HYD-48



HYD 48

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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Memorandum to Chief Designing Engineer

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# HYDRAULIC MODEL STUDIES FOR DESIGN OF HEADGATE ROCK DAM COLORADO RIVER INDIAN IRRIGATION PROJECT - ARIZONA

By

R. A. GOODPASTURE, JUNIOR ENGINEER

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Denver, Colorado
January 20, 1939

HYD 48

Denver, Colorado, January 20, 1939

MEMORANDUM TO CHIEF DESIGNING ENGINEER
(R. A. Goodpasture)

Subject: Hydraulic model studies for design of Headgate Rock Dam - Indian Service.

1. Introduction. The subject of the design of the Headgate Rock Dam to be constructed on the Colorado River was discussed in detail during August 23, 24, and 25, 1938, between Messrs. P. F. Henderson and H. V. Clotts of the Indian Service and engineers of the Bureau of Reclamation. The nature of the hydraulic problems involved led to the consideration of a hydraulic model of the entire structure. Instructions were received September 27, 1938, by the Bureau of Reclamation, to proceed with the design of the model. Through the cooperation of the officials of the Colorado State College Experiment Station, the model was built and tested in the hydraulic laboratory of the Colorado State College at Fort Collins, Colorado.

## SPILLWAY

2. Purpose of studies. The original design of the spillway provided for a maximum discharge of about 160,000 second-feet over a crest 250 feet long. A concrete apron was to extend about 255 feet downstream from the crest to provide for the hydraulic jump. Further consideration of the flood characteristics on the Colorado River showed that the spillway should be designed for 200,000 second-feet with a crest length of about 400 feet. The hydraulic jump as a means of dissipating the energy downstream from the spillway was abandoned in favor of a vertical roller partially confined in a bucket, since this would eliminate the long apron and reduce the uplift under the structure. It was desired, by means of the model studies, to investigate this revised design for tail-water elevations varying between those prevailing at the present time and those corresponding to a maximum retrogression of 35 feet.

3. Description of model. A model scale ratio of 1 to 60 was fixed by limitations imposed by the floor space, available head below the laboratory weir, and the capacity of the laboratory pump. The model was constructed in a 12- by 34-foot metal-lined tank. The exact location, together with topography and other general features, is shown on figure 1. Details of the ogee spillway, bucket, piers, and gates are shown on figure 2. Riprap on the banks extended 150 feet downstream from the sill. The

downstream channel was placed at elevation 300. The first model tests were conducted on this design with the dentated sill and high wing walls extending parallel with the axis from the ends of the sill into the banks. The expected minimum tail-water curve, representing an ultimate retrogression of 35 feet, is shown as curve A on figure 3. Original considerations gave maximum tail-water elevations immediately after construction of the spillway as 35 feet higher than those on curve A. With these higher tail-water elevations, the spillway was found to be completely submerged. The jet did not dive into the bucket with the result that the flow occurred entirely on the surface of the pool. This condition submerged the gate pins. In order to limit the time during which this undesirable condition will exist to a short period immediately after the spillway is first placed in operation, the width and depth of the channel downstream from the spillway were fixed so that the maximum tail-water elevations as controlled by the channel were those which produce satisfactory conditions below the gates. These tail-water elevations are lower than those of the present river, but they will occur as soon as a relatively small amount of retrogression has taken place in the river bed itself. The maximum tail-water elevations were, therefore, taken to be those for which the jet flowing over the crest commenced to dive into the bucket. These limiting tail-water elevations, as determined from the model, are shown in curve B of figure 3. This curve was used as the maximum tail-water curve for future tests. The average of the maximum (curve B) and the minimum (curve A) tail-water elevations for any particular discharge is referred to as the average tail-water elevation for that quantity (curve C).

4. Preliminary studies. Preliminary observations on the operation of the stilling pool showed that critical conditions were obtained with the minimum tail-water curve (curve A, figure 3). Therefore, a series of tests were conducted using tail-water depths obtained from this curve. The stilling pool was rough but satisfactory except for horizontal eddies at the sides of the pool. These eddies caused excessive erosion on the riprapped slopes and, at the higher discharges, exposed the walls below the elevation of the sill. To correct this undesirable condition, several low training walls were tried as a downstream extension of the end walls of the spillway. Training walls nos. 1 and 2 of figure 2 produced a decided improvement, wall no. 1 being the better of the two. However, the undesirable erosion on the end slopes persisted at high discharges. Training wall no. 3 was the result of eliminating the high wing walls extending from the ends of the sill to the banks, raising the low training walls to elevation 365 at the ends of the original training walls, and extending them on a 2 to 1 slope downstream to elevation 300. Riprap backfill

was placed behind the walls, and the riprap on the $1\frac{1}{2}$ to 1 slopes extended 50 feet farther downstream. This arrangement gave excellent stilling-pool conditions up to a discharge of 150,000 second-feet. While erosion on the banks existed, for higher discharges, conditions were improved. It was thought that repairing some damage after an excessive flood would be more economical than providing complete protection in the initial structure. Training wall no. 3 is accordingly recommended for incorporation in the structure and was used during subsequent model studies.

5. Sill studies. The original sill (sill no. 1, figure 2) produced a rough water surface in the stilling pool. However, the toe of the bucket was not endangered by scouring since the major portion of the erosion in the river bed occurred farther downstream. As the bed of the pool in the prototype consists of hard basalt, no concern is felt for the stability of the structure or the satisfactory operation of the stilling pool at minimum tail-water elevations. Because of possible erosion of the sharp corners of the dentates by river-bed material carried back into the stilling pool by the ground roller, it was deemed advisable to develop a solid sill which would give hydraulic conditions equal to those of the dentated sill. Such a sill would also result in a saving in construction. The dentated and four types of solid sills (figure 2) were accordingly tested under the same procedure for water-surface and scour profiles. For each sill, water-surface profiles were taken at successive discharges of 25,000, 50,000, 100,000, 150,000, and 200,000 second-feet with their corresponding minimum tail-water elevation. Preliminary observations showed a stabilized scour pattern for any discharge at the end of a period of one hour. Each discharge was, therefore, maintained for this period, after which the flow was carefully stopped to avoid disturbing the scour pattern. The scour profile was then recorded. The river bed was initially placed at elevation 300 for the test of each sill and was not disturbed between discharges. The results of these sill tests are shown on figure 4. There was little dissimilarity between the results obtained with the five different sills. Original analysis of the data, made during the operation of the model and influenced largely by the construction cost, lead to sill no. 5 being considered as the recommended design. However, final analysis of the data resulted in the recommendation of sill no. 3 as the most satisfactory economical design. Sill no. 5 did produce slightly better scour conditions but gave a reduced depth of water in the bucket.

6. Final studies. The change in the recommended sill design resulted in the final spillway tests being made with sill no. 5. However, the results of these final tests, with the exception of the water-surface and pressure profiles for the bucket,

would be the same regardless of which sill was used. Piezometric pressures on the crest and bucket were obtained at discharges of 50,000, 100,000, 150,000, and 200,000 second-feet for minimum tail-water elevations (figure 5). The water-surface profiles over the bucket are somewhat lower than would be obtained with the recommended design. Discharge curves were obtained with the gates raised, using maximum, average, and minimum tail-water elevations (figure 3). The discharge curves coincided for minimum and average tail-water elevations. For maximum tail-water elevations, the discharge was somewhat less due to submergence. In other words, the capacity of the spillway will increase as the channel downstream erodes.

The flow characteristics for the final design spillway are shown in figures 6 to 11, inclusive, with discharges of 40,000, 80,000, 120,000, and 180,000 second-feet and maximum, average, and minimum tail-water elevations. The maximum design capacity of 200,000 second-feet is shown in figure 12 with average and minimum tail-water heights.

7. Recommendations. In the operation of the spillway, attention is particularly called to the necessity of maintaining all of the spillway gates at the same elevation. Unequal distribution of flow under the gates will create eddies in the stilling pool of a destructive nature, which must be avoided. The roller in the bucket has an upstream velocity component which causes logs and floating debris to be retained on the water surface near the high-velocity water flowing under the gates. The constant pounding of this floating debris may be destructive to the downstream corners of the piers. It is suggested that these corners be protected with a metal face, probably angle irons. The recommended sill may be reduced in size, and hence in cost, without impairing its efficiency by steepening its downstream face. Accordingly, it is recommended that the downstream face of the sill be steepened until it lies in the plane determined by the vertex of the sill (elevation 300) and the center of the circle which forms the bucket. Since the most severe erosion occurred at the ends of the stilling pool near the 2 to 1 sloping training walls, care must be exercised that the footings of these walls are carried into the hard basaltic strata of the foundation.

## CANAL HEADWORKS

8. Studies of canal headworks. Two designs were proposed for the canal headworks, one with two radial gates (design A), and the other with one radial gate (design B) (figure 13). Since it may be necessary for the headworks to operate with a

range of forebay between elevations 362 and 372, it was considered advisable to study the action of the hydraulic jump below the gates and the effectiveness of the transition immediately downstream for each of these extreme conditions. A model of each design was constructed and tested to determine the consistency of the formation of the jump upstream from the transition. Each design was tested with various gate openings using reservoir water surfaces at elevations 362 and 372. Discharges were measured to obtain data from which discharge curves were constructed for both the single- and double-gate designs. Since it was considered desirable that there should be a factor of safety in the design of the pool, tests were run with discharges somewhat in excess of the design capacity. Attention was given to those gate openings at which water overtopped the sides of the pool.

Design A, employing two 22-foot gates and a pool floor at elevation 347, operated satisfactorily for all gate openings with the pond at elevation 362. With the pond at elevation 372 and the gates open 7.5 feet (discharge 7,000 second-feet), water overtopped the sides of the pool and, at larger gate openings, overflowed the canal banks farther downstream, with the jump forming continuously upstream from the transition.

Design B, with one 40-foot gate and a pool floor at elevation 347, also gave satisfactory operating characteristics for the normal pond elevation. With the reservoir at elevation 372 and the gate raised five feet (discharge 4,600 second-feet), the jump swept out of the pool and formed in the canal downstream from the transition. Lowering the floor of the pool two feet to elevation 345 (design C) confined the jump to the pool for both pond elevations and all gate openings. With a pond at elevation 372, water overflowed the pool sides when the gate was raised 10 feet. With the pool floor placed at elevation 346 (design D), the jump did not sweep out of the pool at the higher pond elevation until the gate was opened 10 feet.

Discharge curves for the flow under the single gate in designs B, C, and D and the two smaller gates in design A were obtained for the pond at elevations 362 and 372. The rating curve for the trapezoidal canal section shows the conditions under which the model was tested. These curves are shown on figure 14.

9. Recommendations. Designs A and D are both considered satisfactory solutions of the canal headworks problem. Field operating conditions might make the single-gate design more desirable. Either design is certain to give satisfactory hydraulic conditions and ample protection to the canal if operated with reasonable care. Figure 14 indicates that, with a reservoir at

elevation 362, either design will give an ample canal discharge. With the pond at elevation 372, gate openings should be about 3.0 feet and 2.5 feet for the single- and double-gate designs, respectively, for the normal canal discharge of 2,960 second-feet. Automatic regulation of the discharge might be desirable. It is recommended that sufficient freeboard be allowed on the canal-headworks stilling-pool walls to confine the greater portion of the spray and splash that invariably accompany a hydraulic jump in this type of structure. It is felt that the vertical walls should be extended up to about elevation 370. Flow conditions for the two recommended designs are shown on figures 15 and 16.

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## ABSTRACT OF CORRESPONDENCE ON THE MODEL STUDIES

9-26-38 - Telegram from the Commissioner of the Bureau of Reclamation to the Denver office giving authority to construct the model.

12-15-38 - From Acting Chief Engineer to Office of Indian Affairs concerning design of the wing walls below the spillway and the shape of the downstream bucket, together with other design features including the canal headworks.

12-3-38 - From Acting Chief Engineer to Paul F. Henderson concerning results of model stilling pool tests with the dentated sill (sill no. 1). Flow and scour pictures were attached.

12-22-38 - Telegram from Chief Engineer to the Indian Irrigation Service requesting authority to dismantle the model.

12-22-38 - Telegram from the Indian Irrigation Service to the Bureau of Reclamation replying to above. Additional model flow pictures taken from the upstream and side were requested.

1-10-39 - From Chief Engineer to Paul F. Henderson concerning results of model tests with the horizontal sill (sill no. 5, original recommendation). Flow and scour pictures of the stilling pool together with views of the model from the upstream were attached.

1-27-39 - From Chief Engineer to the Office of Indian Affairs concerning the change in the recommended sill design from sill no. 5 to sill no. 3.

FIGURE 1

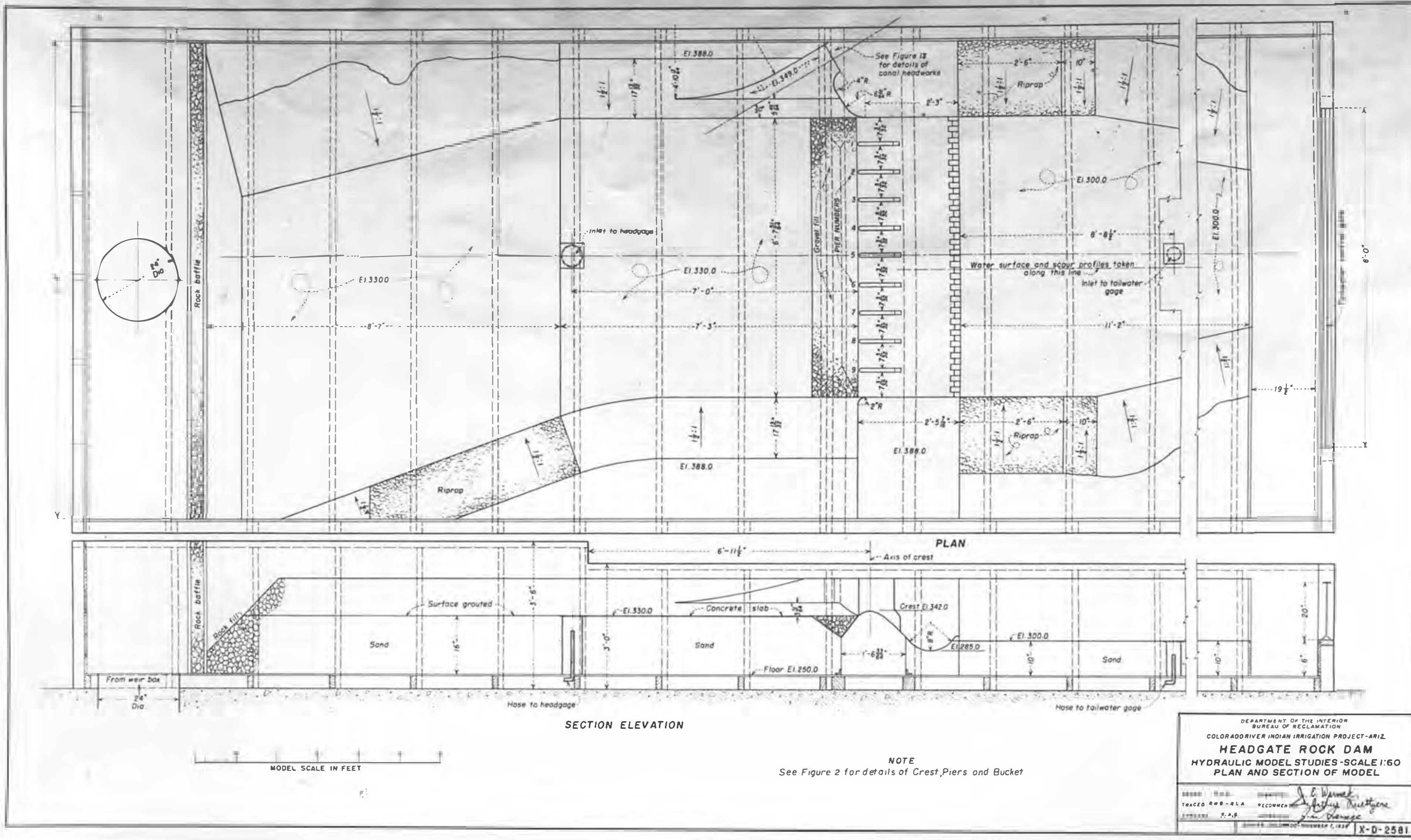
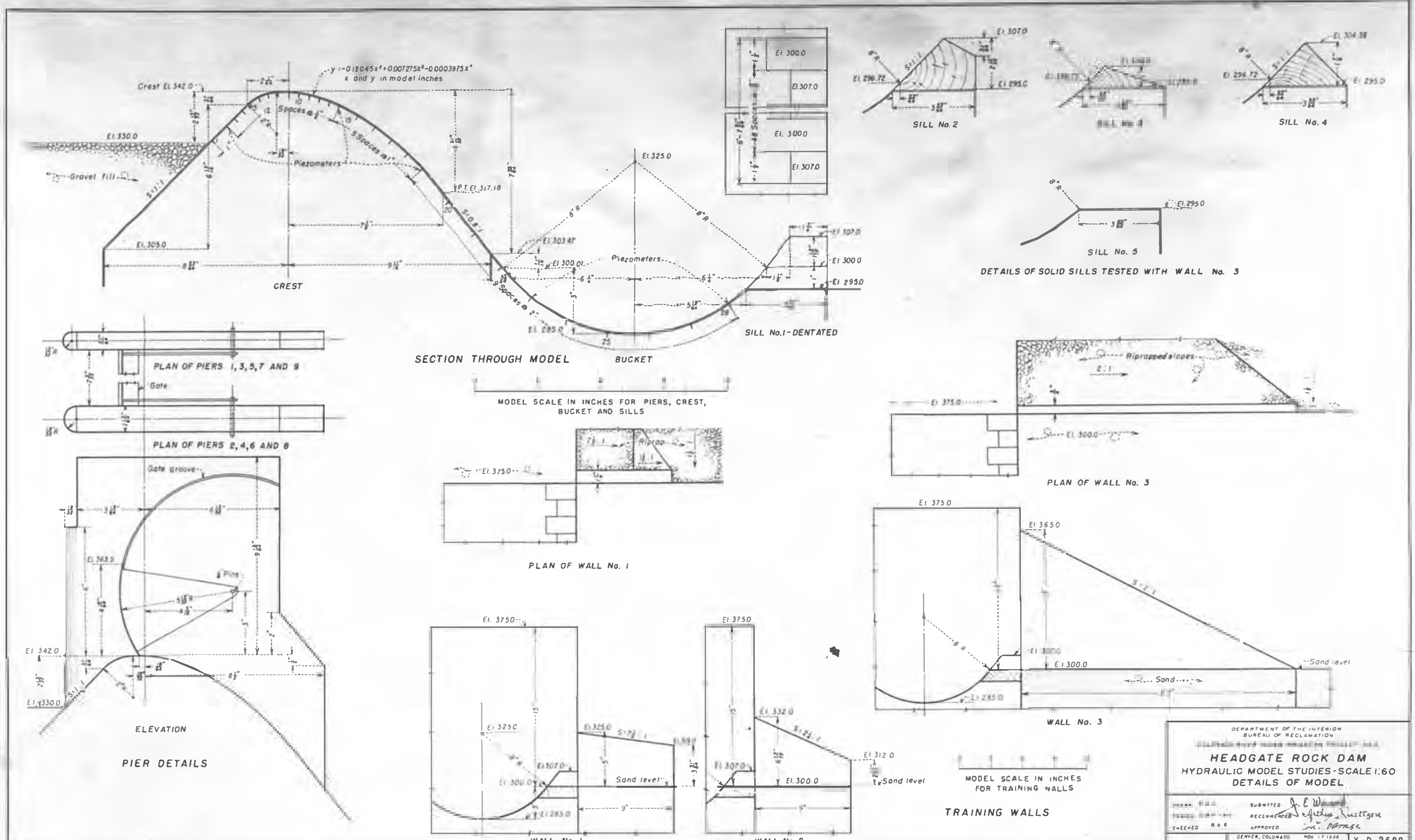
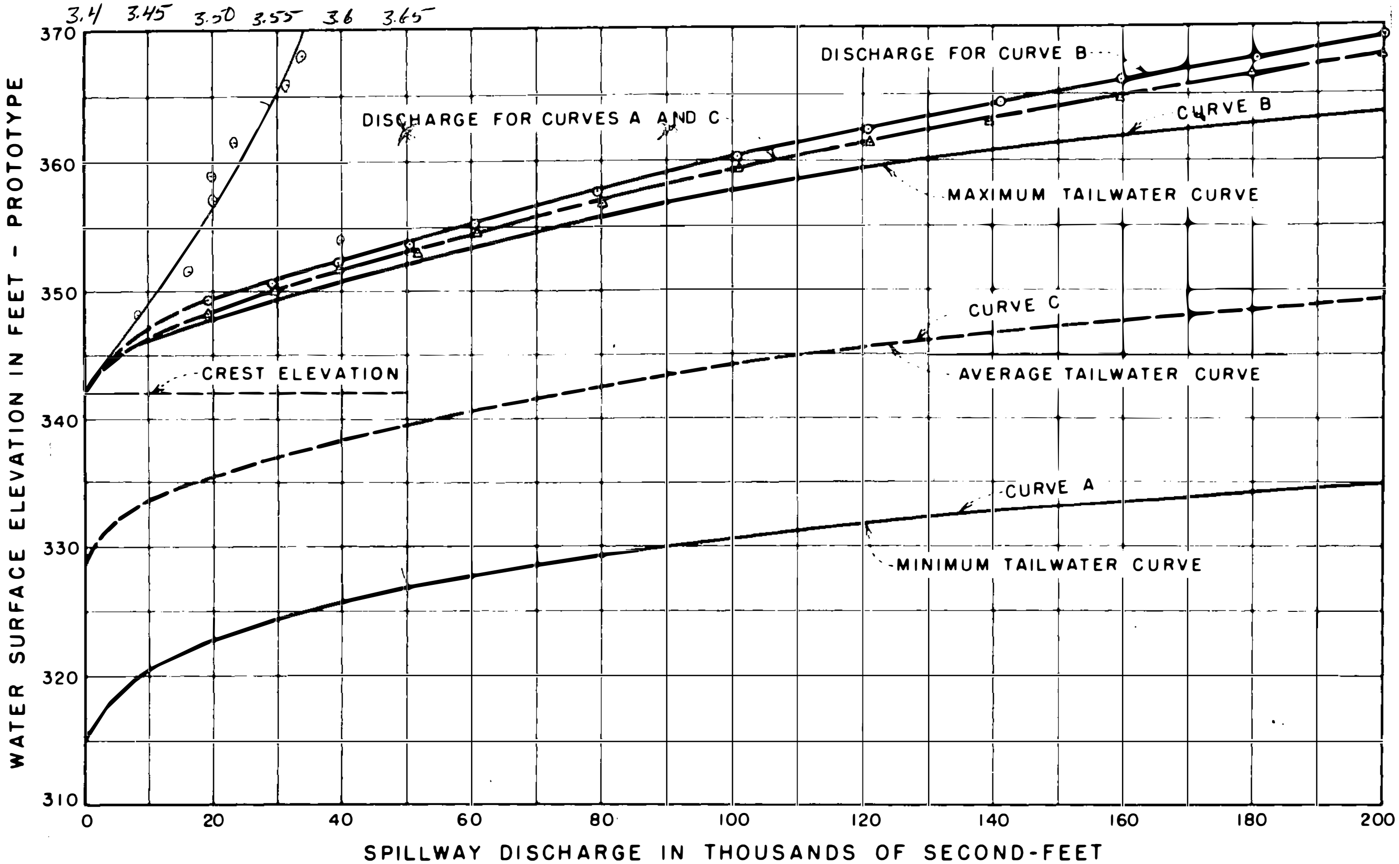

Rock baffle
24" Dia
El. 330.0
Inlet to headgage
El. 388.0
El. 349.0
See Figure 13 for details of canal headworks
Riprap
Gravel fill
PIER NUMBERS
El. 300.0
Water surface and scour profiles taken along this line
Inlet to tailwater gage
8'-7"
7'-3"
7'-0"
11'-2"
6'-0"
PLAN
6'-11½"
Axis of crest
Rock fill
Surface grouted
Concrete slab
Crest El. 342.0
El. 285.0
Sand
Floor El. 250.0
From weir box
Hose to headgage
Hose to tailwater gage
SECTION ELEVATION
MODEL SCALE IN FEET
NOTE
See Figure 2 for details of Crest, Piers and Bucket
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER INDIAN IRRIGATION PROJECT-ARIZ.
HEADGATE ROCK DAM
HYDRAULIC MODEL STUDIES-SCALE 1:60
PLAN AND SECTION OF MODEL
TRACED
X-D-2581

FIGURE 2



Crest El. 342.0
$y = -0.12045x^2 + 0.007275x^3 - 0.0003975x^4$
x and y in model inches
El. 330.0
Gravel fill
Piezometers
P.T. El. 317.18
El. 305.0
CREST
El. 303.47
El. 300.01
El. 325.0
El. 285.0
SECTION THROUGH MODEL
BUCKET
SILL No. 1-DENTATED
El. 307.0
El. 300.0
El. 295.0
SILL No. 2
SILL No. 4
SILL No. 5
DETAILS OF SOLID SILLS TESTED WITH WALL No. 3
PLAN OF PIERS 1, 3, 5, 7 AND 9
Gate
PLAN OF PIERS 2, 4, 6 AND 8
MODEL SCALE IN INCHES FOR PIERS, CREST, BUCKET AND SILLS
Gate groove
Pins
El. 363.5
El. 342.0
El. 330.0
ELEVATION
PIER DETAILS
El. 375.0
Riprap
PLAN OF WALL No. 1
Riprapped slopes
El. 300.0
PLAN OF WALL No. 3
El. 365.0
Sand
Sand level
WALL No. 3
El. 325.0
El. 285.0
WALL No. 1
El. 332.0
El. 312.0
WALL No. 2
MODEL SCALE IN INCHES FOR TRAINING WALLS
TRAINING WALLS
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HEADGATE ROCK DAM
HYDRAULIC MODEL STUDIES-SCALE 1:60
DETAILS OF MODEL
SUBMITTED
RECOMMENDED
APPROVED
CHECKED
DENVER, COLORADO
X-D-2582




## NOTES

Tailwater elevations were obtained 750.0 feet from axis of crest.
Reservoir elevations were obtained 435.0 feet from axis of crest.
Discharge curves were obtained with gates up.

*when determining values of a velocity head was used in formula.*

# HEADGATE ROCK DAM

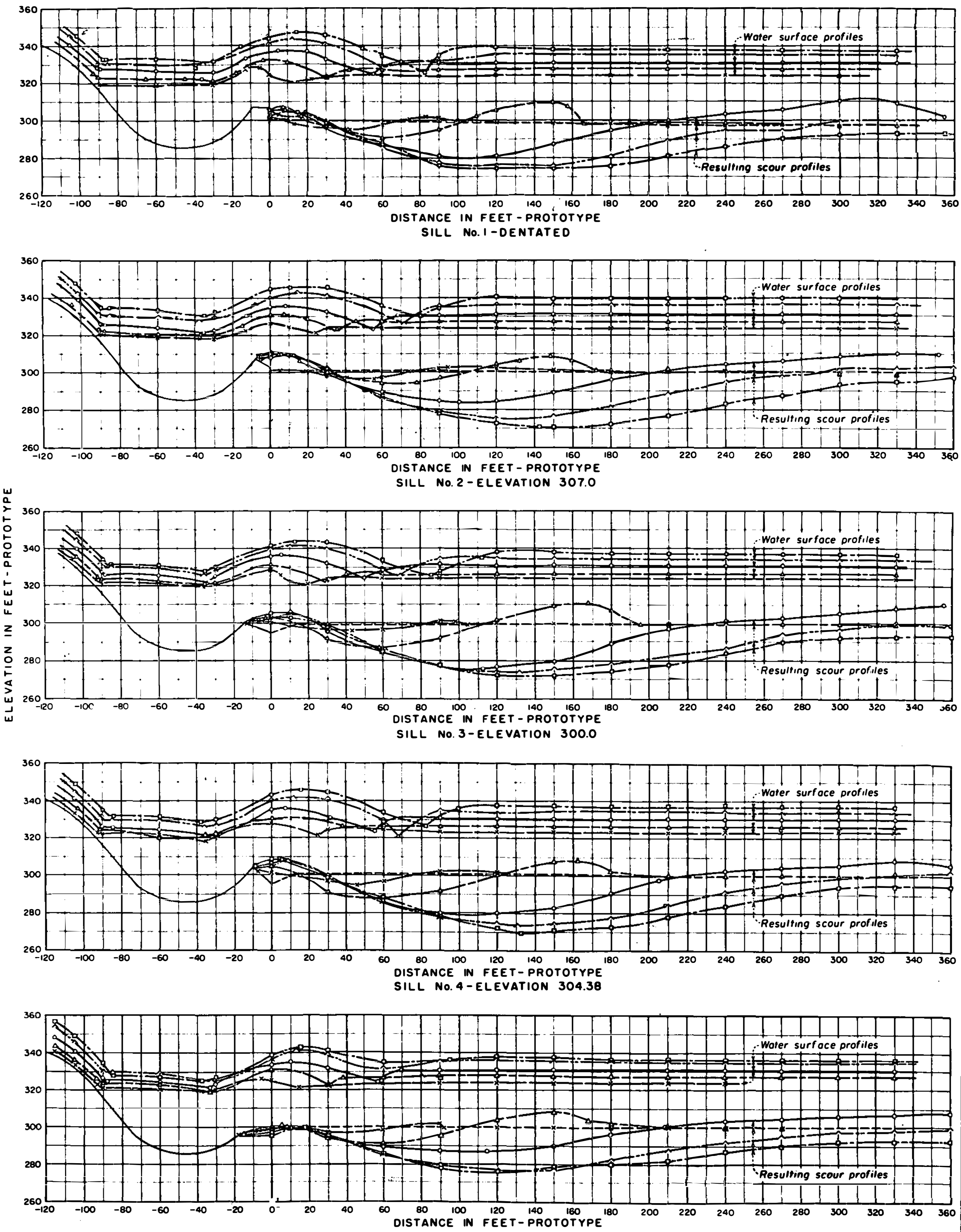
INDIAN SERVICE
SPILLWAY STUDIES
DISCHARGE AND TAILWATER CURVES

D.M.L. 12-28-38

FIGURE 3

X-D-2583

FIGURE 4




**EXPLANATION**

- INITIAL SAND SURFACE
- DISCHARGE 25,000 SECOND-FEET
- DISCHARGE 50,000 SECOND-FEET
- DISCHARGE 100,000 SECOND-FEET
- DISCHARGE 150,000 SECOND-FEET
- DISCHARGE 200,000 SECOND-FEET

**NOTES**

For each sill every discharge was allowed to run for one hour. The discharges were in increasing sequence. The sand was not disturbed between discharges.

The minimum tailwater elevations were used. Tailwater was measured 522 feet downstream.

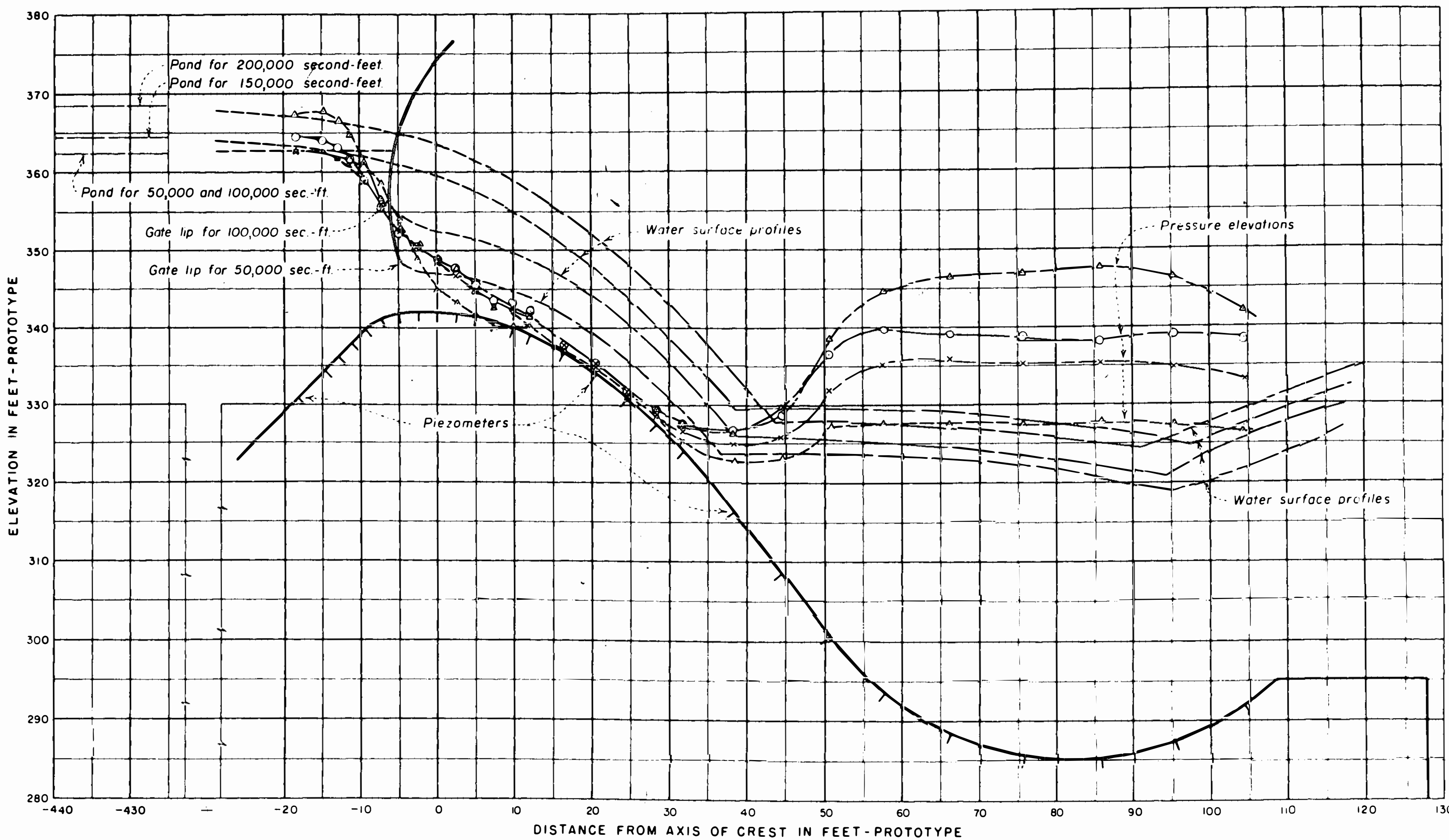
**HEADGATE ROCK DAM**

INDIAN SERVICE

SCOUR AND WATER SURFACE PROFILES

DECEMBER 12, 1938

X-D-2584




PRESSURE ELEVATIONS IN FEET OF WATER PROTOTYPE ON CREST AND BUCKET

## EXPLANATION FOR PRESSURE DATA

^ ——— DISCHARGE 50,000 SEC-FT-TAILWATER EL 327.0
× ——— DISCHARGE 100,000 SEC-FT-TAILWATER EL 331.1
⊙ ——— DISCHARGE 150,000 SEC-FT-TAILWATER EL. 333.7
△ ——— DISCHARGE 200,000 SEC-FT.-TAILWATER EL. 335.6

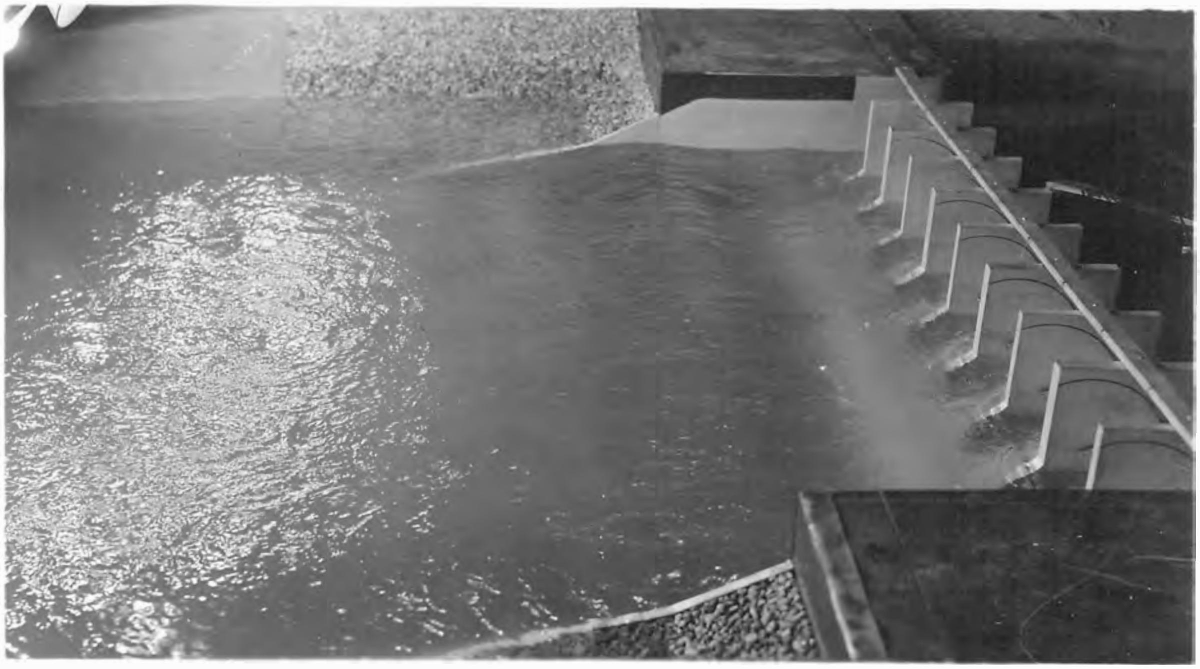
## NOTES

Tailwater elevations were measured 750 feet downstream from the axis of the crest
For discharges of 150,000 and 200,000 second-feet the gates were completely raised
Pressures were measured along center-line of gate No 10, and water surface profiles along center-line of gate No 6

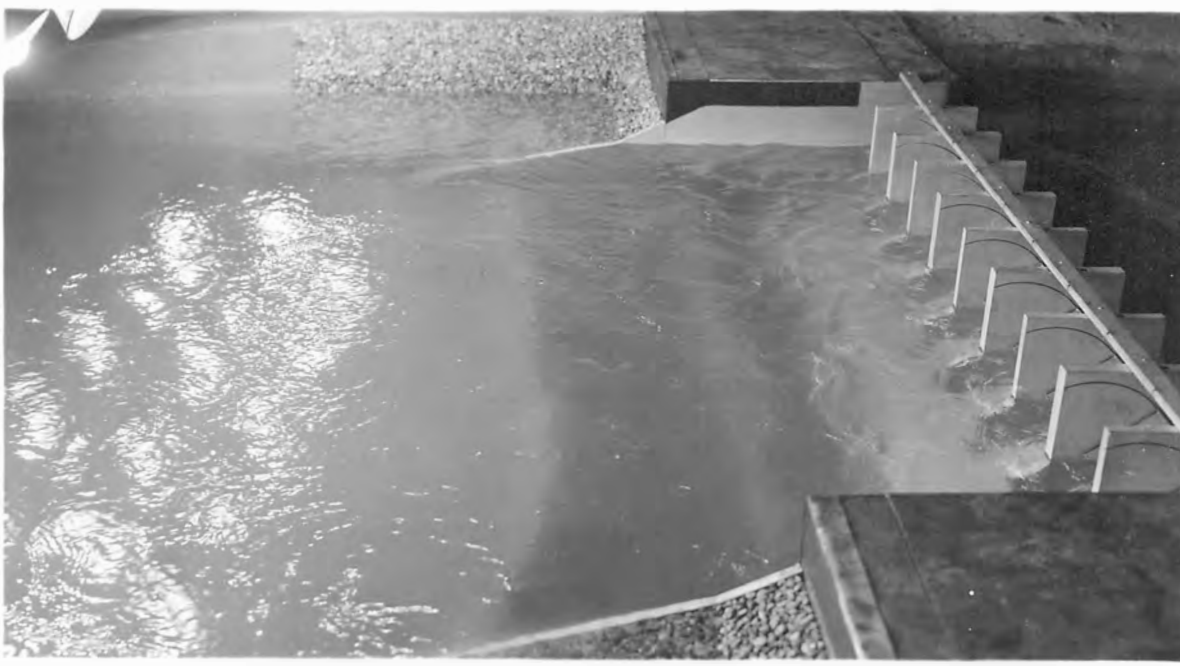
## HEADGATE ROCK DAM
INDIAN SERVICE
## PRESSURE AND WATER SURFACE PROFILES
DECEMBER 17, 1938

FIGURE 5

X-D-2585



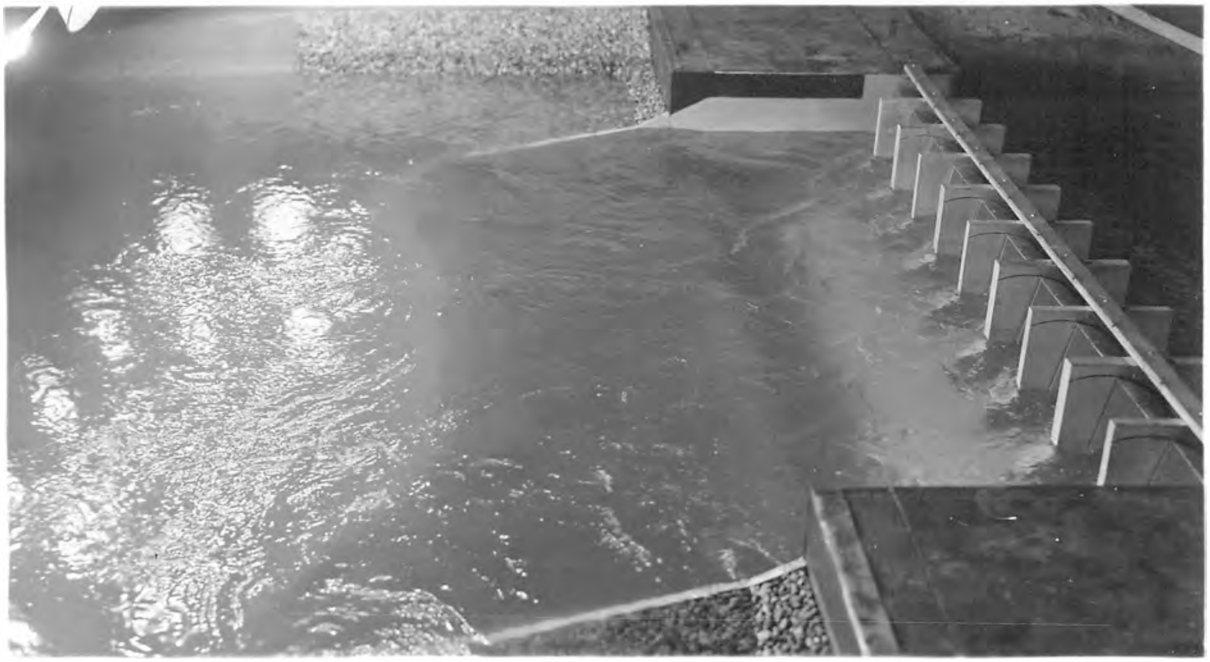
DISCHARGE 40,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 350.8



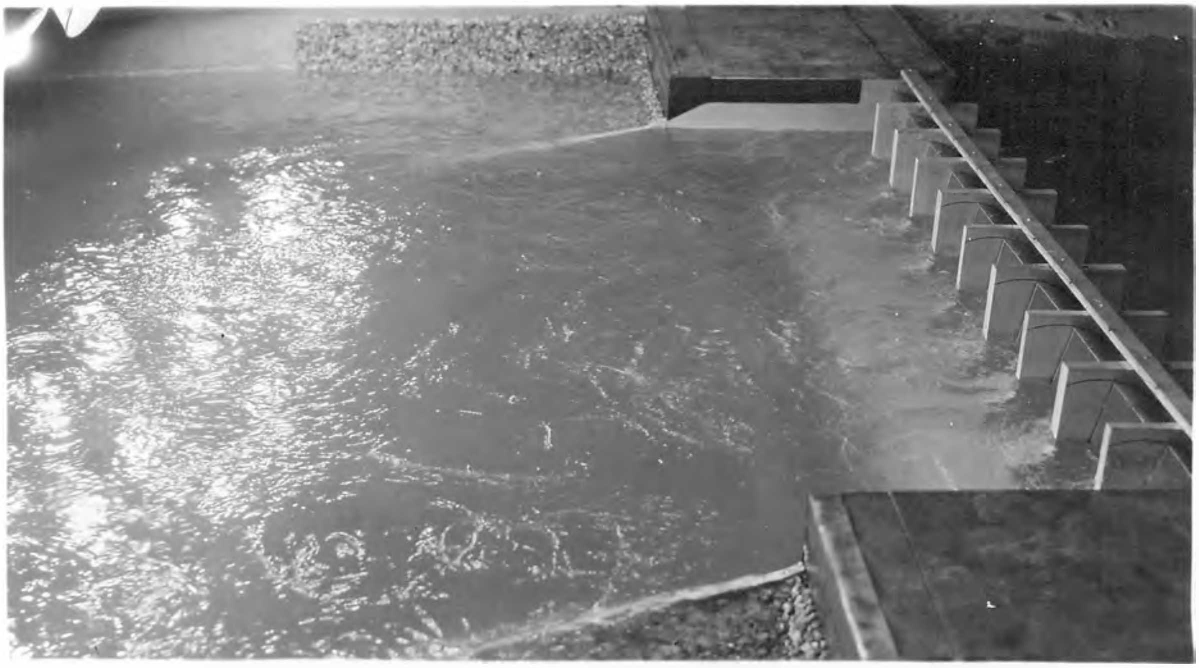
DISCHARGE 80,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 355.7

MAXIMUM TAILWATER ELEVATIONS

RECOMMENDED DESIGN



DISCHARGE 120,000 SECOND-FEET
TAILWATER ELEVATION 359.2



DISCHARGE 180,000 SECOND-FEET
TAILWATER ELEVATION 362.9

MAXIMUM TAILWATER ELEVATIONS

RECOMMENDED DESIGN

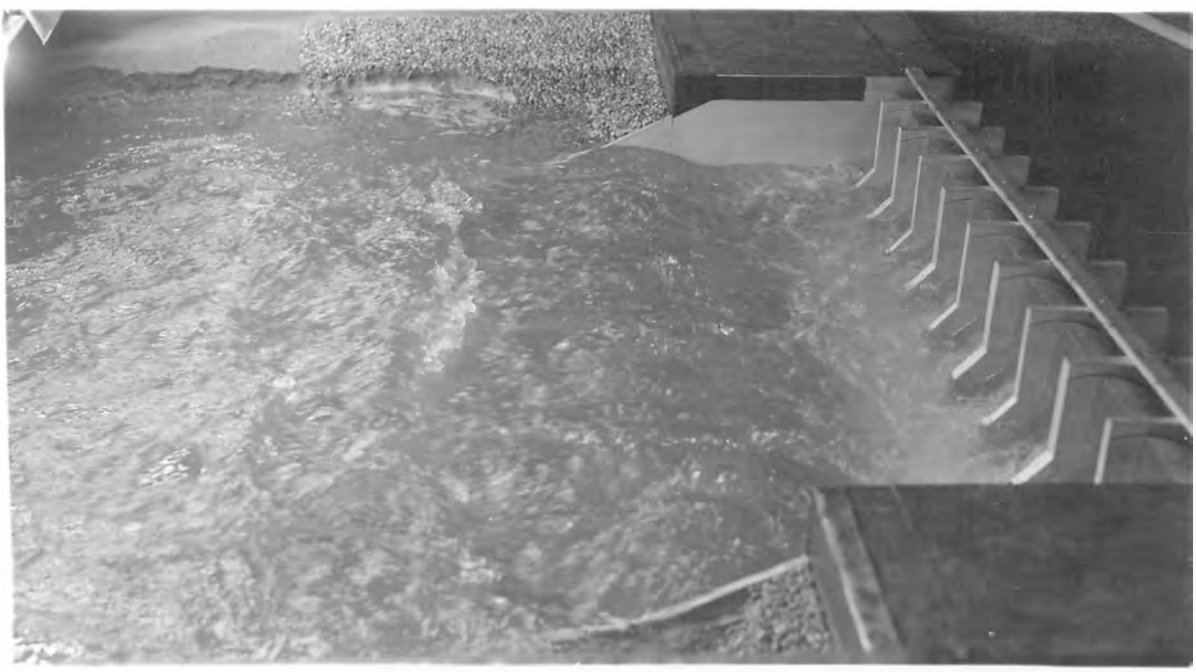
DISCHARGE 40,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 338.2

DISCHARGE 80,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 342.5

AVERAGE TAILWATER ELEVATIONS

RECOMMENDED DESIGN

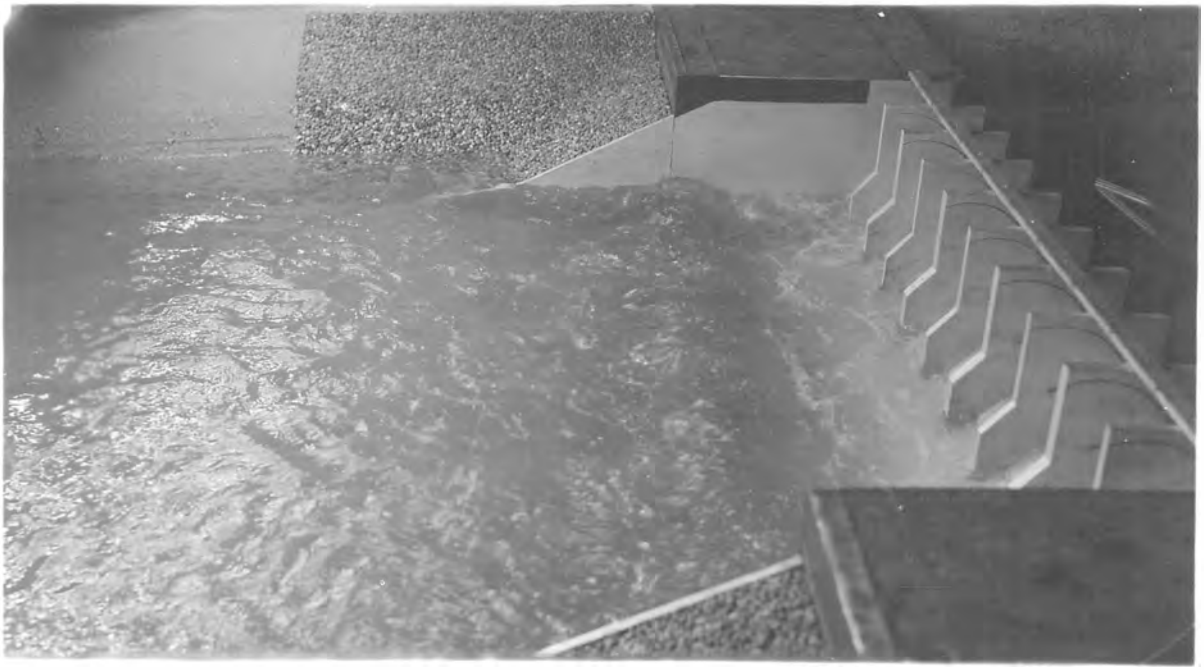
DISCHARGE 120,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 345.5



DISCHARGE 180,000 SECOND-FEET
TAILWATER ELEVATION 348.9

AVERAGE TAILWATER ELEVATIONS

RECOMMENDED DESIGN



DISCHARGE 40,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 325.6

DISCHARGE 80,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 329.2

MINIMUM TAILWATER ELEVATIONS

RECOMMENDED DESIGN

DISCHARGE 120,000 SECOND-FEET - POND ELEVATION 362
TAILWATER ELEVATION 331.7

DISCHARGE 180,000 SECOND-FEET
TAILWATER ELEVATION 334.3

MINIMUM TAILWATER ELEVATIONS

RECOMMENDED DESIGN

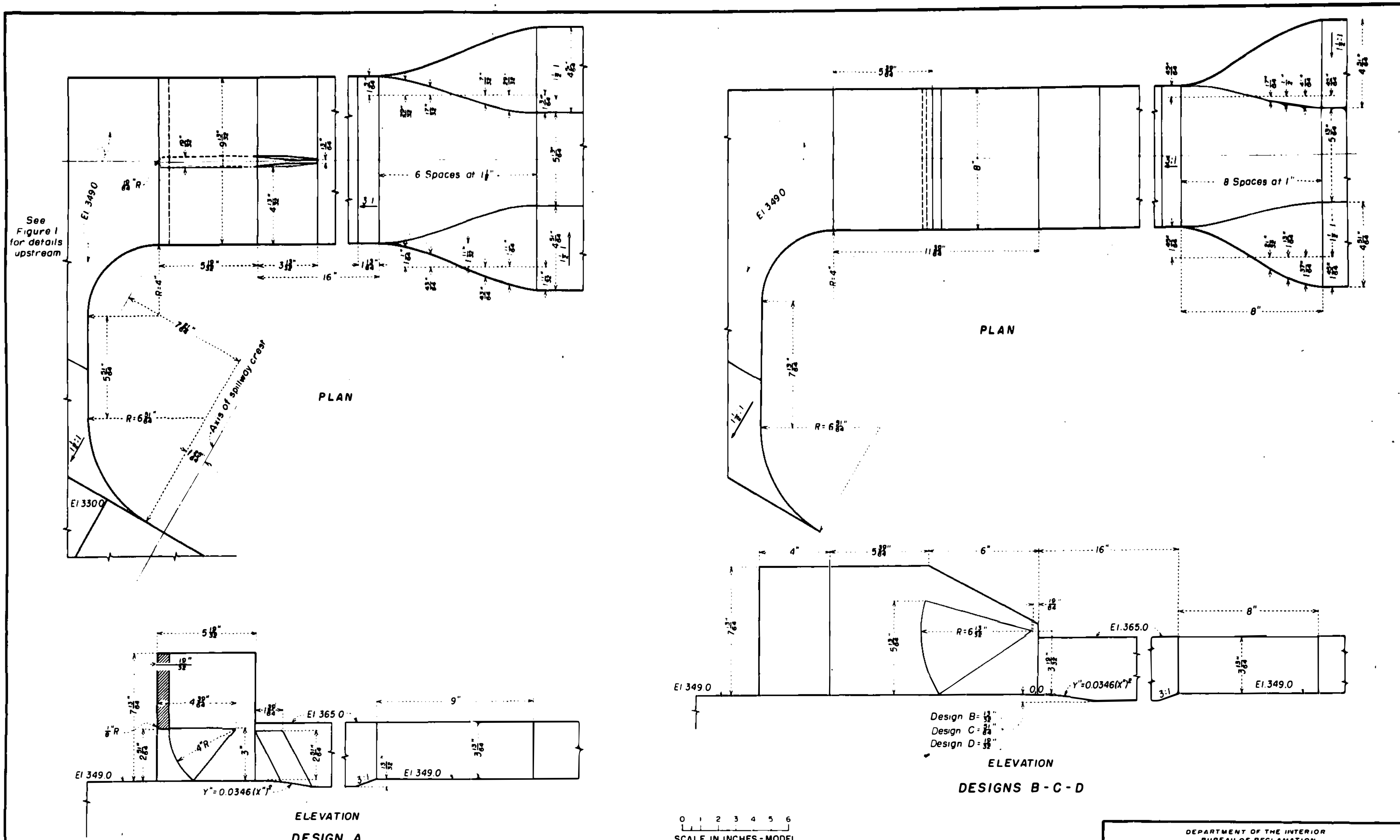
DISCHARGE 200,000 SECOND-FEET
TAILWATER ELEVATION 349.3

AVERAGE TAILWATER ELEVATION

DISCHARGE 200,000 SECOND-FEET
TAILWATER ELEVATION 335.0

MINIMUM TAILWATER ELEVATION

RECOMMENDED DESIGN

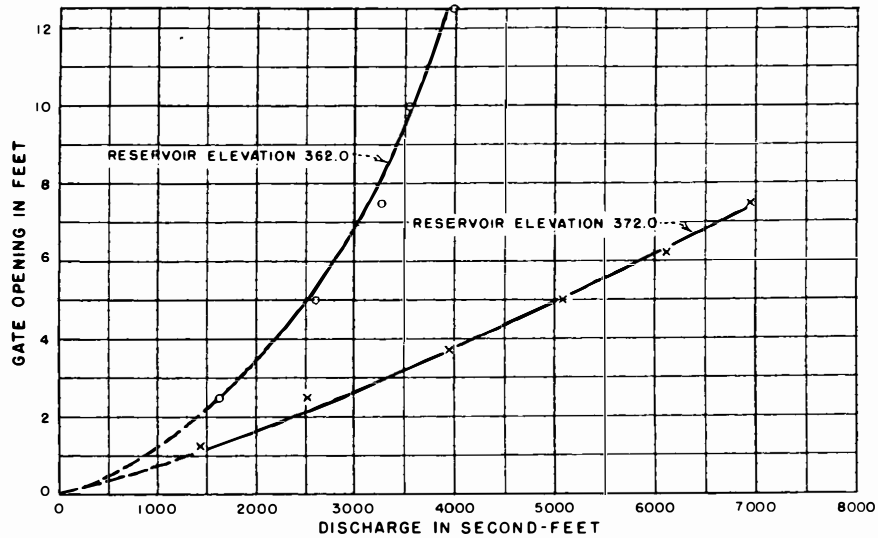



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER INDIAN IRRIGATION PROJECT-ARIZ.

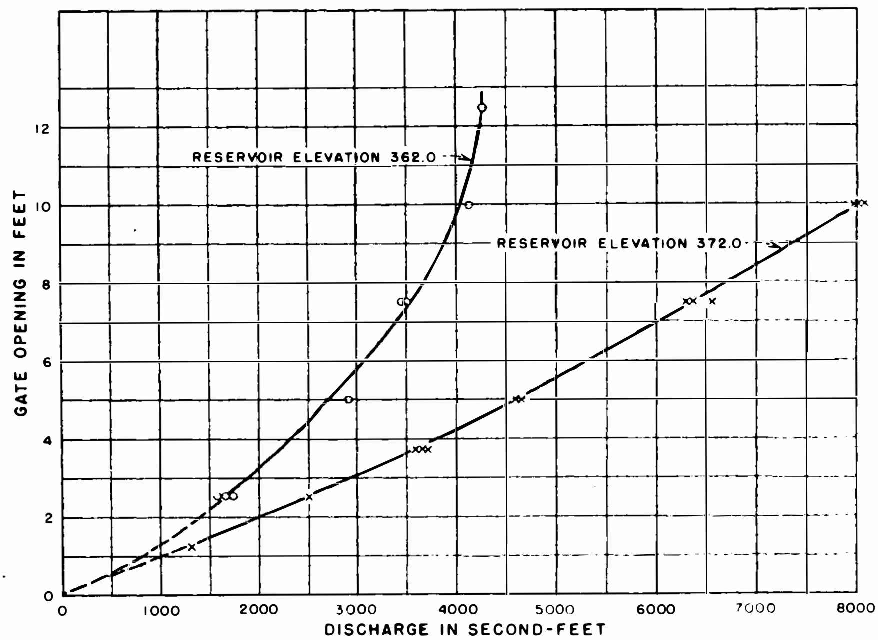
HEADGATE ROCK DAM
HYDRAULIC MODEL STUDIES - SCALE 1:60
DETAILS OF CANAL HEADWORKS

DRAWN D.M.L. SUBMITTED J. E. Warnock
TRACED F.C.-C.I.L. RECOMMENDED Arthur Ruettgers
CHECKED R.A.G. APPROVED J. L. Savage
DENVER, COLORADO, DECEMBER 21, 1938 X-D-2586

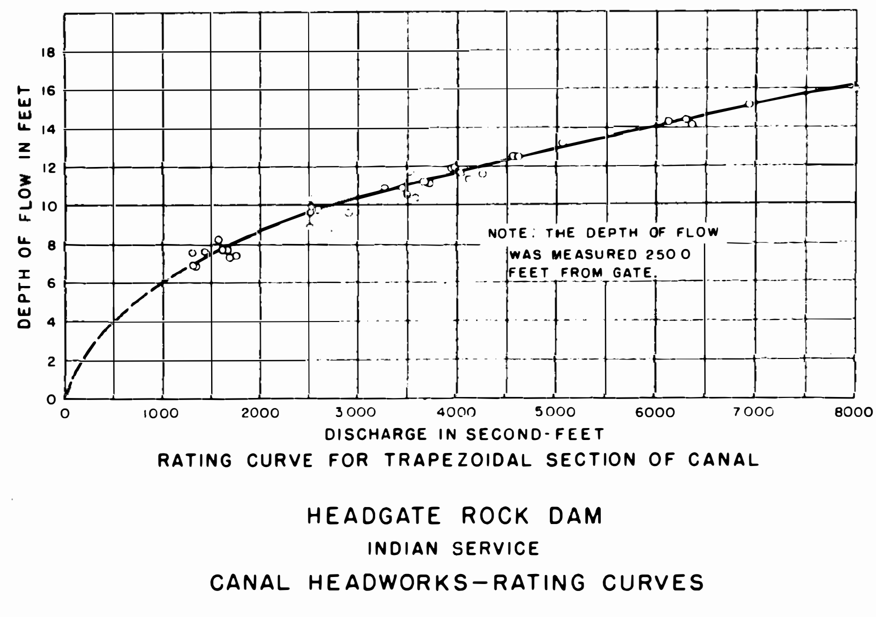
FIGURE 13




RATING CURVES FOR DESIGN A




RATING CURVES FOR DESIGN B, C, AND D




RATING CURVE FOR TRAPEZOIDAL SECTION OF CANAL

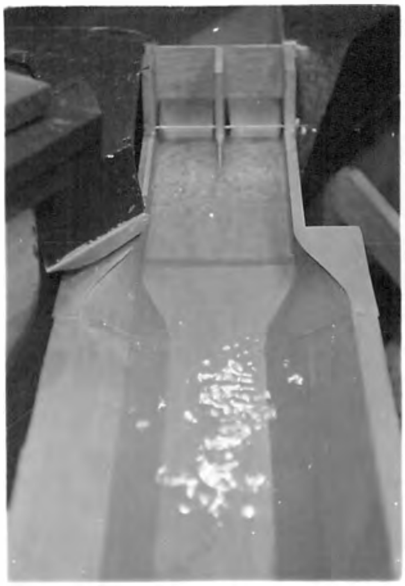
# HEADGATE ROCK DAM

## INDIAN SERVICE

## CANAL HEADWORKS—RATING CURVES

D M L 12-21-38

X-D-2587

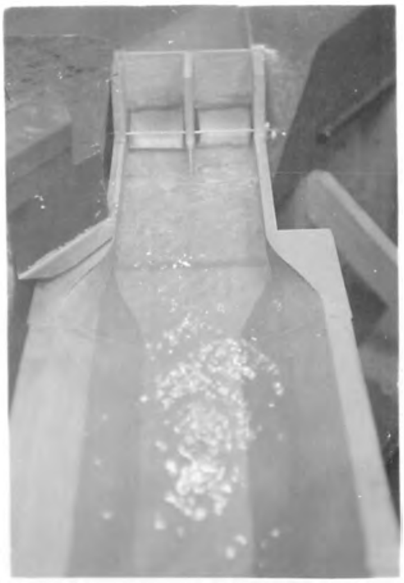


GATES RAISED 2.5 FEET

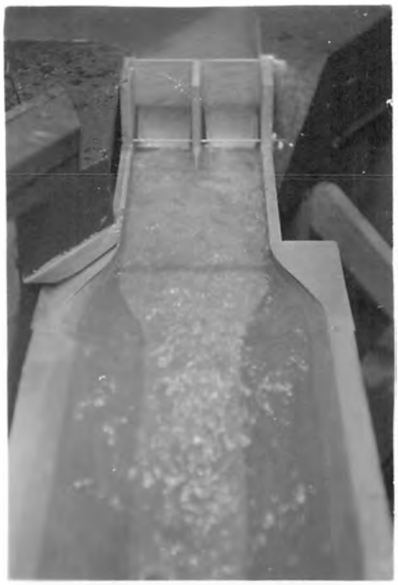
GATES RAISED 7.5 FEET

GATES COMPLETELY OPEN

POND ELEVATION 362



GATES RAISED 2.5 FEET



GATES RAISED 5.0 FEET

GATES RAISED 7.5 FEET

POND ELEVATION 372

DESIGN A - TWO 22-FOOT GATES

CANAL HEADWORK STUDIES

GATE RAISED 2.5 FEET

GATE RAISED 7.5 FEET

GATE COMPLETELY OPEN

POND ELEVATION 362

GATE RAISED 2.5 FEET

GATE RAISED 5.0 FEET

GATE RAISED 7.5 FEET

POND ELEVATION 372

DESIGN D - ONE 40-FOOT GATE

CANAL HEADWORK STUDIES