UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

- - -

HYDRAULIC LABORATORY REPORT NO. 152

- - -

HYDRAULIC MODEL STUDIES
FOR THE DESIGN
OF
MARSHALL FORD DAM - COLORADO RIVER PROJECT, TEXAS

by

R. A. GOODPASTURE, ASSISTANT ENGINEER

- - -

Denver, Colorado
September 25, 1944.

UNITED STATES
DEPARTMENT OF THE INTERIOR
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Under Direction of

J. E. WARNOCK, SENIOR ENGINEER
and
R. F. BLANKS, SENIOR ENGINEER

- - - - -

Denver, Colorado,
Sept. 25, 1944

F O R E W O R D

The hydraulic models for the design of Marshall Ford Dam, as described in this report, were designed and tested and the model studies were made in the Fort Collins, Colorado, laboratory of the Bureau of Reclamation under the supervision of James W. Ball, Associate Engineer, by J. M. Buswell, R. A. Goodpasture, H. W. Brewer, and D. M. Lancaster, Assistant Engineers.

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Engineering and Geological Control
and Research
Denver, Colorado
September 25, 1944

Laboratory Report No. 152
Hydraulic Laboratory

Compiled by: R. A. Goodpasture
Reviewed by: J. E. Warnock

Subject: Hydraulic model studies for the design of Marshall Ford Dam - Colorado River project, Texas.

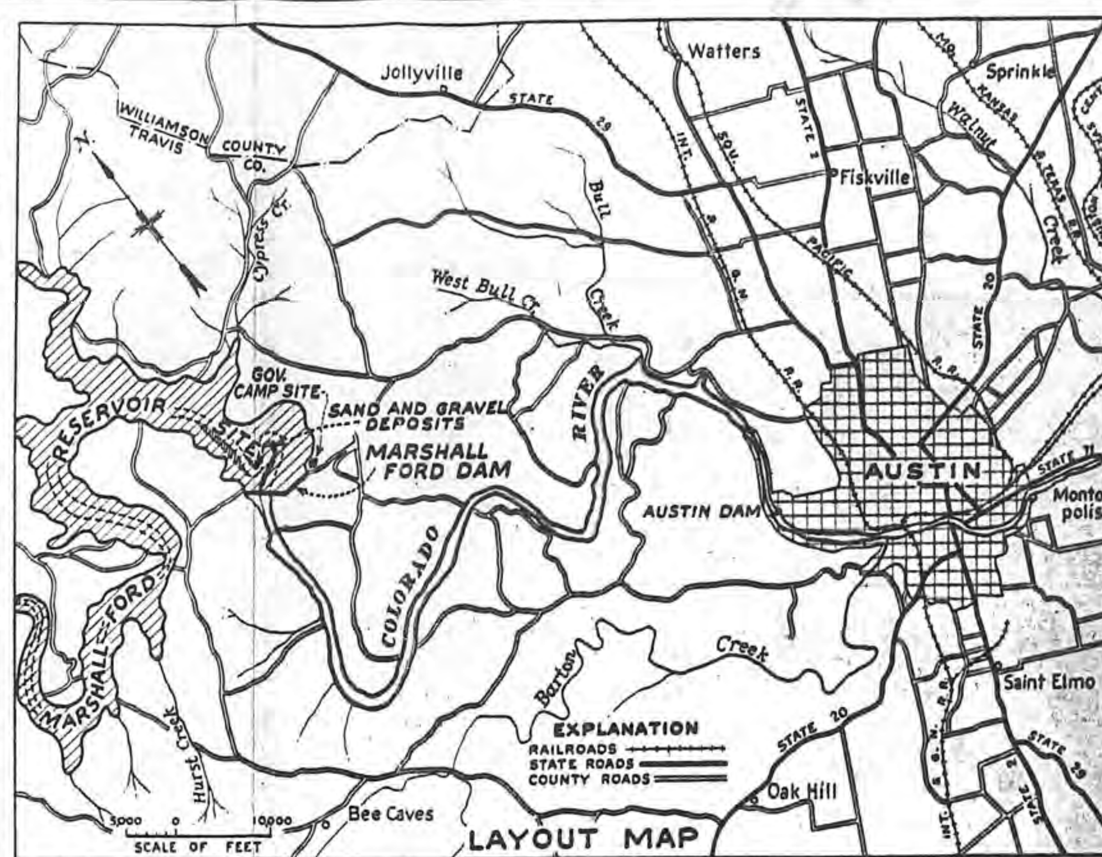
INTRODUCTION

1. Introduction. The Marshall Ford Dam, a part of the Colorado River project, is located in the Colorado River in Texas about 12 miles northwest of Austin (figure 1). The dam will provide flood control and power development. The main portion of the dam is of the concrete, gravity type resting on bedrock with an overflow spillway crest 730 feet long built into this gravity section. On the left of the masonry section an earth-fill dam extends to the high ground some distance from the river.

The dam was constructed in two stages, which will be referred to in this report as the initial and the ultimate development. The initial development spillway had its crest at elevation 640.00, while the crest of the ultimate development spillway is at elevation 714.00 (figure 2). The additional height was gained by placing the upstream face of the high dam about 50 feet upstream from the face of the low dam, making the plane of the downstream face of the dam common to both structures. An ogee crest tangent to the 0.75:1 downstream face was designed for each structure. Piers were not installed on the initial spillway crest, making the net length of this crest 30 feet longer than for the ultimate design. Pertinent data for the layout of the spillways are outlined in table I.

TABLE I

	Initial Dam	Ultimate Dam
Crest elevation	640.0	714.0
Maximum reservoir water surface elevation	670.0	740.0
Maximum head on spillway crest	30.0 ft.	28.0 ft.
Design head on spillway crest	30.0 ft.	30.541 ft.
Net length of spillway	730.0 ft.	700.0 ft.



DEPARTMENT OF THE INTERIOR
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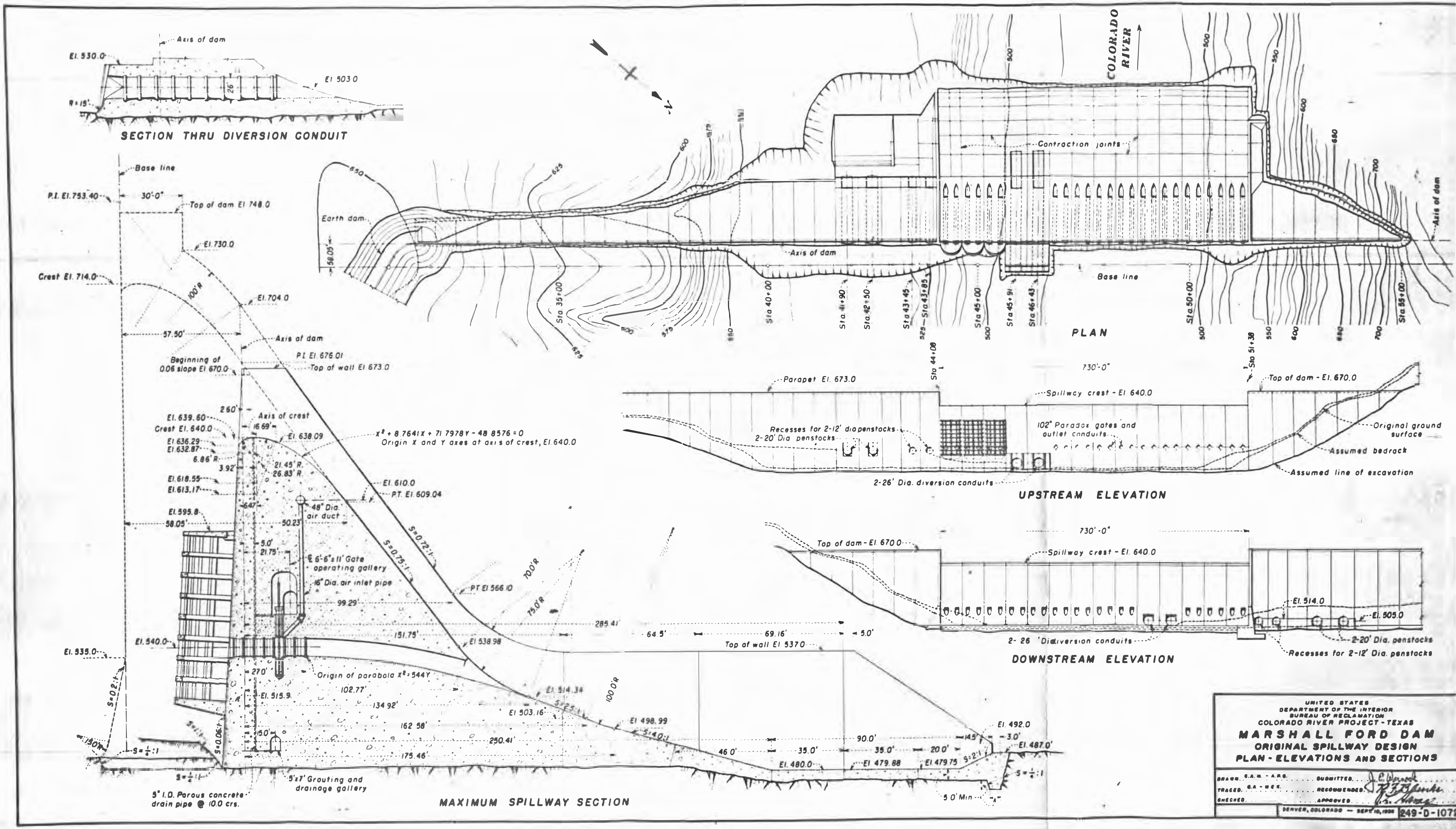
COLORADO RIVER PROJECT-TEXAS

MARSHALL FORD DAM
LOCATION MAP

REV. 12-11-36	DRAWN J.R.G.	SUBMITTED
	TRACED W.M.S.	RECOMMENDED
	CHECKED	APPROVED
28597	DENVER, COLO. SEPT. 10, 1936	249-D-261

FIGURE 2


SECTION THRU DIVERSION CONDUIT
Axis of dam
El. 530.0
El. 503.0
R=15'
26'
MAXIMUM SPILLWAY SECTION
Base line
P.I. El. 753.40
30'-0"
Top of dam El. 748.0
El. 730.0
Crest El. 714.0
100'R
El. 704.0
57.50'
Axis of dam
P.I. El. 676.01
Beginning of 0.06 slope El. 670.0
Top of wall El. 673.0
2.60'
Axis of crest
16.69'
El. 638.09
El. 639.60
Crest El. 640.0
El. 636.29
El. 632.87
6.86'R.
3.92'
21.45'R.
26.83'R.
El. 618.55
El. 613.17
$X^2 + 8.7641X + 71.7978Y - 48.8576 = 0$
Origin X and Y axes at axis of crest, El. 640.0
El. 610.0
P.T. El. 609.04
48" Dia. air duct
El. 595.8
58.05'
50.23'
S=0.72:1
S=0.75:1
6'-6"x11' Gate operating gallery
16" Dia. air inlet pipe
P.T. El. 566.10
70.0'R
75.0'R
99.29'
285.41'
151.75'
El. 538.98
64.5'
69.16'
5.0'
Top of wall El. 537.0
El. 540.0
El. 535.0
Origin of parabola $X^2 = 544Y$
270'
El. 515.9
102.77'
134.92'
El. 514.34
100.0'R
El. 503.16
S=2.5:1
El. 498.99
S=4.0:1
162.58'
250.41'
175.46'
46.0'
35.0'
90.0'
35.0'
20.0'
14.5'
3.0'
El. 492.0
El. 487.0'
El. 480.0
El. 479.88
El. 479.75
S=2:1
S=1/4:1
S=0.2:1
S=0.06:1
S=1/2:1
5'x7' Grouting and drainage gallery
5" I.D. Porous concrete drain pipe @ 10.0 crs.
5.0' Min.
COLORADO RIVER
Contraction joints
Earth dam
Axis of dam
Base line
58.05'
Sta 35+00
Sta 40+00
Sta 41+90
Sta 42+50
Sta 43+45
Sta 43+85
Sta 45+00
Sta 45+91
Sta 46+43
Sta 50+00
Sta 55+00
Sta 44+08
Sta 51+38
PLAN
730'-0"
Parapet El. 673.0
Spillway crest - El. 640.0
Top of dam - El. 670.0
Recesses for 2-12' dia penstocks
2-20' Dia. penstocks
102" Paradox gates and outlet conduits
Original ground surface
Assumed bedrock
Assumed line of excavation
2-26' Dia. diversion conduits
UPSTREAM ELEVATION
730'-0"
Top of dam - El. 670.0
Spillway crest - El. 640.0
El. 514.0
El. 505.0
2-26' Dia. diversion conduits
2-20' Dia. penstocks
Recesses for 2-12' Dia. penstocks
DOWNSTREAM ELEVATION
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
ORIGINAL SPILLWAY DESIGN
PLAN - ELEVATIONS AND SECTIONS
DRAWN S.A.M. - A.R.S.
TRACED S.A. - M.E.S.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO
249-D-1079

The proposed stilling pool was a horizontal apron connected to the downstream face of the dam by a 75-foot radius bucket and a sloping apron. A sill was placed at the end of the apron to assist in dissipating the energy in the spillway flow. Training walls were provided on each end of the spillway stilling pool to confine the hydraulic jump (figure 2).

The first design of the ultimate development provided 28 individual siphons with their exits resting on the crest of the low dam.

River outlets 8 feet 6 inches in diameter were provided so that the structure would serve properly the functions of flow control. The spacing between conduits would allow 28 river outlets to be installed in the spillway section; however, 4 outlets were omitted near the center of the left half of the spillway to provide space for two 26-foot diameter diversion conduits. These diversion conduits passed the river flow during the construction of the low and the high dams, after which they were plugged with concrete (figure 2).

2. The models. Since the same stilling pool was used for both dams, the design was necessarily based on action for the more severe conditions that were expected to exist after completion of the high dam. As a consequence, most of the model testing was confined to the ultimate development. Results of tests on four separate models, covering various phases of the project, are presented in this report. The models discussed are a 1 to 40.8 sectional model of the initial crest, a 1 to 40.8 sectional model of the ultimate spillway, a 1 to 68 model representing half of the ultimate spillway, and a 1 to 25.5 model of one river outlet.

3. Summary of tests. Tests on the 1:40.8 model of the spillway crest of the low dam showed that with the reservoir at elevation 670.2, areas of reduced pressure existed on the crest. Since this pressure was never greater than one pound per square inch below atmospheric pressure and would be present for only short periods, the condition was not considered critical. Measurements of the thickness of the jet flowing over the crest of the low dam at maximum discharge indicated that the height of the proposed training wall should be increased.

The ultimate development crest operated satisfactorily at all discharges after the siphons had been eliminated from the design. With the reservoir at elevation 740.0, a very slight negative pressure occurred on the downstream extremity of the crest. With water flowing over the crest, the discharge of the river outletsremained about constant. Calibration data was obtained from the model with the river outlets and the crest operating separately and together.

The stilling pool studies demonstrated that a 200-foot radius bucket with a 2.094 to 1 slope to the flat apron at elevation 480.0 was superior to the original 75-foot radius bucket and the flatter 4 to 1 slope. This change in slope resulted in a saving in concrete and a longer horizontal apron. Lowering the horizontal apron below elevation 480.0 did not materially improve the stilling pool action. A dentated sill at the downstream end of the horizontal apron produced better conditions than either the original 2 to 1 trapezoidal sill or a 1 to 1 trapezoidal sill. The dentated sill was removed from the entrance of the pool sufficiently to prevent any areas of greatly reduced pressure on the dentates.

Investigations on the 1:68 model of the ultimate development showed that the location of the diversion conduits was satisfactory. Intermediate training walls, sloping from elevation 517.0 at the bucket to the top of the sill, on each side of the diversion conduit were found necessary to prevent undesirable stilling pool currents when the spillway and the outlets were operating.

Operation of the diversion conduits in a manner representing the diversion period during construction of the high dam demonstrated that conditions were better when the intermediate training walls were in place. The diversion conduits were rated so that their capacity might be known for any reservoir elevation.

The more severe stilling pool conditions existing with the high dam were used to determine satisfactory training walls on each end of the spillway. The original left training wall design was unsatisfactory at high discharges because of the excessive flow from the powerhouse tailrace over the wall onto the hydraulic jump. The final designed wall, placed at elevation 548.0 with the downstream portion on a 4.049 to 1 slope ending at elevation 513.0, gave satisfactory stilling pool conditions.

The original training wall on the right end of the spillway did not extend sufficiently far downstream to prevent an undesirable whirl near the right bank. A trapezoidal extension on the end of the wall removed this condition. The final wall was made similar to the left wall in appearance. Where the rock level was higher than the sill elevation, a 3 to 1 cut on the rock extending from the sill downstream was found more desirable than the 1 to 1 slope originally proposed. The backfill behind the training wall was severely eroded at high discharges. Accordingly, it was recommended that the sloping portion of this backfill be riprapped.

The problem of bringing the river outlets into operation at high reservoir and minimum tailwater elevations was solved by installing spreading exit transitions on outlets 10, 12, and 14. It was determined that the most satisfactory operating program would result when these outlets were operated first, in the order named, and then the remaining even-numbered outlets opened, from left to right. The odd-numbered outlets could then be opened in any desired order.

Studies on the 1;25.5 model of one river outlet were primarily for the development of a satisfactory spreading exit for three of the river outlets to be opened first when bringing the outlets into operation with a high reservoir elevation and low tailwater level. Such an exit was evolved, and its operating characteristics were studied.

Observations and pressure measurements on the circular river outlet, when fitted with the spreading exit, indicated that under full capacity better conditions resulted when the air vents were closed. With reservoir levels between elevation 551.78 and 610.0, the air vents produced an objectionable pulsating flow through the outlet. Accordingly, it was recommended that valves be installed in the air lines to control gates and that these valves be closed when the gates are completely open.

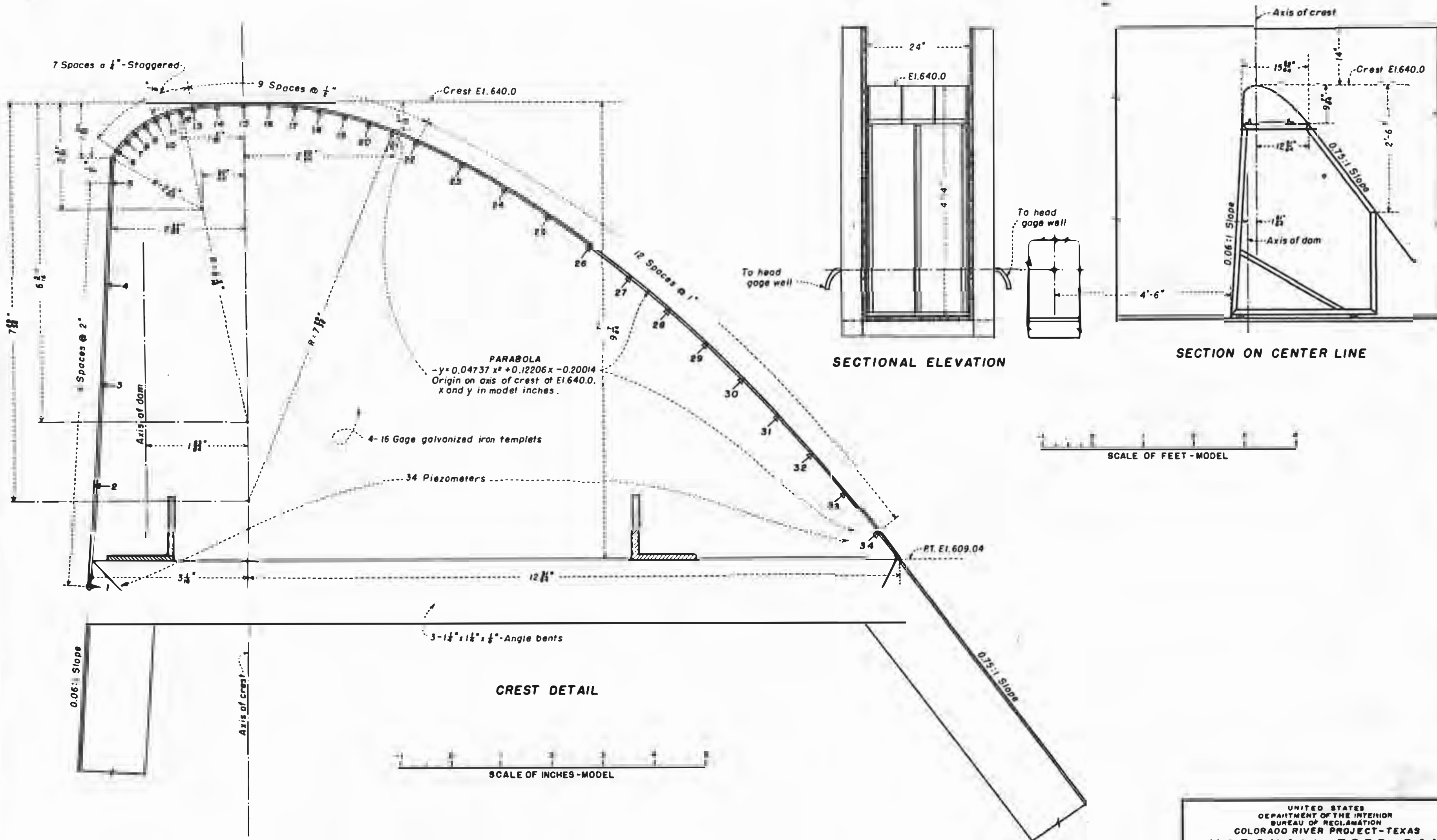
MODEL OF INITIAL DEVELOPMENT

4. The spillway crest. While the greater portion of the model studies were made on the ultimate development, some features made hydraulic model investigations on the low dam desirable. The tests concerned mainly the crest of this structure. A 2-foot sectional model of the crest of the low dam spillway was built on a scale ratio of 1 to 40.8. This model was installed in a metal-lined flume connected to the laboratory supply system. A gage was installed upstream from the model so the head on the crest could be recorded at any discharge. The model stilling pool was not constructed, since this problem was considered only in connection with the ultimate development. Piezometers were installed on the crest for measuring the pressure on the surface of the crest at various discharges (figure 3).

5. Tests and results. Piezometer pressures on the crest were obtained at discharges of 60,000, 120,000, 180,000, 240,000, 300,000, and 360,000 second-feet, respectively. Pressures below atmospheric existed on part of the downstream portion of the crest at maximum discharge (reservoir elevation 670.2). The maximum intensity of this negative pressure was two feet of water (figure 4).

The discharge-coefficient curve obtained for the crest (figure 4) shows a maximum of 3.948 for reservoir elevation 670.0. This high efficiency was due to the reduced pressure on the downstream portion of the crest.

With a discharge of 500,000 second-feet passing over the crest,



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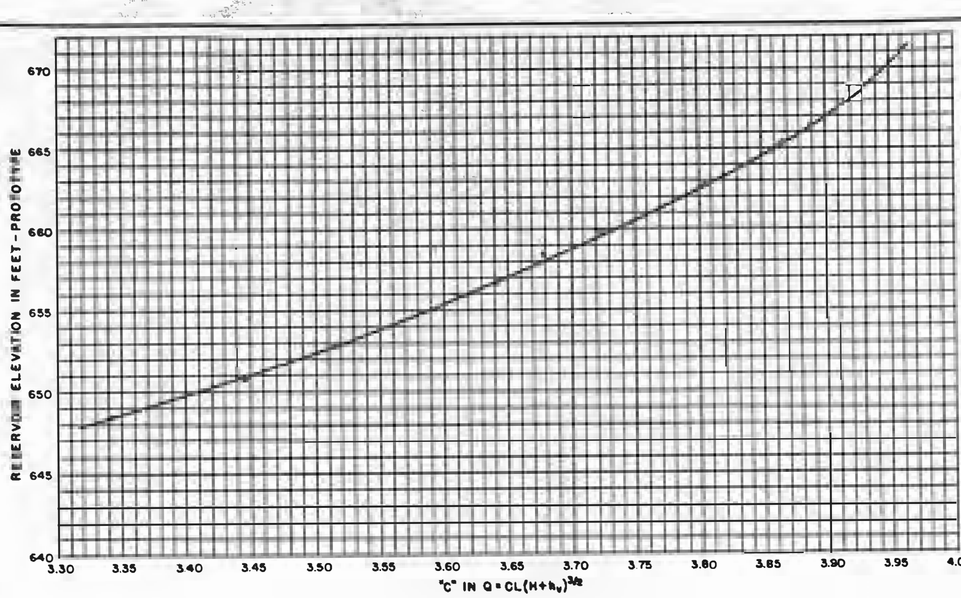
COLORADO RIVER PROJECT-TEXAS

MARSHALL FORD DAM

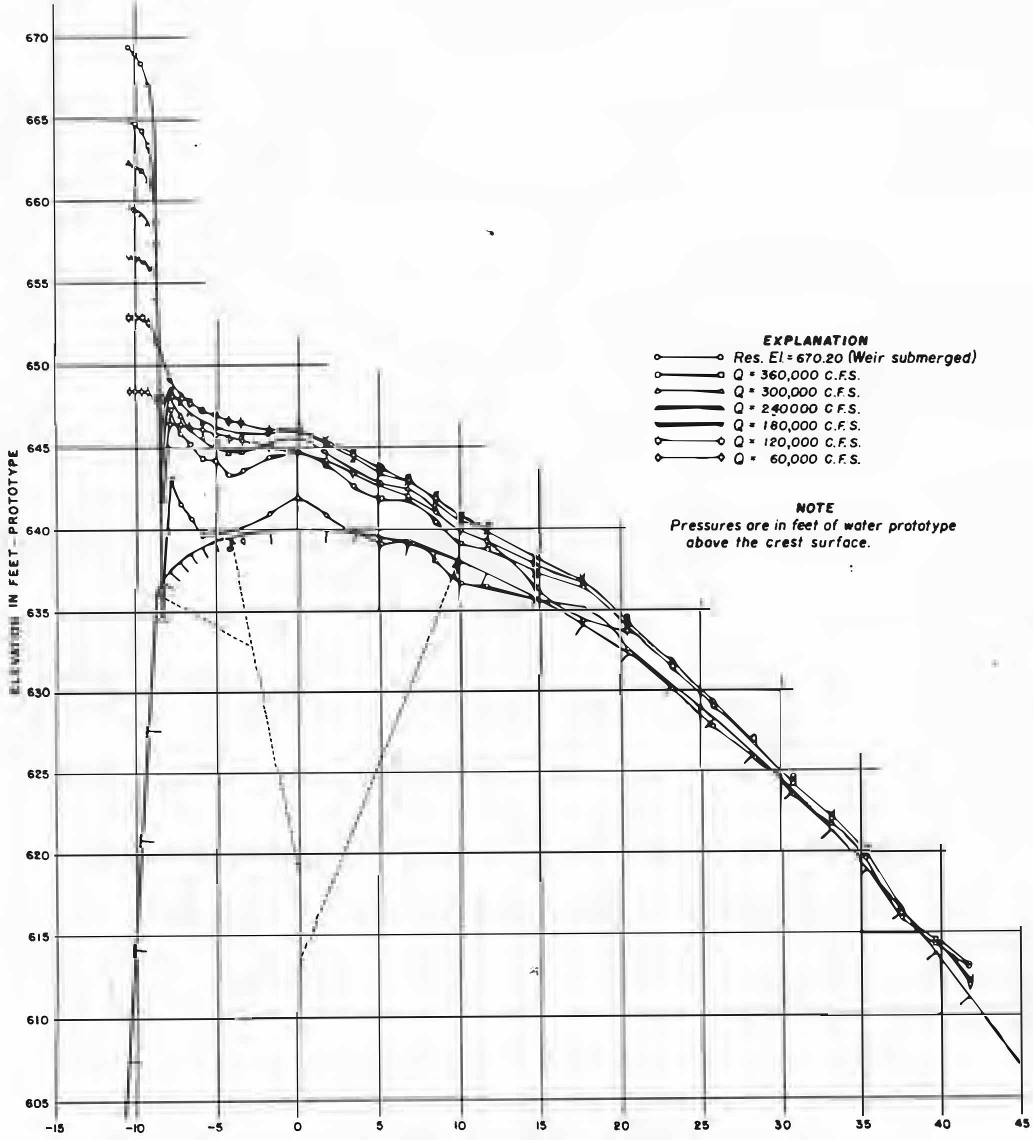
HYDRAULIC MODEL STUDIES

INITIAL DEVELOPMENT- MODEL SCALE 1:40.8

MODEL DETAILS



COEFFICIENT OF DISCHARGE FOR CREST



PRESSURE ELEVATIONS ON CREST IN FEET OF WATER-PROTOTYPE

MARSHALL FORD DAM
INITIAL DEVELOPMENT
HYDRAULIC MODEL STUDIES - SCALE 1:40.8

CREST DISCHARGE COEFFICIENTS AND PRESSURES

D.M.L. 5-13-1937

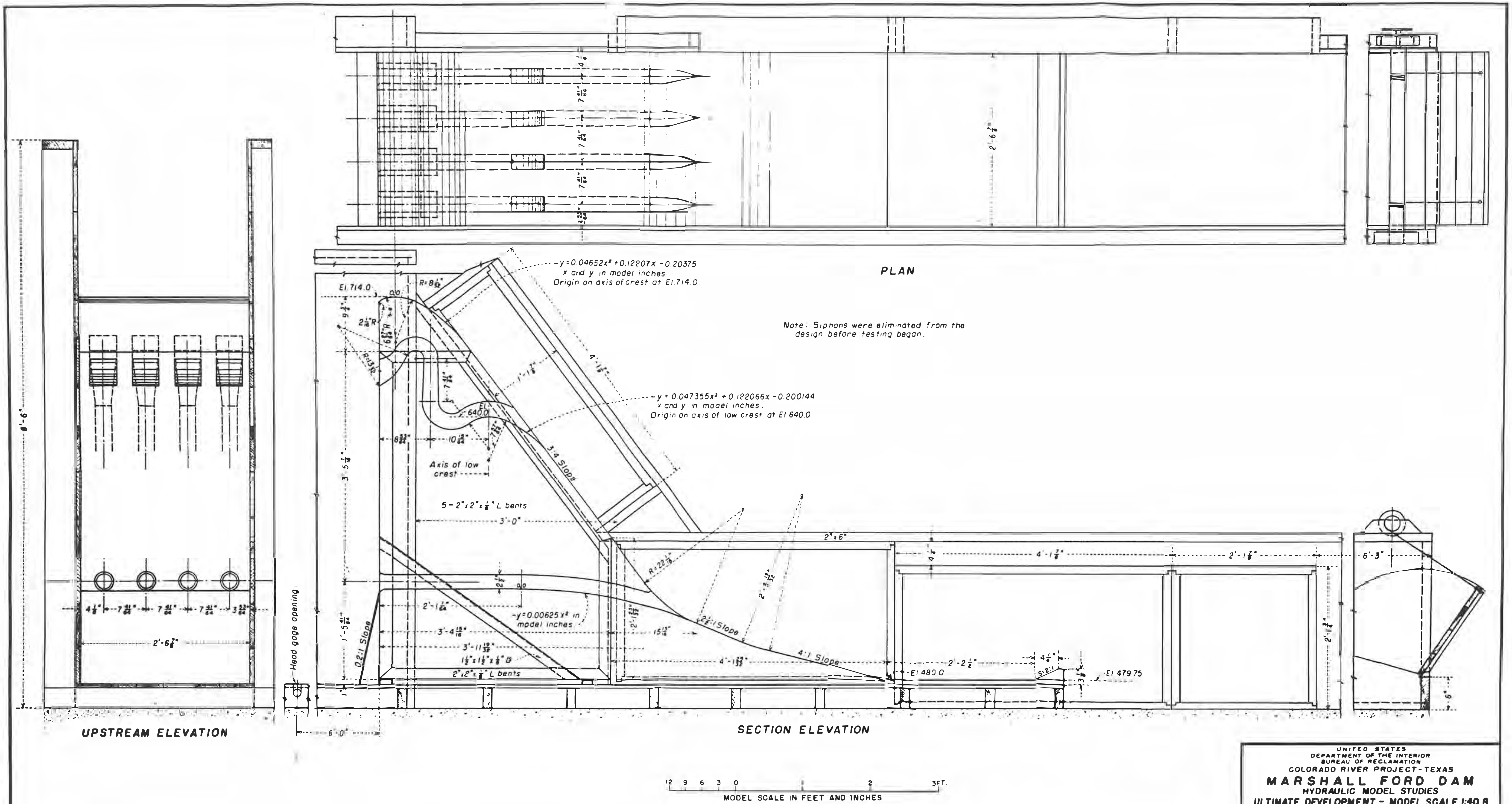
249-D-1074

a profile of the water surface was taken along the face of the dam. This was done to determine the necessary height of the training walls (figure 15). This profile shows that no freeboard existed, the water surface just reaching the top of the wall in one place. No doubt air entrainment would cause a higher water surface, but such a condition would exist only for the low dam; so the occurrence was not considered critical. Moreover, the occurrence of floods of this magnitude was expected to be very infrequent.

STUDY OF SPILLWAY AND STILLING POOL FOR ULTIMATE DEVELOPMENT

6. The spillway model. Design of the stilling basin for the Marshall Ford Dam necessarily depended on the more severe conditions that would exist after completion of the ultimate development. A model of the ultimate development was therefore given early consideration. Three methods of flood regulation in one structure made the stilling pool more complicated and except for the siphons, pointed to a small model including as much of the prototype as possible. However, the subatmospheric pressure present in siphonic action made a large-scale model desirable. Accordingly, a sectional model representing 105 feet of crest on the left end of the dam was constructed, including four siphons and four river outlets. A scale ratio of 1 to 40.8 was chosen to permit use of commercial brass tubing in constructing the outlets. This model required a flow about equal to the maximum capacity of the laboratory pump.

A metal-lined tank 11 feet long, 2 feet 6-7/8 inches wide, and 8 feet deep was connected by a flange to one of the 24-inch conduits of the laboratory supply system. The upstream face of the model dam was installed at one end of the tank. The model understructure was a framework of welded angle iron surmounted by 16-gage galvanized iron bents shaped nearly to the crest outline. A covering of 20-gage sheet metal gave the final crest shape. An apron of sheet iron supported by 16-gage bents was fastened to the floor and soldered to the understructure. The flume contained the apron and extended 10.5 feet beyond to provide a sand bed for erosion studies. Glass panels in one side of the flume permitted visual and photographic studies of the stilling pool action. The tailwater regulator at the end of the flume was a wooden gate hinged at the bottom and regulated by a windlass and a ratchet mounted across the top of the flume. The metal siphons were installed in the model and held in place by flanges. The river outlet conduits of parabolic profile were shaped from brass tubing, soldered into the bucket at the downstream end and held in place by flanges at the upper end. Each tube was fitted with a bell-shaped entrance (figures 5 and 6). A gage in the forebay was provided to measure the water surface elevation at any discharge.



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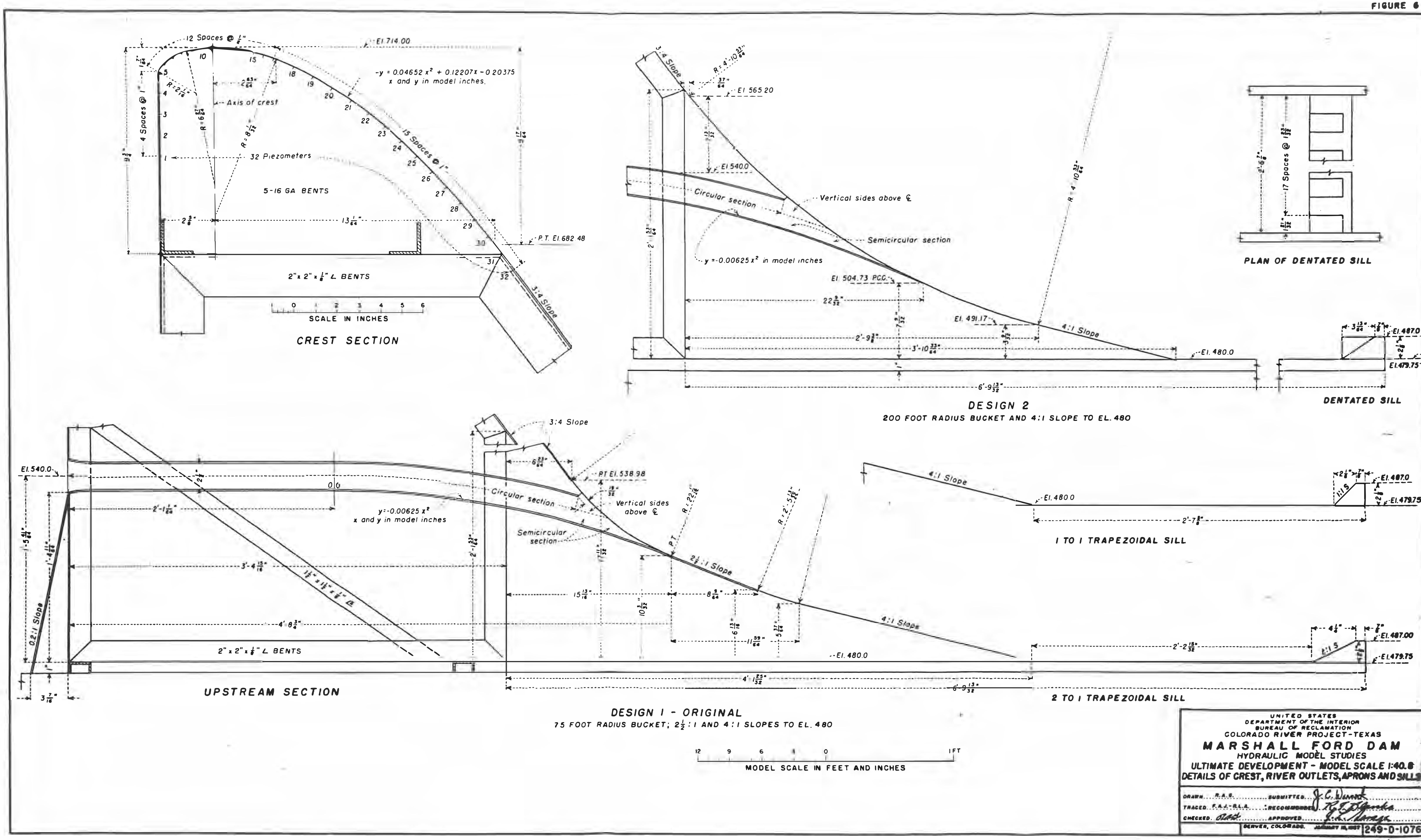
COLORADO RIVER PROJECT-TEXAS

MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:40.8
GENERAL MODEL LAYOUT

DRAWN R.A.E. SUBMITTED J.E. Warnock
TRACED E.V.J.-C.I.L. RECOMMENDED R.F. Blanks
CHECKED [illegible] APPROVED J.L. Savage
DENVER, COLORADO SEPTEMBER 11, 1936 249-D-1075

FIGURE 6


CREST SECTION
12 Spaces @ 1/8"
El. 714.00
-y = 0.04652 x² + 0.12207x - 0.20375
x and y in model inches.
Axis of crest
32 Piezometers
15 Spaces @ 1"
5-16 GA BENTS
P.T. El. 682.48
2"x 2"x 1/8" ∠ BENTS
3:4 Slope
SCALE IN INCHES
DESIGN 2
200 FOOT RADIUS BUCKET AND 4:1 SLOPE TO EL. 480
3:4 Slope
El. 565.20
El. 540.0
Circular section
Vertical sides above ℄
Semicircular section
y = -0.00625 x² in model inches
El. 504.73 PCC.
El. 491.17
4:1 Slope
El. 480.0
PLAN OF DENTATED SILL
17 Spaces @ 1 23/32"
DENTATED SILL
El. 487.0
El. 479.75
UPSTREAM SECTION
El. 540.0
0.2:1 Slope
y = -0.00625 x²
x and y in model inches
2"x 2"x 1/8" ∠ BENTS
DESIGN 1 - ORIGINAL
75 FOOT RADIUS BUCKET; 2½:1 AND 4:1 SLOPES TO EL. 480
3:4 Slope
P.T. El. 538.98
Circular section
Vertical sides above ℄
Semicircular section
2½:1 Slope
4:1 Slope
El. 480.0
1 TO 1 TRAPEZOIDAL SILL
El. 487.0
El. 479.75
2 TO 1 TRAPEZOIDAL SILL
El. 487.00
El. 479.75
MODEL SCALE IN FEET AND INCHES
UNITED STATES
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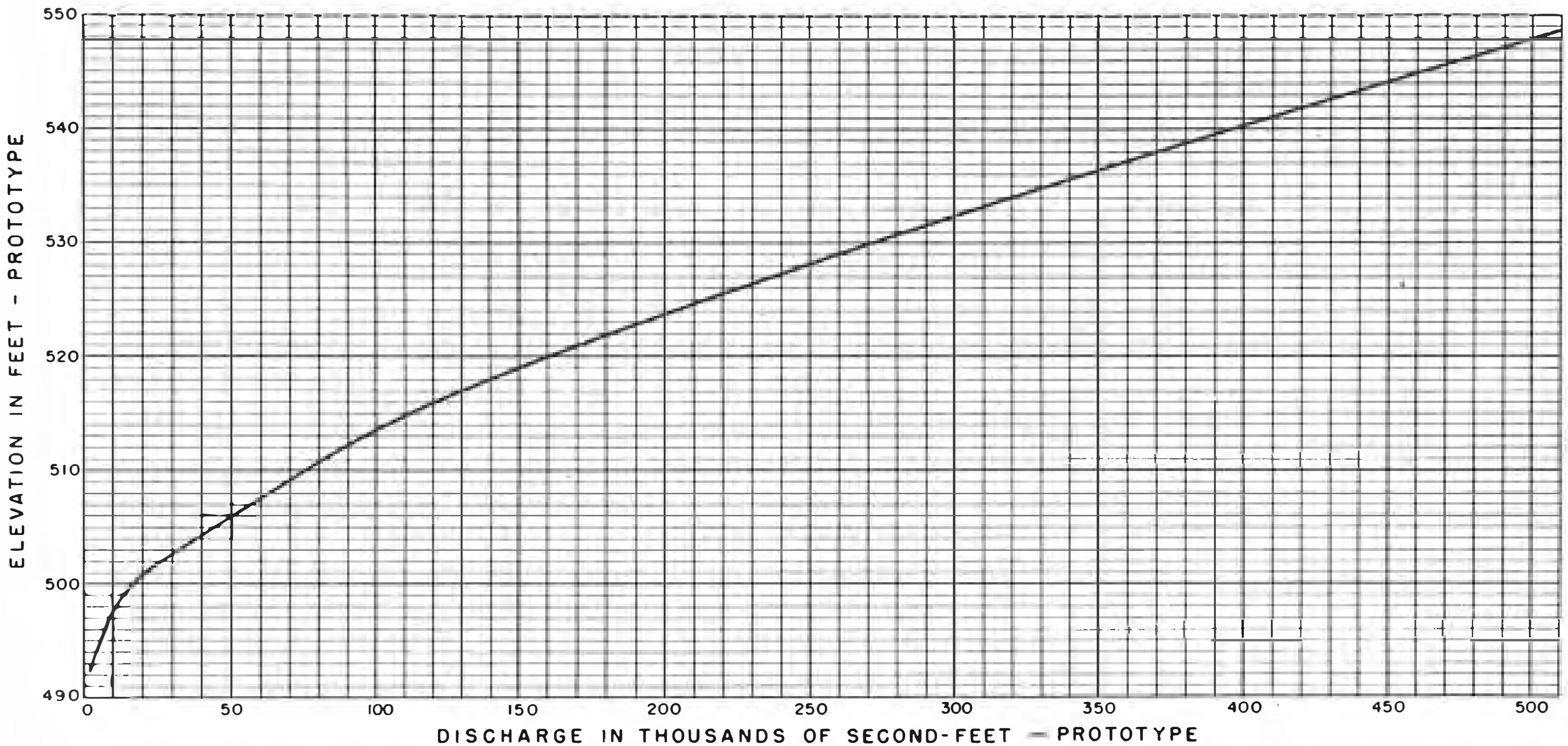
COLORADO RIVER PROJECT-TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:40.8
DETAILS OF CREST, RIVER OUTLETS, APRONS AND SILLS
DENVER, COLORADO
249-D-1076

Stilling Pool Studies

7. The original stilling pool apron. Visual studies on the original model (design 1) showed that the siphon jets produced very rough flow on the spillway. However, the siphons were eliminated from the design before organized testing began. The crest and the outlets were calibrated individually and collectively to ascertain the proper model quantity to represent a given prototype discharge. Throughout all tests the tailwater elevation for any discharge was maintained in accordance with the normal tailwater shown on figure 7.

Very satisfactory conditions existed in the stilling pool at all discharges, with only the river outlets operating. The remote possibility of operating all outlets with a low reservoir elevation led to further tests in which a satisfactory procedure was determined for placing the outlets in operation with the reservoir near the crest elevation. Outlet 1, nearest the powerhouse, was operated with minimum tailwater, to represent conditions when it was opened with no flow in the river below the dam. At 5,000 second-feet the jet swept down the apron, over the sill, and scoured excessively the river bed immediately downstream from the sill. When the dentated sill was placed on the apron (figure 6), slight improvement was noted and a series of tests was made with the left river outlet discharging at normal tailwater. A discharge of 2,000 second-feet gave conditions which were not altogether undesirable. The return flow onto the apron from the right crowded the jet against the wall, where rough conditions and considerable splash occurred. Although the jet continued downstream over the sill, very little erosion was noted on the stream bed. Sand was deposited on the apron to the right of the outlet. Discharges of 3,000 and 4,000 second-feet, respectively, produced excessive scour below the sill with an increased amount of sand deposited on the apron. Conditions were practically the same with outlets 1 and 2 operating. A visual test with the two center outlets discharging gave fairly good results and indicated that a systematic operation of the river outlets from the center of the spillway might eliminate any danger to the toe of the apron. It also indicated that depressing the horizontal apron immediately downstream from outlets 5 and 6 and operating these first might produce a satisfactory operating program. The short sectional model prevented a satisfactory investigation either of the river outlet operating program or of the depressed area. Accordingly, these features were studied on the 1 to 68 model.

The stilling pool conditions were studied next, with the crest and the river outlets operating. With a 2 to 1 trapezoidal sill, conditions in the pool were very rough at high discharges, although the jump did not sweep off the apron. With various discharges, the tailwater level was raised above normal until a good pool was obtained. This procedure (figure 7) was used to determine approximately the distance the apron should be lowered to obtain good conditions. Results showed



MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
TAILWATER CURVE

R.A.G. 8-24-37

249-D-1077

FIGURE 7

that with the original 2 to 1 trapezoidal sill the apron should be lowered about 3-1/2 feet. With the 7-1/4 foot, 1 to 1 trapezoidal sill, the pool became rougher and a slight additional depth was necessary to produce a satisfactory jump. When the dentated sill was installed on the apron, conditions were improved, with a satisfactory jump forming on the apron at normal tailwater.

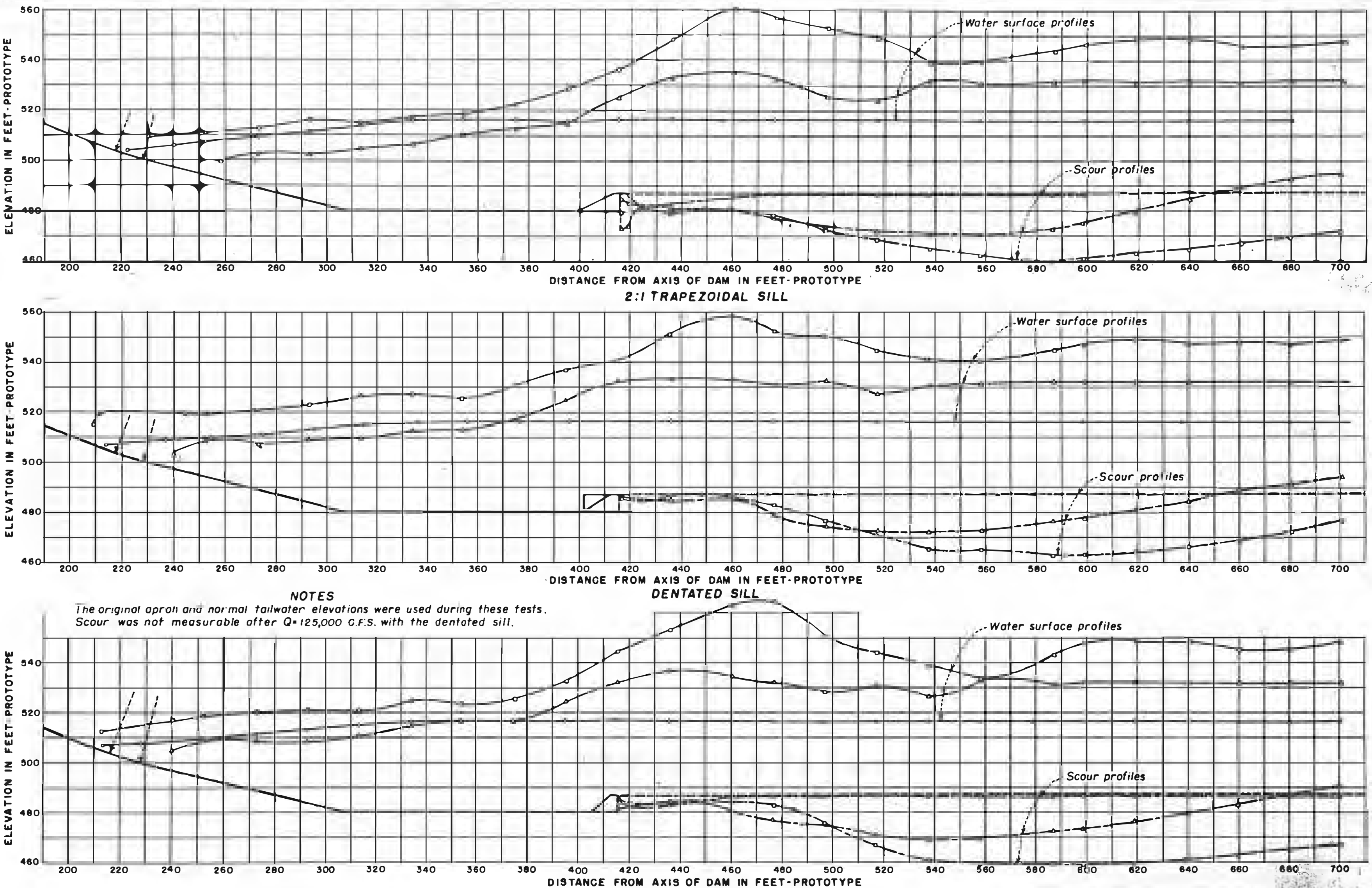
Comparative scour and erosion profiles were taken along the center line of the model flume for the three sills at discharges of 125,000, 300,000, and 500,000 second-feet, respectively, (figure 8). The dentated sill gave the best scour conditions and the smoothest water surface, with the 2 to 1 trapezoidal sill being somewhat more efficient than the 1 to 1 sill. The dentated sill was accepted as the final design and was used during the remaining studies.

8. Apron design 2. A 75-foot radius bucket appeared to turn the flow too abruptly as it entered the pool. The apron design was changed to a 200-foot radius bucket tangent to the face of the dam, the invert of the parabolic river outlets, and a 4:1 sloping apron. It was believed that the decrease in pressure on the outlet exits would increase the quantity flowing through them when the crest was discharging. The 4:1 sloping apron was moved upstream from its original position, producing a longer horizontal apron and resulting in a saving in concrete (design 2, figure 6). The scour and water surface profiles, taken at discharges of 125,000, 300,000, and 500,000 second feet, respectively, are shown on figure 9, together with the corresponding data for the original design. While there was little difference in the results at the first two discharges, design 2 produced less scour and a greater depth of water on the apron for the 500,000 second-foot discharge.

Although this improvement was encouraging, means of further improving the stilling pool action at maximum discharge were sought. Two methods of accomplishing this were considered. One was to lengthen the horizontal apron either by moving the sill downstream or by steepening the sloping apron; the other was to lower the horizontal apron, thus increasing the effective tailwater depth. Moving the sill downstream would increase the cost of the spillway, and it was therefore not considered a satisfactory solution.

9. Apron design 3. Design 3 was formed by removing the downstream portion of the bucket and placing the sloping apron tangent to the 200-foot radius bucket and the parabolic outlets. This resulted in a 2.094:1 sloping apron which lengthened the horizontal portion of the apron (figure 10).

The action of the outlet jets with a discharge of 125,000 second-feet was practically the same as for the preceding design except that the more abrupt change in direction at the toe of the 2.094 to 1 slope gave a greater spreading effect. This condition produced a satisfactory



EXPLANATION

- ------- Initial bed.
- o———o Water surface Q=125,000 C.F.S.
- o-----o Scour after Q=125,000 C.F.S.
- △———△ Water surface Q=300,000 C.F.S.
- △-----△ Scour after Q=300,000 C.F.S.
- □———□ Water surface Q=500,000 C.F.S.
- □-----□ Scour after Q=500,000 C.F.S.

MARSHALL FORD DAM
ULTIMATE DEVELOPMENT
HYDRAULIC MODEL STUDIES - SCALE 1:40.8
COMPARISON OF SILLS

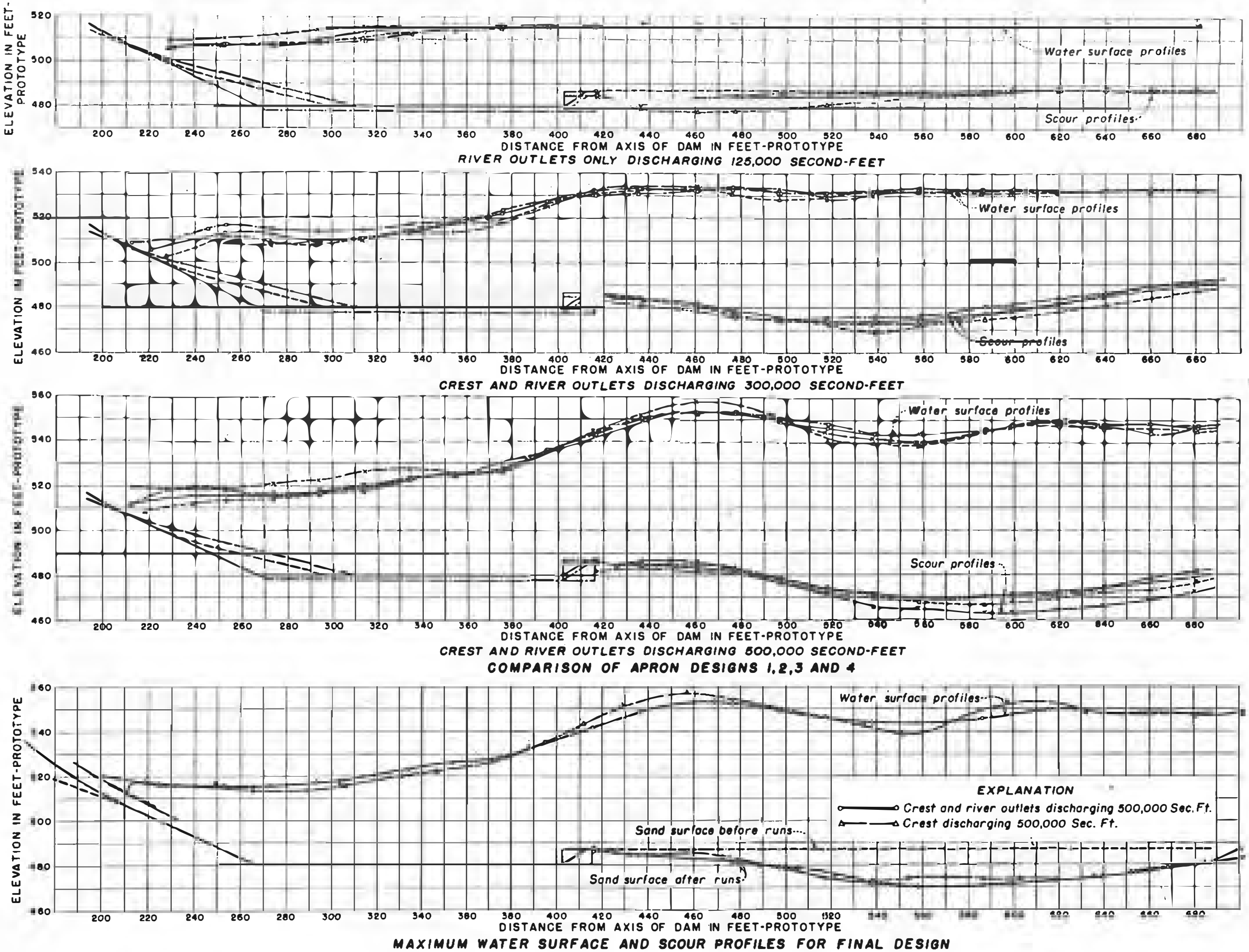
TABULATION OF MODEL FLOW DURATION

DISCHARGE SEC. FT.	DURATION OF FLOW	ACCUMULATED TIME
125,000	25 MIN.	25 MIN.
300,000	45 MIN.	1 HR.-10 MIN.
500000	35 MIN.	1 HR.-45 MIN.

D.M.L.-R.A.G. - 1-30-37

FIGURE 6

249-D-1078



TABULATION OF MODEL FLOW DURATION

DISCHARGE SEC. FT.	DURATION OF FLOW	ACCUMULATED TIME
125,000	25 MIN.	25 MIN.
300,000	45 MIN.	1 HR. 10 MIN.
500,000	35 MIN.	1 HR. 45 MIN.

EXPLANATION

- Design No. 1
- Design No. 2
- Design No. 3
- Design No. 4
- Initial sand El.

NOTE

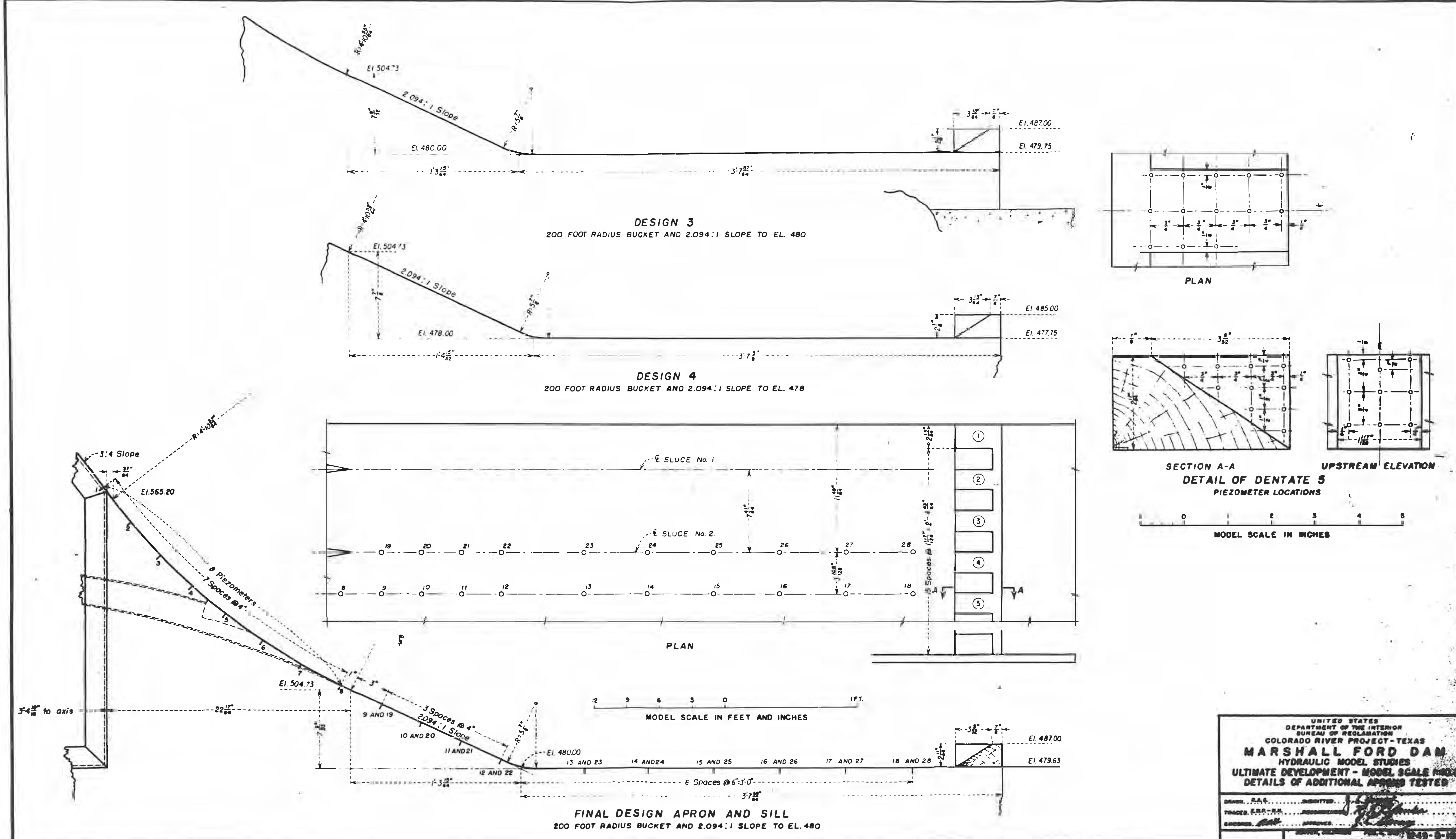
The dentated sill and normal tailwater elevations were used during these tests.

MARSHALL FORD DAM
ULTIMATE DEVELOPMENT
HYDRAULIC MODEL STUDIES - SCALE 1:40.8
APRON STUDIES

H.W.B.-R.A.G. - 1-30-37

249-D-1079

FIGURE 9



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE
DETAILS OF ADDITIONAL APRONS TESTED

DRAWN R.A.C. SUBMITTED
TRACED E.B.P.-R.R. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO
249-D-1

jump and reduced the impact on the sill. An improved stilling pool with less scour downstream resulted for all discharges (figure 9).

The appearance of the stilling pool with only the spillway discharging was improved, a very satisfactory jump forming for discharges up to 200,000 second-feet. Increasing the flow to 360,000 second-feet (reservoir elevation 740.0) gave a rough water surface in the pool, but the jump formed well up on the apron. The resulting scour appeared less than with the maximum discharge when the river outlets were also operating.

10. Apron design 4. This design was formed by lowering the horizontal apron and the sill of design 3 two feet, to elevation 478.0. Flow conditions were not improved over those for the preceding design. The water surface and the scour profiles gave no improvement over design 3 (figure 10). These results demonstrated that the improvements obtained with the 2.094 to 1 slope were due to the increased length of the horizontal apron. Accordingly, the lower apron would not be justified.

11. Final design apron. Design 3 was considered a satisfactory solution of the stilling pool problem. Accordingly, this apron, with slight changes, was installed on the model and termed the final design. The downstream end of the apron was lowered from elevation 479.75 to 479.63, and the spacings of the dentals on the sill were altered (figure 10). These slight changes were not sufficient to alter the action of the stilling pool in scour and water surface profiles from those for design 3.

A condition might arise on the prototype whereby a maximum flood must pass the dam before the river outlets could be opened. Accordingly, with 500,000 second-feet passing over the crest, the resulting water surface and the scour profiles were obtained (figure 9). These profiles were practically identical with those for the crest and the sluices both operating. While the water surface was very rough, the jump formed well up on the apron, demonstrating that the apron and the sill were effective when the maximum discharge passed over the crest. However, conditions should improve when the spillway and the river outlets operate together.

Piezometers were installed in the final design apron and sill to investigate the distribution of pressures throughout the stilling pool. Two rows of piezometers were installed, one along an extension of the center line of one river outlet and another equally spaced between a pair of outlets. Piezometers were also installed on the upstream face, the top, and one side of a dentate to investigate the existence of negative pressures. Pressures on the apron and the sill were obtained at discharges of 125,000, 300,000, and 500,000 second-feet with crest and outlets operating, and at discharges of 200,000, 360,000, and 500,000 second-feet with the outlets closed (figure 11). At no discharge did any negative pressures exist on the apron or the sill. Stilling pool conditions are shown on figure 12.

Spillway Studies

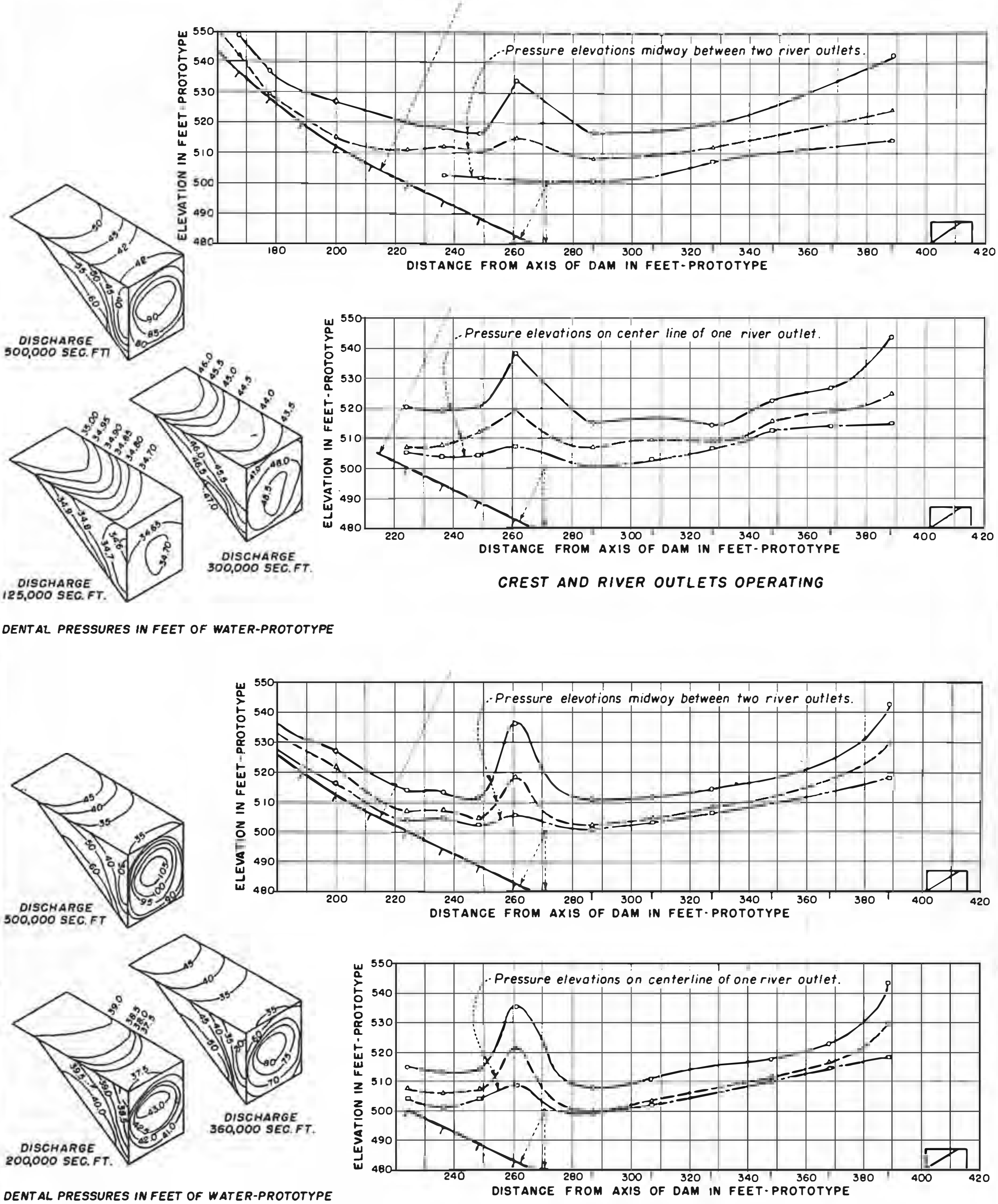
12. Discharge calibrations. Calibrations on the 1 to 40.8 ultimate model included calibration of the river outlets with the crest closed, calibration of the crest alone, and determination of the discharge characteristics of the outlets with the crest discharging. Discharge coefficients for the crest and the river outlets are shown on figure 13. From these coefficient curves, prototype discharge curves were calculated for 700 feet of crest and 24 river outlest (figure 14). These curves do not include the effect of piers that will be present on the prototype crest. At a reservoir elevation of 740.0, a discharge of about 490,000 second-feet could pass the dam with both the crest and the river outlets operating. A reservoir elevation of 740.4 would be required for a discharge of 500,000 second-feet.

13. Spillway training walls. Tests were made next to determine the adequacy of the proposed training walls on the spillway. This was accomplished by recording water surface profiles along the side of the flume at various discharges (figure 15). The proposed training walls for the ultimate development were considered to be sufficiently high to pass 500,000 second-feet over the crest without overtopping the wall.

14. Crest pressures. Piezometers were installed in the curved portion of the crest (figure 6) to determine pressures at crest discharges of 60,000, 120,000, 240,000, and 300,000 second-feet, and at reservoir elevation 740.063 (figure 16). A very slight negative pressure was found to exist on the downstream portion of the crest at high discharges.

MODEL OF LEFT HALF OF SPILLWAY FOR ULTIMATE DEVELOPMENT

15. Description of the model. The 1:40.8 sectional model of the Marshall Ford ultimate development was necessarily limited in its usefulness to study of the stilling pool and the crest. To study river outlet combinations, the effect of the training walls on the stilling pool, and conditions in the diversion conduit area, a model was built representing half of the spillway. A model scale of 1 to 68 permitted use of commercial tubing in the construction of the circular river outlets. With a model of half the spillway section on this scale, the model discharge represented about the maximum capacity of



NOTES

Pressures are in feet of water prototype.

Normal tailwater elevations were used on these tests.

EXPLANATION

- Discharge 500,000 second-feet
- Discharge 360,000 second-feet
- Discharge 200,000 second-feet

MARSHALL FORD DAM

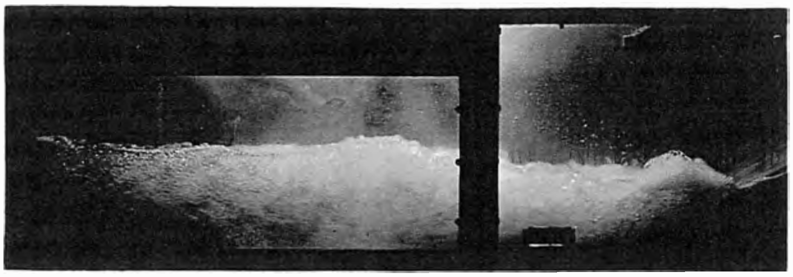
ULTIMATE DEVELOPMENT

HYDRAULIC MODEL STUDIES - SCALE 1:40.8

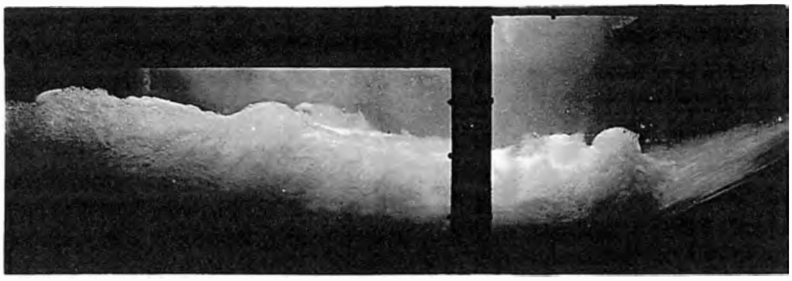
APRON AND SILL PRESSURES

J.M.B.-H.W.B. 6-26-37

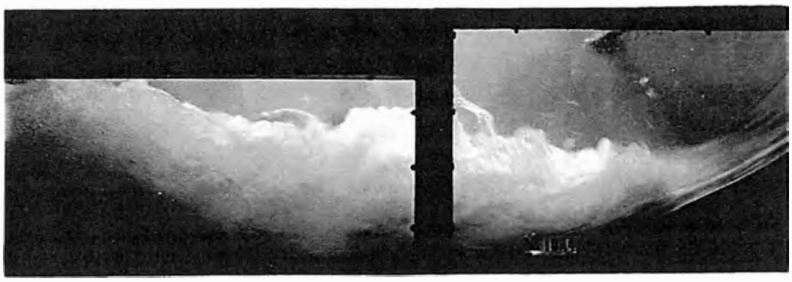
249-D-1081



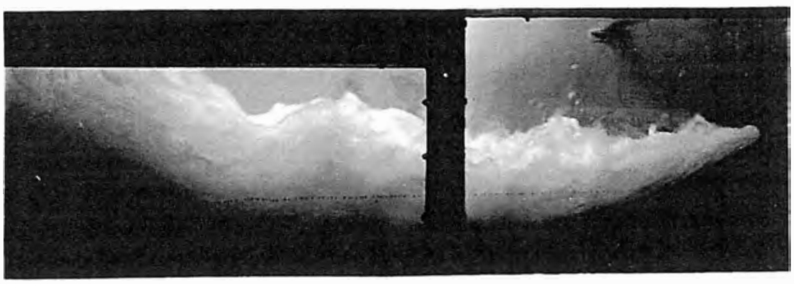
Discharge 125,000 Second-Feet



Discharge 240,000 Second-Feet



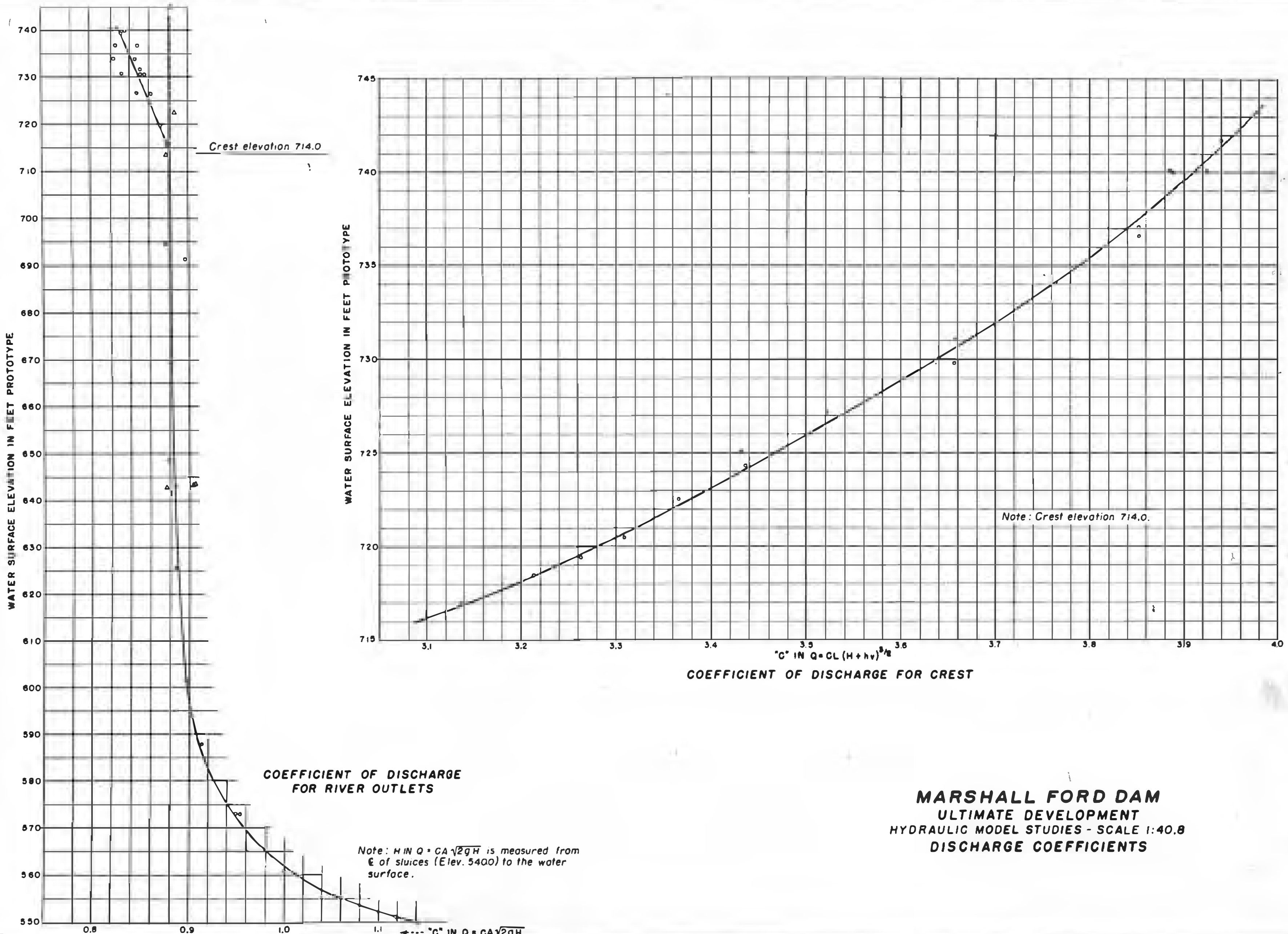
Discharge 360,000 Second-Feet



Discharge 500,000 Second-Feet

Scour After 500,000 Second-Feet

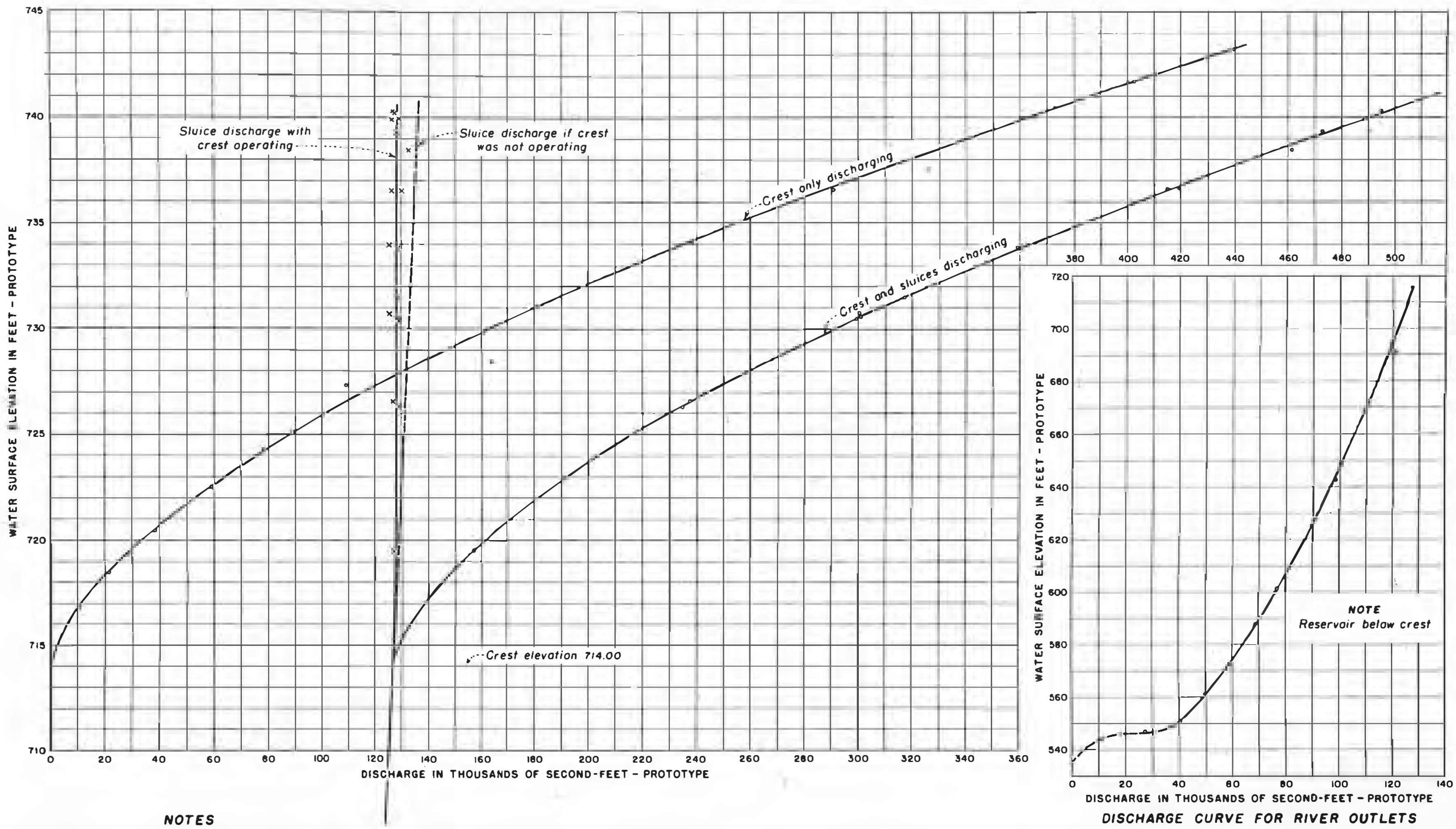
STILLING POOL ACTION - RECOMMENDED DESIGN



MARSHALL FORD DAM
ULTIMATE DEVELOPMENT
HYDRAULIC MODEL STUDIES - SCALE 1:40.8
DISCHARGE COEFFICIENTS

249-D-1082

FIGURE 13



NOTES

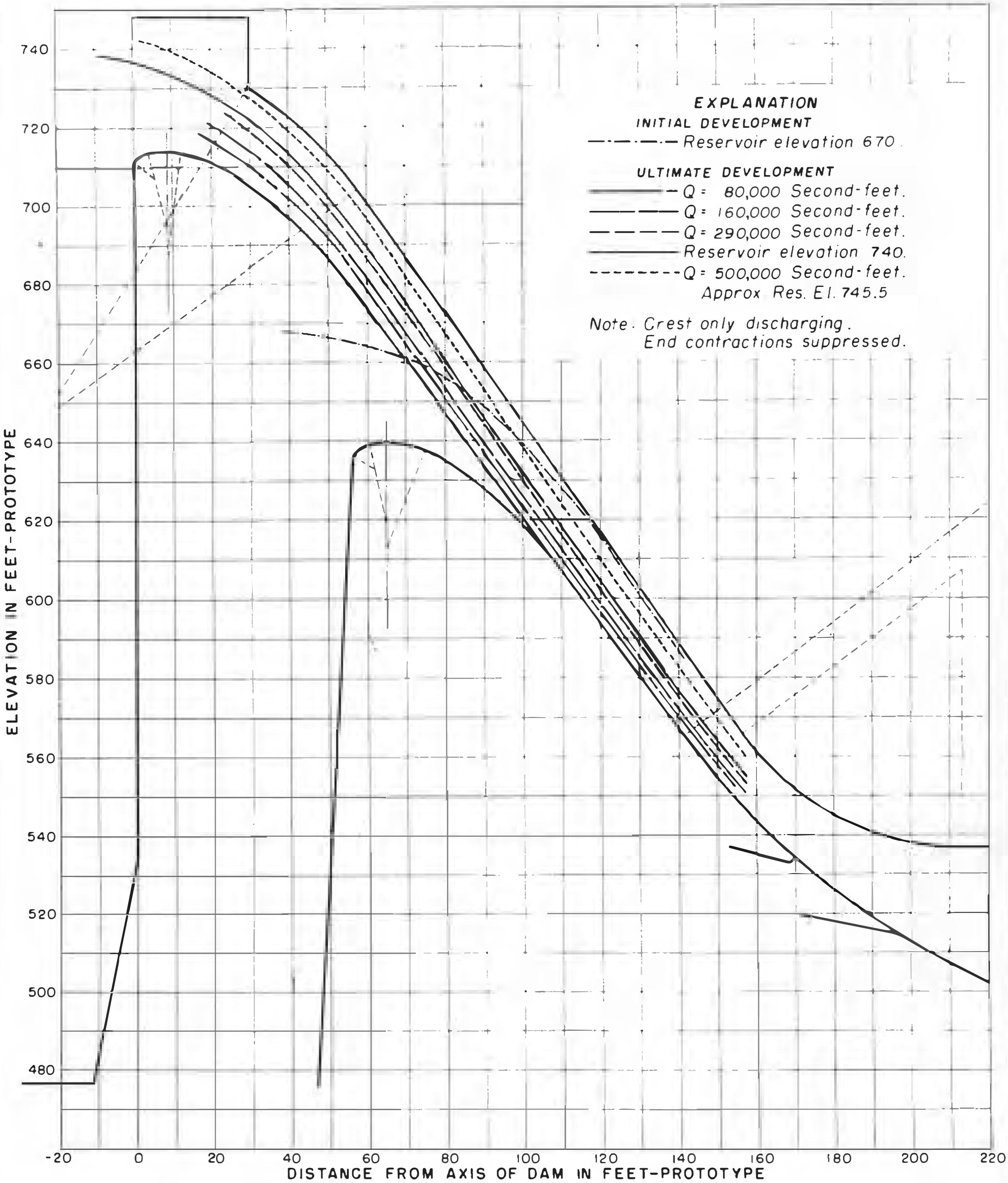
Curves are for 700 foot crest with no piers, 200 foot radius bucket, and 24-8'-6" diameter circular river outlets.
Curves were calculated from the coefficient curves.
Observed points are shown.

MARSHALL FORD DAM
ULTIMATE DEVELOPMENT
HYDRAULIC MODEL STUDIES - SCALE 1:40.8
DISCHARGE CURVES

R.A.G. 5-20-1937

249-D-1083

FIGURE 14



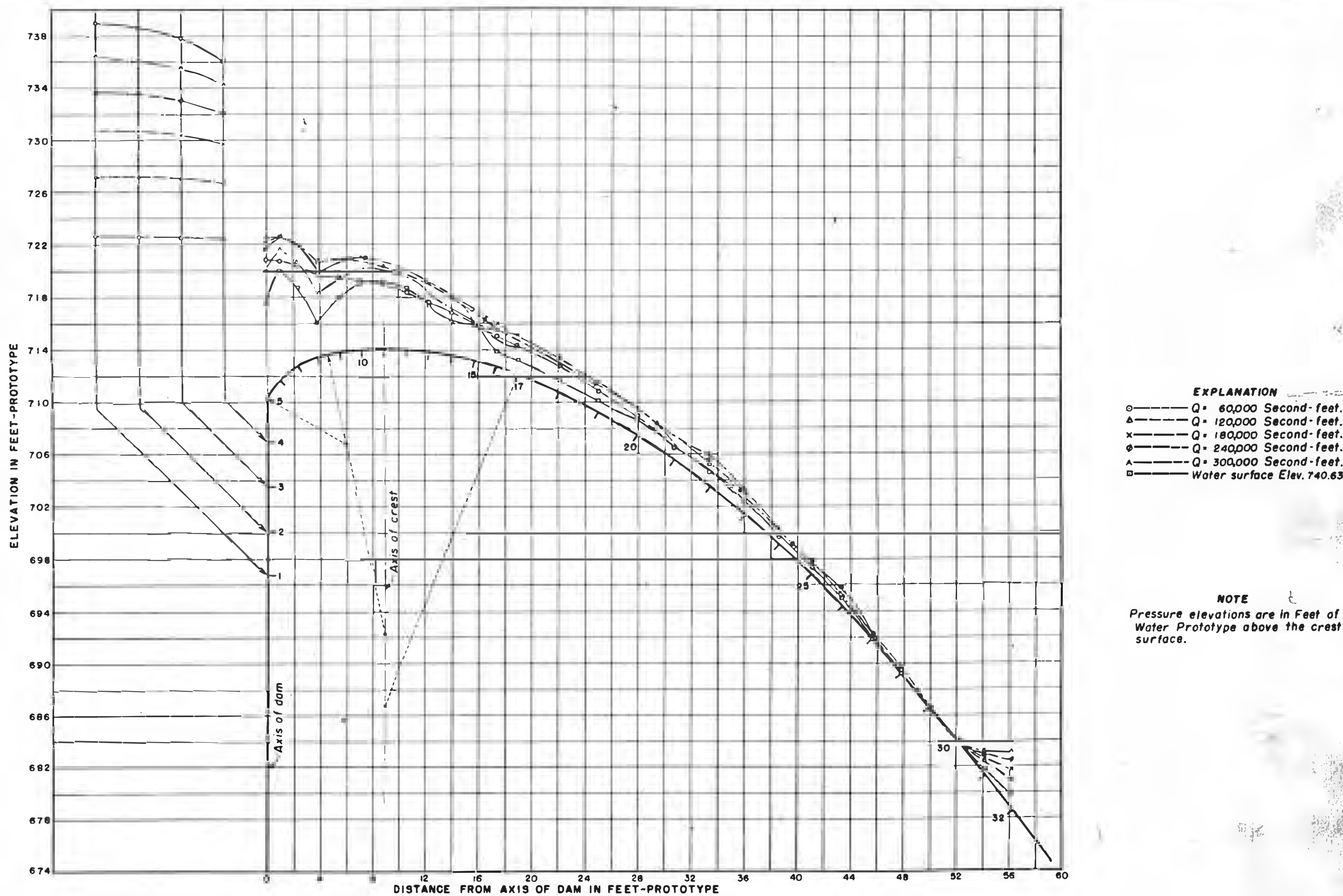
MARSHALL FORD DAM

HYDRAULIC MODEL STUDIES - SCALE 1:40.8

WATER SURFACE PROFILES ALONG TRAINING WALLS

R.A.G.-D.M.L. 1-30-37

249-D-1084



MARSHALL FORD DAM
ULTIMATE DEVELOPMENT
HYDRAULIC MODEL STUDIES - SCALE 1:40.8
PIEZOMETRIC PRESSURES ON CREST

R.A.G. 1-11-37

249-11-1085

FIGURE 16

the laboratory pump and supply system.

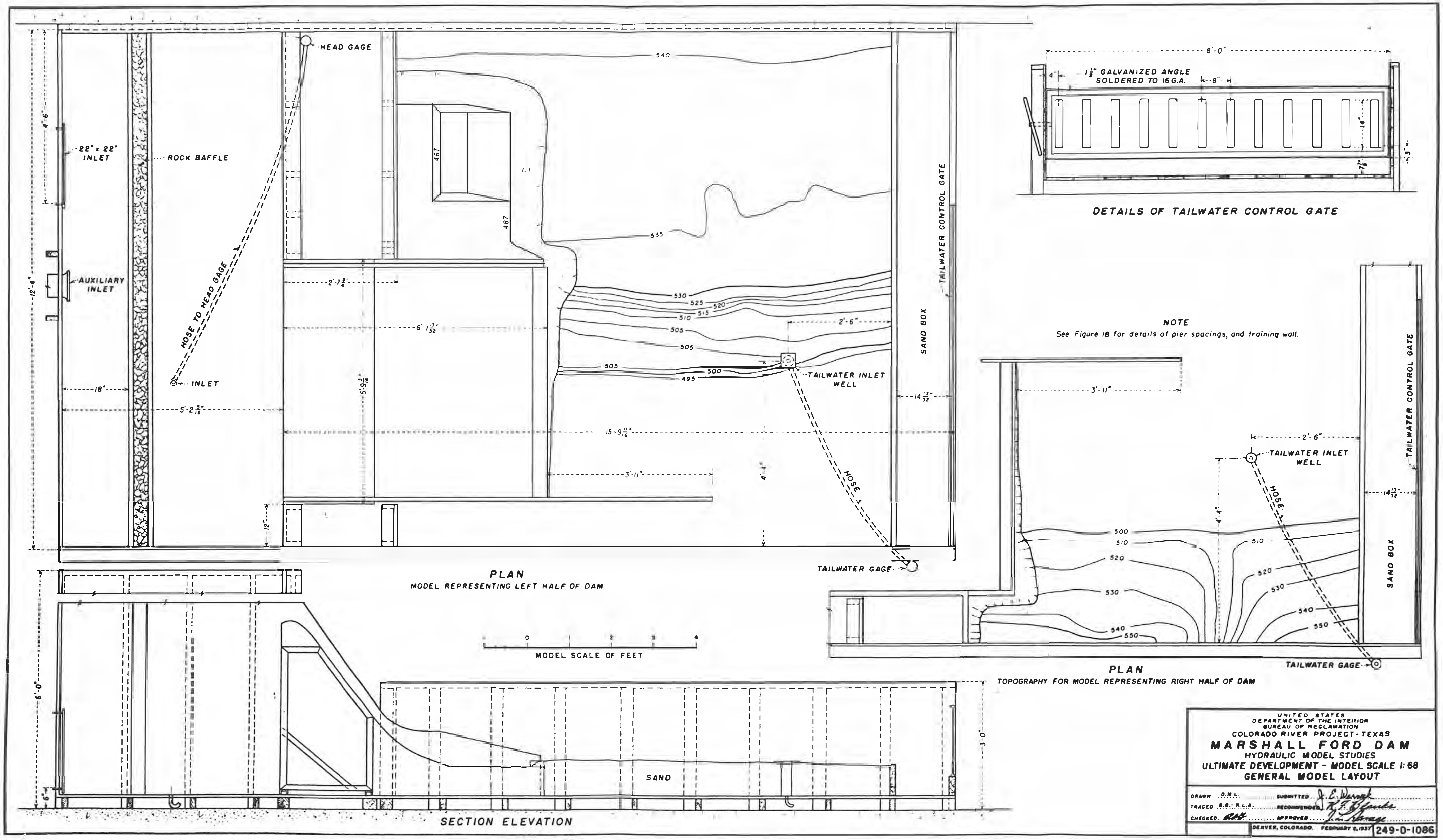
Two metal-lined tanks (figure 17) were constructed; one to provide a forebay for the model and the other to contain the topography downstream from the dam representing half of the width of the river bed, with a reach of 800 feet. The topography was molded in sand. A gate at the downstream end of the tailrace tank controlled the tailwater level, and a hook gage measured its elevation. The forebay tank was connected to the laboratory supply system and fitted with baffles to quiet the inflowing water. A forebay gage was installed so that the reservoir elevation could be measured.

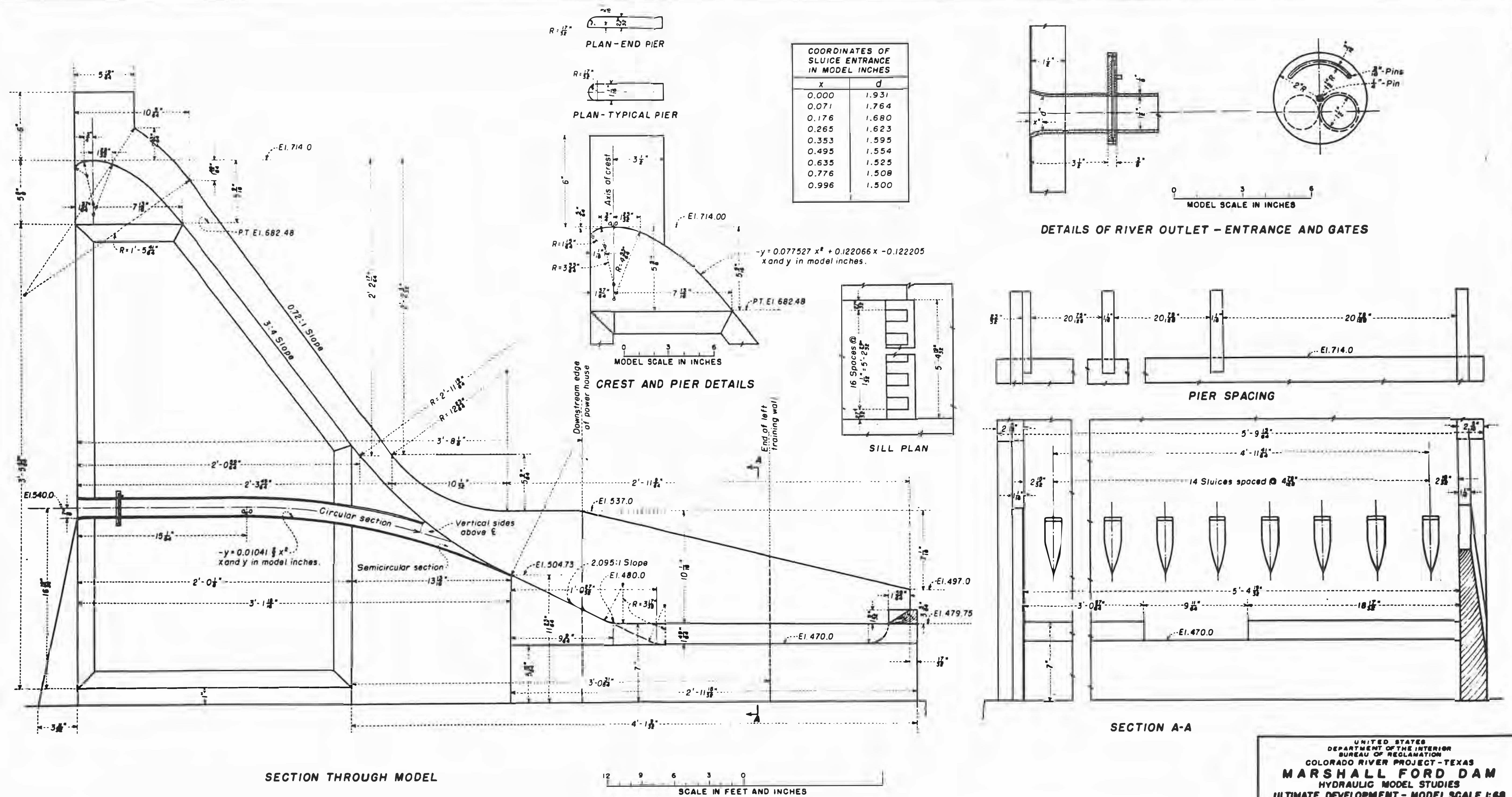
The model was constructed of angle-iron bents covered with sheet iron (figure 18). The 2.094 to 1 slope was installed on the apron and connected to the spillway face by a 200-foot radius bucket. The horizontal apron extended from elevation 480.0 at its upstream end to elevation 479.75 at its downstream extremity. A dentated sill, with dimensions proportional to those for the first sill used on the 1 to 40.8 ultimate model, was installed at the end of the apron. Fourteen of the 24 river outlets were installed in the model, making it possible to represent either half of the spillway. The circular outlets were fitted with bell entrances, while the downstream portions were parabolic curves tangent to, and terminating at, the point where the inverts intersected the 200-foot radius bucket. The outlets were fitted with small gates near the upstream end so that any flow combination could be studied. The outlets were numbered from left to right, looking downstream.

The pier spacings were so arranged that the crest represented the left end of the spillway. A wall, extending above the tailwater, was placed on the right end of the apron and extended 3 feet 11 inches downstream from the sill (figure 17). Training wall design 1 was placed on the left side of the apron (figures 18 and 22). Since consideration was being given to operating river outlets 5 and 6 first, when the outlets were brought into operation at minimum tailwater and high reservoir elevations, that portion of the horizontal apron immediately downstream from these outlets was placed at elevation 470.0 to provide deeper tailwater.

16. River outlet stilling basin. With only outlets 5 and 6 discharging into the depressed stilling basin at normal tailwater, the flow conditions were not satisfactory. Severe return eddies on each side carried sand and gravel upstream onto the apron where it would have caused severe abrasion in the prototype. Low, intermediate training walls (design 1, figure 19) were installed on each side of the depressed basin. With a total discharge of 5,000 second-feet through the two outlets, little scour occurred at the toe of the sill although some sand was carried onto the left portion of the apron. When the discharge was increased to 10,000 second-feet, the

FIGURE 17

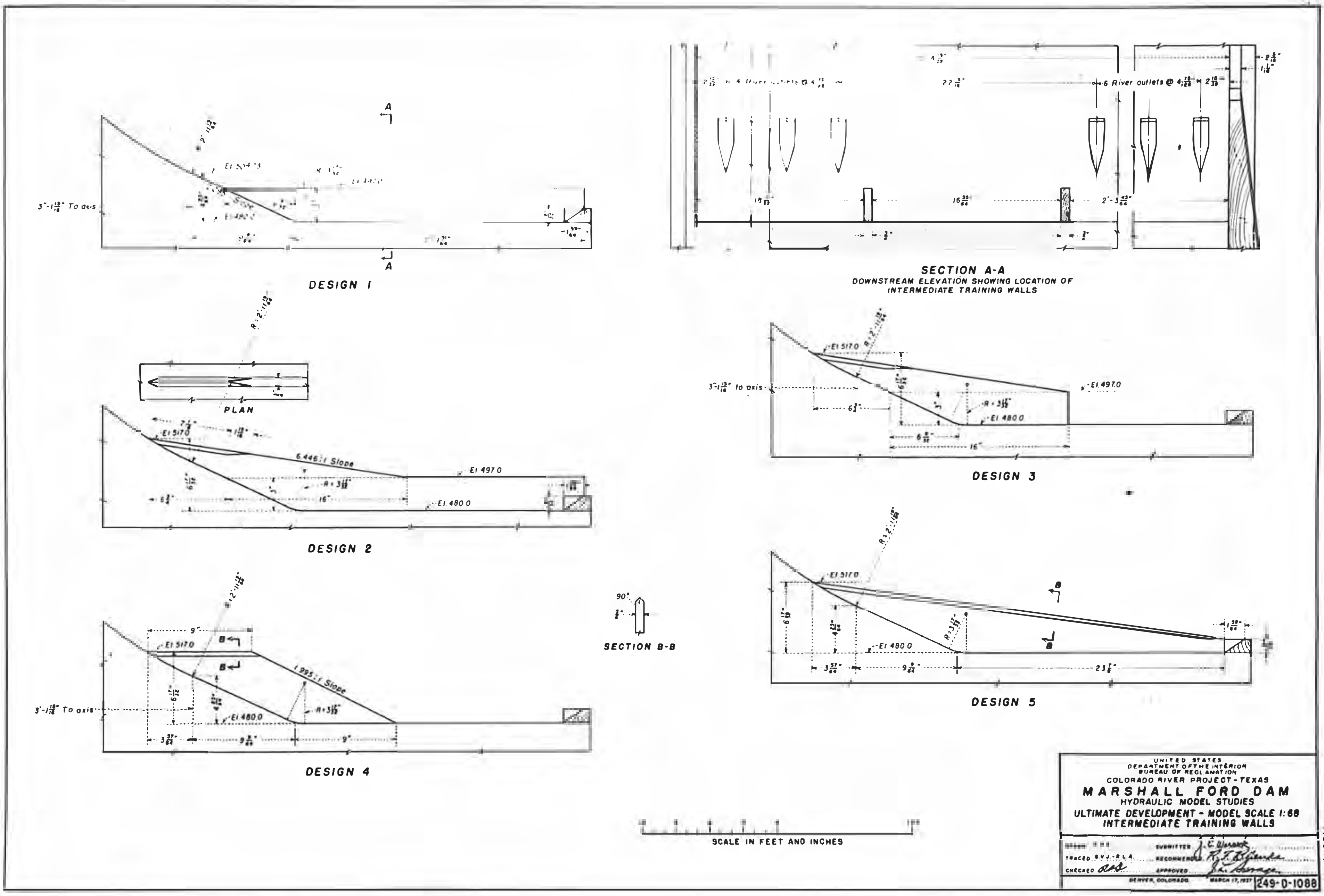

HEAD GAGE
22" x 22" INLET
ROCK BAFFLE
AUXILIARY INLET
HOSE TO HEAD GAGE
INLET
TAILWATER INLET WELL
HOSE
TAILWATER GAGE
SAND BOX
TAILWATER CONTROL GATE
PLAN
MODEL REPRESENTING LEFT HALF OF DAM
MODEL SCALE OF FEET
SECTION ELEVATION
SAND
1½" GALVANIZED ANGLE SOLDERED TO 16 G.A.
DETAILS OF TAILWATER CONTROL GATE
NOTE
See Figure 18 for details of pier spacings, and training wall.
PLAN
TOPOGRAPHY FOR MODEL REPRESENTING RIGHT HALF OF DAM
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT-TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:68
GENERAL MODEL LAYOUT
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO. FEBRUARY 2, 1937
249-D-1086



COORDINATES OF SLUICE ENTRANCE IN MODEL INCHES	
x	d
0.000	1.931
0.071	1.764
0.176	1.680
0.265	1.623
0.353	1.595
0.495	1.554
0.635	1.525
0.776	1.508
0.996	1.500

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:60
MODEL DETAILS

DRAWN ... R.A.B. ... SUBMITTED ... J. E. Warnock
TRACED ... R.V.L.-R.L.L. ... RECOMMENDED ...
CHECKED ... ... APPROVED ...
DENVER, COLORADO FEBRUARY 2, 1937 | 249-D-1067



DESIGN 1
SECTION A-A
DOWNSTREAM ELEVATION SHOWING LOCATION OF INTERMEDIATE TRAINING WALLS
6 River outlets
PLAN
DESIGN 2
6.446:1 Slope
El 517.0
El 497.0
El 480.0
DESIGN 3
SECTION B-B
90°
DESIGN 4
1.995:1 Slope
DESIGN 5
SCALE IN FEET AND INCHES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:68
INTERMEDIATE TRAINING WALLS
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO
MARCH 17, 1937
249-D-1088

FIGURE 19

erosion was materially increased, with sand being carried onto the apron on each side of the stilling basin. An undesirable grinding action on the apron was produced by a large whirl on the right. A large amount of sand was deposited on the downstream left portion of the apron, completely covering the sill in this region. With one outlet discharging 5,000 second-feet, conditions improved but were still undesirable.

Consideration was given to the possibility of opening either alternate outlets or even every third outlet when outlets were placed in operation. Further study was deferred, however, until other details in the stilling pool design were determined. The depressed area was removed to permit these other studies.

17. Location of diversion conduits. At the completion of the construction, according to the specifications, the diversion conduits in the ultimate development would be filled with concrete. Outlets 7, 8, 9, and 10 in the model were closed to represent this condition. Visual observations demonstrated that with the river outlets only discharging, good conditions in the pool could not be obtained unless low training walls were placed on each side of that portion of the apron immediately downstream from the diversion conduits. A proposal to place the diversion conduits on the left end of the spillway, since only one intermediate training wall would be needed, was studied. This condition was represented by closing outlets 1, 2, 3, and 4 instead of outlets 7, 8, 9, and 10.

To facilitate testing, the intermediate training walls were removed, it being planned to develop the correct shape for these walls after the conduit locations had been fixed. Observations were made with various discharges passing the dam; first with the diversion conduits on the left end of the spillway and then by repeating the tests with the conduits located according to the specifications. The left training wall design had not been decided. Since its shape would have an effect on the stilling pool, several walls were tested to obtain comparisons. Training wall designs 1, 2, and 3 were tested under the above procedure (figure 22).

Stilling pool conditions were always better when the diversion conduits were located according to the specification drawings. With the diversion conduits on the left end of the spillway, a strong reverse current deposited sand in the powerhouse tailrace. This condition became worse at high discharges when water began spilling over the training wall onto the hydraulic jump. Raising the top of the wall to the maximum tailwater surface did not eliminate these undesirable conditions, demonstrating that a sloping training wall could be developed which would give satisfactory conditions. Location of the diversion conduits according to specification drawings would consequently produce a saving in cost of the large training wall which would partially compensate for the necessary additional intermediate training wall.

18. Intermediate training wall studies. The complexity of the stilling pool problem did not permit the independent development of any particular feature. While the initial tests for the location of the diversion conduits served their immediate purpose, they also furnished material for the design of a better training wall on the left side of the spillway. A new training wall (design 4, figure 22) was placed on the left of the spillway. This wall was near enough to its final shape so that tests to determine a design for the intermediate training walls could proceed unhindered. The powerhouse wall was installed before testing was started.

With no intermediate training walls, an upstream flow onto the apron toward the region of the diversion conduits carried sand onto the apron. The flow then divided, turning outward and crowding the outlet jets toward the sides of the spillway (figure 20). A discharge of nearly 300,000 second-feet over the spillway was required to prevent this return flow. Necessity for the intermediate training walls was obvious. Tests were made to determine the design of these walls, using discharges of 130,000, 200,000, and 240,000 second-feet, respectively, since the stilling pool conditions were most unsatisfactory at those flows. Five designs were studied before an acceptable one was evolved.

In design 1, low training walls with their tops horizontal at elevation 497.0 were placed on each side of the diversion conduits and extended from the 2.094 slope to the downstream extremity of the sill (figure 19). Conditions were somewhat improved but still remained unsatisfactory, the general flow conditions being similar to those with no walls. Above a discharge of 200,000 second-feet the spillway flow was sufficient to produce an acceptable pool.

Design 2 was formed by adding triangular pieces on the upstream portion of the walls of design 1 to prevent the excessive flow over the walls onto the jump in the upper end of the pool. These additions extended on a 6.46 to 1 slope, from elevation 517.0 at the bucket to the horizontal wall at elevation 497.0 (figure 19). Elevation 517.0 was chosen as the maximum elevation of the wall, since this will be the tailwater for a discharge of 130,000 second-feet. The top of the wall next to the spillway face was beveled to minimize interference with the flow down the spillway face. This bevel was used on all subsequent tests. Higher discharges over the crest prevented excessive flow over the training walls near the face of the spillway. In this design stilling pool conditions were much improved over those for the preceding design. With a discharge of 130,000 second-feet, the jump was improved and less sand was carried onto the apron. Slight improvement was also noticeable at a discharge of 200,000 second-feet. At higher discharges no difference could be detected.

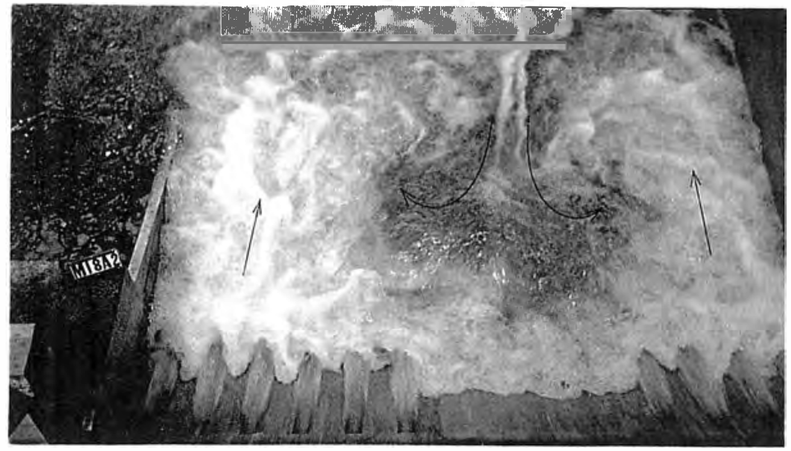
The walls downstream from the intersection of the 6.446 to 1 slope and the horizontal portion at elevation 497.0 were removed, resulting in design 3 (figure 19). With discharges of 130,000 and 200,000 second-feet, the undesirable whirl on the apron was more pronounced, depositing sand well up on the apron. A flow of 240,000 second-feet gave about the same characteristics as the preceding designs except that most of the sand deposited on the apron was removed.

The training walls for design 4 had horizontal portions at elevation 517.0 extending 51 feet out from the face of the bucket, thence on a 1.995 slope to the horizontal apron at elevation 480.0 (figure 19). With this design the whirl on the apron was distinctly worse than in any previous trial. When the discharge was increased to 240,000 second-feet, the sand deposited on the apron by the lower flow was removed.

Comparison of designs 3 and 4 with design 2 showed that the extension of the walls downstream to the sill materially improved the hydraulic jump. Design 2 demonstrated the advisability of building the walls higher near the bucket. These two ideas were combined in design 5 by sloping the training walls from elevation 517.0 at the bucket to the top of the dentated sill (figure 19). Good conditions were obtained at a discharge of 130,000 second-feet, with some sand carried onto the apron between the walls. The water flowing over the walls did not prevent a good jump from forming. This condition was better than for design 2. With higher discharges, practically the same conditions existed as with design 2. Since these walls presented a pleasing appearance and their flow conditions were satisfactory, this design was chosen for future tests with other spillway features.

The left training wall design was fixed, and it was installed on the model with piezometers on both sides of the left intermediate training wall. Pressures were obtained with discharges of 125,000, 200,000, 300,000, 400,000, and 500,000 second-feet, respectively, with outlets 7, 8, 9, and 10 closed. These pressures (figure 21) were used in the stress analysis on the design of the walls.

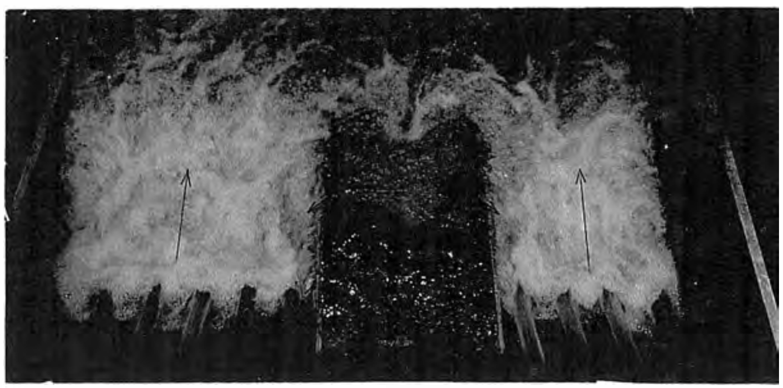
19. Studies on left training wall. During the tests to determine the correct location of the diversion conduits, studies of left training wall designs 1, 2, and 3 (section 17 and figure 22) indicated the need for further development of the training wall to prevent adverse flow conditions in the stilling pool. While better scour conditions existed at the toe of the sloping wall of design 1 than for the other two walls, with discharges up to 300,000 second-feet, water spilled over

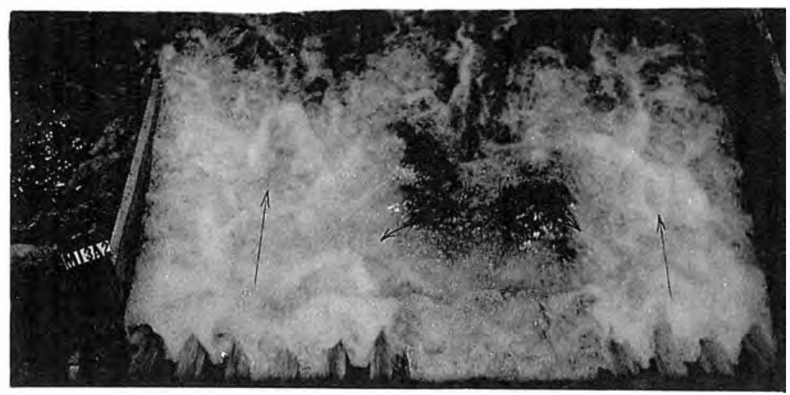


A. Discharge 130,000 Second-Feet

B. Discharge 200,000 Second-Feet

No Walls





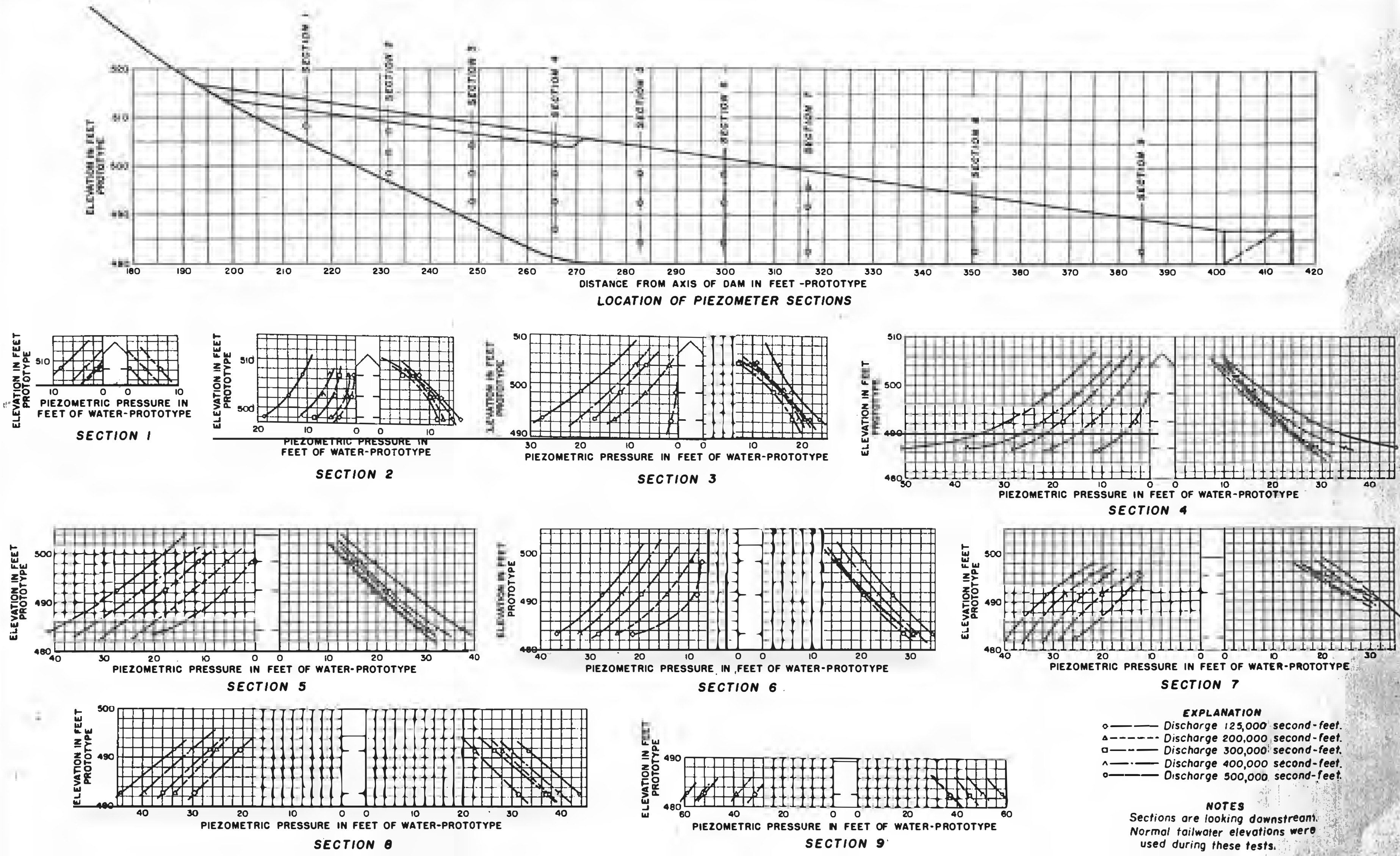
C. Discharge 130,000 Second-Feet

D. Discharge 200,000 Second-Feet

Final Design

INTERMEDIATE TRAINING WALLS

FIGURE 20



MARSHALL FORD DAM

ULTIMATE DEVELOPMENT

HYDRAULIC MODEL STUDIES - SCALE 1:68

PRESSURES ON LEFT INTERMEDIATE TRAINING WALL

R.A.G. 5-6-1937

249-D-1089

FIGURE 21

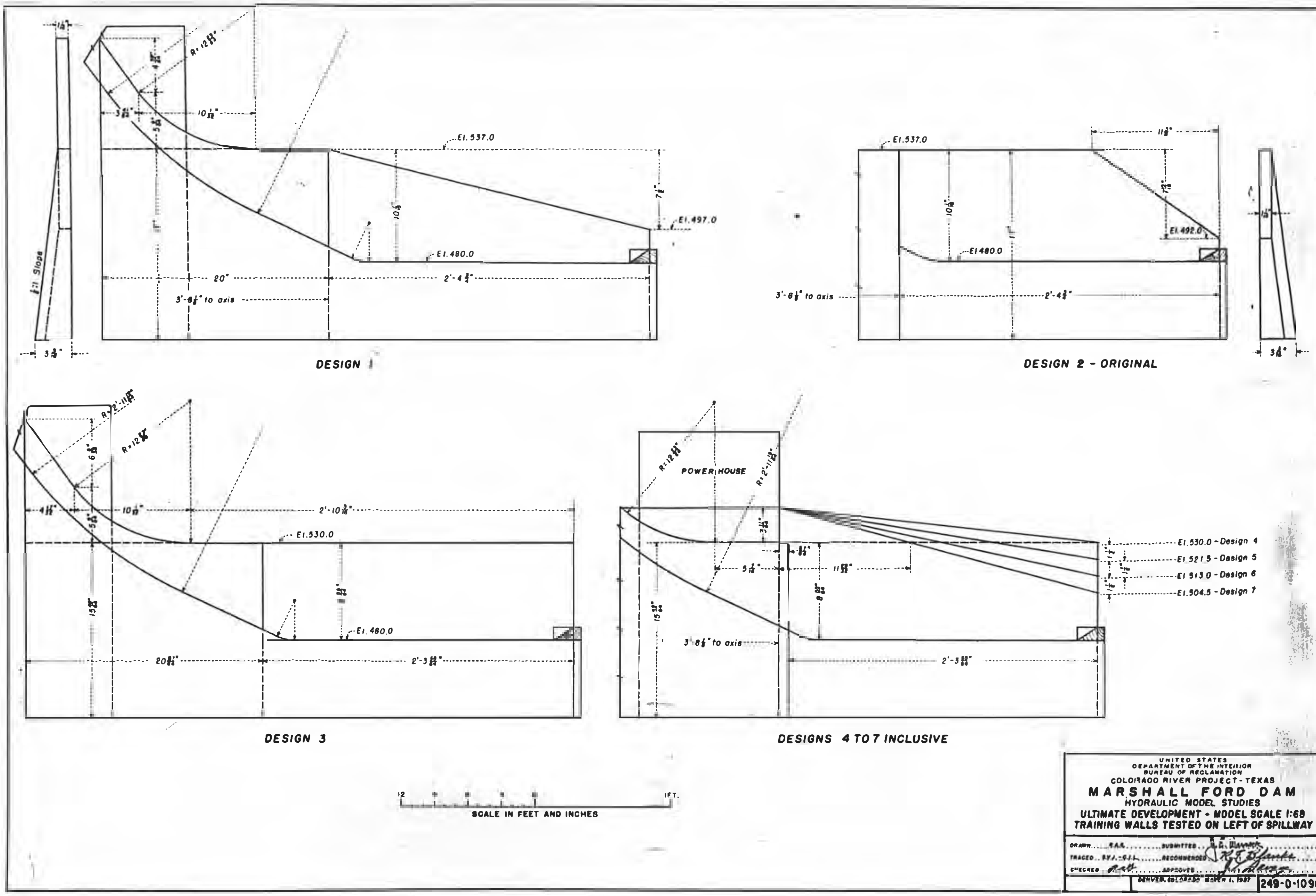

DESIGN 1
El. 537.0
El. 497.0
El. 480.0
3:1 Slope
20"
2'-4 3/4"
3'-8 1/2" to axis
DESIGN 2 - ORIGINAL
El. 537.0
El. 492.0
El. 480.0
3'-8 1/2" to axis
2'-4 3/4"
DESIGN 3
El. 530.0
El. 480.0
2'-10 3/16"
DESIGNS 4 TO 7 INCLUSIVE
POWER HOUSE
El. 530.0 - Design 4
El. 521.5 - Design 5
El. 513.0 - Design 6
El. 504.5 - Design 7
3'-8 1/2" to axis
SCALE IN FEET AND INCHES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:68
TRAINING WALLS TESTED ON LEFT OF SPILLWAY
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO
249-D-1090

FIGURE 22

the walls from the powerhouse area, drowning the jump adjacent to the training wall. Test results obtained with design 1 suggested the use of a sloping wall with its upstream portion at the elevation of the maximum tailwater. Design 4 was a wall of this type, sloping from elevation 548.0 at the downstream face of the powerhouse to elevation 530.0 at the end of the apron (figure 22). This wall, with the end of the powerhouse in place, was used in connection with the studies on the intermediate training walls. After a satisfactory design for the intermediate training walls had been developed and installed on the model, testing was continued to obtain an improved shape for the left training wall with all discharges. The flow conditions in the spillway stilling pool were good. A small amount of sand was carried into the powerhouse tailrace, and a hole was scoured at the downstream toe of the left wall at high discharges. Nevertheless, this design was considered a satisfactory solution. However, additional testing was desirable to determine how much the wall could be lowered in the region where the difference between spillway stilling pool and powerhouse tailrace water-surface elevations was small. This would effect a saving in construction costs.

In designs 5, 6, and 7, the downstream end of the wall was lowered progressively to elevations 521.5, 513.0, and 504.5, thus steepening the slope of the wall (figure 22). With a discharge of 300,000 second-feet no difference could be detected between designs 4 and 5. At a discharge of 500,000 second-feet more water spilled over the wall onto the jump, but with no pronounced effect on the pool. The hole scoured at the toe of the wall was larger in plan but not so deep. With design 6, no change in the pool could be detected at a discharge of 300,000 second-feet, but improvement was noted in the scour. While more water spilled over the wall onto the jump when the discharge was increased to 500,000 second-feet, the pool was very satisfactory, with less scour than in any preceding design. The objectionable hole at the end of the wall had almost disappeared. Design 7 produced conditions differing but little from those of design 6. The increased slope on the wall allowed more water to spill onto the hydraulic jump from the powerhouse tailrace, but this action did not noticeably affect the jump. A slightly increased amount of sand was carried into the tailrace at the higher discharges. Scour conditions were very similar to those in design 6.

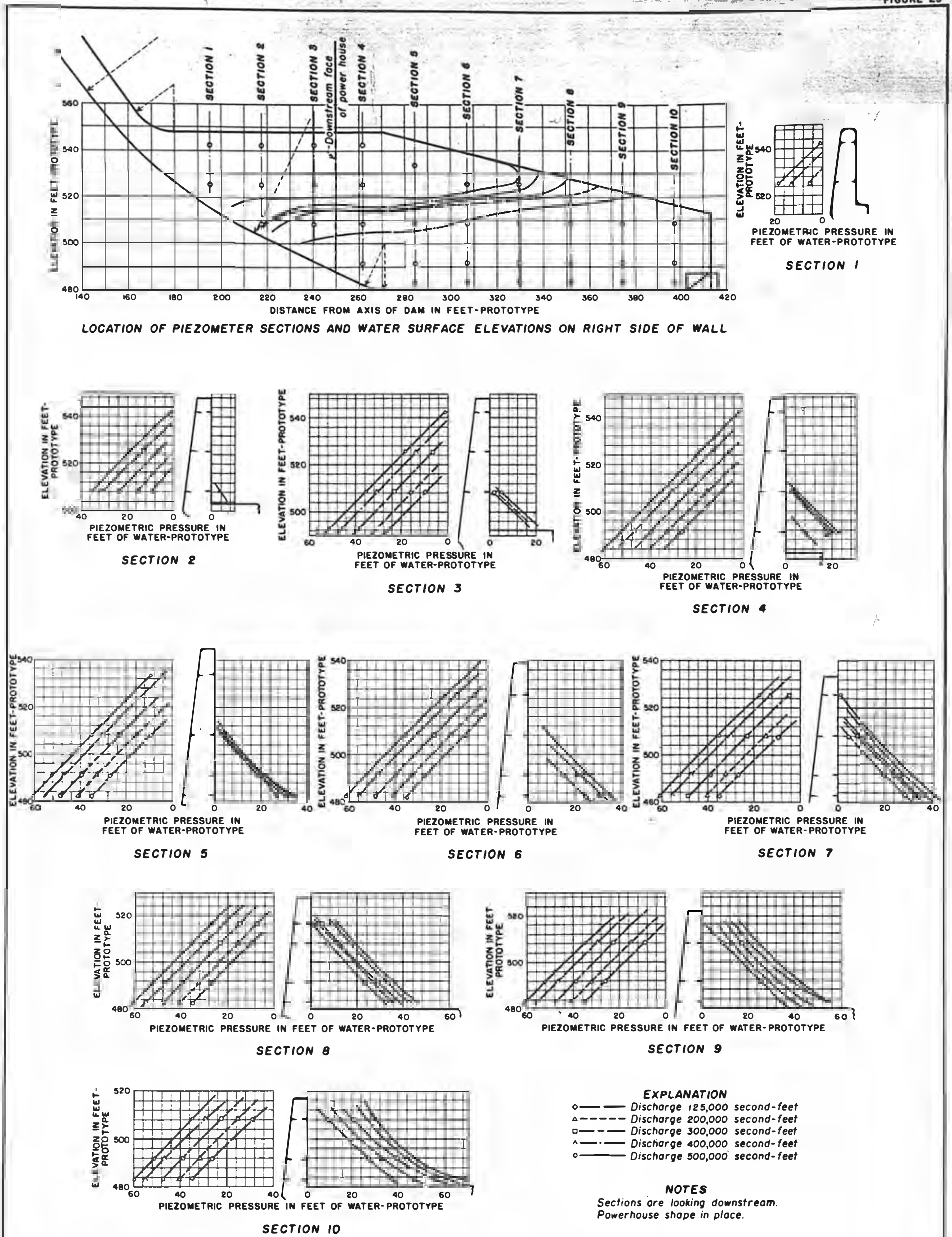
The tests demonstrated that the training wall should be built to elevation 548.0 in the region of the powerhouse, with a sloping downstream extension. Both designs 6 and 7 were considered satisfactory solutions, with scour conditions slightly better with design 6. Since the tailwater will be at or near elevation 505.0 a large portion of the time, the wall with its downstream extremity at elevation 513.0 would present a better appearance than one whose downstream end would be submerged a portion of the time. Design 6 was altered slightly by construction considerations and accepted as the final design.

The final wall extended level, at elevation 548.0, a distance of 20.67 feet downstream from the face of the powerhouse, thence on a 4.049 to 1 slope to elevation 513.0 on the apron. The model wall was fitted with piezometers to obtain pressure data for structural-design purposes. Pressures were obtained at discharges of 125,000, 200,000, 300,000, 400,000, and 500,000 second-feet, respectively. Water-surface profiles along the spillway face of the wall were also measured at these discharges and are shown, together with the pressures, on figure 23. Water was admitted into the model powerhouse so that pressures were obtained on both sides of the wall in this region. Flow conditions with the final design are shown on figure 24.

20. Diversion conduit studies. After the location of the diversion conduits, the type of intermediate training walls, and the shape of the left training wall had been determined, the diversion conduits were installed in the model to determine their operating characteristics. In previous tests the intermediate training walls had been placed with their outside faces along construction joints in the apron. To strengthen these walls by making them part of a T-beam, they were moved in toward the diversion conduits. This necessitated moving the conduits closer together than was proposed in the original specifications. The revised conduits were installed in the model and the dentates for the sill were respaced in accordance with the new training wall locations (figure 25).

21. Evacuation of the reservoir storage through the diversion conduits. It was originally planned that between the time of completion of the initial dam and that of the ultimate structure the two diversion conduits would be closed by wooden bulkheads. Each bulkhead was to contain two 3- by 3-foot gates, to be used in the evacuation of storage water in the extreme bottom of the reservoir after most of the evacuation had been made through the river outlets. The studies of the diversion conduits were made on that assumption. Later, plans were changed, whereby the diversion conduits were not closed until the base on the addition to the upstream face of the initial construction had been poured to a point above the intake to the river outlets and the diversion could be made through them. The diversion conduits were then plugged permanently.

To provide hydraulic data for the original plan small plates, with holes in them to represent the small 3- by 3-foot gates, were placed over conduit entrances of the model to represent these bulkheads (figure 25). Observations with the small gates discharging when the reservoir was below the invert of the river outlets showed that satisfactory conditions existed either with or without the intermediate training walls. A rating curve was prepared for the four 3- by 3-foot gates (figure 26).



MARSHALL FORD DAM

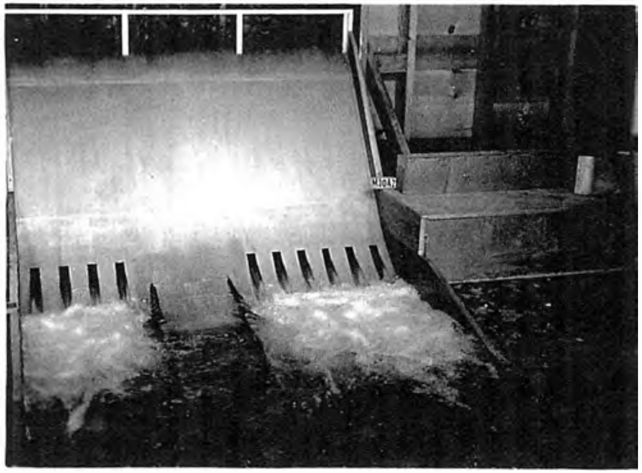
ULTIMATE DEVELOPMENT

HYDRAULIC MODEL STUDIES - SCALE 1:68

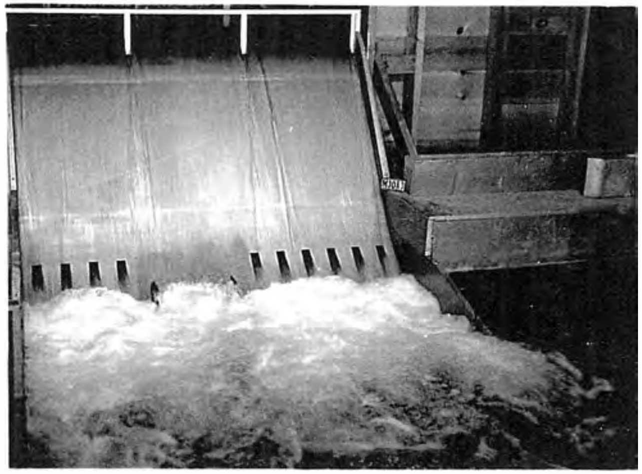
PIEZOMETRIC PRESSURES ON LEFT TRAINING WALL

R.A.G. 4-24-1937

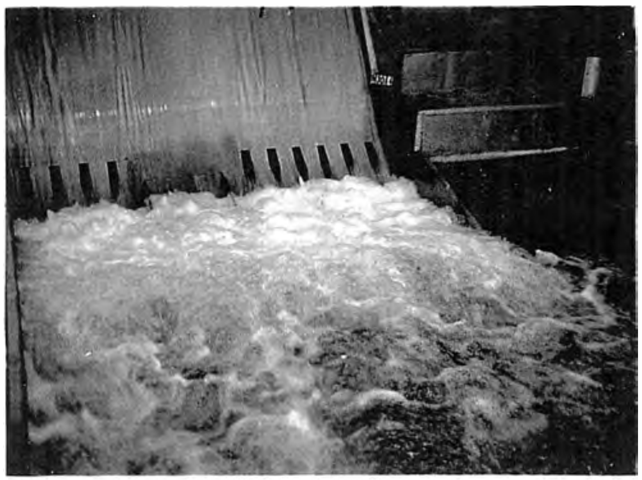
249-D-1091



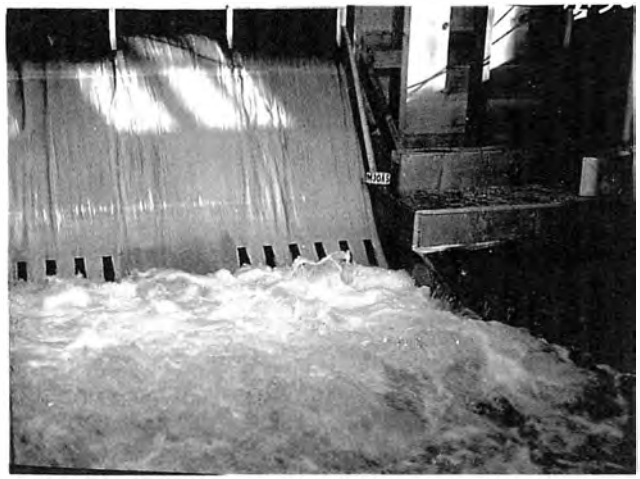
A. Discharge 125,000 Second-Feet



B. Discharge 300,000 Second-Feet



C. Discharge 400,000 Second-Feet

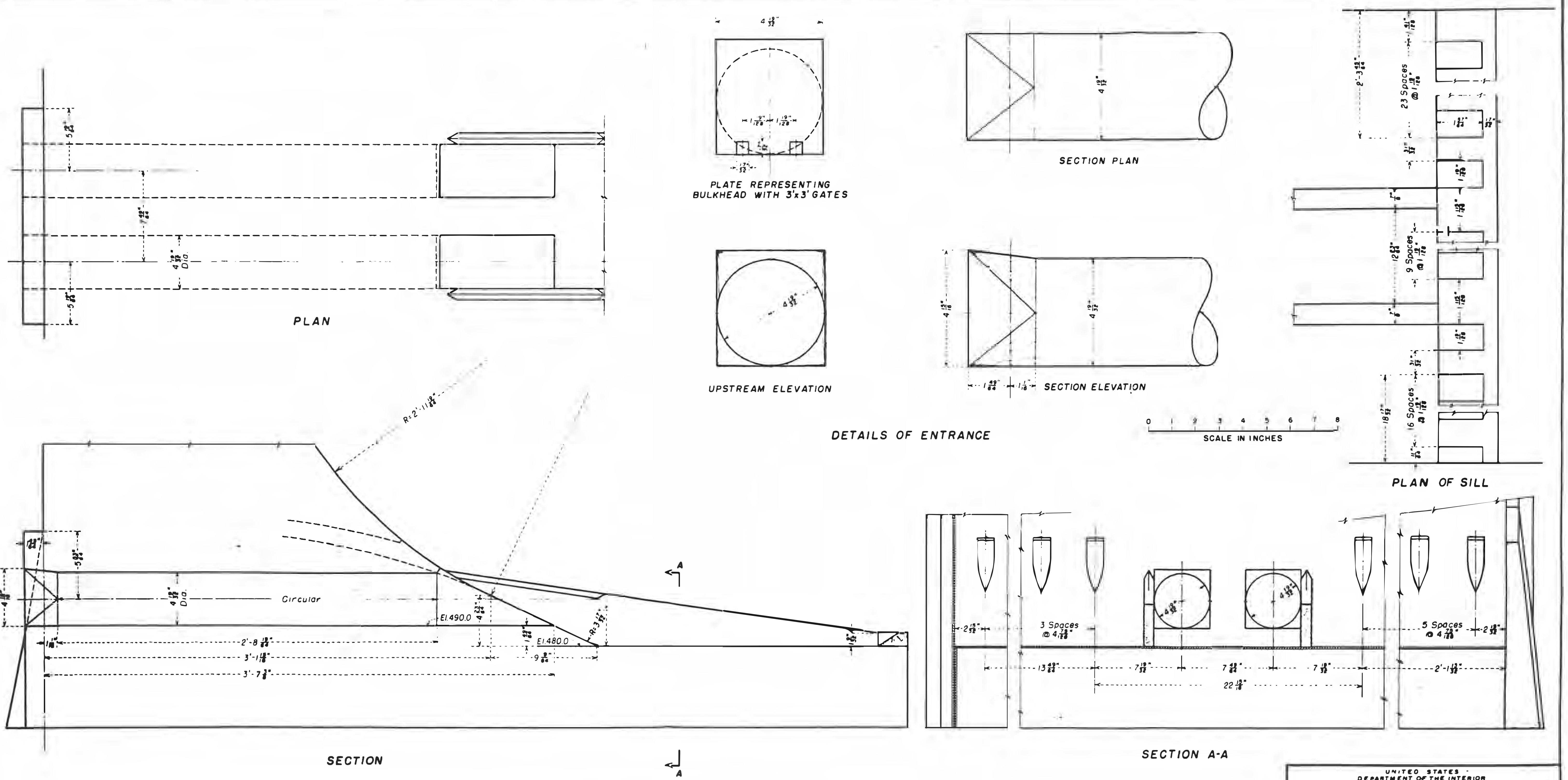


D. Discharge 500,000 Second-Feet

FLOW CONDITIONS - FINAL DESIGN TRAINING WALLS

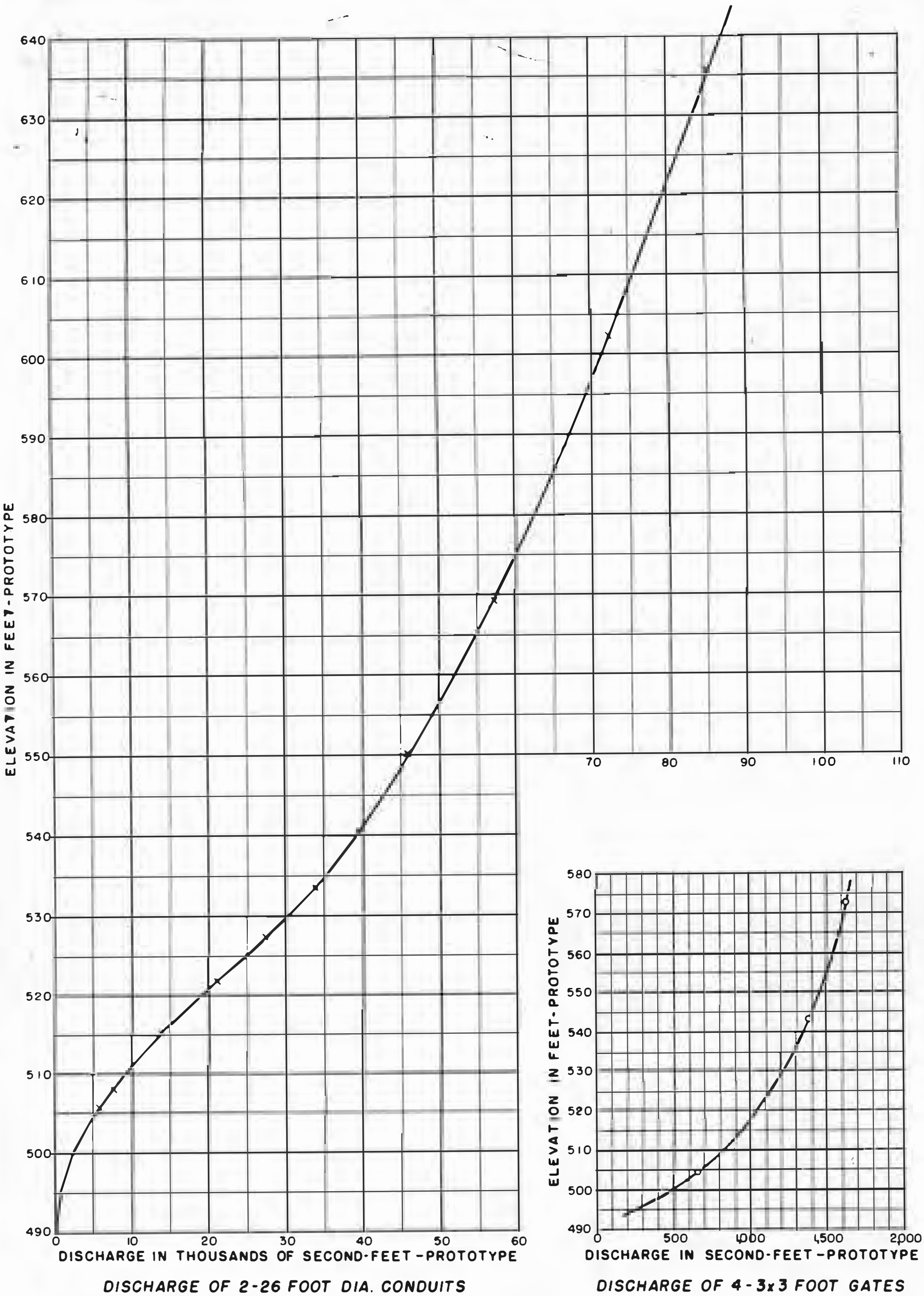
FIGURE 24

FIGURE 25



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:68
DETAILS OF DIVERSION CONDUITS

DRAWN R.A.B.	SUBMITTED J. E. Warnock
TRACED B.B.-C.I.L.	RECOMMENDED K. F. Blanks
CHECKED	APPROVED J. L. Savage
DENVER, COLORADO MARCH 24, 1937	249-D-1092



DISCHARGE OF 2-26 FOOT DIA. CONDUITS

DISCHARGE OF 4-3x3 FOOT GATES

MARSHALL FORD DAM
DIVERSION PERIOD
HYDRAULIC MODEL STUDIES - SCALE 1:68
RATING CURVES

R.A.G.-H.W.B. 10-5-37

249-D-1093

22. Diversion. Investigations of the flow conditions through the diversion conduits during the proposed period of construction of the second stage, according to the original plan, led to the operation of the diversion conduits at various reservoir elevations below the invert of the river outlets, with and without the intermediate training walls.

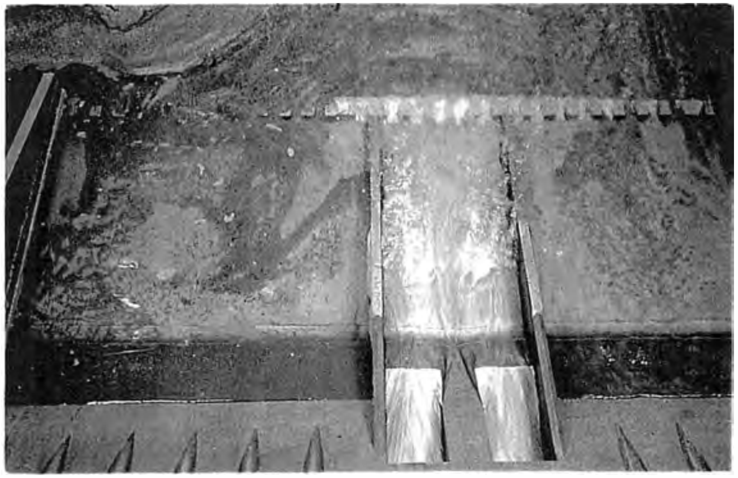
With the intermediate training walls in place, discharges in the region of 5,000 second-feet gave satisfactory stilling pool conditions. Very little sand was carried onto the apron, and practically no erosion occurred downstream from the sill. When the discharge was increased to 14,500 second-feet, the pool was satisfactory, with a small increase in the amount of sand carried onto the apron. A discharge of 36,200 second-feet, which was approximately the capacity of the diversion conduits with the reservoir at the invert of the river outlets, gave an increased amount of material deposited on the apron. The excessive scour below the sill on the right probably would not exist if the complete model were installed, as the whirl would extend farther to the right and would be reduced in intensity (figure 27).

The intermediate training walls were removed, and the test was repeated. Discharges of 5,000 and 14,500 second-feet, respectively, gave satisfactory scour conditions downstream from the sill, with more sand carried onto the apron than when the walls were in place. A discharge of 36,200 second-feet gave conditions similar to those obtained when the walls were in place, with no apparent increase in the amount of sand deposited on the apron, and with less scour downstream from the sill. The heavy scour downstream would have been less if the entire apron had been included on the model, since the adverse currents in the stilling pool would then have been reduced. These studies indicated that the intermediate training walls should be installed during construction of the low dam.

23. River outlet studies. Studies on the 1 to 40.8 model to determine some satisfactory program for bringing the river outlets into operation with a full reservoir and a minimum tailwater were unsuccessful. Initial tests on the 1 to 68 model with a stilling basin downstream from outlets 5 and 6 also gave unsatisfactory conditions when various programs were investigated. Observations on the models have shown that after three outlets are in operation the tailwater is sufficiently deep to prevent excessive erosion from any additional outlets that may be opened.

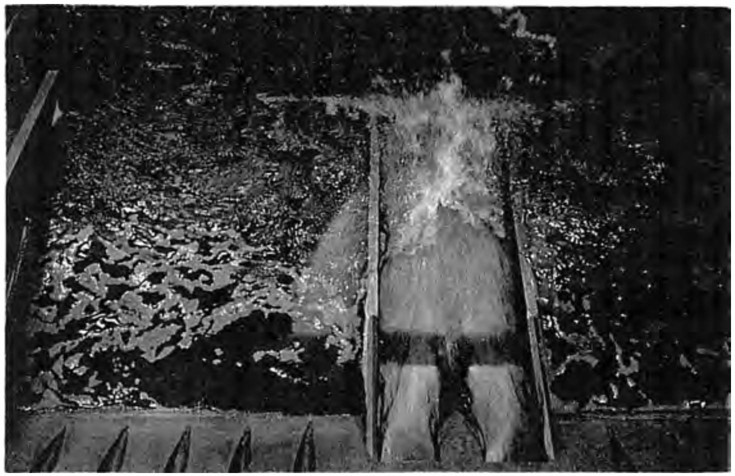
A transition at the exit of the outlet was developed which could spread the jet more evenly over the apron (figure 28). Original recommendations were that three alternate outlets, between the left intermediate training wall and the left spillway training wall, be constructed with this spreading exit and opened first when placing the river outlets in operation. By the time the desired spreading

DIVERSION CONDUITS - RECOMMENDED



A. Discharge 14,500 Second-Feet
Reservoir Elevation 516

B. Scour After 14,500 Second-Feet



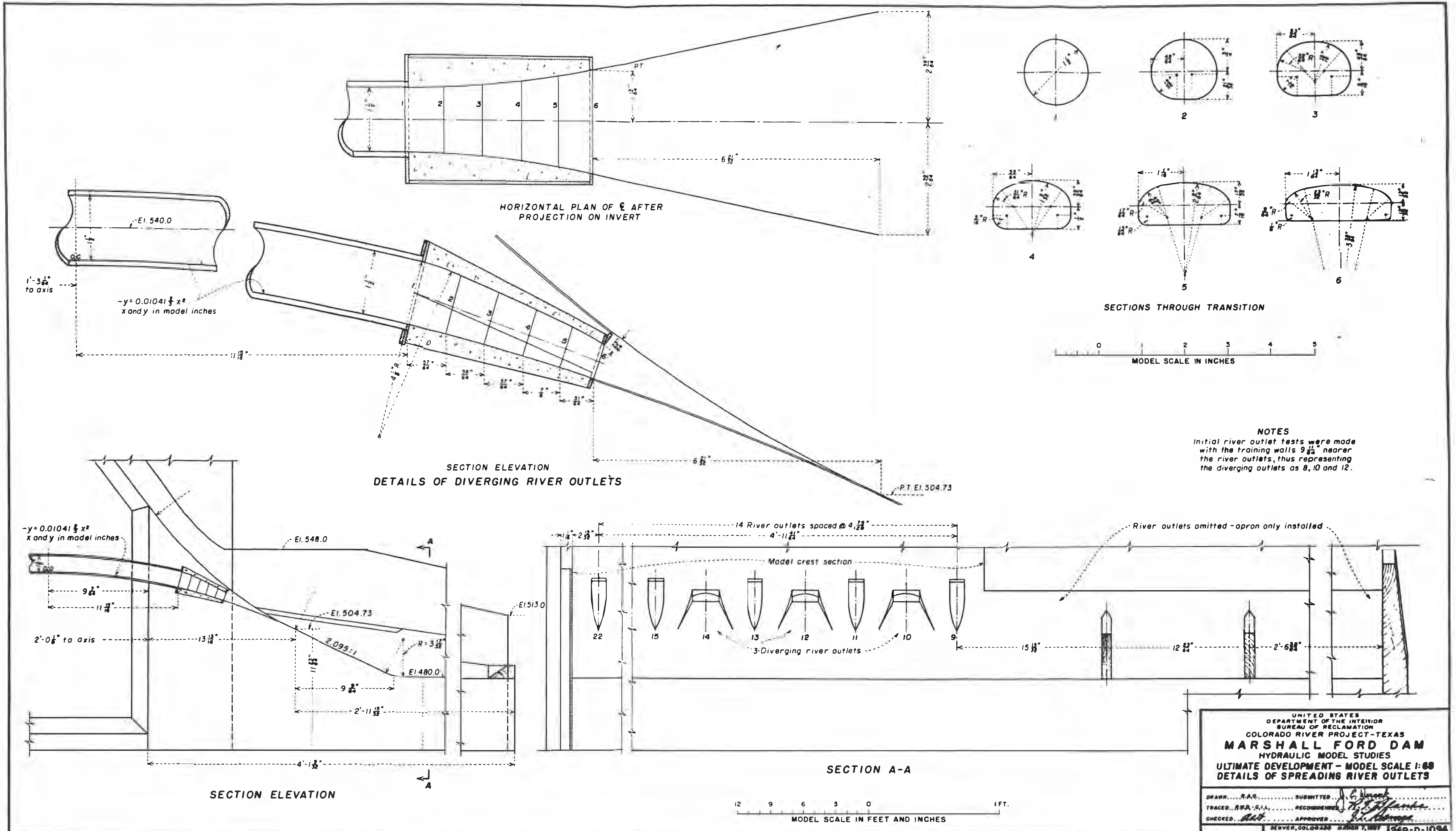
C. Discharge 36,200 Second-Feet
Reservoir Elevation 535

D. Scour After 36,200 Second-Feet

FLOW CHARACTERISTICS

FIGURE 27

FIGURE 28


HORIZONTAL PLAN OF ℄ AFTER PROJECTION ON INVERT
SECTIONS THROUGH TRANSITION
MODEL SCALE IN INCHES
El. 540.0
$-y = 0.01041\frac{5}{3}x^2$ x and y in model inches
SECTION ELEVATION
DETAILS OF DIVERGING RIVER OUTLETS
P.T. El. 504.73
NOTES
Initial river outlet tests were made with the training walls $9\frac{11}{64}$" nearer the river outlets, thus representing the diverging outlets as 8, 10 and 12.
El. 548.0
El. 504.73
2.095:1
El. 480.0
El. 513.0
2'-0$\frac{1}{8}$" to axis
SECTION ELEVATION
14 River outlets spaced @ 4.$\frac{73}{128}$"
Model crest section
River outlets omitted - apron only installed
3-Diverging river outlets
SECTION A-A
MODEL SCALE IN FEET AND INCHES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT-TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:60
DETAILS OF SPREADING RIVER OUTLETS
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO
249-D-1094

exit had been completely developed, construction on the left end of the spillway had progressed too far to permit its inclusion in that area. It was necessary, therefore, to place the spreading exits to the right of the intermediate training walls, nearer the center of the spillway.

The development of the spreading exit was studied on a 1 to 25.5 model of one river outlet and will be described in detail in subsequent sections.

24. Exit transitions on outlets 8, 10, and 12. The spreading-exit transitions were installed initially near the left end of the model spillway. When it was learned that the construction had progressed too far to permit such construction in the prototype, the exits were moved to outlets 8, 10, and 12 by extending the apron, fitted with the proper training walls, farther to the left (figure 28). While this arrangement did not allow detailed testing of all combinations, since outlets 1 to 6 and the extreme right end of the spillway were not represented in the model, it was considered the most practical for determining the flow action on the apron to the right of the spreading-exit outlets.

The problem resolved itself naturally into two parts - the sequence of opening the outlets with the spreading exits and the sequence of opening the other outlets. Throughout all the outlet tests the reservoir was maintained at approximately elevation 714.0, the elevation of the fixed crest of the spillway, while the tailwater was varied according to the quantity of water being discharged by the outlets.

With one spreading outlet in operation, best conditions were obtained with outlet 10, while least desirable conditions occurred when outlet 8 operated alone. In the two-outlet combination, best conditions were obtained with outlets 10 and 12 operating together. With outlet 8 discharging, the flow pattern was crowded toward the intermediate training wall, and this condition persisted with the three-outlet combination although some of the unsymmetrical flow was eliminated. Accordingly, the best initial operating program for this arrangement would be to open the spreading outlets in the order 10, 12, and 8. Results indicated that improved flow patterns would be obtained if the spreading exit on outlet 8 were placed on outlet 14. It would improve the spreading outlets to be opened from left to right.

With the three spreading-exit outlets in operation, even-numbered outlets to the right of outlet 12 were opened next, proceeding from left to right. The flow conditions were rough, and the return flow onto the apron from the right become more pronounced as additional outlets were opened. Better results were obtained when three of the outlets to the left of the intermediate training walls were opened following the opening of the spreading outlets and before any of the regular outlets on the right were opened. When the latter method was represented by raising the tailwater, the action of the spreading outlets was improved and the undesirable eddies on the right were reduced.

25. Exit transitions on outlets 10, 12, and 14. The preceding studies illustrated the desirability of placing the spreading exits on outlets 10, 12, and 14. To represent this condition on the model, the intermediate and the left training walls were moved 9-11/64 inches toward the left. Outlets 1 to 8 and 23 and 24 were not represented because of the restricted length of the model.

The results of the previous test suggested the desirability of opening outlets 10, 12, and 14 in that order, which proved correct. As in the preceding test, observations indicated that best results would be obtained if three outlets to the left of the intermediate training walls were to be opened following the opening of the three spreading outlets and before any further outlets were opened to the right. This produces a deeper tailwater which improves the action below the spreading outlets and reduces undesirable eddies. The remaining even-numbered outlets should then be opened, beginning with outlet 8 and proceeding toward the right. The odd-numbered outlets can then be operated in any order. Figure 29 shows the general flow characteristics with this operating program.

26. Pressures in outlet conduits. A circular and a spreading-exit river outlet were equipped with air vents and all outlets were closed. The air vents were connected to manometers so that the pressures within the outlets could be measured. With the spillway only operating, negative pressures were present in both outlets. These negative pressures varied from 6.5 to 10.2 feet of water, prototype, for discharges between 200,000 to 500,000 second feet. These objectionable pressures will be relieved by having the air vents open when the outlet control gates are closed.

MODEL OF RIGHT HALF OF SPILLWAY FOR ULTIMATE DEVELOPMENT

27. Description of the model. The 1 to 68 model was modified to represent the right half of the spillway (figure 30). The piers were rearranged and the high wall, formerly on the right end of the spillway, was moved to the left end. The topography was placed as shown on figure 17. Tests on this model were for the purpose of establishing satisfactory flow conditions in the spillway around the right wall. Since these studies were made before the installation of the diversion conduits in the model, there were 14 outlets in the model.

FIGURE 29

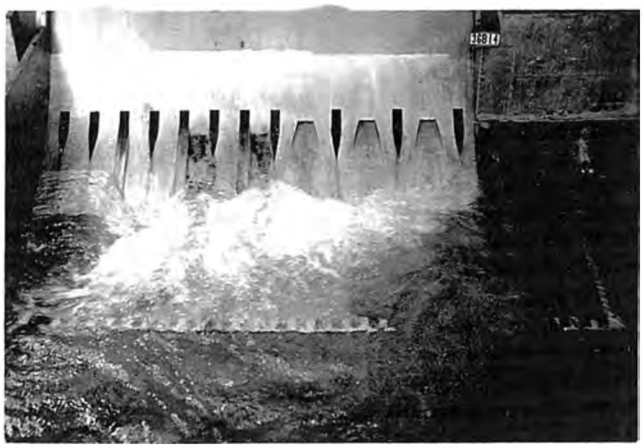
Outlet 10 Operating

Outlets 10 & 12 Operating

Outlets 10, 12 & 14 Operating

Outlets 10, 12, 14 & 16 Operating

Outlets 10, 12, 14, 16 & 18 Operating



Outlets 10, 12, 14, 16, 18 & 20 Operating

Reservoir Elevation 714 - Normal Tailwater Elevations

RIVER OUTLET COMBINATIONS

28. Studies of right training wall. The original design of the right training wall was tested first. This wall ended 72.5 feet upstream from the end of the spillway apron and was at elevation 537.0 throughout, with an additional wing wall extending at right angles from the end of the main training wall to the side of the model tank. The original information indicated that the river bank was rock. Loose rock was used in the model to represent a strip of topography 102 feet wide, downstream from the apron sill. A discharge of 125,000 second-feet gave satisfactory stilling pool conditions. With discharges of 300,000 and 500,000 second-feet, a large flow onto the apron from the right drowned the hydraulic jump near the training wall. The loose rock was scoured excessively at moderate discharges, indicating that the rock was not represented correctly on the model. The strip of topography downstream from the sill was replaced on the model by concrete, and the tests were repeated. At the higher discharges, the return flow onto the apron was very pronounced. The 1 to 1 slope on the hill downstream seemed to turn the water upward too suddenly, resulting in a shortening effect on the stilling pool in that area.

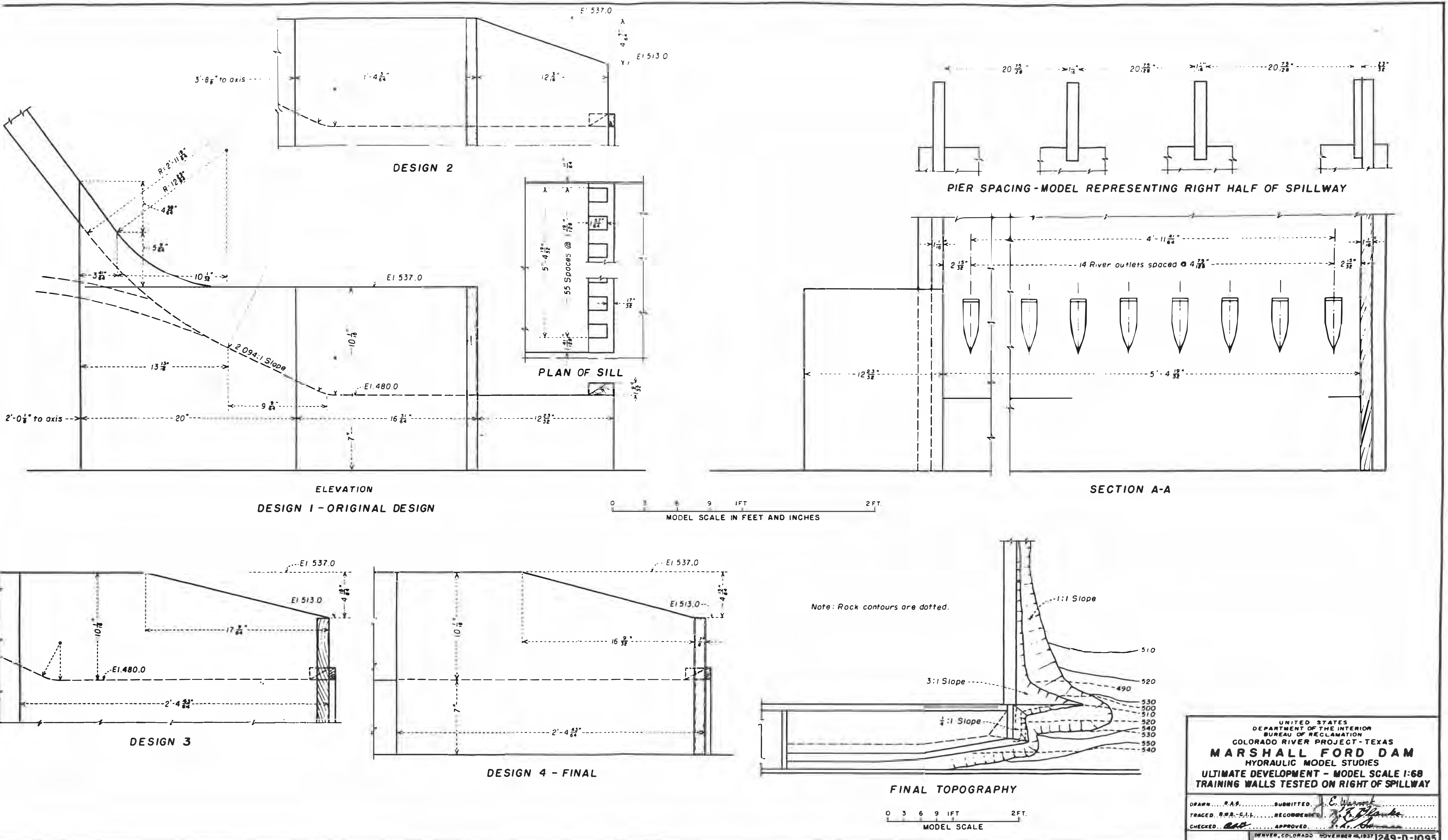
To minimize the excessive flow onto the apron from the right, a trapezoidal shape was added to the training wall (figure 30). This second design sloped from elevation 537.0 at the end of the original wall to elevation 513.0 at the end of the apron. The objectionable return flow was reduced materially. The shortening of the pool on the right was still present, indicating that the slope of the topography should be flattened.

Design 3 was obtained by moving the cutoff wall to the downstream end of the apron and placing the wall on a flatter slope with the downstream end at elevation 513.0 (figure 30). The cut on the rock topography downstream from the sill was flattened to a slope of 3 to 1. With discharges of 300,000 and 500,000 second-feet, conditions in the right portion of the stilling pool were materially improved. The jump formed with no undesirable return flow over the training wall onto it.

29. Final design of right training wall. Complete information on the rock conditions along the right bank were lacking during the initial studies on the training wall. It was known that a large portion of the bank was solid rock; but the extent of the overburden was not known. When detail information on the position of the rock surface became available, it was represented in the model in concrete. The cut in the rock, downstream from the apron sill, was placed on a 3 to 1 slope, and around the toe of the wall on a 1/4 to 1 slope (figure 30). The earth overburden was placed on a 1 to 1 slope. A large portion of the earth overburden would be moved downstream during medium and high discharges. The training wall for the final design was altered little from the design 3 (figure 30).

Lowering the rock topography to its correct position materially

FIGURE 30


DESIGN 2
El 537.0
El 513.0
3'-8 1/8" to axis
PLAN OF SILL
55 Spaces @
PIER SPACING - MODEL REPRESENTING RIGHT HALF OF SPILLWAY
14 River outlets spaced @ 4 75/128"
SECTION A-A
R 2'-11 19/64"
R 12 63/64"
2.094:1 Slope
El 537.0
El. 480.0
2'-0 1/8" to axis
ELEVATION
DESIGN 1 - ORIGINAL DESIGN
0 3 6 9 1FT 2FT.
MODEL SCALE IN FEET AND INCHES
El 537.0
El 513.0
El.480.0
DESIGN 3
El 537.0
El 513.0
DESIGN 4 - FINAL
Note: Rock contours are dotted.
1:1 Slope
3:1 Slope
1/4:1 Slope
FINAL TOPOGRAPHY
0 3 6 9 1FT 2FT.
MODEL SCALE
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
ULTIMATE DEVELOPMENT - MODEL SCALE 1:68
TRAINING WALLS TESTED ON RIGHT OF SPILLWAY
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
DENVER, COLORADO
249-D-1095

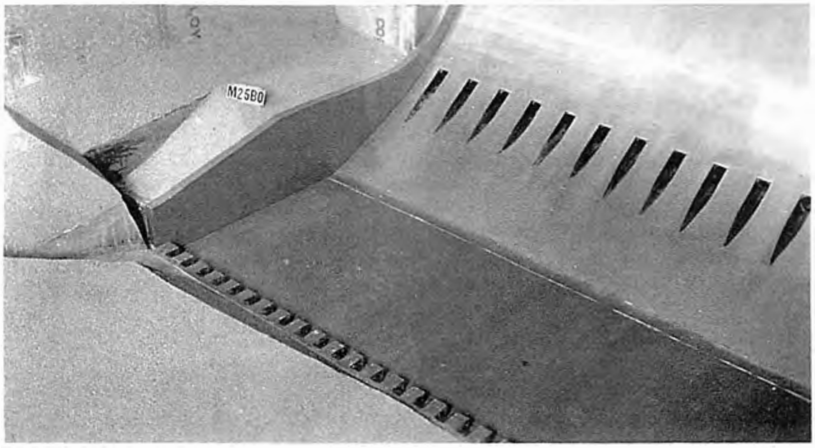
improved the stilling pool conditions at high discharges. The upstream flow over the wall from the right was not present at discharges of 125,000 and 300,000 second-feet. At 500,000 second-feet this flow was small and had little effect on the jump (figure 31). With a discharge of 300,000 second-feet, erosion was noticeable on the downstream portion of the backfill on the right of the training wall. When the discharge increased to 500,000 second-feet, this erosion increased in intensity; so a large portion of the backfill was removed near the wall. This condition can be prevented by riprapping the downstream slope of the backfill.

MODEL OF RIVER OUTLET

30. Description of the model. The problem of the spreading of the jet from certain river outlets led to a more detailed study of a single outlet to develop pressure data and the most effective shape both hydraulically and structurally. The model was constructed to a scale of 1 to 25.5, which permitted the use of standard 4-inch brass tubing in the construction of the model. The upstream face of the dam was represented by one side of a rectangular, metal-lined tank. The outlet, fitted with a bell entrance, extended out from the tank and downward on a parabolic curve. A section of the apron was installed downstream with a grid painted on it so that the spread of the jet could be measured (figure 32). Piezometers were installed in the outlet to measure the pressures within it, and in the sides, the crown, and the invert of the exit so that pressure distributions in that region could be investigated.

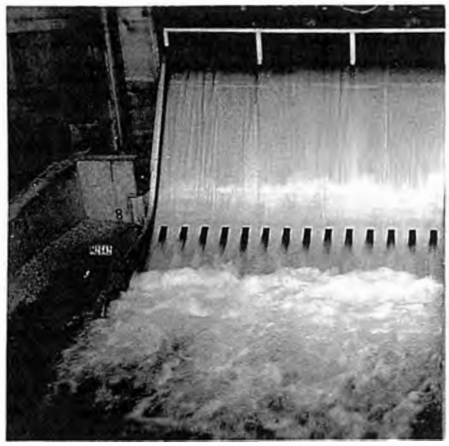
31. Outlet exit studies. The exit design used on 21 of the 24 river outlets was installed first. The invert of this design was circular and tangent to the apron. With reservoir water surfaces near the initial and the ultimate spillway-crest elevations, the pressures were measured on the sides and at the invert of the exit and within the outlet (figures 33 and 34). While some negative pressures occurred within the outlet, they were not excessive to the extent of causing cavitation. Pressures and water surface profiles were also taken on the sloping apron at right angles to the center line of the jet (figure 34).

The exit was then modified to cause the jet to spread on the apron, to induce the formation of a hydraulic jump more quickly than it could be formed with the jet from the circular exit. Design 2 was a transition from a semicircular invert at the end of the tube to a horizontal line at the point of tangency. The sides of the exit were vertical (figure 32). Design 3 was essentially the same except that the sides were given a warp, changing from vertical at the exit to horizontal at the point of tangency. Observations were also made on

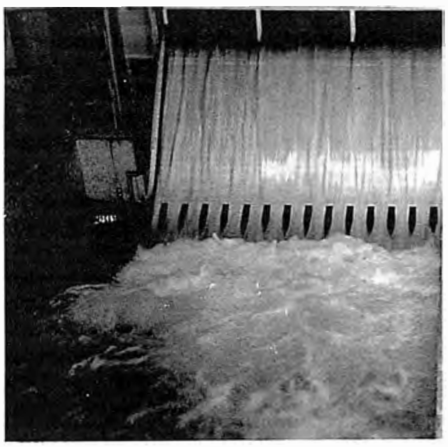


No Flow

Discharge 125,000 Second-Feet



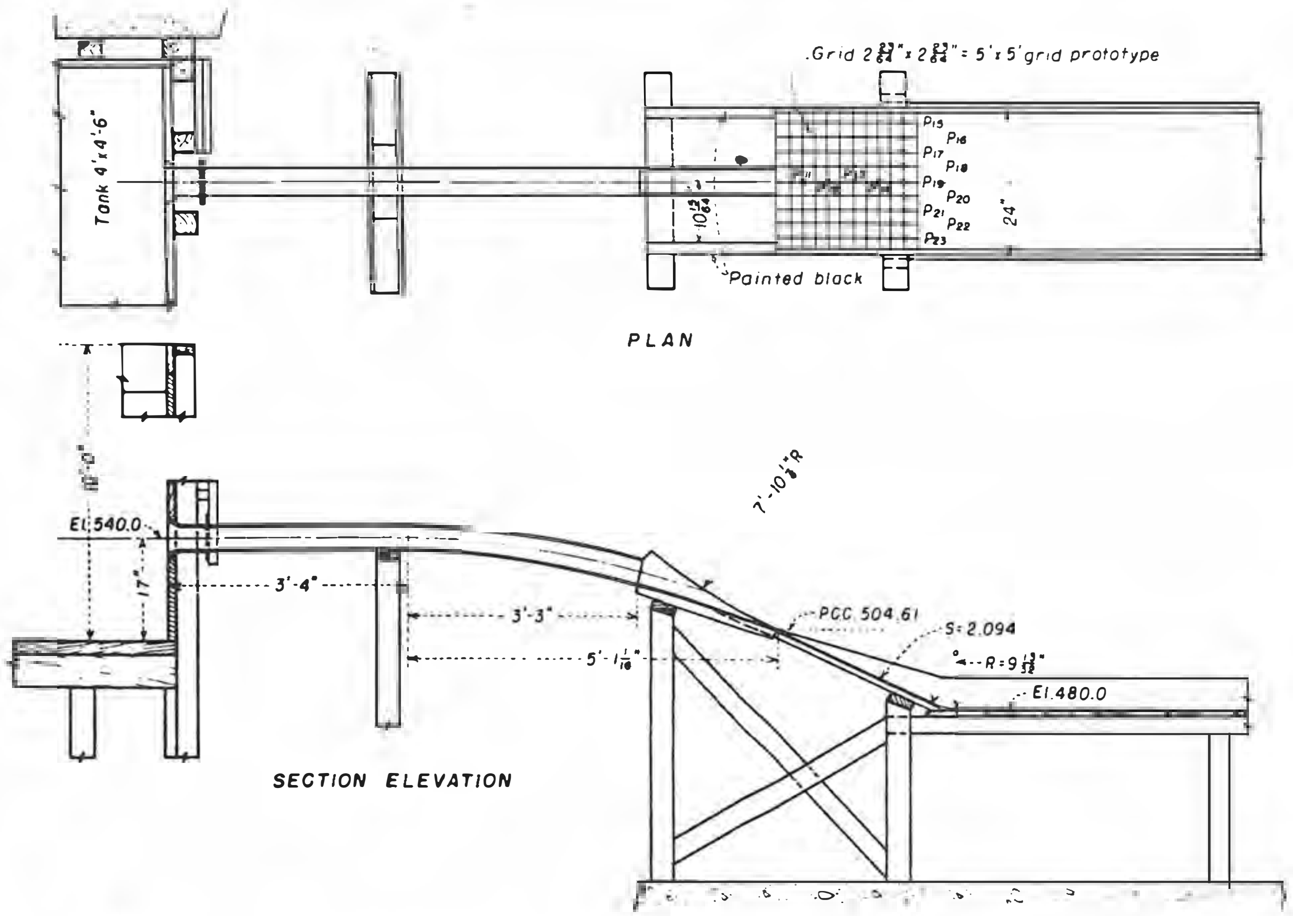
Discharge 300,000 Second-Feet



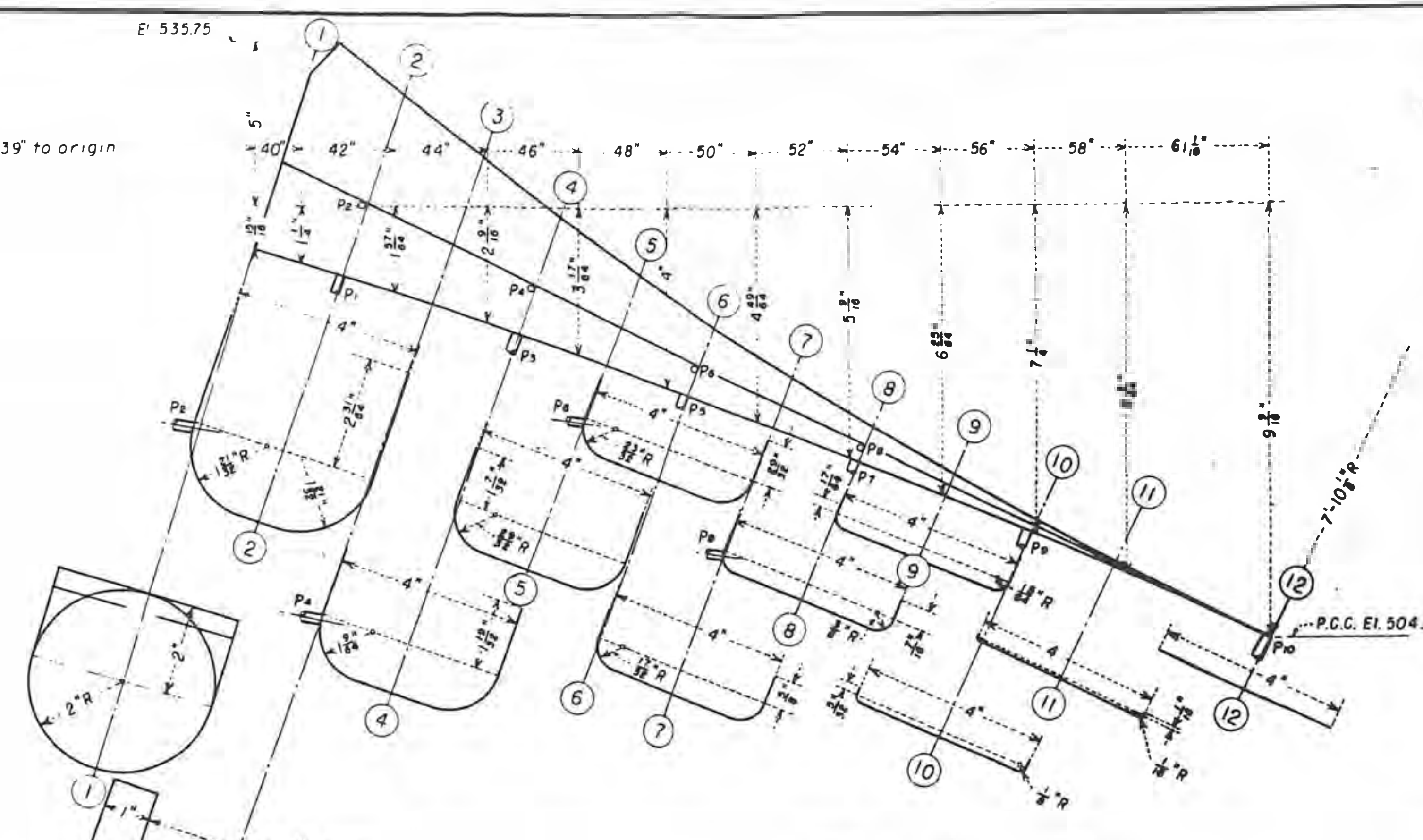
Discharge 500,000 Second-Feet

Scour After 500,000 Second-Feet

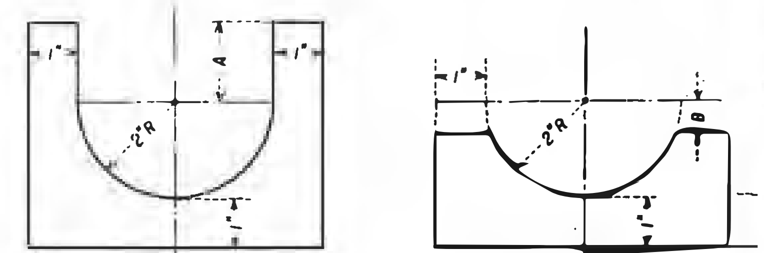
RIGHT TRAINING WALL - FINAL DESIGN



GENERAL MODEL



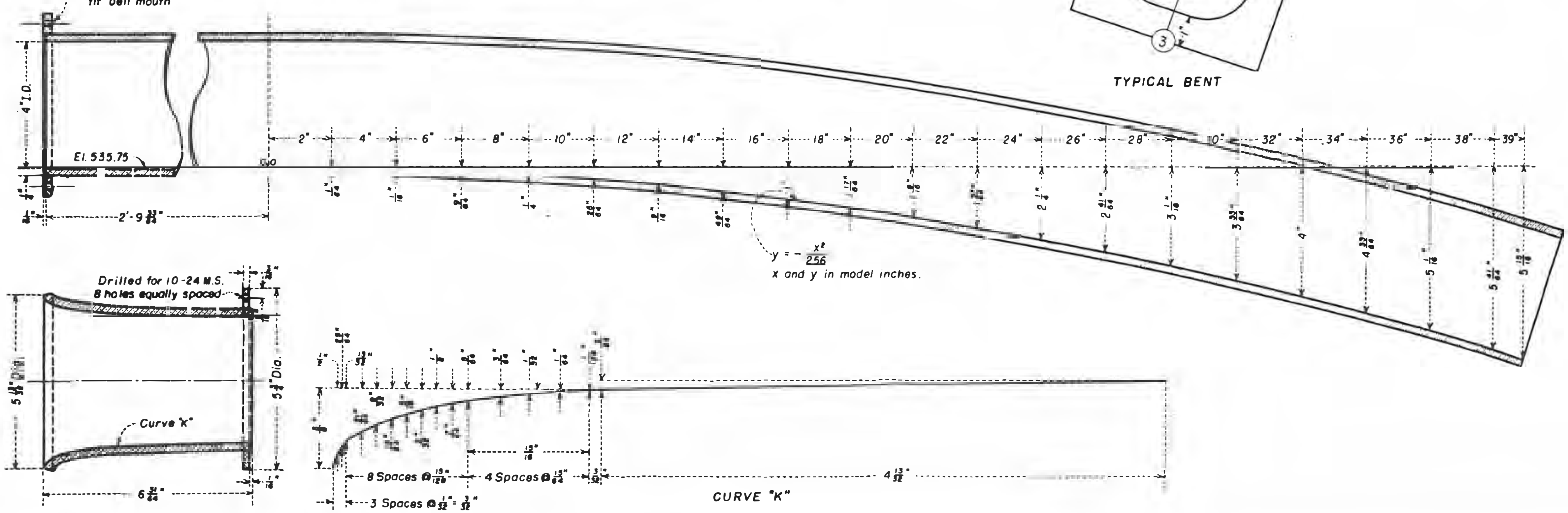
TYPICAL BENT



BENT NO.	A
1	2"
2	2 9/64"
3	1 5/8"
4	41/64"
5	1/4"

BENT NO.	B
6	3/8"
7	27/32"
8	1 3/8"
9	1 1/2"
10	1 53/64"
11	1 5/8"
12	2"

DESIGN 1 - EXIT SECTIONS

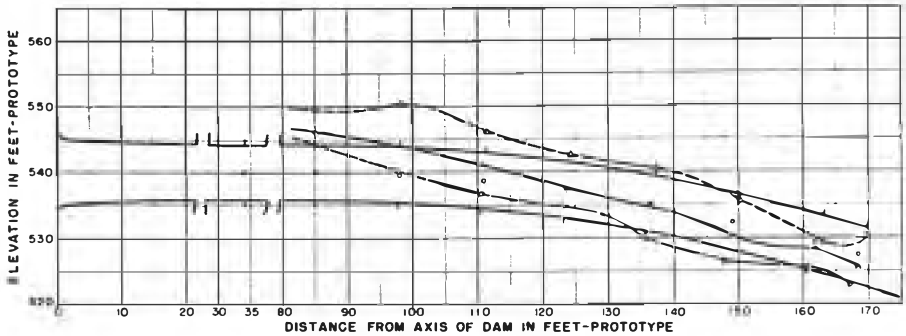


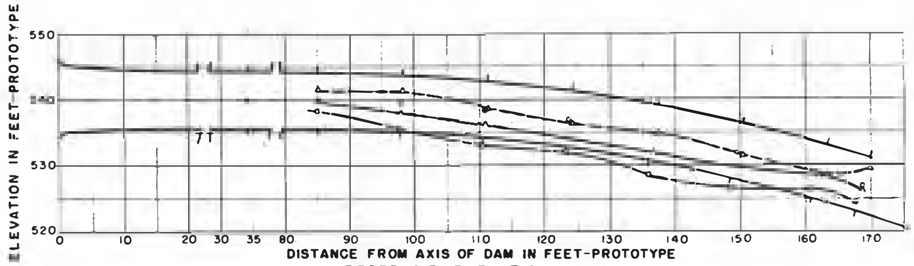
RIVER OUTLET DETAILS

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS

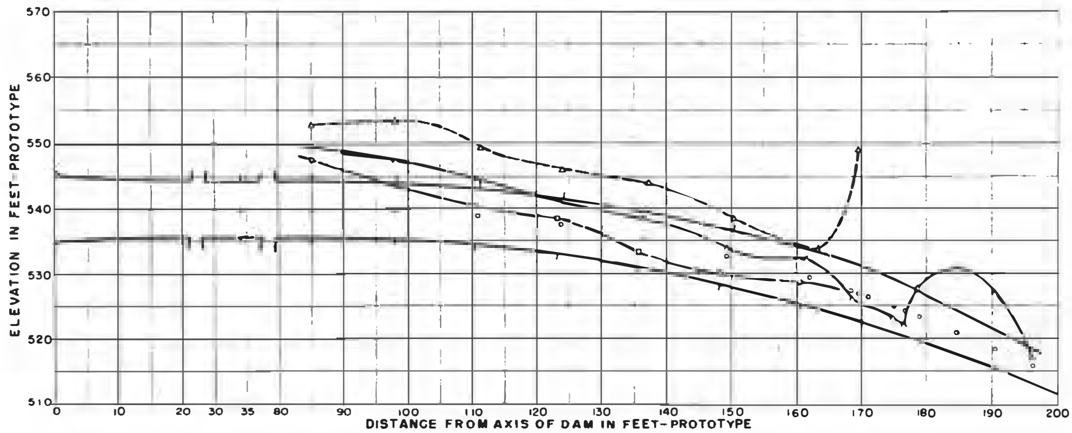
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
MODEL SCALE 1:25.5
MODEL LAYOUT AND DETAILS

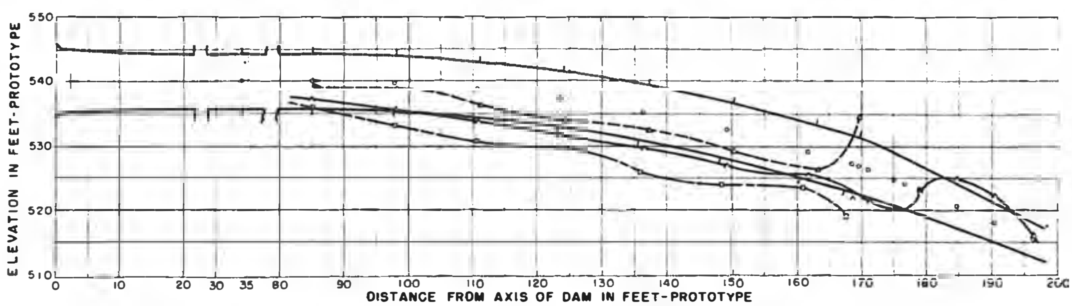
DRAWN... J.W.B. ... SUBMITTED ...
TRACED... ... RECO. ...
CHECKED... ... APPROVED ...
DENVER, COLORADO ... 249-D-1066





PRESSURES IN CIRCULAR PARABOLIC RIVER OUTLET - DESIGN 1





PRESSURES IN RIVER OUTLET WITH EXIT - DESIGN 5

NOTES

Pressure elevations are in feet of water prototype.
No air was admitted into sluice.
The pressures for Design 2 and 3 will be those for Design 1.

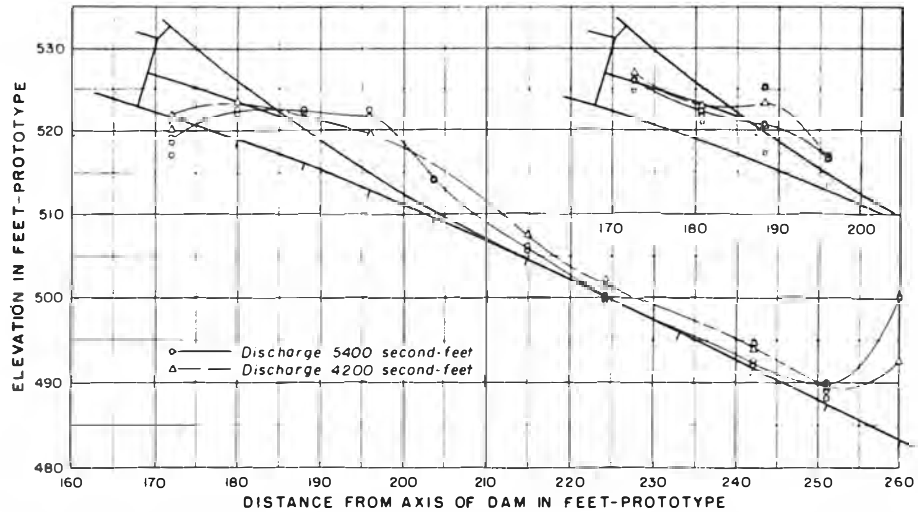
MARSHALL FORD DAM
RIVER OUTLETS
HYDRAULIC MODEL STUDIES - SCALE 1:25.5
PRESSURES IN OUTLETS
DESIGNS 1 AND 5

EXPLANATION

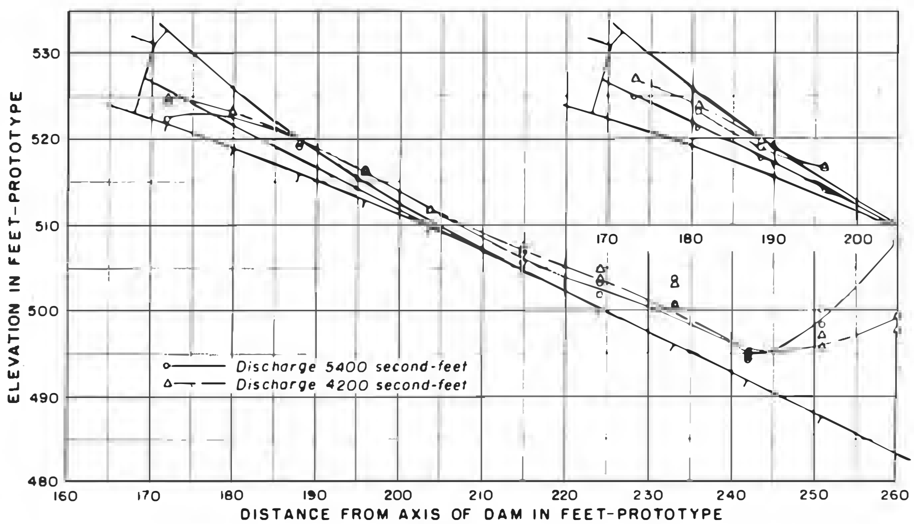
Δ- - - - Pressure elevations along crown.
□——— Pressure elevations along invert.
∧——— Pressure elevations along sides.
o Indicates side piezometers

R.A.G. 12-27-37

249-D-1097



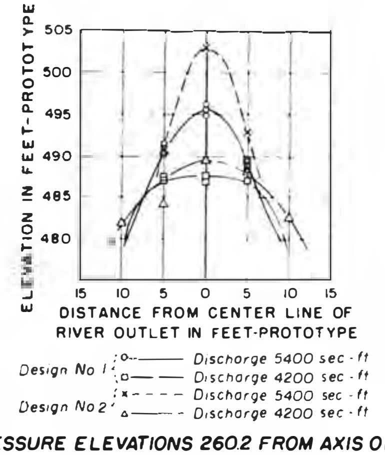
DESIGN No. 1



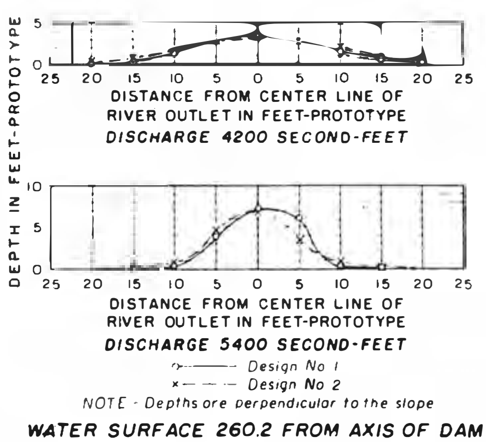
DESIGN No. 2

PRESSURE ELEVATIONS ALONG INVERT OF RIVER OUTLET

NOTE
All pressures are in feet of water prototype



PRESSURE ELEVATIONS 260.2 FROM AXIS OF DAM



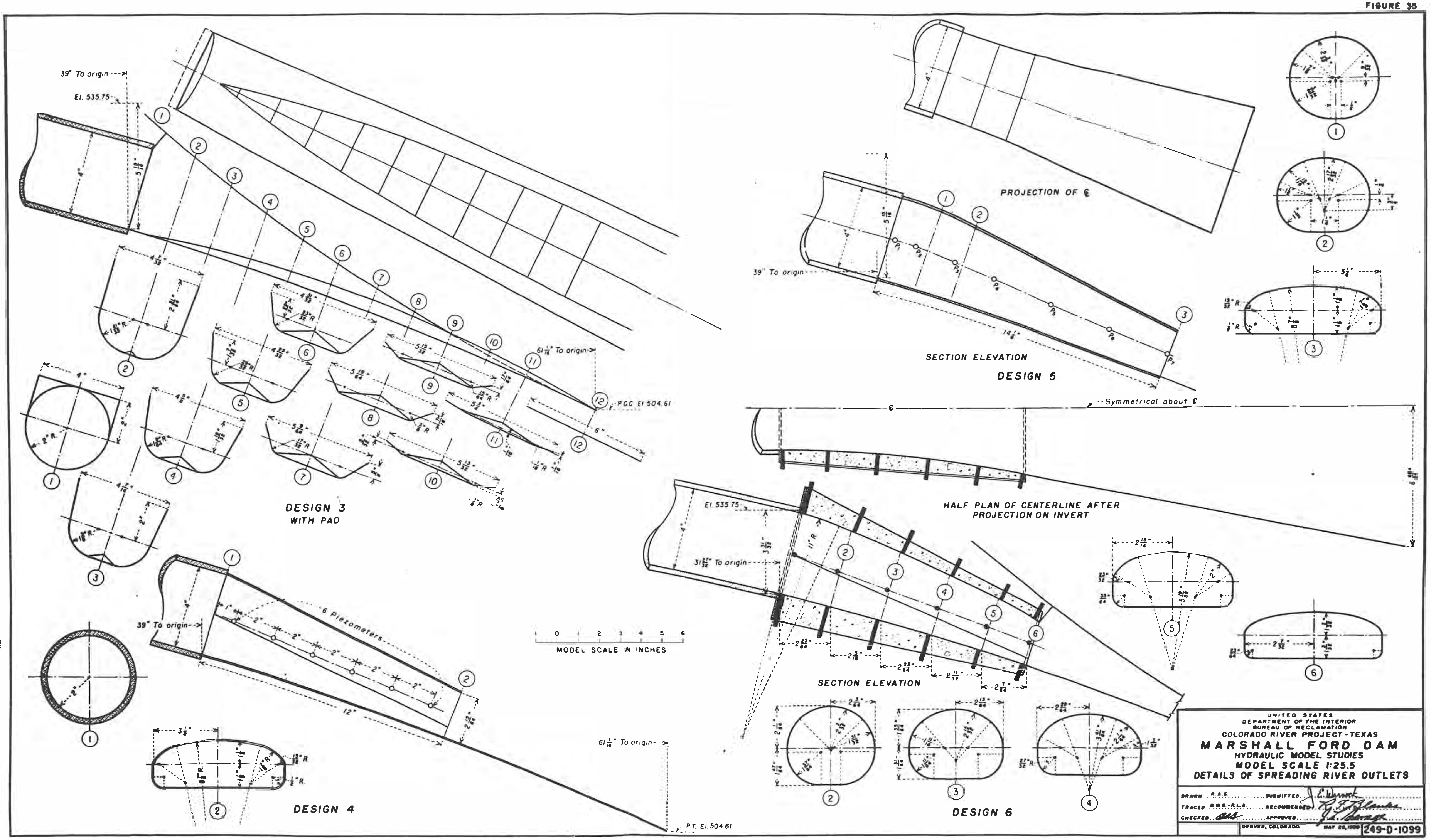
WATER SURFACE 260.2 FROM AXIS OF DAM

MARSHALL FORD DAM

RIVER OUTLETS

HYDRAULIC MODEL STUDIES - SCALE 1:25.5

PRESSURE AND WATER SURFACE PROFILES FOR DESIGNS 1 AND 2


DESIGN 3
WITH PAD
DESIGN 4
6 Piezometers
PROJECTION OF ℄
SECTION ELEVATION
DESIGN 5
Symmetrical about ℄
HALF PLAN OF CENTERLINE AFTER
PROJECTION ON INVERT
SECTION ELEVATION
DESIGN 6
MODEL SCALE IN INCHES
39° To origin
El. 535.75
PCC El 504.61
PT El 504.61
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT-TEXAS
MARSHALL FORD DAM
HYDRAULIC MODEL STUDIES
MODEL SCALE 1:25.5
DETAILS OF SPREADING RIVER OUTLETS
DENVER, COLORADO.
249-D-1099

design 3 with a pad in the invert downstream from the circular outlet (figure 35). Exit pressures and water surface profiles on the apron were measured for design 2 (figure 34). All of these designs were ineffective in spreading the high-velocity jet over the apron; but they did demonstrate that effective spreading could be accomplished only by constructing a transition exit within the dam.

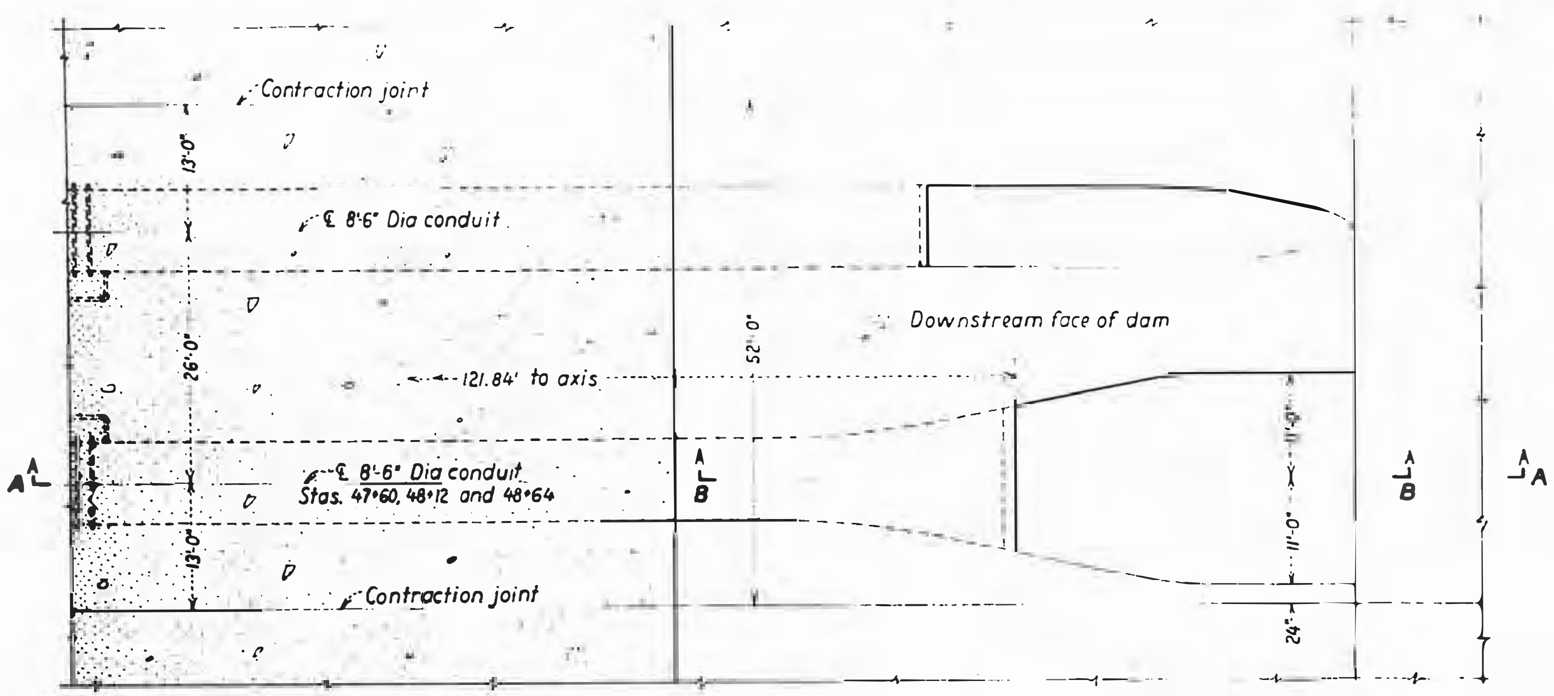
Design 4 was a preliminary design of transition attached to the end of the circular outlet to determine the effectiveness of such a structure (figure 35). The results were gratifying, with no adverse pressures indicated.

Design 5 was similar to design 4 but constructed more carefully (figure 35). It was added to the end of the original outlet; hence the pressure and the discharge characteristics were slightly in error. The spread of the jet was greater than necessary to give a good flow distribution on the apron. As spread on the apron would be increased with the transition in its correct position farther upstream, the width of the transition could be reduced. The pressures through the outlet with this exit are shown on figure 33.

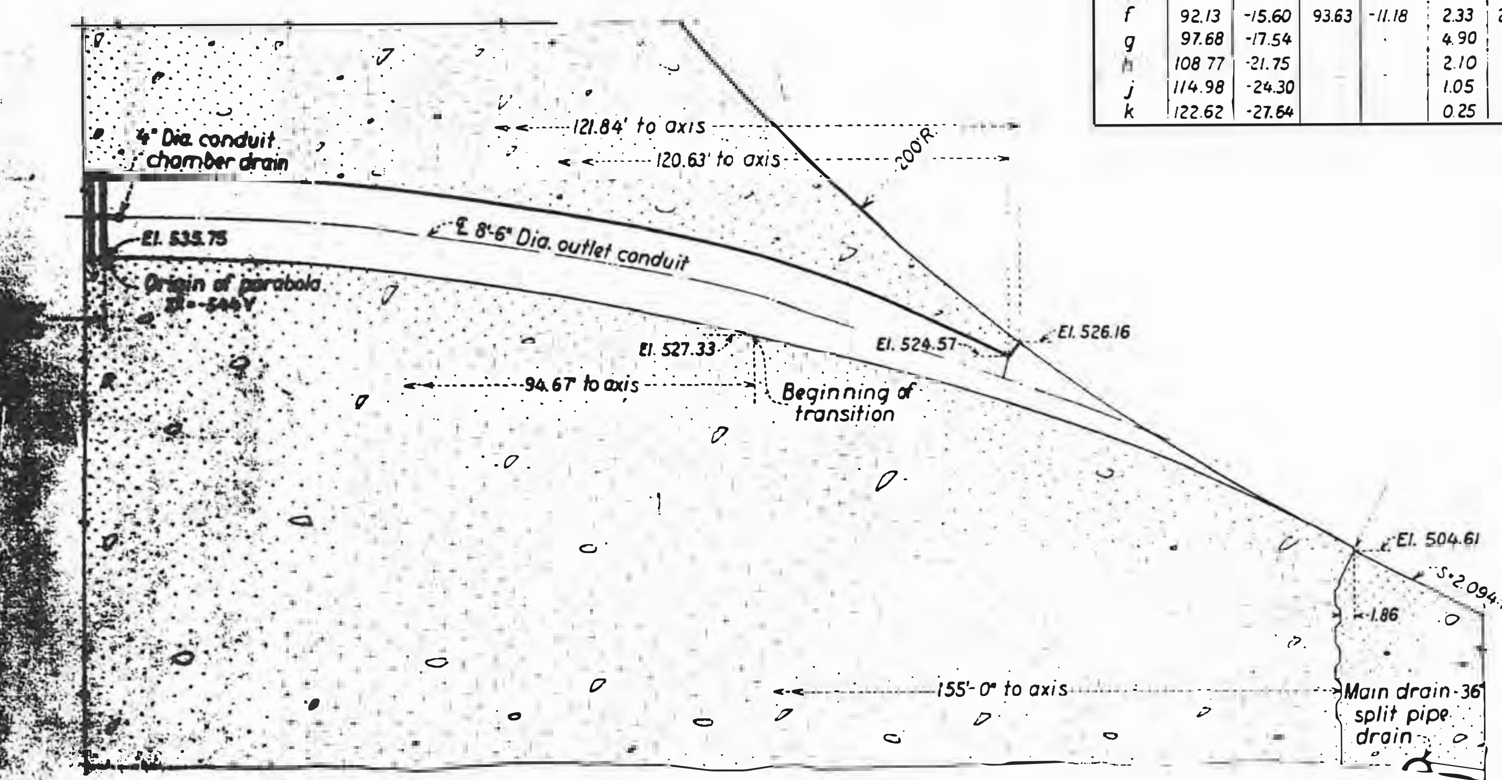
The results of the preceding tests were used in designing a constant-area spreading exit (design 6, figure 35). A portion of the circular pipe was removed and the transition installed in its proper relation to the face of the dam. Discharge characteristics of this transition were good, and the design was adopted as final. The details of the prototype conduit are shown on figure 36.

The proposed air vent was installed in the outlet to study its effectiveness. Pressure measurements were made with discharges of 2,500, 3,000, and 4,000 second-feet, respectively, with the air vent both open and closed, while corresponding data was taken at higher discharges with the vent closed (figure 37). The outlet was calibrated at the same time (figure 38). The lowest negative pressure was -12 feet of water with the air vent closed and the reservoir at elevation 567.8, and a discharge of 2,500 second-feet. The minimum reservoir water surface at which the outlet would flow full continually with the air vent closed is shown on the discharge curves as 2,000 second-feet with reservoir elevation 553.0. At this flow the minimum pressure would be slightly lower than for 2,500 second-feet. Average pressures at low flows, with the vent open, were higher than with it closed. The pressures became zero or positive throughout the outlet when the discharge reached 4,000 second-feet. High pressures were present on the top of the transition for the larger discharges, the maximum - 40 feet of water - being near the upstream end when the water surface in the reservoir was elevation 714.0.

The operating characteristics of the outlet were investigated



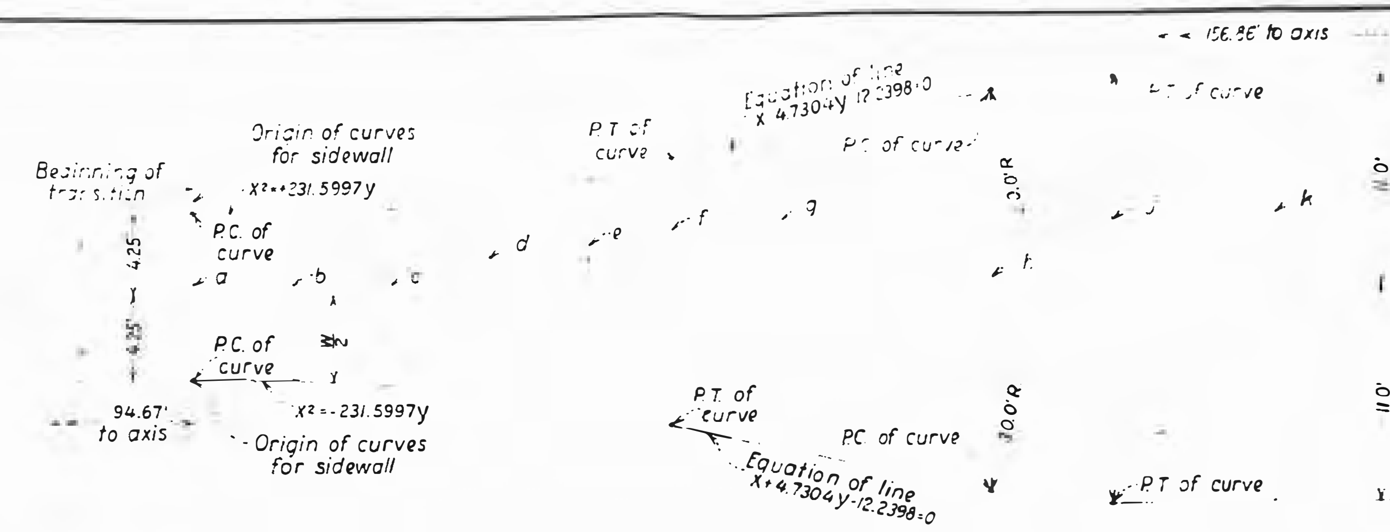
PLAN



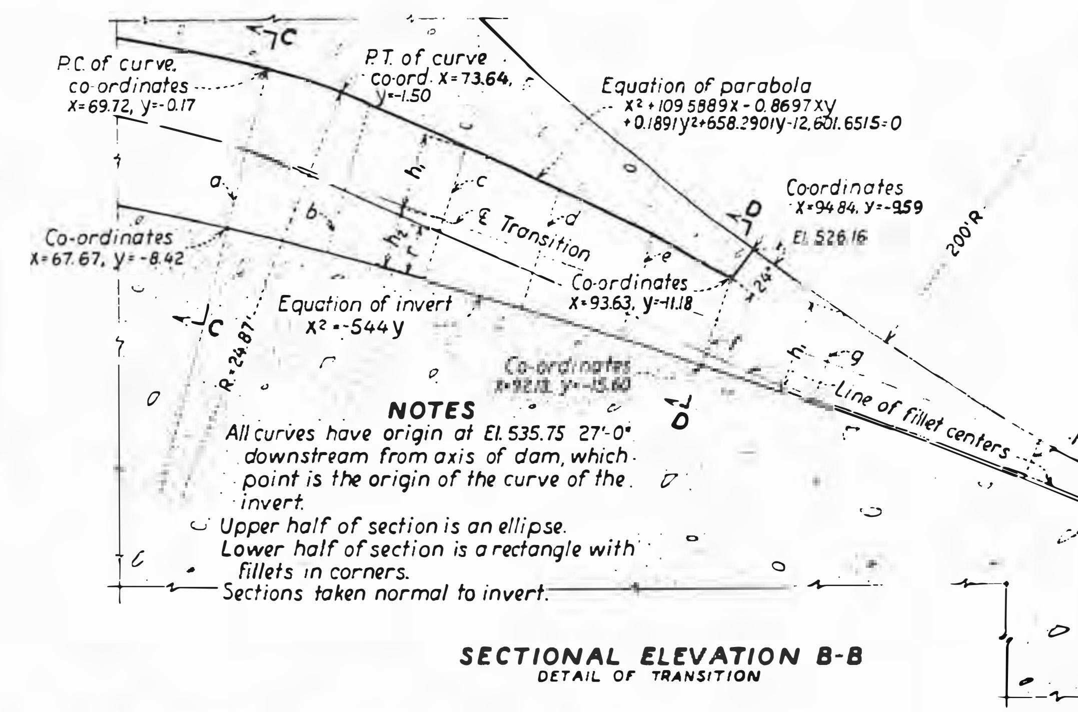
SECTIONAL ELEVATION A-A

TABLE OF TRANSITION DIMENSIONS

SECTION	CO-ORD. INVERT x	CO-ORD. INVERT y	CO-ORD. ROOF x	CO-ORD. ROOF y	h_1	h_2	r	$\frac{W}{2}$
a	67.67'	-8.42'	69.72'	-0.17'	4.25'	4.25'	4.25'	4.25'
b	72.67	-9.71	74.74	-1.98	4.31	3.68	3.36	4.36
c	77.67	-11.09	79.64	-4.20	3.96	3.21	2.56	4.68
d	82.67	-12.56	84.51	-6.51	3.50	2.83	1.85	5.22
e	87.67	-14.13	89.35	-8.94	2.93	2.53	1.23	5.98
f	92.13	-15.60	93.63	-11.18	2.33	2.33	0.75	6.83
g	97.68	-17.54			4.90		0.25	8.01
h	108.77	-21.75			2.10		0.25	10.35
j	114.98	-24.30			1.05		0.25	11.0
k	122.62	-27.64			0.25		0.25	11.0



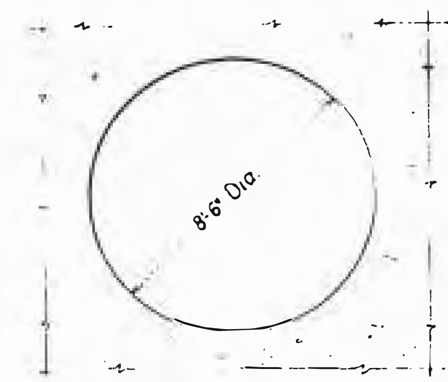
PROJECTION OF INVERT ON HORIZONTAL PLANE



NOTES

All curves have origin at El. 535.75 27'-0" downstream from axis of dam, which point is the origin of the curve of the invert.
Upper half of section is an ellipse.
Lower half of section is a rectangle with fillets in corners.
Sections taken normal to invert.

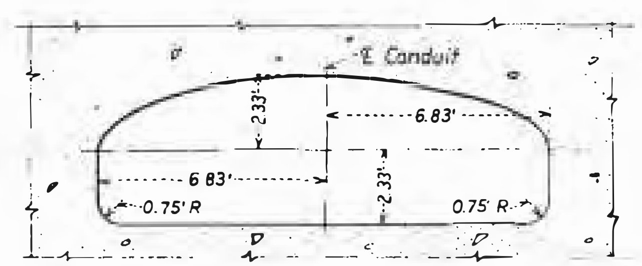
SECTIONAL ELEVATION B-B
DETAIL OF TRANSITION



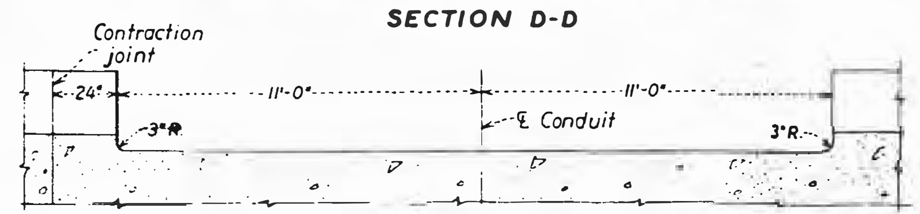
SECTION C-C

NOTES

Chamfer all exposed corners 1" unless otherwise specified.



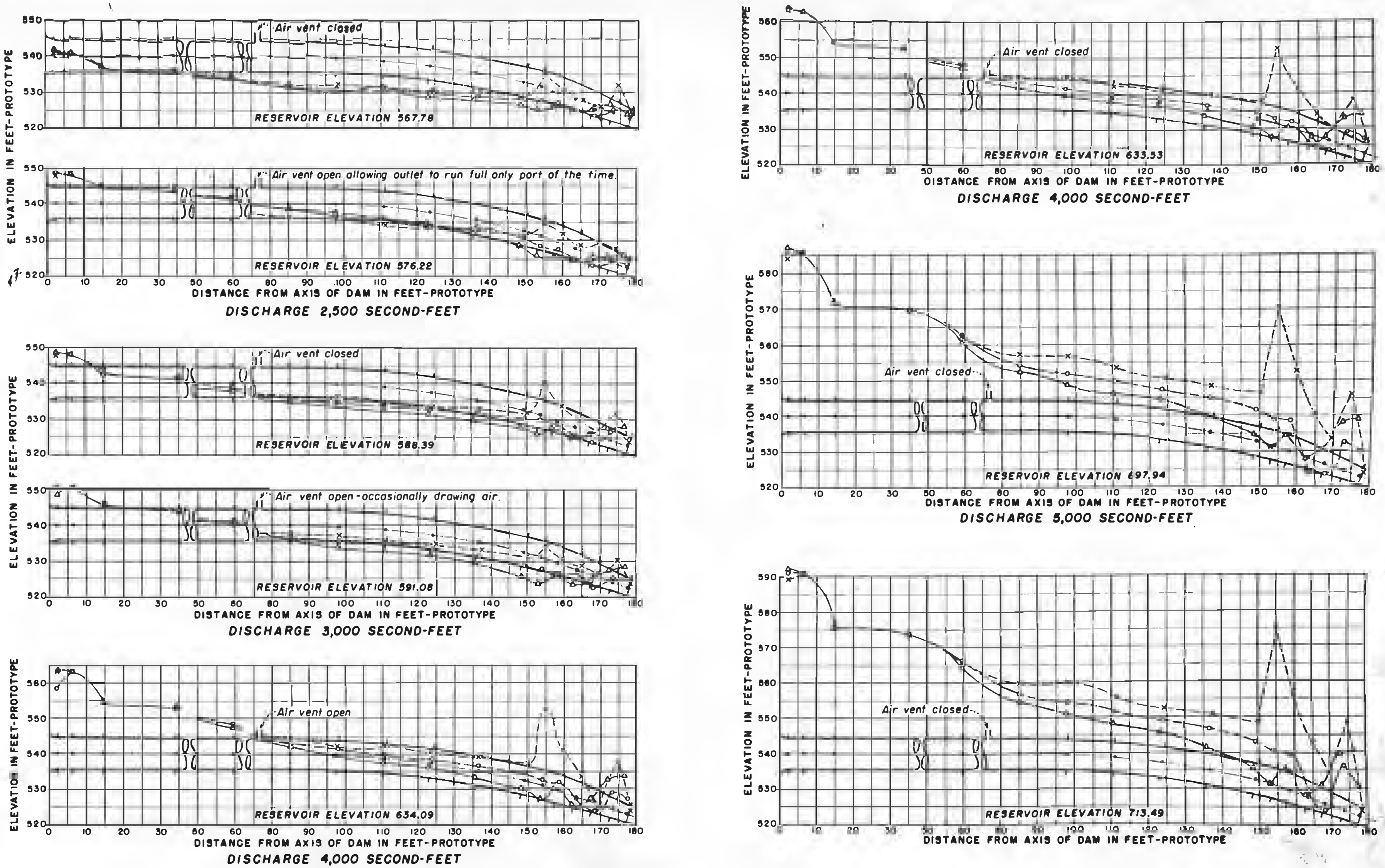
SECTION D-D



SECTION E-E

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT-TEXAS
MARSHALL FORD DAM
OUTLET WORKS
CONDUITS No. 10-12 AND 14
PLAN AND SECTIONS

DRAWN J.R. SUBMITTED
TRACED J.E.M. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO MAR 28, 1938 249-D-642



MARSHALL FORD DAM

RIVER OUTLETS

HYDRAULIC MODEL STUDIES - SCALE 1:25.5

PRESSURES IN OUTLET WITH SPREADING EXIT DESIGN 6

EXPLANATION

×— — — Pressure elevations along crown.
o——— Pressure elevations along sides.
△——— Pressure elevations along invert.

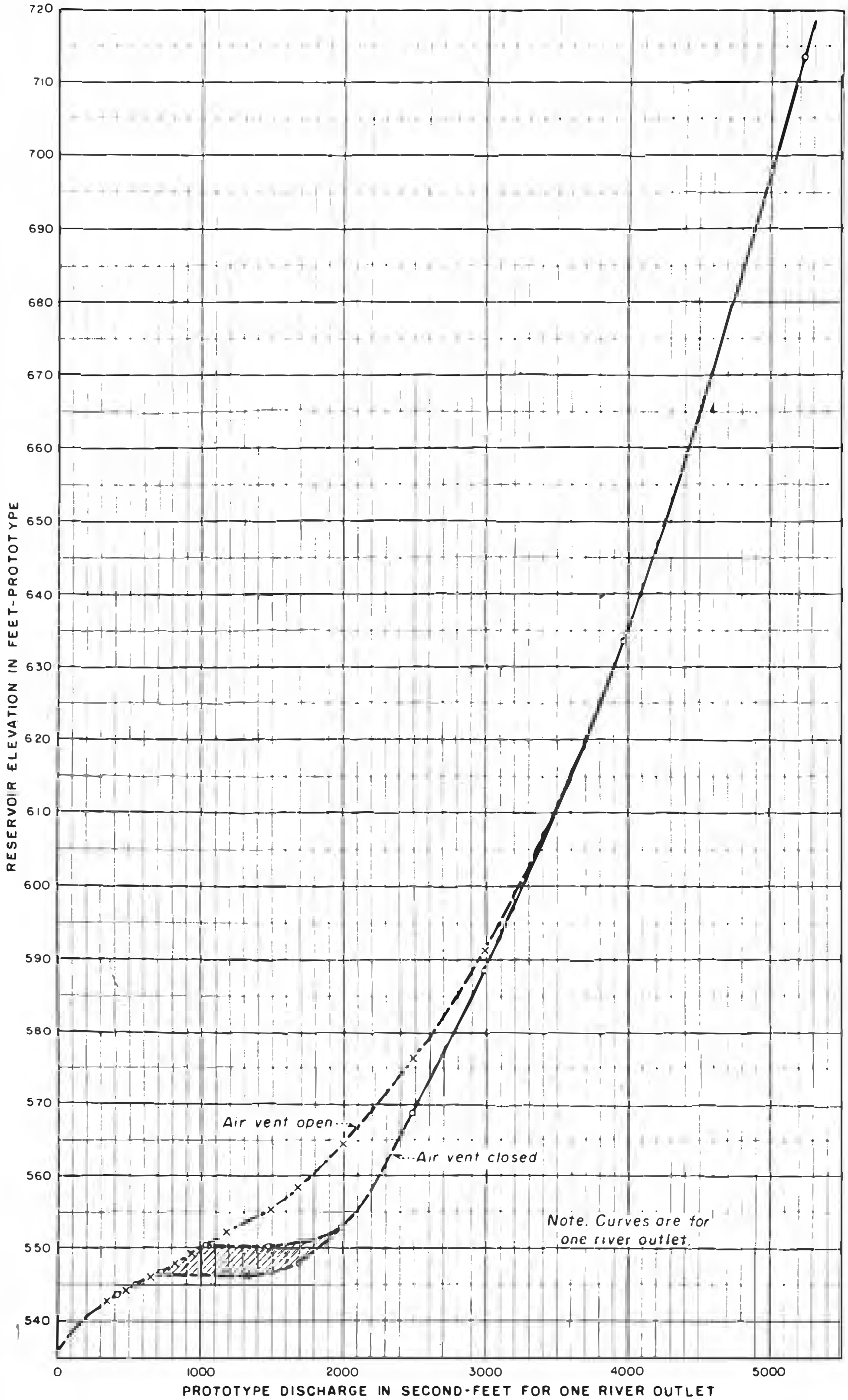
NOTES

Pressures are in feet of water prototype.
Discharges are for one outlet.
With 2,500 second-feet and the air vent open, the outlet ran alternately full and free.

R.A.G. 3-7-39

249-D-1100

FIGURE 37



MARSHALL FORD DAM
RIVER OUTLETS
HYDRAULIC MODEL STUDIES - SCALE 1:25.5
DISCHARGE CURVES FOR SPREADING OUTLET
DESIGN 6

RAG 2-28-38

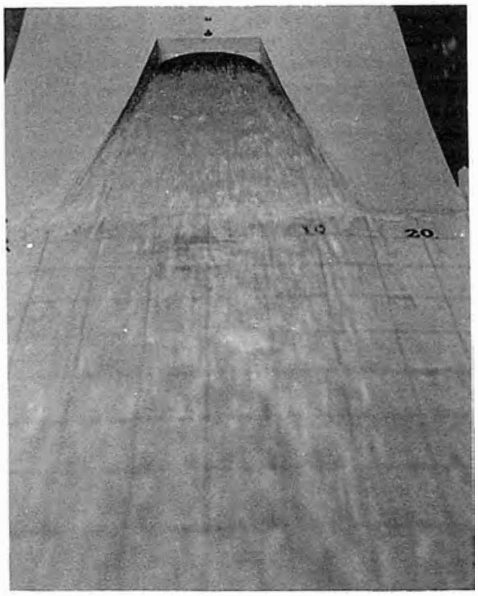
249-D-1101

with and without air being admitted through the vent. This was done by increasing the discharge, by small increments, from reservoir elevation 542.7 to 714.0. With the air vent open, acceptable conditions existed until the reservoir reached elevation 551.8. The outlet flow then began to pulsate rapidly, with the outlet flowing alternately full and free. This condition, which prevailed until the reservoir water surface reached elevation 610.0, was considered undesirable because of the vibrations which might be created in the prototype structure.

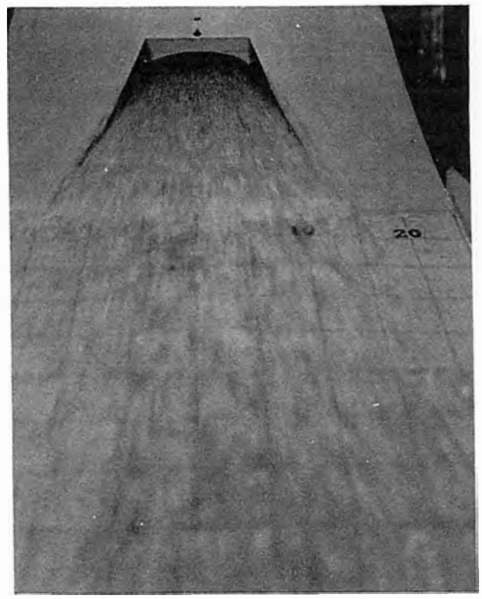
Acceptable conditions were obtained with the air vent closed until the water surface in the reservoir reached elevation 550.35. At this point the outlet began to flow completely full and the discharge increased, drawing the forebay of the model down rapidly to elevation 546.43 where the conduit became aerated and the free flow stage reoccurred. The discharge was decreased by this aeration, the water level in the small forebay of the model gradually rose to elevation 550.4 where the outlet again flowed full, and the cycle was repeated. This condition could not occur in the prototype because of the size of the reservoir and the remote possibility of operating at this quantity and head. When the reservoir reached elevation 552.8, the outlet flowed full continually.

Because the reservoir probably never will be at these low elevations after it is filled, and, since it is likely to be receding or raising while operating near this level, it does not seem essential to provide air to the river outlets when the gates are completely open. However, air should be supplied to the outlets while the gates are being opened and the vents should be left open when the gates are closed. Flow characteristics of the recommended design of outlet and outlet exit are shown on figure 39.

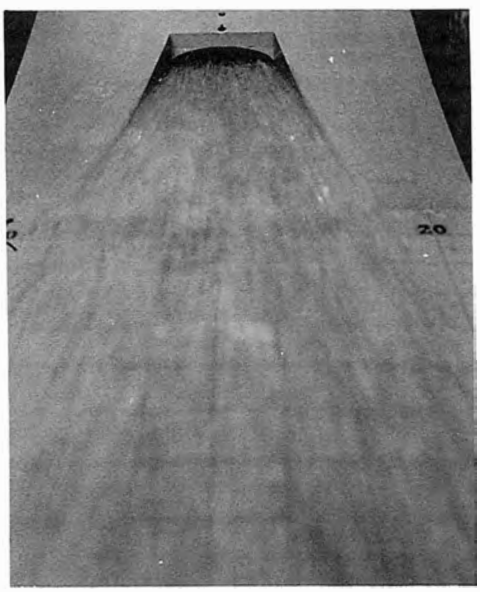
- - -



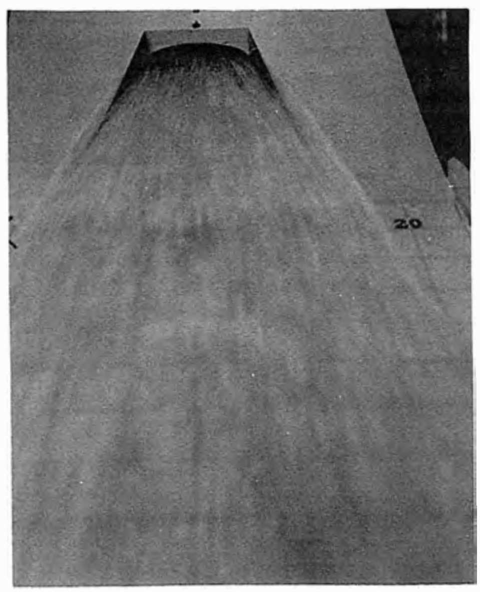
Discharge 2,000 Second-Feet



Discharge 3,000 Second-Feet



Reservoir Elev. 640



Reservoir Elev. 714

SPREADING RIVER OUTLET - RECOMMENDED DESIGN