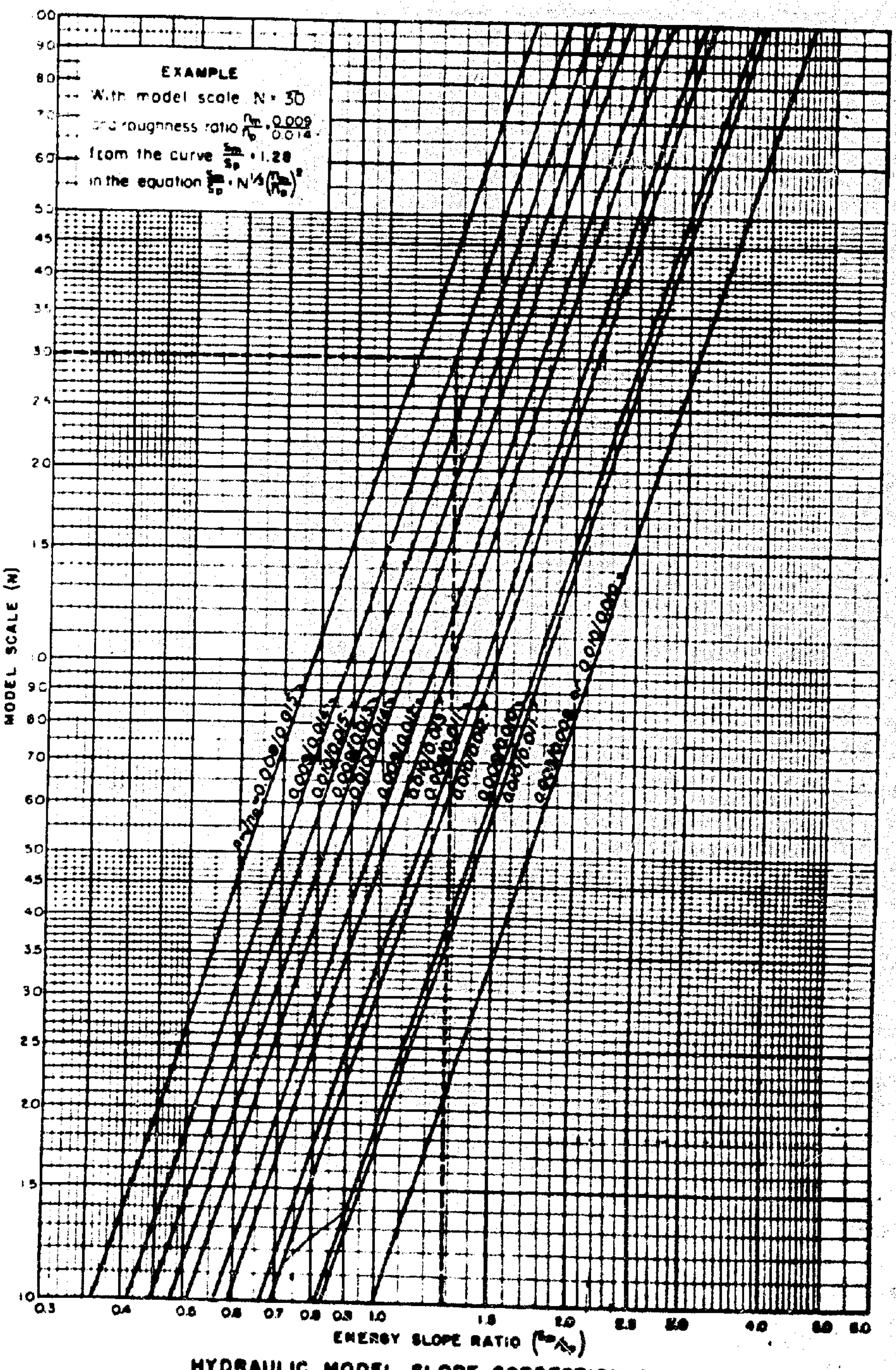
$$\text{or} \quad S_m = N^{1/3} \left(\frac{n_m}{n_p}\right)^2 S_p \qquad [illegible]$$

In a model dynamically similar to the prototype, [illegible] between model and prototype energy gradient slopes [illegible] function of the scale ratio and the roughness [illegible] equation 13. The graphic representation of the [illegible] S_m/S_p and N for different values of the roughness [illegible] shown in figure 40. The values shown [illegible] assuming values of n_m, n_p, and N, and [illegible] S_m/S_p ratios.

The roughness coefficient for the [illegible] type spillway n_p was assumed as 0.014 and [illegible] portion as 0.015. This coefficient for the model [illegible] 0.010 and the scale ratio, N, was [illegible].

$$\frac{n_m}{n_p} = \frac{0.010}{0.014} \quad \text{and} \quad [illegible]$$

FIGURE 40



HYDRAULIC MODEL SLOPE CORRECTION CURVES

Referring to figure 40 with the ratio $\frac{10}{14}$, $\frac{S_m}{S_p}$, = 2.0 or

$S_m = 2.0\ S_p$ and for the ratio $\frac{10}{13}$, $S_m = 2.3\ S_p$

where $S_m = \frac{H_{fm}}{L_{sm}}$ and $S_p = \frac{H_{fp}}{L_{sp}}$ (figure 38B) in which the subscripts m and p designate model and prototype, respectively.

The amount that the floor of the model must be dropped to duplicate depths and velocities in the prototype is $H_{fm} - H_{fp}$ or K for the length of reach L (figure 38B). The computations for this correction are shown in table 2. Columns 2 to 7, inclusive, constitute prototype values which were obtained from table 1. Columns 8 through 15 demonstrate the method used in obtaining the additional slope of the channel floor by reaches in the model.

The values in column 8 were obtained from figure 40 using the $\frac{n_m}{n_p}$ ratios shown in table 2 and a model scale of 60. The values in column 9 constitute the product of the respective values in columns 6 and 8. Values listed in column 10 represent the friction loss in the model expressed in terms of the prototype and was obtained from the product of the values in columns 4 and 9. The additional friction loss in the model, caused by the fact that the surface roughness in model and prototype are not to scale, is obtained from the difference between the values in columns 10 and 7 and these differences are listed in column 11. Column 12 is self-explanatory. The values in column 13 represent the corrected drop for the model (measured downward from the crest) for the various stations listed and constitute the sum of the respective values in columns 5 and 12. From these values and the stationing, the model was constructed. Column 14 represents the drop in the channel floor between stations and column 15 gives the corrected slopes of this floor for the model. The amount of correction necessary at the downstream end of the chute computed in this manner is 6.35 feet (expressed in prototype) or 0.106 feet in the model constructed on a scale of 1 to 60. Velocities in the model were computed from measurements of the cross section and the discharge. The average depth of each cross section was determined by a point gage and the discharge was measured over the laboratory V-notch weir. A comparison of the measured model velocities (expressed in prototype) with the computed prototype velocities is shown for a number of sections in table 3.

TABLE 3

COMPARISON OF VELOCITIES IN MODEL AND PROTOTYPE
MOON LAKE DAM SPILLWAY

Prototype Computations		Model Results Without Slope Correction Run 1-1		Model Results With Slope correction Run 2-1		
Station	Velocity Ft. per sec.	Station	Velocity Ft. per sec.	Station	Velocity Ft. per sec.	Remarks
6+53.10	27.35	6+33.10	25.85*	6+33.10	24.99*	Run Discharge
		6+66.84	12.99	7+04.30	26.19	1-1 10,200 sec. ft.
7+79.75	27.80	7+68.20	17.07 26.38*	7+78.77	25.48	2-1 9,983 sec. ft.
8+90.59	26.25	8+67.53	20.80	8+89.56	26.34	Prototype Computations 10,000
		10+73.80	22.75	10+73.44	27.45	
12+76.79	26.25	12+79.60	23.45	12+79.44	26.99	*Indicates that roller is out.
		13+98.99	33.04	13+99.24	35.40	
15+07.98	39.91	15+04.82	36.59	15+07.98	39.60	
17+19.17	76.04	17+21.98	71.84	17+21.98	80.82	
		17+51.98	53.57	17+51.98	60.88	

The measured velocities taken before the model was corrected for excess friction (column 2), are compared with the measured velocities obtained after the correction was made (column 4), and the computed prototype velocities (column 6).

The agreement between the prototype velocities as computed from the measurements of the model with a corrected slope and the prototype velocities as computed using Chezy's formula with either Kutter's or Manning's formula does not mean that the latter will yield correct results on the actual prototype. As previously discussed, Manning and Kutter and Ganguillet derived their formulas from low-velocity data. Until that time in the future when high-velocity data can be obtained those equations are the best available and must be used keeping in mind the shortcomings and pitfalls discussed in this section.