THE PANAMA CANAL

MADDEN DAM PROJECT

REPORT TO THE GOVERNOR

SUBJECT: MADDEN DAM SPILLWAY MODEL TESTS

Conducted in the Hydraulic Laboratory of the

Colorado Agricultural College at

Fort Collins, Colorado.

By R. R. RANDOLPH, JR., ASST. ENGINEER

Under direction of the

BUREAU OF RECLAMATION OF

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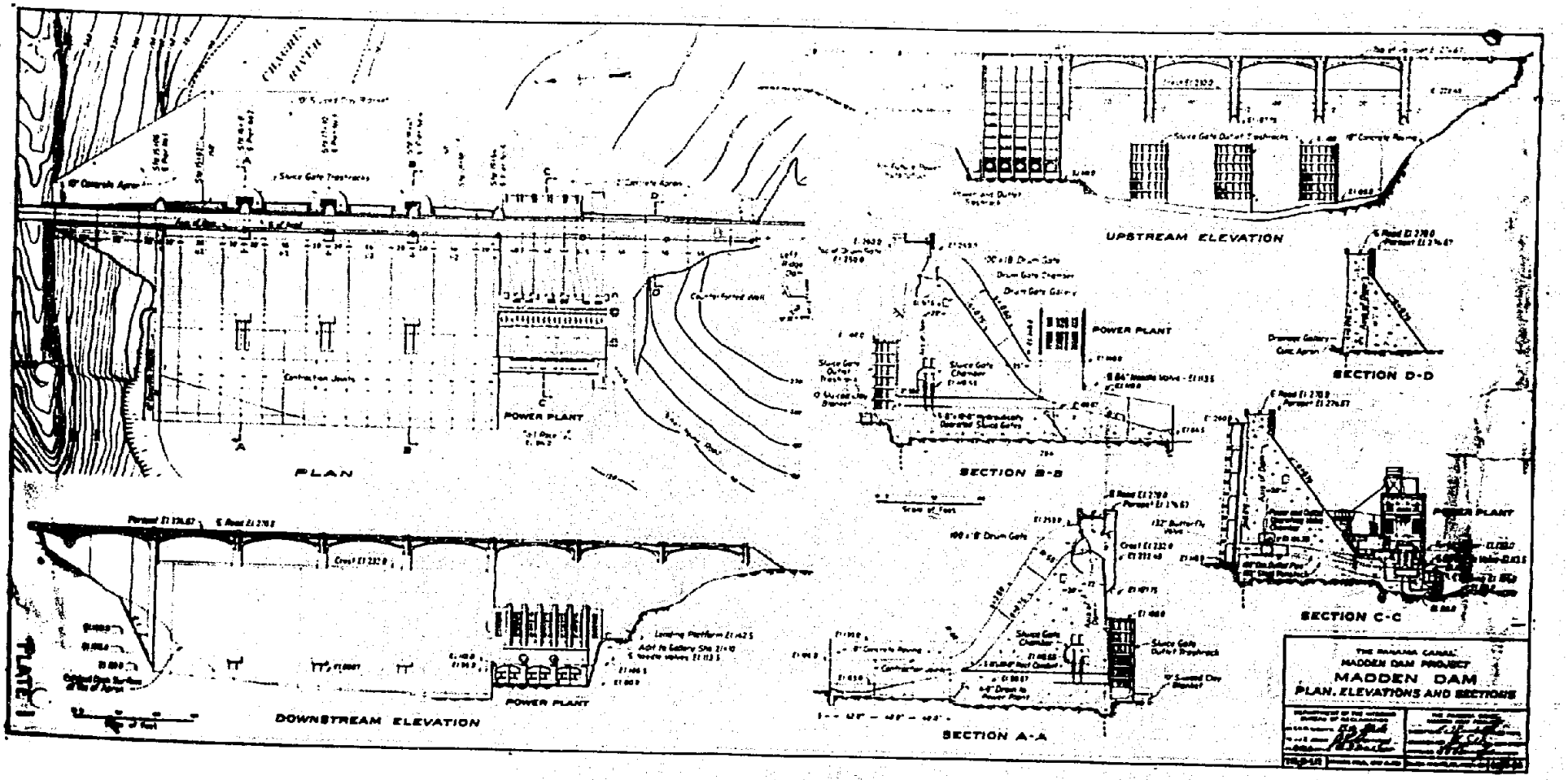

PLAN
POWER PLANT
UPSTREAM ELEVATION
SECTION B-B
SECTION D-D
SECTION C-C
SECTION A-A
DOWNSTREAM ELEVATION
POWER PLANT
THE PANAMA CANAL
MADDEN DAM PROJECT
MADDEN DAM
PLAN, ELEVATIONS AND SECTIONS
PLATE

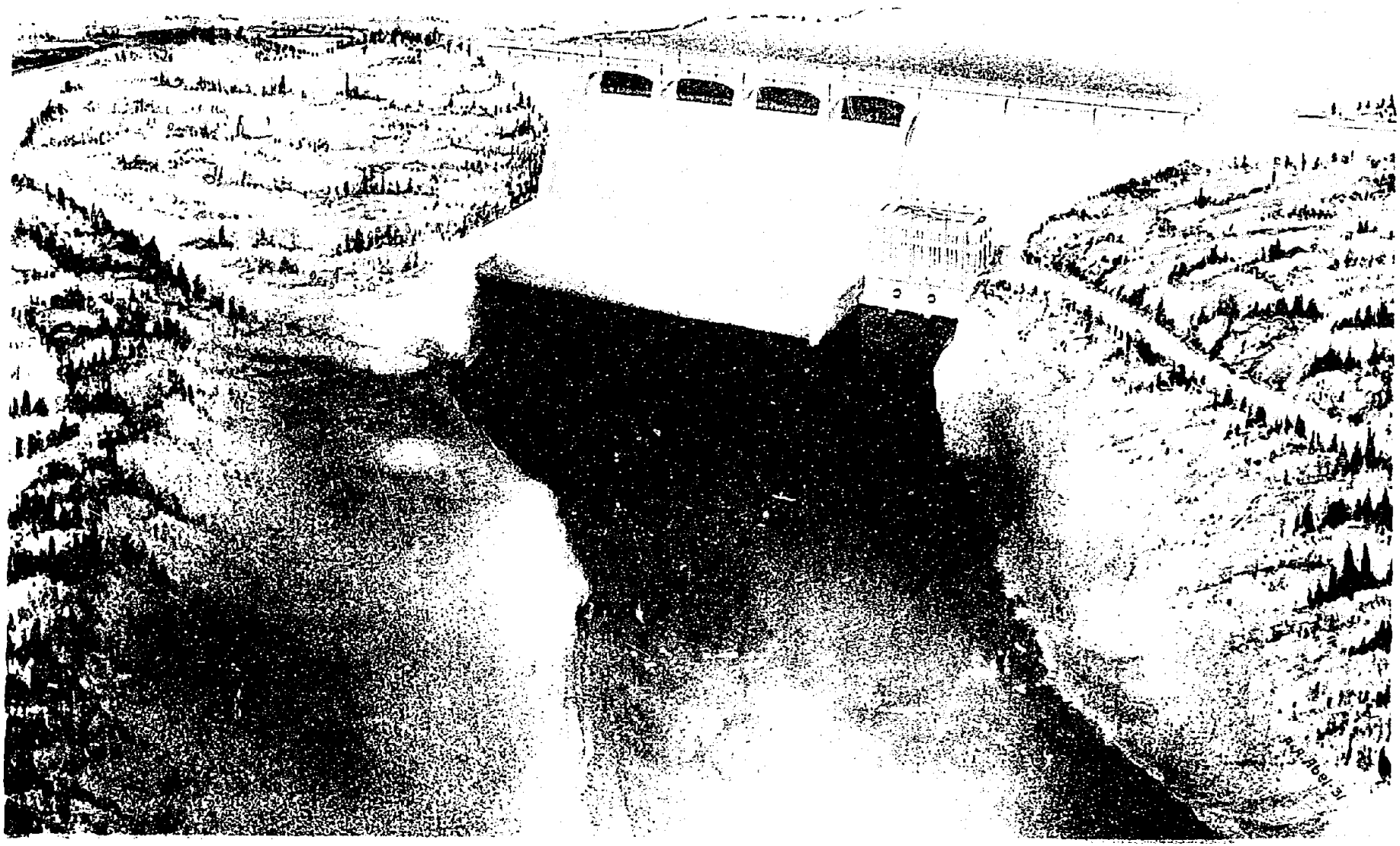
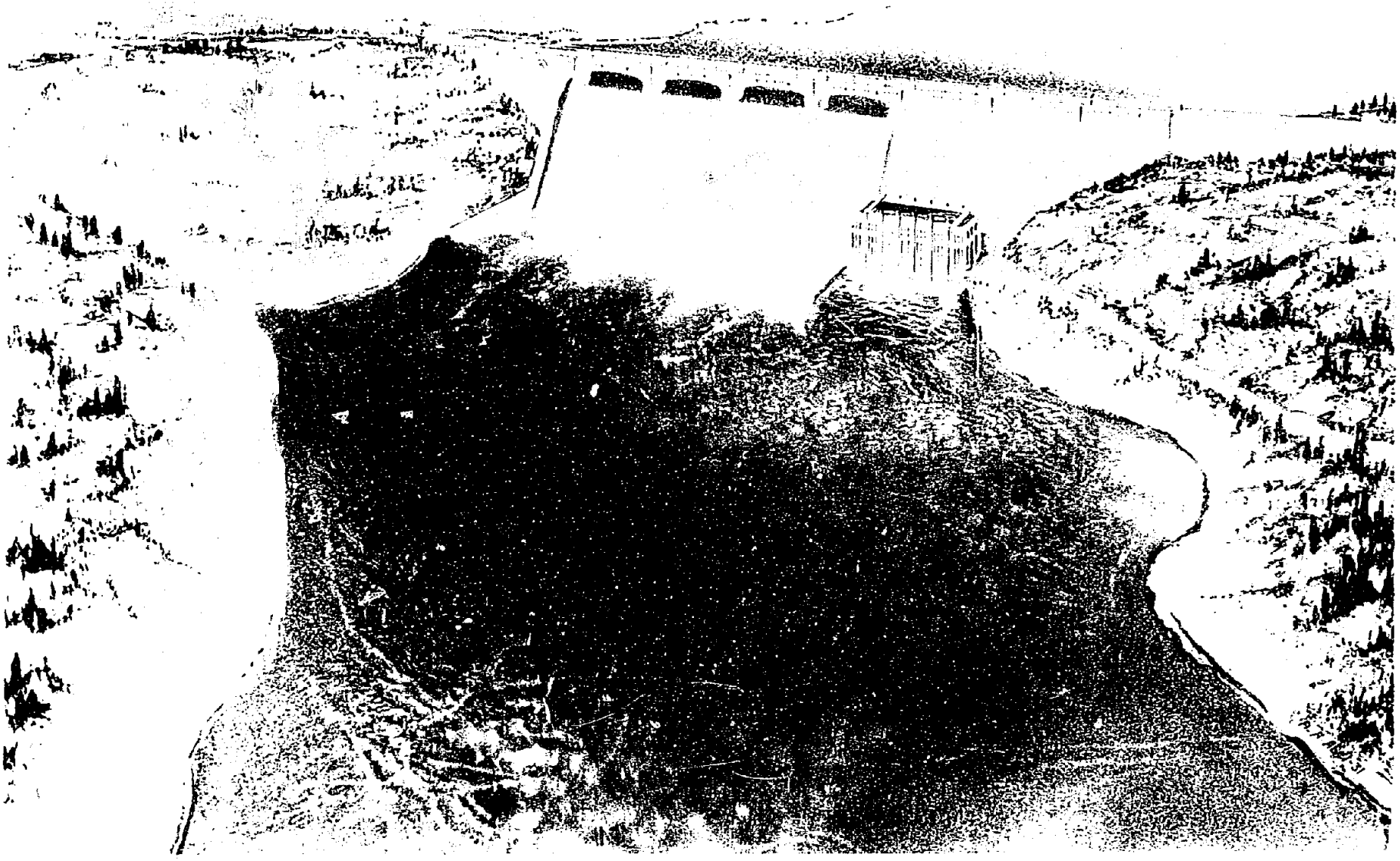


PLATE 1



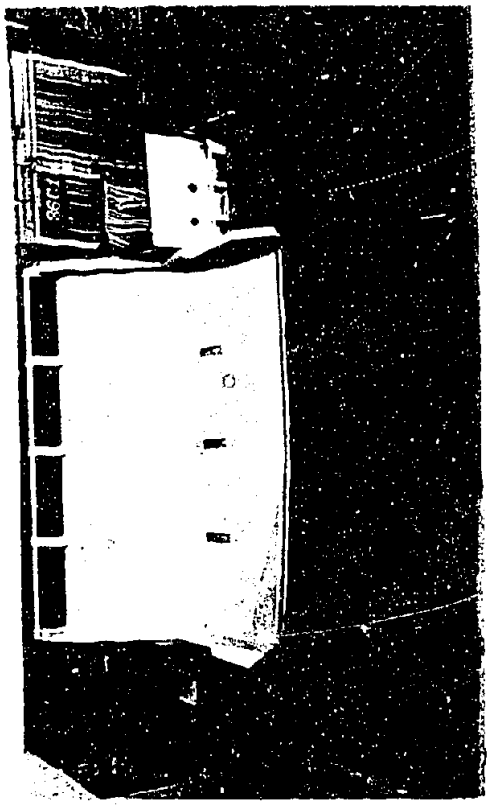


PHOTO 2
SPILLWAY MODEL
WITHOUT TAILWATER

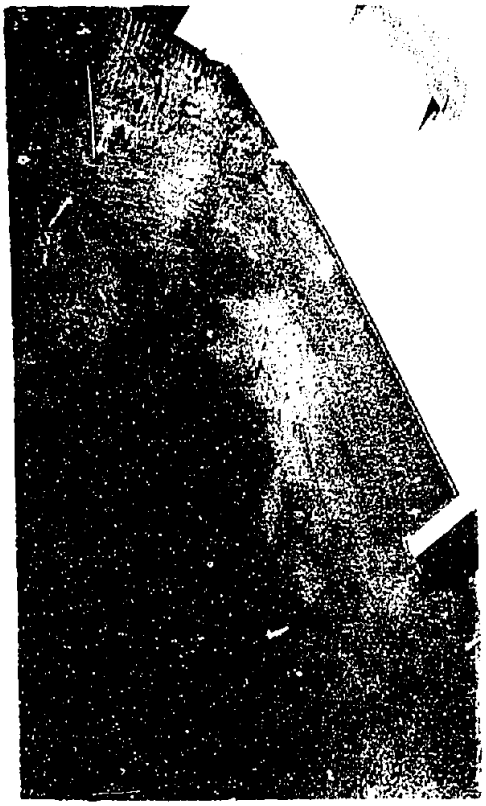


PHOTO 4
LOOKING DOWN FROM BRIDGE
WITHOUT TAILWATER

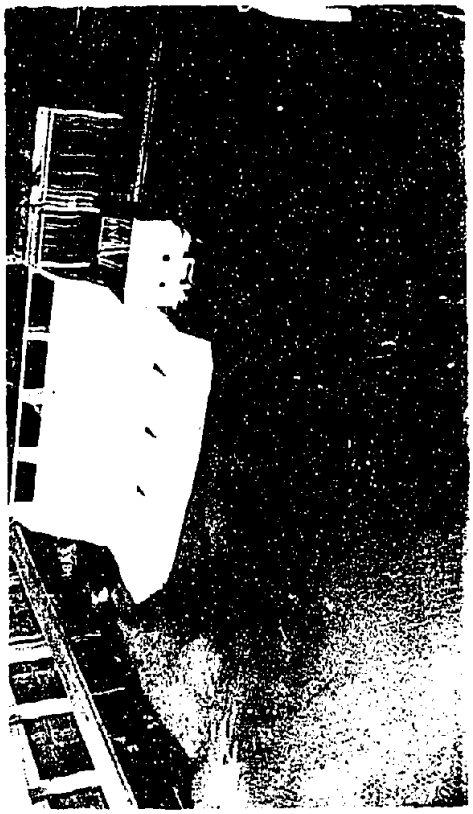


PHOTO 6
RIVER BED AND MODEL
WITHOUT TAILWATER

PHOTO 7
WITH NO FLOW
TAILWATER

FINAL DESIGN-MODEL LAYOUT

PLATE 5

PHOTO 6
FLOW 50,000 C.F.S
JUMP ON APRON

PHOTO 8
FLOW 100,000 C.F.S
JUMP ON APRON

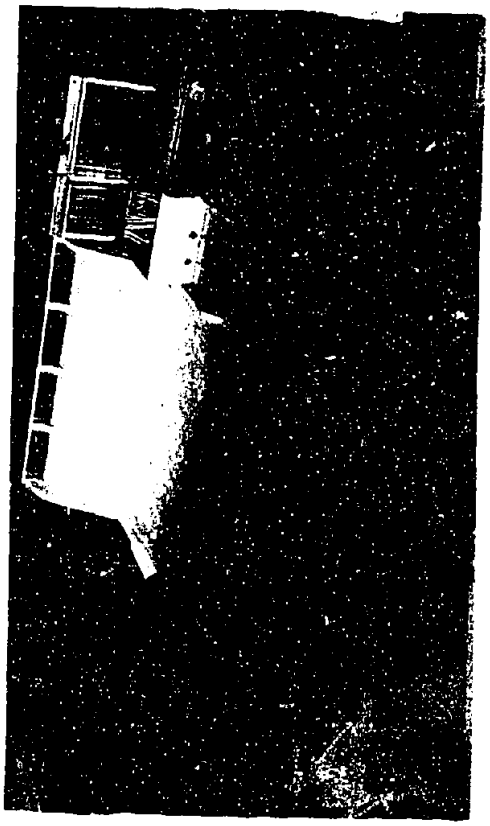


PHOTO 5
FLOW 50,000 C.F.S

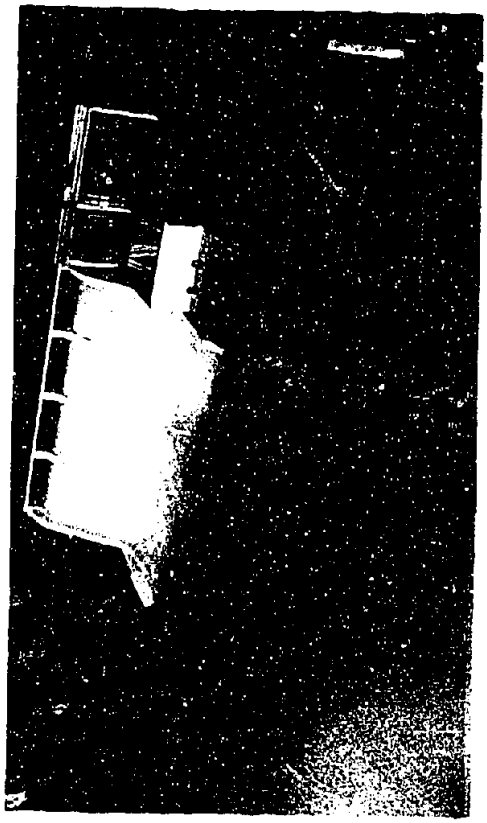


PHOTO 7
FLOW 100,000 C.F.S

FINAL DESIGN PERFORMANCE

PLATE 6

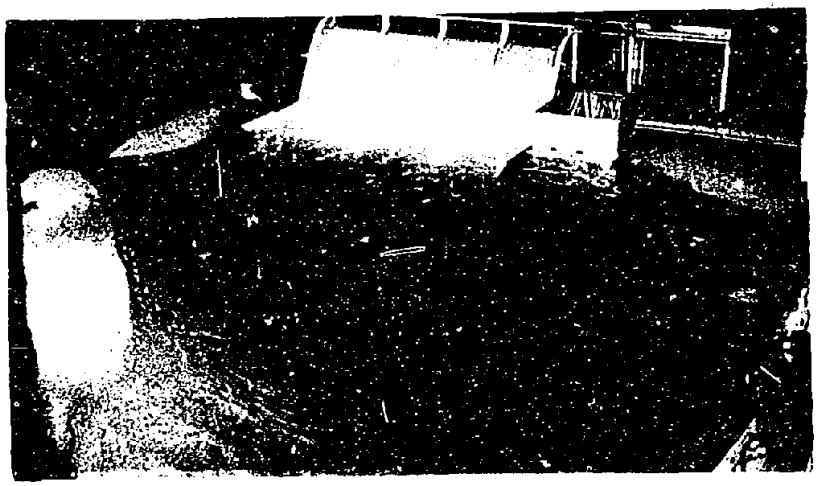


PHOTO 9
FLOW 150,000 C.F.S.

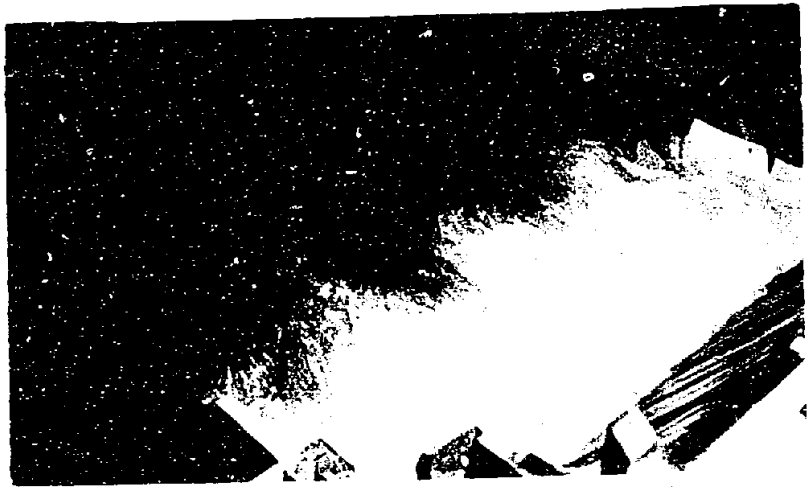


PHOTO 10
FLOW 150,000 C.F.S.
JUMP ON APRON

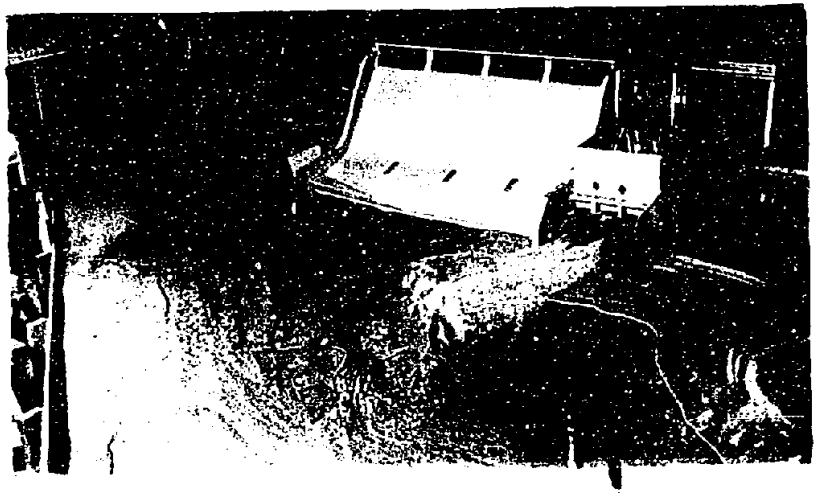


PHOTO 11
AFTER 150,000 C.F.S. FLOW
SAND DEPOSITION

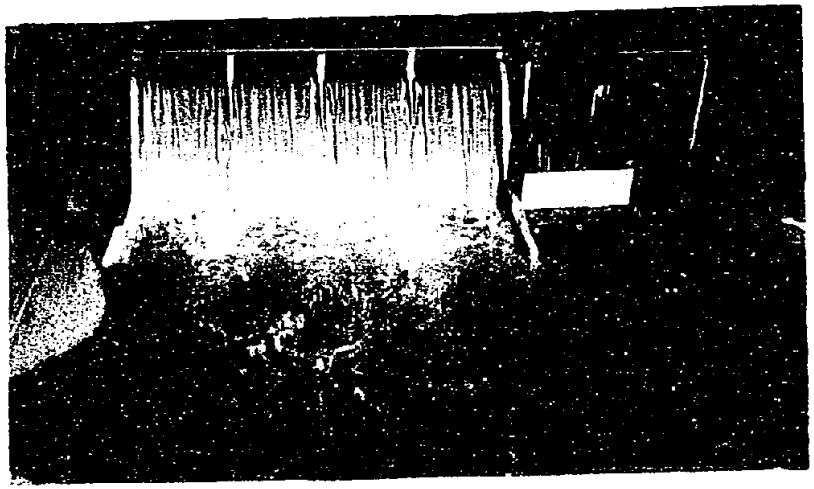


PHOTO 12
FLOW 175,000 C.F.S.

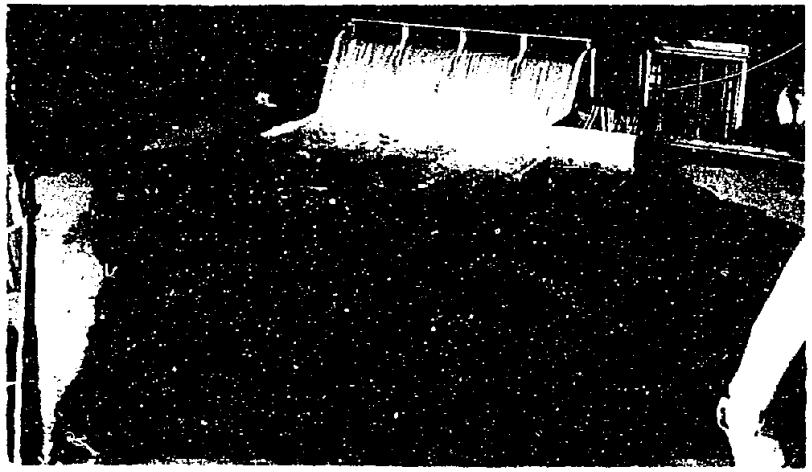


PHOTO 13
FLOW 200,000 C F S

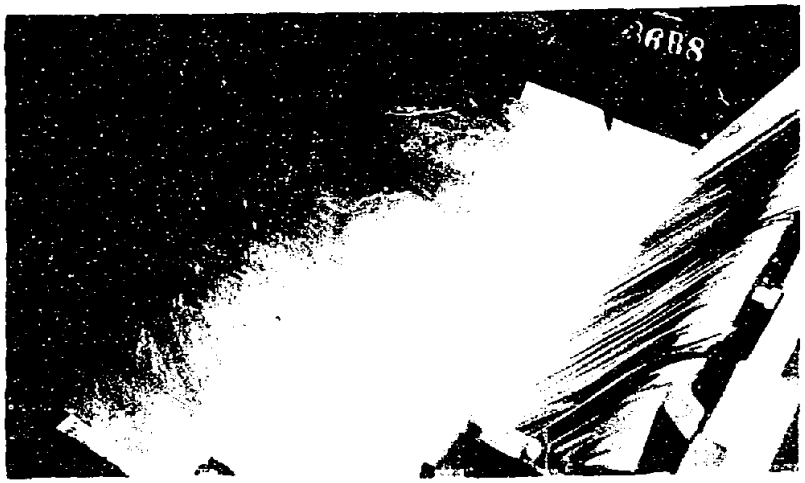


PHOTO 14
FLOW 200,000 C F S
JUMP ON APRON

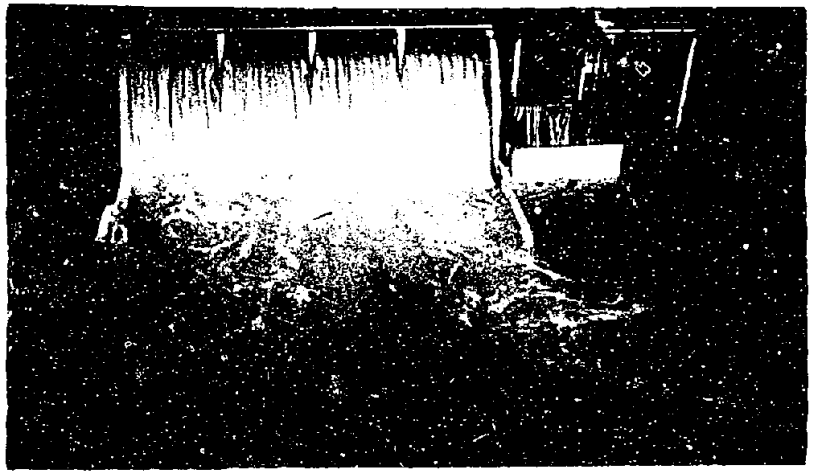
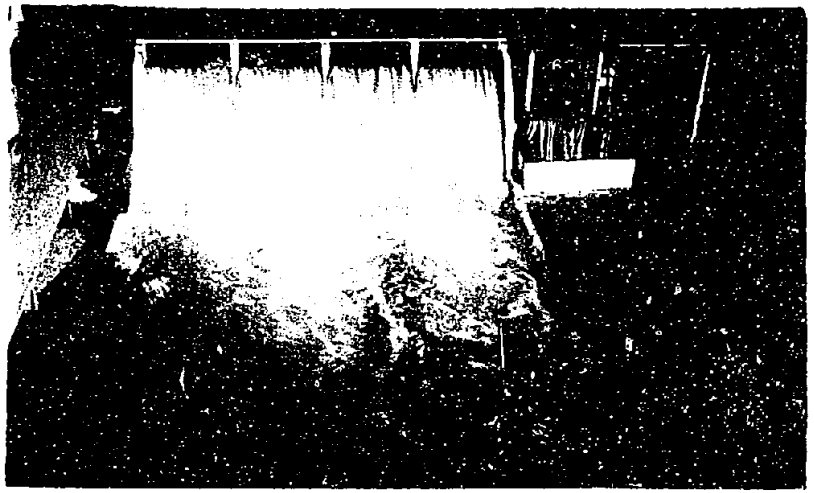


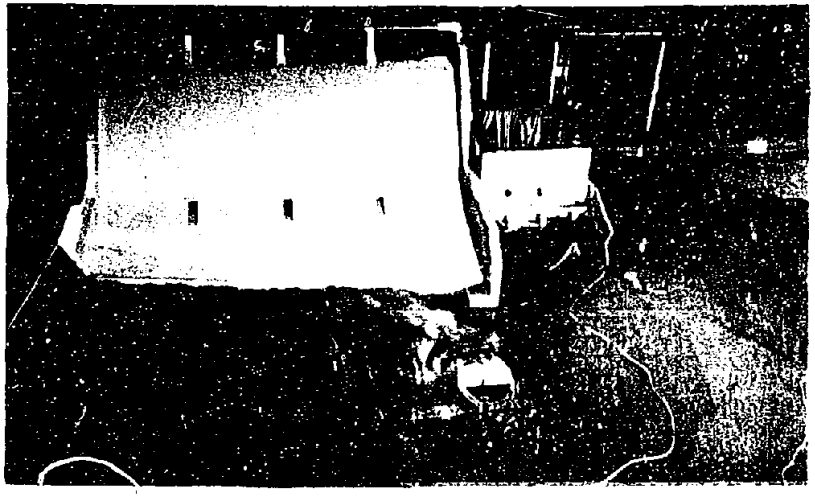
PHOTO 15
FLOW 260,000 C F S

PHOTO 16
FLOW 260,000 C F S
JUMP ON APRON



SLUICES & NEEDLE VALVES OPEN
MAX FLOW OF 280000 CFS

PHOTO 18
JUMP CONDITIONS ON APRON
MAX FLOW OF 280000 CFS



SAND DEPOSITION AFTER
MAX FLOW OF 280000 CFS

PHOTO 20
VIEW OF RIGHT BANK
AFTER FLOW OF 280000 CFS

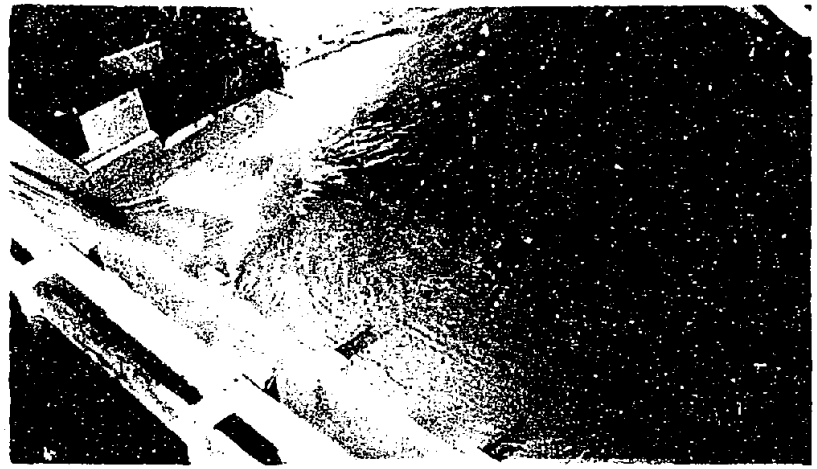


PHOTO 21
GATE NO 1 ONLY OPEN
FLOW 27,000 CFS

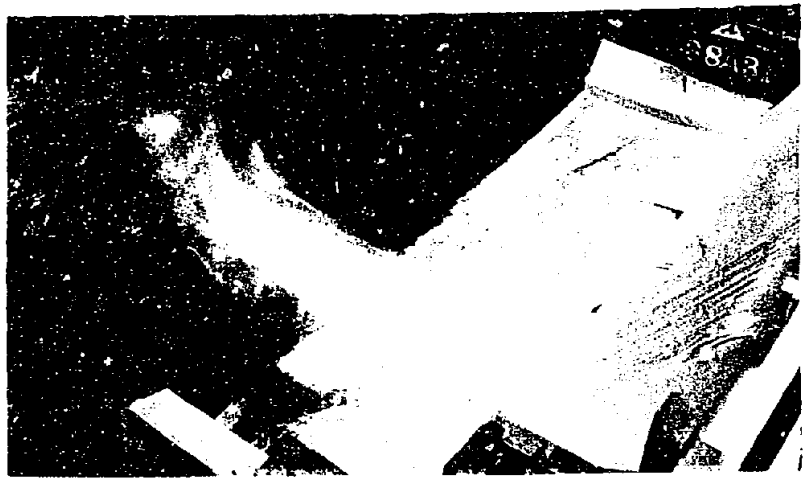


PHOTO 22
GATE NO 2 ONLY OPEN
FLOW 27,000 CFS

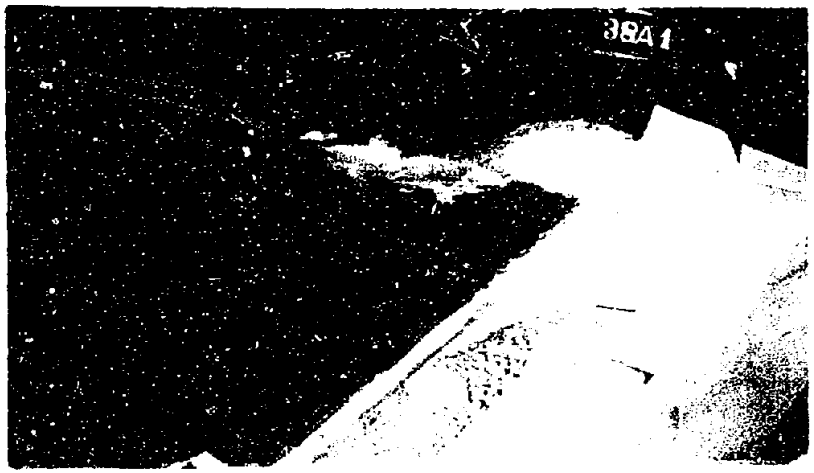


PHOTO 23
GATE NO 4 ONLY OPEN
FLOW 27,000 CFS

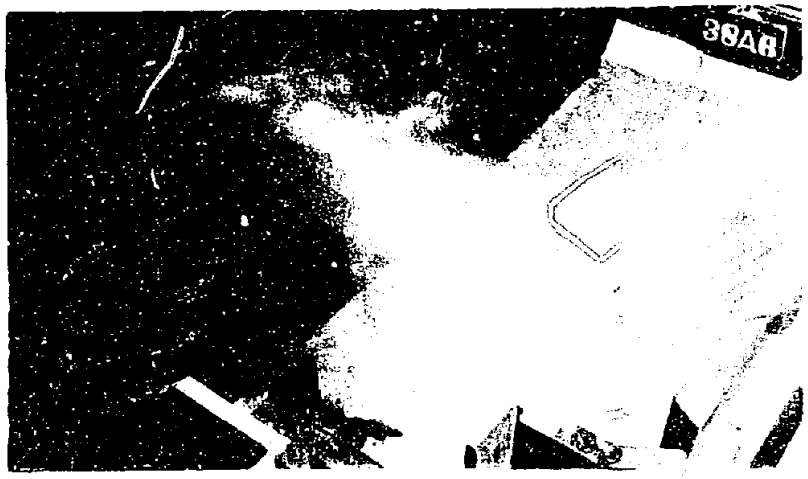


PHOTO 24
GATES 2 & 3 ONLY OPEN
FLOW 54,000 CFS

The pictures showing the sand deposition after the run indicate the nature, though not necessarily the amount, of erosion that may be expected to occur. It is seen that the right bank is pretty well cleaned of the overlying earth and gravel and that the flow cuts a channel along that side leaving the gravel deposits along the left bank practically undisturbed.

The conditions of one or two gate operations are seen to be unsatisfactory. There is a concentration of flow and although the jump does not sweep off the apron, its effectiveness is greatly impaired by the overpour of the adjacent tailwater. In order to secure a balanced effective dissipation of the energy, the drum gates must be operated simultaneously allowing the same quantity of overflow per linear foot of the spillway.

Compare the pictures showing practically the same discharge through two gates and through all four gates; photo 24 with photo 6.

The first two large size views included, plates 2 and 3, are retouched enlargements of photos 3 and 9, from which a general conception of the project can be obtained.

Plate 4 is a photograph of the final model set-up.

SUMMARY

Features of the Design Developed from the Model Tests:

1. Elevation, slope and length of spillway apron.
2. Height and shape of sill at end of apron.
3. Height and length of training wall below power house.
4. Location of sluiceways through the spillway section.
5. Deflecting lip over sluiceway exits.
6. Height and shape of sidewalls down spillway face.
7. Treatment of right bank excavation.
8. Proper operation of drum gates during flood.
9. Location of intake for drum gate control float.

Alterations of the Specification Design Indicated Beneficial by the Tests:

1. Raise power house tailrace division wall to El. 128 and extend 30 feet downstream (Incorporated)
2. Flatten spillway apron slope to 1:4.3 and use concrete to extend apron and right bank paving 30 feet downstream. (Recommended to be incorporated; see page 151.)
3. Build apron in rows of staggered offset steps instead of smooth slope. (Not recommended for adoption.)
4. Place vents down through sidewalls to assure thorough aeration at outside ends of drum gates. (Recommended for adoption; see page 130.)
5. Build earth or rockfill jetty, top at El. 140, out into stream from left bank below power house to prevent reverse flow. (Not recommended for adoption.)

Points of Design Checked by Results from the Model Tests:

1. Height and shape of spillway retaining wall and pavement along right bank.
2. Capacity of the spillway.
   a. Discharge at various heads over crest and positions of drum gates.
3. Shape and location of nose of spillway piers in reference to loss of discharge capacity.
4. Amount and extent of pressures and vacuum on spillway crest.
5. Nappes over fixed crest and over drum gates at various positions.
6. Proper aeration of nappe over the drum gates.
7. Check on pressure or vacuum developed in sluiceways by flows past exits.
8. Check on possible interference with overflow dissipation on spillway apron by abrupt bend in river channel below dam.

Other Data Obtained:

1. Nature of sand and gravel erosion in river bed below spillway.
2. Position and water surface profiles of jump on apron through all stages of the maximum flood.
3. Velocity contours through jump on 1:4 sloped apron at various stages of flow.
4. Pressures on apron at various stages of flow.
5. Photographic record, both moving and still-pictures, of flow conditions at various discharges.

6. Tailwater required for hydraulic jump on level aprons, and various sloped aprons.

7. Velocity contours through flow over dentated sill.

8. Comparison of flow conditions with dentated sill and plain sills. Determination of sill heights.

9. Various flow actions with jump-off bucket.

10. Various flow actions with dentated bucket.

CONCLUSIONS

1. The hydraulic jump is very effective in dissipating the energy of the overflow, the resultant velocities after completion of the jump being little greater than those normally present in the river channel during a given flow, were no dam present.

2. The spillway apron must be placed in respect to the tailwater so that the correct depths over it are obtained in order that the hydraulic jump action may function.

3. The slope of spillway apron should not be made steeper than 1:4.

4. The elevation of the sloped apron as shown on Plate 1 is such that the hydraulic jump will form well up on it at all stages of discharge over the spillway, except at the lower flows there is a tendency for the jump to move down the slope somewhat. Even then, the velocities leaving the end of the apron are comparatively small but it would be of an advantage to lower the start of the slope slightly to pull the jump farther upstream on it. The apron may be lowered as much as 2.5 feet without appreciably changing the conditions of flow.

5. The thickened apron adds to the stability of the dam, affording a greater factor of safety against earthquake movements and decreases the sliding factor.

6. The length of the jump or distance downstream required for full recovery varies as its magnitude, increasing with the discharge.

7. The highest velocities through the jump follow close to the floor until they are gradually retarded.

8. The length of the spillway apron then should cover the range of velocity retardation until a point is reached where the velocities are low enough not to cause scour of the river bed below.

9. The apron length may be shortened by placing a small sloped face sill at the end to lift what velocities that are present just clear of the river bed below until they are retarded in the natural course of the jump action.

10. The sill should be of proper height. If too high, it will receive terrific impact and also lift the high velocity flow from the bottom to the surface where retardation is much slower, and also expose the river banks to erosive forces. If made only high enough to deflect the flow just off the river bed the velocity is quickly retarded by the static pressure of the tailwater.

11. The apron as shown on Plate 1 does not extend for the full length of the jump occurring at the higher flows but covers the range of great turmoil characteristic of the beginning. The velocity leaving the end of the apron may exceed 50 feet per second at the maximum flow but is lifted by the sill from the river bottom below until retarded in the jump. The hillside along the right bank, protruding across the line of flow, however, is exposed to these velocities. The greater the length of apron the more protection is gained, of course, but it would seem advisable to extend the apron at least thirty feet farther downstream. Little or no additional cost is necessary as the concrete can be taken from the apron by flattening its slope which would be beneficial.

12. If the floor of the apron is roughened by building it in steps very little resultant loss in velocity is recorded although the tendency of the jump to move its position on the slope for tailwater variations is lessened. Small size steps seem to be as effective as larger ones, although any steps should be built in staggered offsets to overcome the possibility of setting up a periodic vibration. The whirls formed behind the steps increase the possibility of cavitation effects and pitting of the concrete. The form work required would be additional expense. Altogether the advantages to be gained by their use in this set-up were so small that no recommendation was made for their adoption in the design. They may be used to greater advantage in an installation where there is a greater variation between the depth required for the hydraulic jump and those actually furnished by the natural tailwater.

13. The jump should be protected or enclosed by vertical side-walls at a height, at least, following the rise in water surface through the jump, to prevent "washing out" by the inpour of the adjacent tail-water. It would be better if the walls were high enough to exclude the adjacent tailwater altogether. The walls should extend downstream until full recovery of the jump where the water surface levels off to the same elevation as that of the adjacent tailwater.

14. The power house wall on the left side of the spillway should be raised to El. 128, which about follows the water surface through the jump at maximum discharge. There will still be some overpour onto

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14. The power house wall on the left side of the spillway should be raised to El. 128, which about follows the water surface through the jump at maximum discharge. There will still be some overpour onto

the top of the jump but no particular harm seems to result. The wall should be extended at least 30 feet farther downstream than shown on Plate 1 to prevent an unbalanced condition caused by the cross flow set up when the eddy occurs. This eddy or whirl is caused principally by the flow from the higher adjacent tailwater surface to the lower elevation of the water in the jump but is accentuated by the reverse flow of the river occurring along the left bank. The reverse flow upstream will pick up sand from the river bed below and deposit it in the draft tube exits and the eddying will cause a pulsation of the tailwater which is detrimental to the speed regulation of the turbines. The wall as finally recommended is of the least size that will afford ample protection at the maximum discharge but will cause absolute elimination of the eddy for flows up to 160,000 c. f. s. corresponding to a 160-year flood.

15. With the protection of high walls on either side of the apron there is a balanced, effective hydraulic jump produced at all flows. Although the apron rises on the right side and the river bank below protrudes out across the jump there is no indication of a concentration of flow at that point and the dissipation seems to be as effective there as on the left side. However, at the maximum flow of 280,000 c. f. s., indications are that this hillside will be subjected to velocities as high as 35 feet per second as an average over the whole area and 50 feet per second just below the end of the apron, but if the bank washes back a better jump is formed and the velocities are

lessened. At flows up to about 150,000 c. f. s. the bank will be subjected to velocities less than 10 feet per second.

16. It may be expected that the gravel deposits in the river channel and the earth overburden on the right bank will be washed a considerable distance downstream during a flow of any magnitude. These deposit as a bar across the channel downstream, through which the receding flood waters cut a passage along the right bank. No detrimental effect on the performance of the jump was noticed.

17. Prototype performance similar to that of the model may be reasonably assured due to the well established theory and existing proofs of the hydraulic jump action at various velocities and depths. The Madden Dam model was built and tested with precision and care. The surfaces exposed to the flow of water were practically frictionless so that any friction existing on the prototype structure will be on the safe side. Due to the sloping apron even at the maximum flow the tailwater may be lowered as much as twenty feet below normal before the jump swoops off enough to dangerously increase the velocities at the end of the apron.

18. The dentated sill or any wall of concrete placed across the apron will, of course, check the velocity of the overflowing sheet of water. Such a sill, as was used in the original design with the apron at El. 60, will receive tremendous impact as the high velocity carries to the floor with little retardation. This calls for a heavy sill structure with reinforcement and anchorage. The whole dependence for

energy dissipation is then placed on the sill, a small break in which would endanger the entire scheme. The dentated sill prevented any serious erosion from occurring below the apron but a vertical faced solid sill of lesser height was almost as effective.

19. The discharge capacity of the crest is ample. For a maximum pond elevation of 263 the crest will discharge better than 255,000 c. f. s. and with the capacity of the needle valves and sluices at 30,000 c. f. s. the requirement of 280,000 c. f. s. discharge is exceeded. However, with the drum gates raised to their maximum position at El. 250 and the pond at El. 263 the flow can only be throttled to 70,000 c. f. s. The pond must recede to El. 260 or less before the flow can be regulated to 50,000 c. f. s.

20. There is a vacuum developed on the downstream face of the crest amounting to about 12 feet of water at the maximum point for a head of 31 feet over the crest. This, presumably, has been taken into account in the stability design. Pressures on the upstream face near the top decrease as the gate is lowered, under a given head. This means that the intake for the drum gate control float will have to be located at a point in the bulkhead face away from the spillway drawdown. Pressures on the crest obtained from the experiments checked very closely those computed by the flow-net method.

21. There will be practically no side thrust on the spillway training walls built down the face of the dam. The velocity of the jet carries it almost to the toe before there is enough spread to

cause an impact on the wall and then the jet is very thin and the force of the impact is near the base of the wall.

22. With a deflecting lip of the proper radius over the sluice openings through the face of the dam the overflow from the spillway is deflected tangentially enough so as not to cause direct impact on the floor or the sluiceway. However, for all stages of flow a piezometer in the closed sluice conduit indicated a pressure showing that the jet from the deflector is depressed by the weight of the tailwater over it. With no tailwater over it there was a vacuum in the closed conduit and indications were that it would take a 3-1/2-foot diameter pipe to aerate the jet so that it would leap entirely clear of the floor. No concentration of flow or disturbance to the jump performance is caused by the sluice openings through the face of the spillway.

23. The drum gates should be operated simultaneously for the best jump performance. When one or two gates are operated separately the jump on the apron is to an extent drowned out by the overpour of the adjacent tailwater and a concentration results causing dangerous erosion at the end of the apron.

24. Some other devices for dissipating the overflow were tried but did not prove practically successful. One of these was a high bucket with a drop off to the rock line. When the bucket is placed at the correct elevation in the tailwater, the back pressure deflects the overflowing jet upwards thus relieving the river bed of any impact.

There is a back-roll created underneath the lip of the bucket which, in the tests, brought sand up from the river bed below and deposited it against the face of the bucket, forming a slope from the bucket lip to the apron. The overflow jet, then, released of the back-pressure at the lip of the bucket, was deflected downward along the slope, striking the river bed with full force and producing dangerous erosion. Due to the instability of performance this set-up was not considered feasible.

25. Dentates or slots were cut in this high bucket down to the original apron level. This killed the spray action of the higher, solid, bucket and both jets were deflected down to the apron with serious erosion resulting in the river bed below. It would seem advantageous to make the slots or dentates narrower.

26. A pocket or channel cut in the apron along the point of tangency of the bucket proved unsuccessful in breaking the energy of the overflowing water. Further investigation might have developed a shape of pocket in which a whirl could be created to act as a drag underneath on the overflowing sheet, but it was decided that it would be detrimental to the stability of the dam to take this concrete from the apron and the experiments were discontinued.

MADDEN DAM
SPILLWAY MODEL TESTS

OUTLINE

I The Project

A Purpose

B Design

1 Spillway layout - Drawing 213-D-212 - Plate 1

Retouched enlargement of photos 36D1 and 36A6 - Plates 2 and 3

Retouched enlargement of photo 36A1 - Plate 4.

2 Object of spillway experiments

II The Laboratory

A Layout showing location of model - Plate 2

B Measuring weir

1 Discharge curve -- Fig. 1 - Plate 12) Plate 12

2 Prototype discharge and natural tailwater curves)

III The Model

A Scale - Prototype relations

B Details - Plate 13 and Plate 14

1 Original apron - Photo C - Plate 15

Photo 19C1 - Plate 15

2 Final apron - Photo 1 - Plate 15

Photo 4 - Plate 15

III (Cont.)

C Apparatus - Plate 16

1 Forebay head gage - Photo 36A1

2 Piezometer gage boards - Photo 36D1

3 Forebay point gage - Photo 25A0

4 Tailwater gage - Photo 16E13 and 36D6

5 Regulating tailwater weir - Photo 6

6 Automatic tailwater weir - Photo 16E13

7 Pitot tube - Photo 5

IV The Dentated Sill Apron

A Apron level at El. 60 (original design) - Drawing 213-D-69 - Plate 17

1 Record of flow

a No flow - Photo 19C1 - Plate 15

b 260,000 c. f. s. - Photo 20A2 - Without sill - Plate 18

c 260,000 c. f. s. - Photo 20A4 - With sill - Plate 18

2 Analysis of flow (260,000 c. f. s.)

a Stream lines - Photo 6A10 and 6B10 - Plate 18

b Velocity contours - T. 20 - R. 4 - Plate 19

c Velocity against sill - T. 20 - R. 2 - Plate 19

B Apron level at El. 70, El. 60, El. 70 and El. 80 in respective bays

1 Record of flow (260,000 c. f. s.)

a Separated gate operation - Photos 9A3 and 8A23 - Plate 20

b Dentated sill - Photo 8 - Plate 20 - Photos 8A2 and 8B2 - Plate 21

c Turn-up sill - Photos 8A3 and 8B3 - Plate 21

C Apron level - Rising from El. 60 to El. 80 max.

1 Erosion after max. flow - Photo 13A1 - Plate 20

V The Hydraulic Jump Apron - Development to Obtain Correct Elevation

A Apron A - Apron level at El. 60 - No sill

1 Tailwater depth variations

a Depth giving greatest energy dissipation

1 Velocity measurements - T. 19 - R. 17, 18, 19 - Plate 24

2 Observation - T. 2 - R. 4, 8, 12, 16, 20, 24, 28, 29, 30 - Plate 23

3 Computation - Plate 25

b Surface conditions and velocity profiles on apron

1 80-ft. depth - Photo 20A2 - Plate 18 - T. 20 - R. 2 - Plate 19

2 70-ft. depth - Photo 19D1 - Plate 22 - T. 19 - R. 1 - Plate 25

3 55-ft. depth - Photo 19A2 - Plate 22 - T. 19 - R. 2 - Plate 25

4 50-ft. depth - Photo 19A3 - Plate 22 - T. 19 - R. 3 - Plate 25

5 No tailwater - Photo 19B1 - Plate 22

B Apron B - Apron level at El. 80

1 Jump observations - Test 5 and Test 6 - Movie T5R1

2 Jump computations - Plate 26

3 Apron lengths and sills

C Apron C - 1:6.6 sloped apron starting at El. 84

1 Adjustment of tailwater to obtain hydraulic jump - T. 10 - R. 8-12 - Plate 27

a Typical jump - Photo 10A13 - Plate 28

b Erosion - Photo 10B13 - Plate 28

2 Separate gate operation

a Erosion caused - Photos 11A4 and 11A3 - Plate 28

V (Cont.)

D - Apron D - 1:4 sloped apron starting from level bucket at El. 90

1 Tailwater adjustment to obtain jump - Test 4 - Runs 1-5 - Plate 29

E - Apron E - 1:4 sloped apron starting at El. 97.3 (final design)

1 Tailwater adjustment to check jump action

a Observation - Test 18 - Runs 27-32 - Plate 32

b Min. velocity measurements - Plate 34

Crest constant - Pond varied - Sec. centerline gate 4 - T. 30 - R. 1-6 - Plate 30

Crest constant - Pond varied - Sec. centerline gate 2 - T. 33 - R. 1-6 - Plate 31

Gates varied - Pond constant - T. 23 - R. 1, 5, 9, 13, 17 - Plate 33

For erosion, see test 24 following

2 Slope variation

a 1:2 slope - Photo 14A7, 14 and A8 - Plate 50

b 1:4 slope constant across apron

1 No flow - Sand bin - Photo 18B49 - Plate 35

2 After max. flow - Sand erosion - Photo 18D49 - Plate 35

c Apron rising at right side

1 After maximum flow - Sand erosion - Photo 18B51 - Plate 35

2 After flow - Sand erosion - Contour block removed - Photo 18B48 - Plate 35

V (Cont.)

3 Sill variation

a 6-ft. sill (same as under b and c above)

b 4.5-ft. sill

1 After max. flow - Sand erosion - Photo 18B50 - Plate 36

With P. H. wall raised to El. 140 - Photo 18B52 - Plate 36

4 Apron extension (see also under VIII-c)

a 20-ft. extension

6-ft. sloped face sill above and below T. 18 - R. 7, 9, 11, 12, 13

6-ft. vertical face sill above T. 18 - R. 13

(Observation data not included in report)

5 Position of jump under depths higher and lower than normal

a No tailwater - Photo 18A53 - Plate 36

b Tailwater 5 ft. below normal - Photo 18A54 - Plate 36 - T. 18 - R. 54 - Plate 38

c Tailwater 10 ft. below normal - Photo 18B55 - Plate 37 - T. 18 - R. 55 - Plate 38

d Tailwater 20 ft. below normal - Photo 23C1 - Plate 37

e Tailwater normal - Photo 23A1 - Plate 37

f Tailwater 20 ft. above normal - Photo 23B1 - Plate 37

g Tailwater erosion - 18B50, 18B52 - Plate 36 and 18B48, 18B49, 18D49, 18B51 - Plate 35

For other erosion runs, see test 24, under stepped apron

V (Cont.)

6 The stepped apron

a Various size steps - Fig. 1 - Plate 39

b Velocity contours through jump - Test 22 - Runs 4 and 5 - Figs. 2 and 3 - Plate 39

c Tailwater variation - Test 22 - Run 15 - Plate 40

d Steps 2.25' x 9.2'

Erosion, gates various positions - Sand bin contours before run - Plate 86

Pond constant at El. 263 -

Gate at El. 232 - No sill - Photo 24C1 and 24D1 - Plate 41

Erosion contours - Plate 86

Gate at El. 232 - With 6-ft. sill - Photo 24C6 and 24D6 - Plate 87

Sand bin erosion contours - T. 24 - R. 6 - Plate 44

Gate at El. 232 - With 4.5-foot sill

Tailwater normal - Photos 24B7 and 24C7 - Plate 42

Erosion contours - Plate 88

Tailwater 12' below normal - Photo 24A28, 24B28 - Plate 42

Erosion contours - Plate 89

Tailwater 12' above normal - Photo 24A29, 24B29 - Plate 43

Erosion contours - Plate 90

Gates at El. 236 - Erosion contours - Plate 91 - Photos 24A8 and 24B8 - Plate 43

V (Cont.)

Gates at El. 240 - Erosion contours - Plate 92 - Photos 24A13 and 24B13 - Plate 44

Gates at El. 245 - Erosion contours - Plate 93 - T. 24 - R. 13 - Photos 24A18 and 24B18 - Plate 44

Gates at El. 250 - Erosion contours - Plate 94 - T. 24 - R. 18 - Photos 24A23 and 24B23 - Plate 45

Gates at El. 232 - P. H. wall extended 60 ft. downstream - Erosion contours - Plate 95 - Photos 24C30, 24D30 - Plate 45

7 The hydraulic brake pocket - Photos 21A1 and 21A10 - Plate 47

a Velocity profiles on apron - T. 21 - R. 9 - Plate 46

VI The Jump-Off Bucket

A Stages of flow - Bucket at El. 90

1 Best jump conditions - Photos 3B2 and 3A2 - Plate 48 - T. 3 - R. 2, 5, 8, 12 - Plate 51

a Stream lines photo - Plate 49

2 Jet sprayed upward - Photo 3B4 and 3A4 - Plate 48 - T. 3 - R. 1, 4, 9, 13 - Plate 51

a Stream lines photo 14A3 - Plate 49

3 Jet deflected downward - T. 3 - R. 3, 6, 10, 14 - Plate 51 - Photo 5A6 - Plate 49

a Stream lines - Photo 14A2 - Plate 49

b Against dentated sill - Photo 14A5 - Plate 49

VI (Cont.)

B Bucket at El. 80 - Photo 5A6 - Plate 49

C Bucket at El. 87 - 1:2 slope to apron

Sill at foot of slope - Photo 14A7 - Plate 50

Sill on slope - Photo 14A8 - Plate 50

D Dentated bucket - Photo 7A1 and 7A3 - Plate 50

VII Drum Gate Discharge and Operation

A Coefficient of discharge curves

1 4 - 100-ft. openings - T. 24 - R1-27 - Plates 52, 53 and 54 - Tests 28, 29 and 30 - Plate 58

2 Pier contraction suppressed - T. 23 - R. 36-60 - Plates 56 and 57

a Flat gates - T. 23 - R. 61-91 - Plates 55 and 57

B Nappes and pressures on crest

1 Crest at El. 232 - T. 29 - R. 1-6 - T. 30 - R. 1-6 - Plate 60

Gates at El. 232 - T. 25 - R. 1-6 - Fig. 1 - Plate 59

Gates at El. 232 - Comparison with computed values - Plate 61

2 Gates at El. 240 - T. 25 - R. 7-10 - Plate 59

3 Gates at El. 250 - T. 25 - R. 11-13 - Plate 59

C Nappes and pressures along side walls

1 Gates at various positions - Pond constant at El. 263 - Photos 17A1 and 17B1 - Plate 47

Test 14 and Test 17 - Plate 62

VII (Cont.)

D Simultaneous gate operation

1 See photo of test 24 - Plates 41 to 45

E Separate gate operation

1 See photos 8A23 and 49A3 - Plate 20 and 11A3 and 11A4 - Plate 28

2 Photo - Test 38 - Plate 10

VIII Sluice Openings and Deflectors

A Types - Photos 18A22 and 18A35 - Plate 63

Photos 31A1 and 31A3 - Plate 63

1 Dimensions - Plate 66

B Operation - Photos 18F26 and 18G26 and 18A24 and 18B24 - Plate 64

T. 18 - Runs 20, 21 and 24 - Plate 65

T. 18 - Runs 22, 25 - Plate 66

IX Right Bank Excavation

A Cut No. 1 - Photos 15B1 and 15D1 - Plate 68

B Cut No. 2 - Photos 36E11 and 36K11 - Plate 9

1 Flow conditions and velocities - Photos - Test 36 - Plates 5-10 - T. 34 - R. 1 and 2 - Plate 69

IX (Cont.)

C Apron extension and alteration

1 Sand erosion with and without extension - Photos 18A13 and 18A19 - Plate 68 - Test 18 - Runs 13 and 19 - Plates 96 and 97

2 Proposed layout - Plate 72

a Effect of lowering apron - Plate 71

X The Power House Wall

A Views showing flow conditions

	Without Wall	Low Wall	High Wall	Plate No.
1 Set-up	32B3 Pl. 75	29C6 Pl. 74	36A1	4
		4 Pl. 15	33A0	76
2 150,000 c. f. s. flow	---	29A3	29A9	73
3 150,000 c. f. s. - Erosion	---	29B3	36F6	73
4 260,000 c. f. s. - Close-up	---	27A2	(27B6 (18B52 (18A24	75 36 64
a Wall extended downstream	---	24C30 24D30 Pl. 45		
5 280,000 c. f. s. flow	32A3 Pl.75	29A6	29B12 36C11	74 9
6 280,000 c. f. s. - Erosion	32B3 Pl. 75	29C6	29D12 36F11	74 9

X (Cont.)

B Energy and hydraulic gradients along apron

1 Low wall - T. 29 - R. 1, 3, 5 - Plate 70

2 High wall - T. 29 - R. 7, 9, 11

XI The Final Design

Photo 7 and 38A0 - Plate 76

A Views showing flow conditions

	Looking Upstream	Closer View	Looking Down from Roadway
1 No flow	Plate 5 36A1	36D1 36C1	36B1
2 50,000 c. f. s.	Plate 6 36A2	----	36B2
3 100,000 c. f. s.	Plate 6 36A4	----	36B4
4 150,000 c. f. s.	Plate 7 36A6	----	36B6
a Erosion	Plate 7 36D6	36F6	36E6
5 175,000 c. f. s.	Plate 7		
6 200,000 c. f. s.	Plate 6 36A8	----	36B8
7 260,000 c. f. s.		36C10 Pl.8	36B10
8 280,000 c. f. s.	36C11	36C11 Pl.9	36K11
a Erosion		36F11 Pl.9	36E11
8 27,000 c. f. s.	38A4, 38A3, 38A1 - Plate 10 - One-gate operation		
9 54,000 c. f. s.	38A6 - Plate 10 - Two-gate operation		

XI (Cont.)

B Velocity profiles through jump

1 In 50,000 c. f. s. stages to 280,000 c. f. s. - Sec. 2 - Gates 2 and 4 - T. 30 - R. 1-6 - Plates 77 to 80

2 200,000 and 260,000 c. f. s. flows - Without sill - T. 30 - R. 7 and 8 - Plates 81 and 82

C Stream lines through jump

1 258,000 c. f. s. - Photo 18G3 - Plate 76

2 76,000 c. f. s. - Photo 18E6 - Plate 76

D Velocities at toe (P. T. of apron)

1 50,000 to 260,000 c. f. s. stages - T. 28 - R. 1D to 5D - Plates 83 to 84

E Hydraulic relations - Plate 85

1 Friction loss - Photos 31A1 and 31A4 - Plate 63

TEST PROCEDURE

Foreword:

The main intent of the text following herewith is to present as clearly as possible a record of the apparatus, method and procedure of the tests, and make what explanation necessary of conditions, not self-evident, of the data shown on the included curves.

No special attempt has been made to analyze the results in application to installations other than Madden, but it is believed that much general information can be obtained from them, so the results are given here in as complete a form as expedient.

The model was accurately built and the tests carefully and precisely performed and the data is presented here for what it is worth.

The Project:

The Madden Dam will be built on the Chagres River about twelve miles above the point where it joins the Panama Canal in Gatun Lake. The reservoir will afford a reserve water supply for operation of the locks and for maintaining adequate depth for navigation during the four months' dry season. It will also provide hydro-electric power to replace the Diesel electric plant now in use. Storage will be obtained for control of floods on the Chagres River.

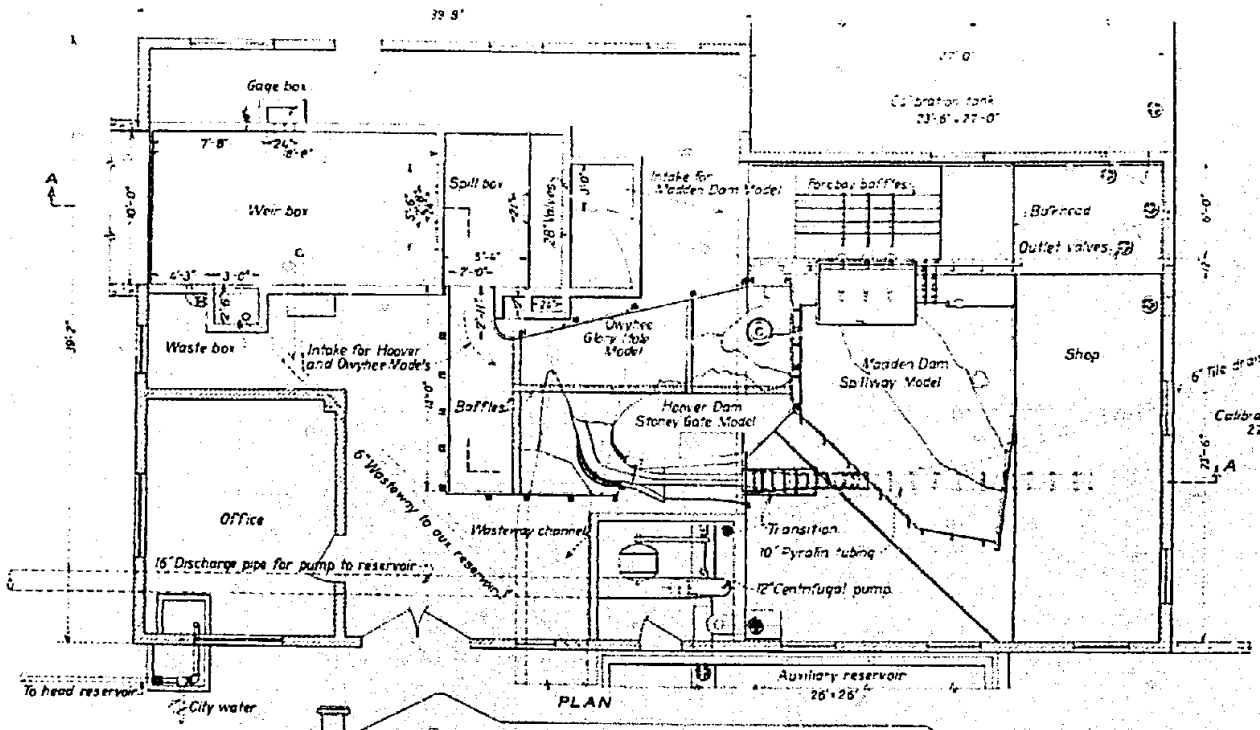
The main section of the dam will be straight concrete gravity type with a spillway overflow, controlled by four 100 ft. x 18 ft. drum gates on the crest. The drop from the maximum reservoir water surface to the river bed below the dam will be about 200 ft. which would subject the stream bed to velocities around 115 ft. per sec. (about 80 mi. per hr.) if not checked by some means. The total energy of this overflow at the maximum expected flood of 280,000 c. f. s. would amount to 6 million h. p. With such tremendous energy resulting at the base of the dam the necessity of protecting the foundation rock is readily seen.

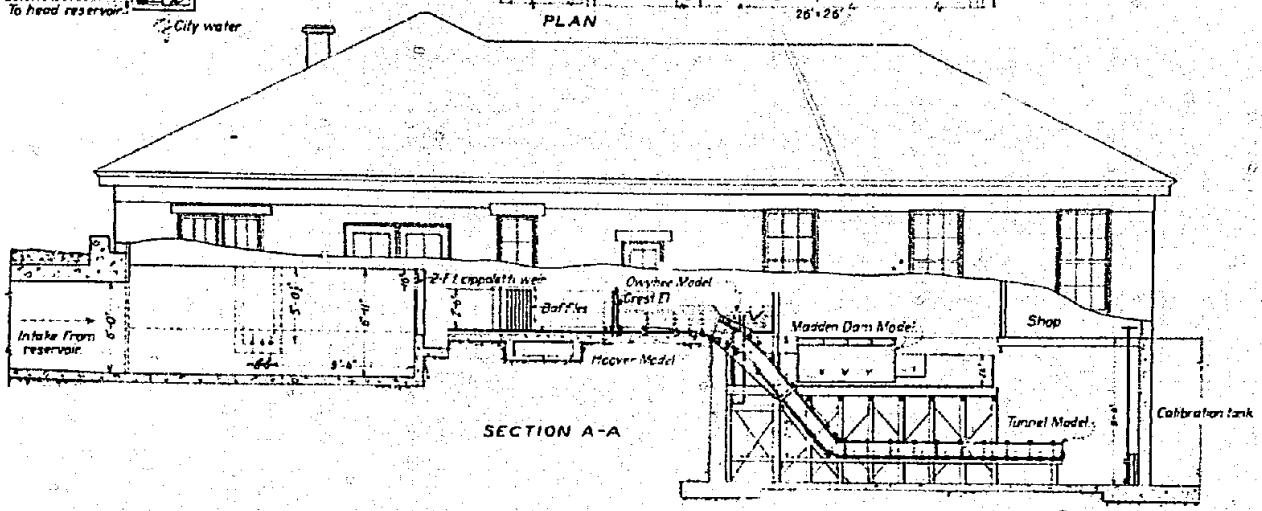
The object of these experiments was to develop a means of dissipating this energy of the overflow to an extent that the velocities along the river bed downstream would be so reduced that no dangerous erosion would occur.

The Laboratory:

The experiments were conducted in the hydraulic laboratory of the Colorado Agricultural College at Fort Collins, Colorado. The college very generously allows the Bureau of Reclamation the use of the laboratory for payment of operating expenses and replacement of tools and equipment broken or worn out in use. The laboratory is very well adapted to the testing of small models, having a water reservoir of ample capacity to obtain a steady flow and a return pump of about 8 cu. ft. per sec. capacity. The weir channel affords an excellent set-up for precise measurement of the quantity of water flowing. The measuring weir was calibrated in place by measuring the flow in volumetric tanks built for this purpose; see Plate 12 for discharge curve of the weir. The discharges shown on this curve must be multiplied by the five-halves power of the model scale (72) or 44,000 to obtain the prototype relation of the quantity of water flowing.

Layout of the laboratory is shown on Plate 11 following.





Scale of feet

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HYDRAULIC MODEL EXPERIMENTS
COLORADO AGRICULTURAL COLLEGE
HYDRAULIC LABORATORY
LOCATION OF MODELS

DRAWN WFB SUBMITTED
TRACED ERC RECOMMENDED
CHECKED APPROVED
213-D-352 DENVER, COLO. JAN. 13, 1932

PLATE II

The Bureau of Reclamation has been using this laboratory for some time for hydraulic investigation, the work being in charge of their Research Engineer E. W. Lane. The writer was detailed from the Panama Canal forces to collaborate with the bureau engineers to work out the design of the Madden spillway apron, and was placed in charge of the laboratory from May to November, 1931. During this time models of the Hoover Dam and Owyhee Dam spillways were built and tested along with the Madden Dam spillway model, so that, although the Madden model was started in February and tests run up to December, the testing was not continuous or full time due to the program of testing for the other projects.

Model Details and Apparatus:

The model was built on a scale of 1 to 72, one foot on the model equaling seventy-two feet on the prototype or one inch equaling six feet. Other relations follow:

	Model	Prototype
Linear dimensions	1	72
Area	1	72^2 (squared)
Velocity	1	$\sqrt{72}$ (square root)
Discharge	1	$72^{5/2}$ (five-halves power)
Time	1	$\sqrt{72}$ (square root)

The spillway section of the dam with drum gates installed in the crest and sluice openings through the dam were accurately reproduced. The power house was represented with the needle valves discharging from it.

Due to the fact that the river channel below the damsite made a rather abrupt bend, this was reproduced in the model for some distance downstream to check the effect of such a bend on the flow conditions at the toe of the dam.

The rock contours, as mapped at the site from drill holes, etc., were reproduced on the model by cribbing up boards sawed out along the contour line of a thickness representing a five-foot contour interval. These were covered with sand to the correct depth to represent the overlying earth and gravel deposits.

The main section of the model was made of wood framing covered with galvanized iron, which was later painted with gray enamel and sanded down until it was practically frictionless.

Details of the model construction are shown on the following drawings, Plates 13 and 14, and photographs, Plate 15.

The discharge over the model was measured by means of the 2-ft. Cippoletti weir, as described above.

The head over the crest was measured by means of a hook gage located in a stilling-pot which was connected to a perforated pipe that ran parallel to the crest and 29 inches upstream therefrom. Pot and gage may be seen in right corner, Plate 4.

Pressures on the crest and apron were recorded by water-heights in glass tubes that were connected to small piezometer pipes opening at various points along the dam face.

There was also installed on the final set-up fixed Pitot tubes along the apron. They consisted of a 1/8-inch copper tube, bent upstream, and connected to the glass water gage tubes by means of rubber tubing. See Photo 28, Plate 15. See Photo 3, Plate 5, showing gage board.

Nappes over the crest were measured by the point gage slide shown in Photo 29, Plate 16.

The tailwater elevation was measured by a float gage connected to a perforated pipe opening in the river channel about 700 ft. (prototype scale) downstream from the dam. The pipe can be seen in Photo No. 11, Plate 7, and Photo 31, Plate 16.

For regulating the tailwater depth a slat gate was installed across the channel downstream. See Photo 30, Plate 16.

Another gate was also developed which would create the natural tailwater, according to the rating curve, for any flow over the crest. See Photo 31, Plate 16, following.

Velocities of the flow on the apron were measured with the Pitot tube shown in Photo 32, Plate 16. The tube was mounted on leveled railings so that the depth and position of the point could be determined. The static and dynamic columns were sucked up into the tubes by a vacuum pump and the differential read on the scale. The tube was previously rated in the flow from an orifice and also from the carriage during a flow meter rating and was found to have a coefficient of true velocity = .975 observed velocity. Moving and still-pictures were taken of nearly every set-up and run.

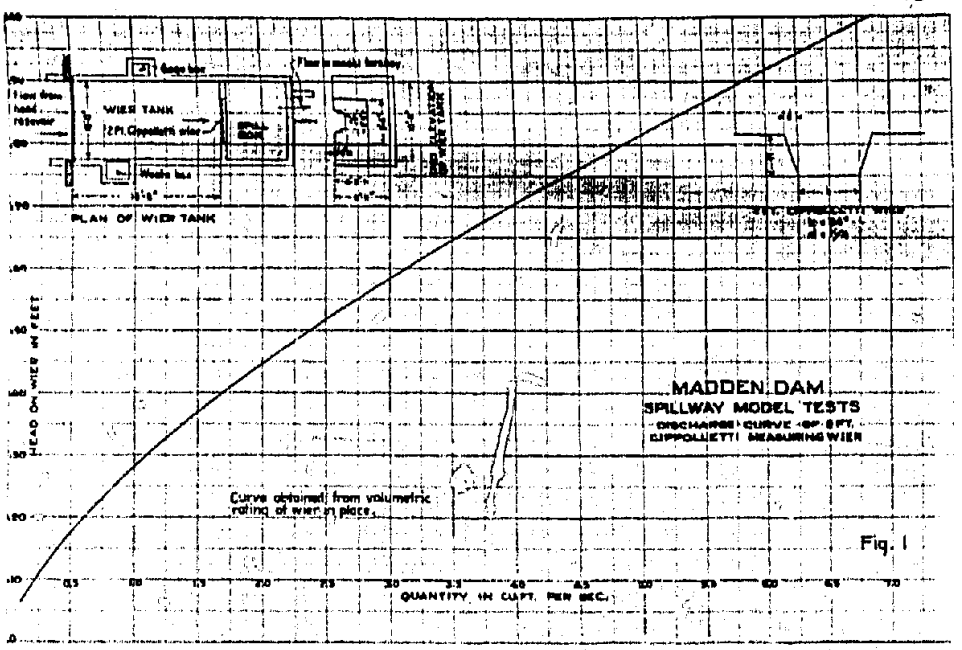


Fig. 1

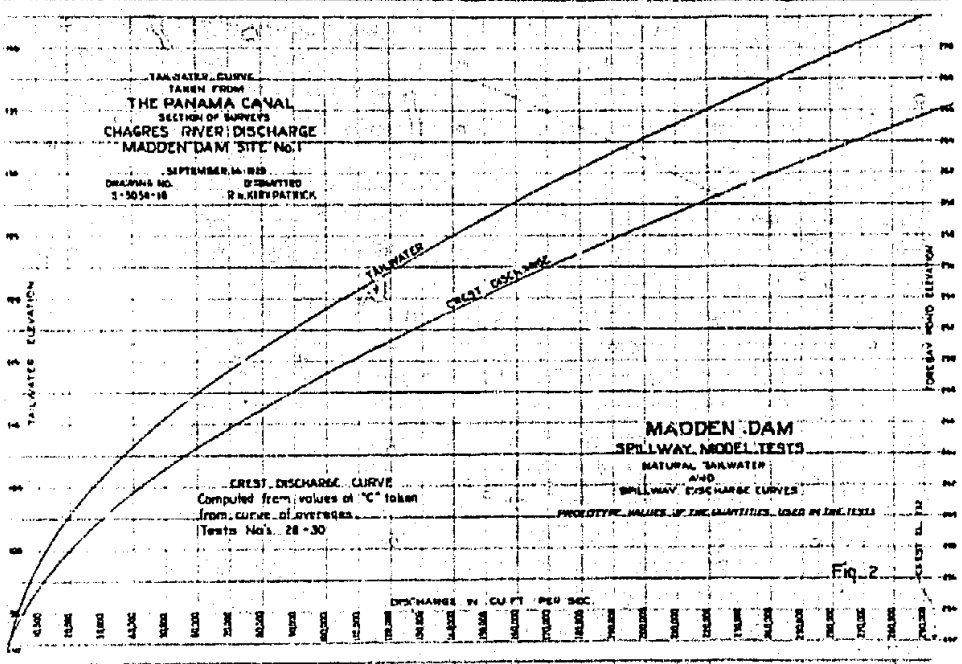


Fig. 2

NOTE

This weir constructed by cut and try experimentation so that it would back the water up in the river bed to correspond to the correct tailwater elevation for any flow over the spillway.

See photo Plate

MADDEN DAM
SPILLWAY MODEL TESTS
AUTOMATIC TAILWATER WEIR

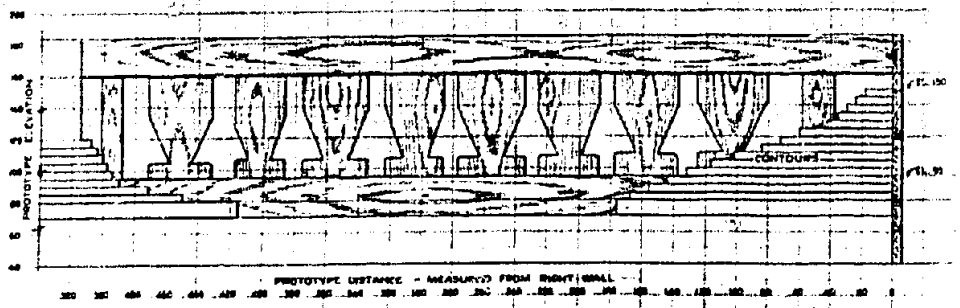


Fig. 3

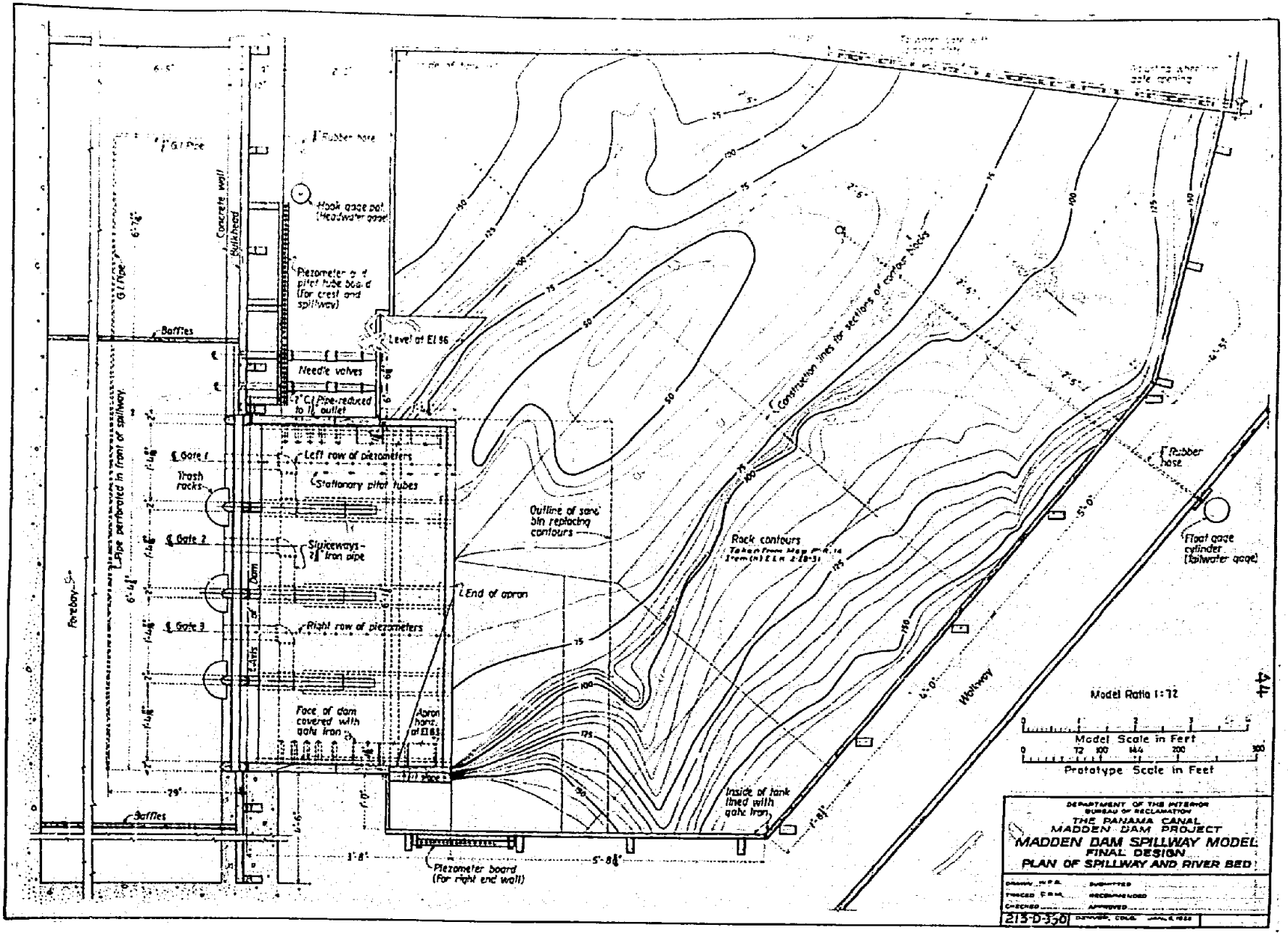

Hook gage pot (Headwater gage)
Piezometer and pitot tube board (for crest and spillway)
Level at El 96
Needle valves
2" C.I. Pipe reduced to 1" outlet
Baffles
Concrete wall
Bulkhead
Gate 1
Gate 2
Gate 3
Trash racks
Left row of piezometers
Stationary pitot tubes
Sluiceways - 2" iron pipe
Right row of piezometers
Face of dam covered with gahz iron
Apron horiz. at El 83
End of apron
Outline of sand bin replacing contours
Rock contours
Construction lines for sections of cofferdams
Inside of tank lined with gahz iron
Piezometer board (for right end wall)
Forebay
Pipe perforated in front of spillway
Rubber hose
Float gage cylinder (Tailwater gage)
Walkway
Model Ratio 1:72
Model Scale in Feet
Prototype Scale in Feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
THE PANAMA CANAL
MADDEN DAM PROJECT
MADDEN DAM SPILLWAY MODEL
FINAL DESIGN
PLAN OF SPILLWAY AND RIVER BED
213-D-350

PLATE 13

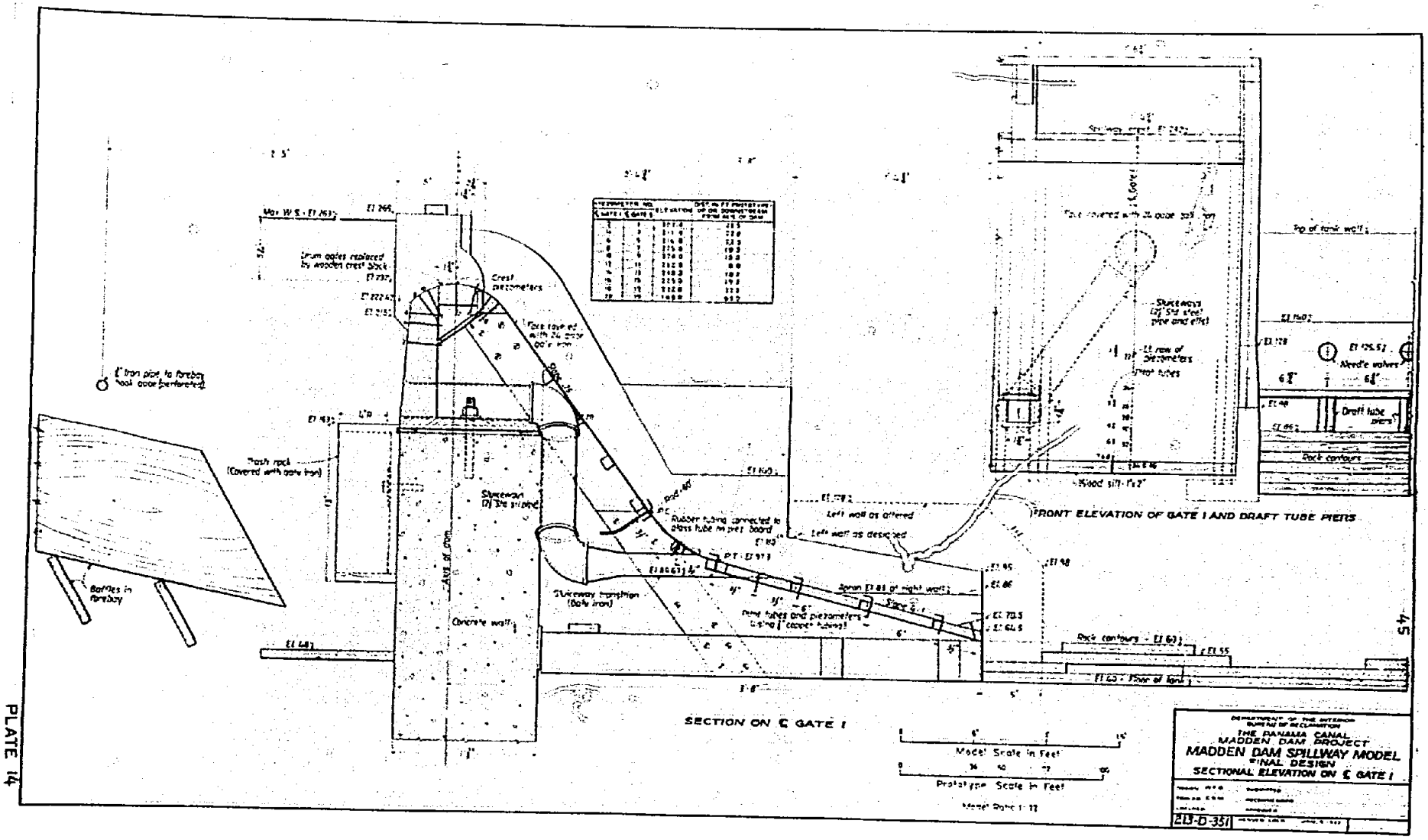

Max W.S. - El 263
El 266
Drum gates replaced by wooden crest block
Crest piezometers
Face lined with 24 gage galv. iron
Trash rack (Covered with galv. iron)
Baffles in forebay
Stoneway (1) 3/8" plywood
Axis of dam
Concrete wall
Rubber tubing connected to glass tube in pet board
Stoneway transition (Galv. iron)
Piezometer tubes and piezometers (using 1/4" copper tubing)
Left wall as offered
Left wall as designed
Stoneways (2) 3/4" steel pipe and elbows
Left row of piezometers (Pet tubes)
Plywood sill - 1'x2'
Top of tank wall
Needle valves
Draft tube piers
Rock contours
Rock (contours - El 63)
El 63 - Floor of tank
FRONT ELEVATION OF GATE 1 AND DRAFT TUBE PIERS
SECTION ON ℄ GATE 1
Model Scale in Feet
Prototype Scale in Feet
Model Ratio 1:12
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
THE BANNAMA CANAL
MADDEN DAM PROJECT
MADDEN DAM SPILLWAY MODEL
FINAL DESIGN
SECTIONAL ELEVATION ON ℄ GATE 1
213-D-351

PLATE 14

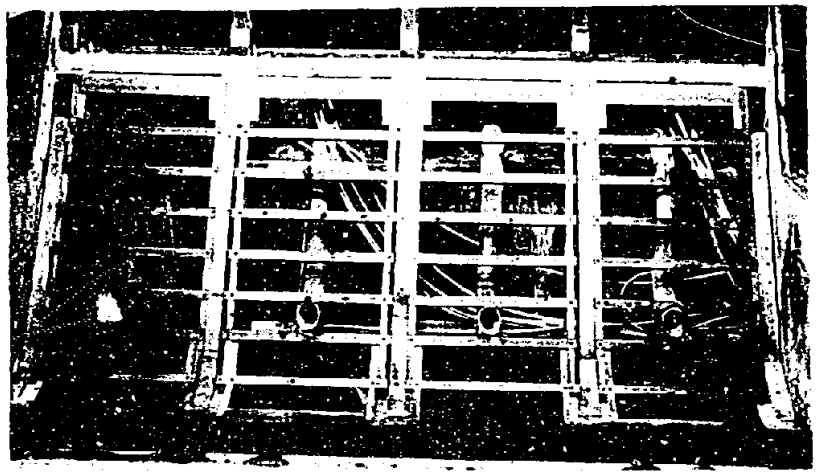


PHOTO 25
ORIGINAL DESIGN
MODEL FRAMING

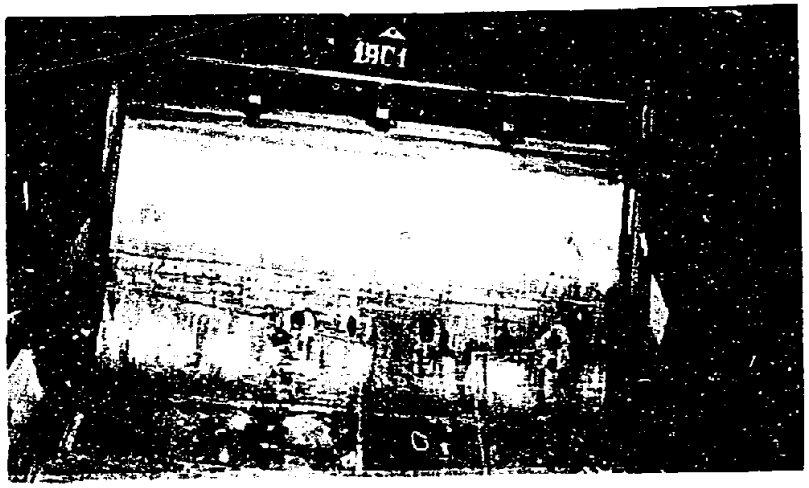


PHOTO 26
ORIGINAL DESIGN
FRAMING COVERED

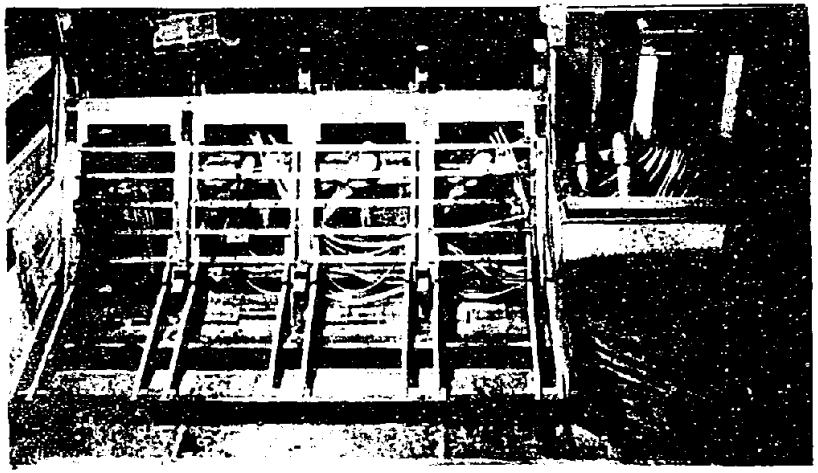


PHOTO 27
FINAL DESIGN
FRAMING DETAIL

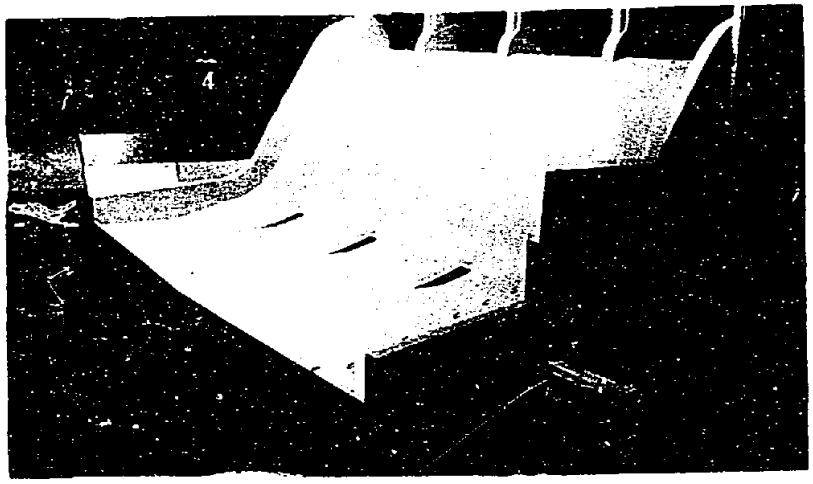


PHOTO 28
FINAL DESIGN
FRAMING COVERED

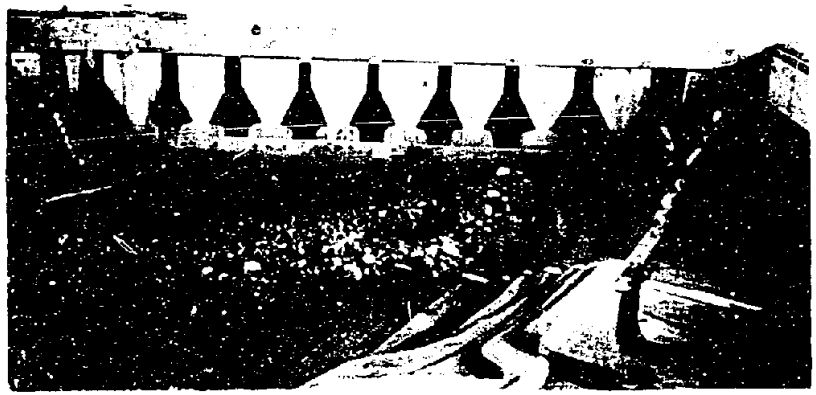


PHOTO 31
AUTOMATIC TAILWATER WEIR

PHOTO 29
NAPPE POINT GAGE

PHOTO 32
PITOT TUBE-MOUNTED

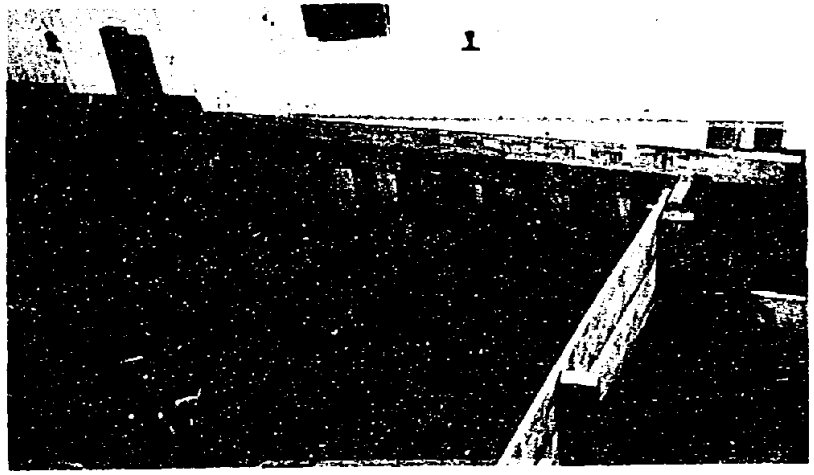


PHOTO 30
REGULATING TAILWATER WEIR

DETAILS OF SET-UPS AND TESTS

The Dentated Sill:

Original design; apron level at El. 60. In the first layout the spillway apron was drawn in level at El. 60 as shown on Plate 17, following. At all stages of flow there was a considerable depth of tailwater over this low apron but this depth was not very effective in checking the velocity of the overflowing sheet of water. The jet sweeps down to the bottom, with very little disturbance or consequent loss of energy and persists downstream for a considerable distance at a high velocity. Photo 33, Plate 18, following, shows the comparative quiet nature of the surface flow and Fig. 1, Plate 19, traces the high velocity sheet under the tailwater. At a point about 30 ft. upstream from the end of the apron where the face of the dentated sill would come, there is a measured velocity of about 70 ft. per sec. It would seem that the sill should be designed to take the force of impact due to this velocity.

With the dentated sill in place the line of maximum velocity is lifted to the surface of the stream relieving the bottom of any erosive velocities. See Figs. 2 and 3, Plate 19.

The terrific impact on the sill is evidenced by the boiling which occurs as seen in Photo 34, Plate 18, also Photo 41, Plate 21. The path of the high velocity water is etched on the iron plate, that was immersed in the stream when freshly painted; seen in Photos 35 and 36, Plate 18.

The batter wall along the right bank, as shown on the drawing, Plate 17, was too steeply sloped for the water to "ride," consequently, it fell into a thick concentrated jet, at the base of the slope, and had very high erosive velocities. It was considered advantageous to excavate this obtrusion although an attempt was made to save some of the rock by stepping up the bays towards the hillside. Photo 37, Plate 20, following, illustrates one scheme, in which bay 4 was raised to El. 80, bay 3 to #1. 70, bay 2 was at El. 60, and bay 1 at El. 70.

In this set-up there was a sand bin built in at the end of the apron to ascertain the erosive tendencies of the flow. The bin was filled with ordinary screened building sand, no attempt being made to have the particles sized to the model scale, consequently, the resulting erosion did not represent necessarily the amount that would result in the prototype but the general tendency was indicated and comparative merits of different set-ups could be determined.

With only two gates open and the pond at its maximum elevation there resulted a concentration of flow with dangerous erosive tendencies, as seen in the following Photos 39 and 40 on Plate 20.

This shows the danger of separate gate operation. As the drum gates can easily be operated simultaneously it was assumed that this would be the normal procedure and, unless noted, the flow, in all tests following was passed over the crest or gates in equal quantities per linear foot.

Photo 38, Plate 20, shows the most economical set-up with the dentated sill. The apron was raised at both ends of the spillway to follow the natural rock lines with least excavation. Flow conditions were as satisfactory as with the horizontal apron and the dentated sill apparently stopped all dangerous erosion below the apron.

However, with the apron so far down under the tailwater, the hydraulic jump is drowned out and the overflowing sheet slips under the backwater with very little loss in velocity. Even at the lowest flows there is considerable impact on the sill which means that dependence for the dissipation of the energy must be placed entirely on the sill. A small break at any point lets through a high velocity jet starting erosion that endangers the entire structure and scheme. To unwater the tailrace to make repairs on the sill would be quite an expensive job.

The sill then must necessarily be a heavy reinforced structure, well anchored to the bottom, with probably metal armor on the block faces as it was seen that the high velocity jet would sweep any debris directly down against it. It was found necessary to armor the faces of blocks at the foot of spillways such as Gatun, where the velocity of the water striking the sill is much less than it is here.

The sill proposed was ten feet high and 30 feet wide involving rather large quantities of concrete and form work which together with the reinforcement and anchorage required would make it expensive.

A royalty would also have to be paid for the use of the dentated sill patent.

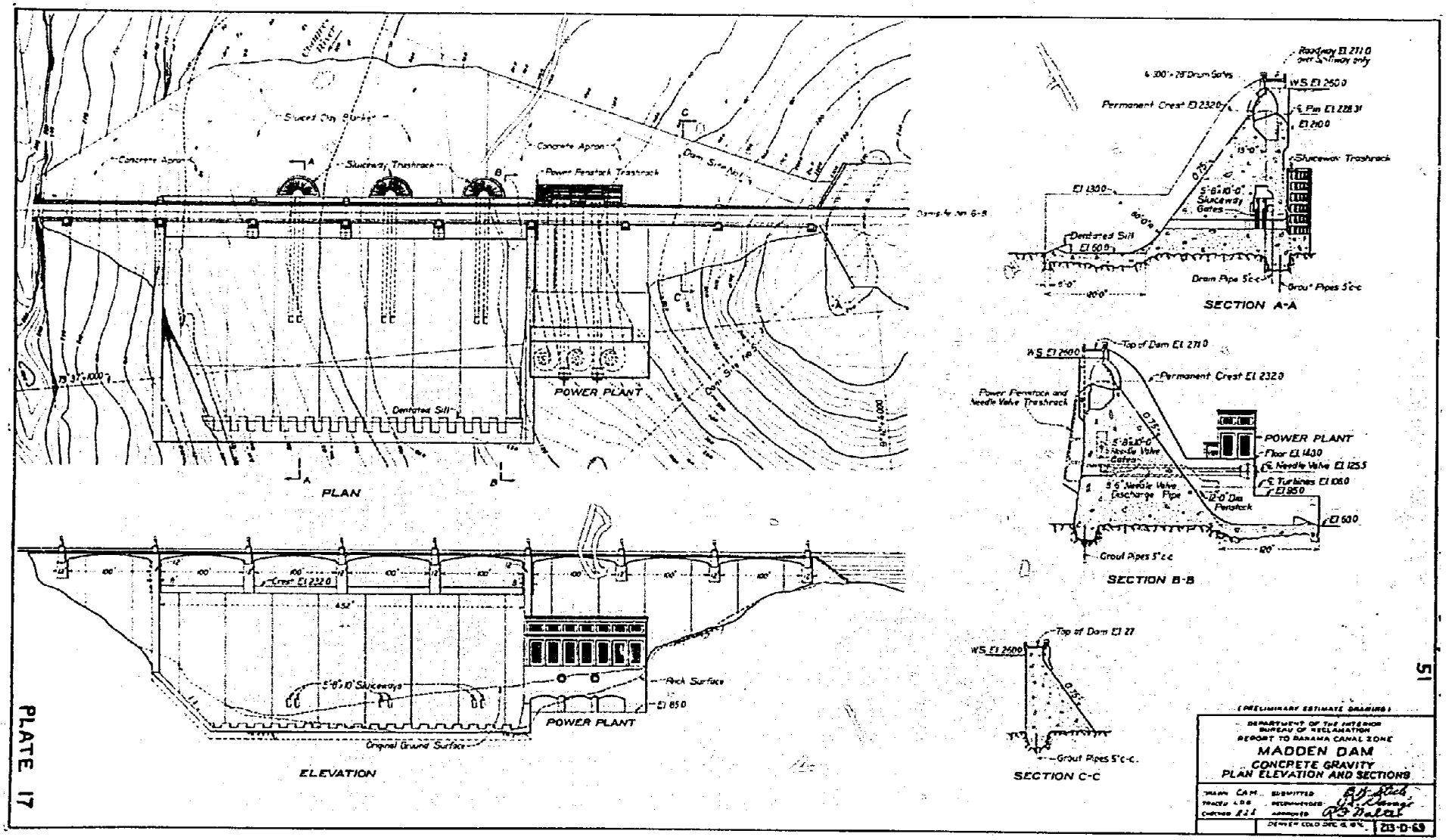

Sluice Dis Bunker
Concrete Apron
Sluiceway Trashrack
Power Penstock Trashrack
Dam Site No. 1
Dam Site No. 6-B
POWER PLANT
Dentated Sill
PLAN
Roadway El 271.0
6-300'x28' Drum Gates
W.S. El 260.0
Permanent Crest El 232.0
℄ Pin El 228.31
El 210.0
Sluiceway Trashrack
El 130.0
5'-6"x10'-0" Sluiceway Gates
Dentated Sill
El 60.0
Drain Pipe 5' c-c
Grout Pipes 5' c-c
SECTION A-A
Top of Dam El 271.0
W.S. El 260.0
Permanent Crest El 232.0
Power Penstock and Needle Valve Trashrack
POWER PLANT
Floor El 143.0
℄ Needle Valve El 125.5
℄ Turbines El 106.0
El 85.0
5'-6"x10'-0" Needle Valve Gates
5'-6" Needle Valve Discharge Pipe
12'-0" Dia Penstock
El 60.0
Grout Pipes 5' c-c
SECTION B-B
Crest El 232.0
452'
5'-6"x10' Sluiceways
Rock Surface
El 85.0
POWER PLANT
Original Ground Surface
ELEVATION
Top of Dam El 27
W.S. El 260.0
Grout Pipes 5' c-c.
SECTION C-C
(PRELIMINARY ESTIMATE DRAWING)
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
REPORT TO PANAMA CANAL ZONE
MADDEN DAM
CONCRETE GRAVITY
PLAN ELEVATION AND SECTIONS
DRAWN CAM
TRACED LBB
CHECKED RJJ
SUBMITTED
RECOMMENDED
APPROVED
DENVER COLO DEC 6, 1948
203-D-69

PLATE 17

PHOTO 33
FLOW 260,000 C.F.S
WITHOUT SILL

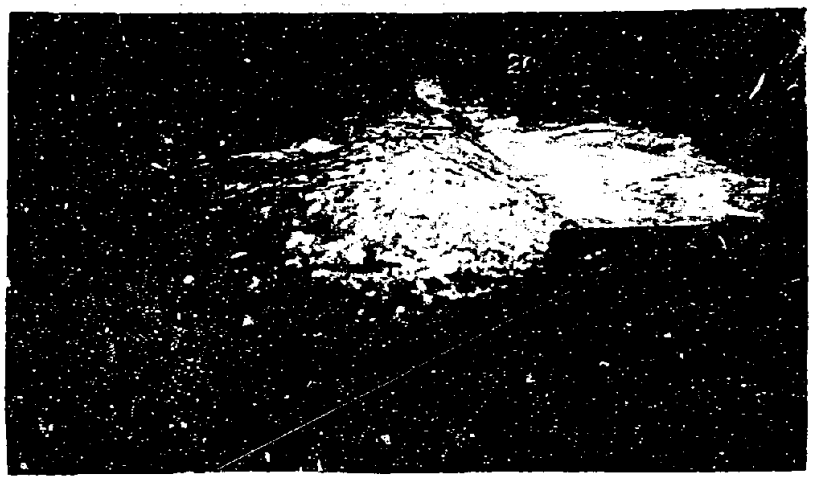


PHOTO 34
FLOW 260,000 C.F.S
WITH SILL

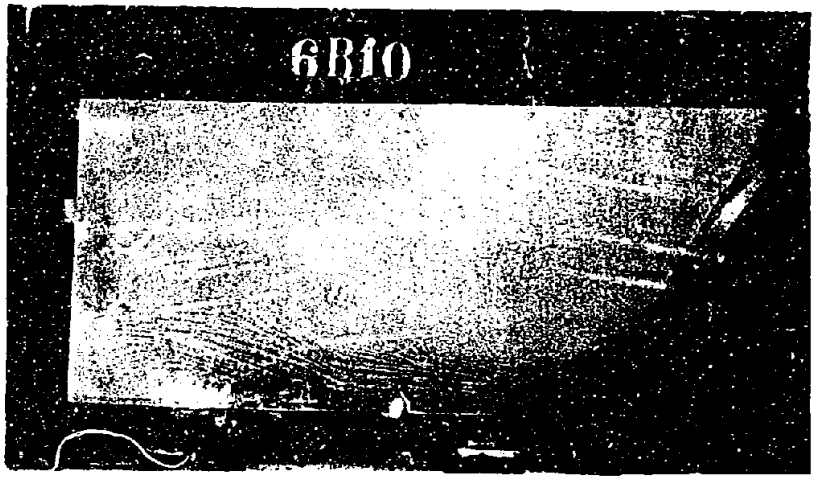


PHOTO 35
STREAM LINE FLOW
ETCHING BETWEEN DENTATES

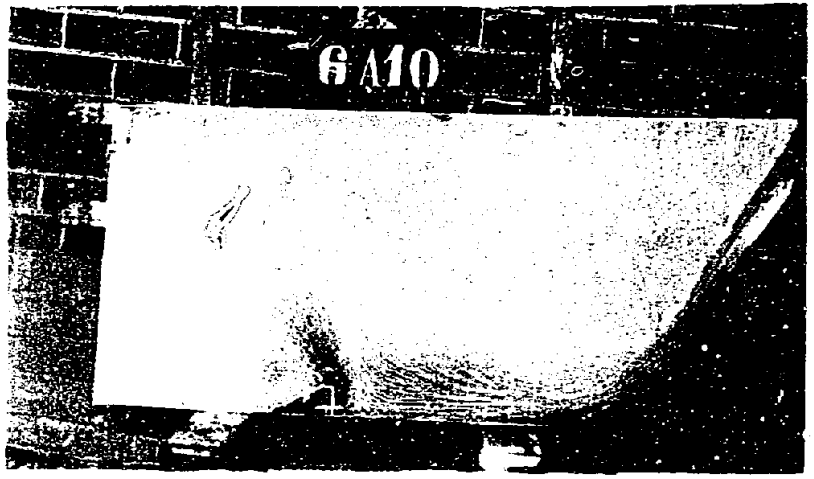


PHOTO 36
STREAM LINE FLOW
ETCHING AT BLOCK

ORIGINAL DESIGN-APRON LEVEL AT EL 60

PLATE 12

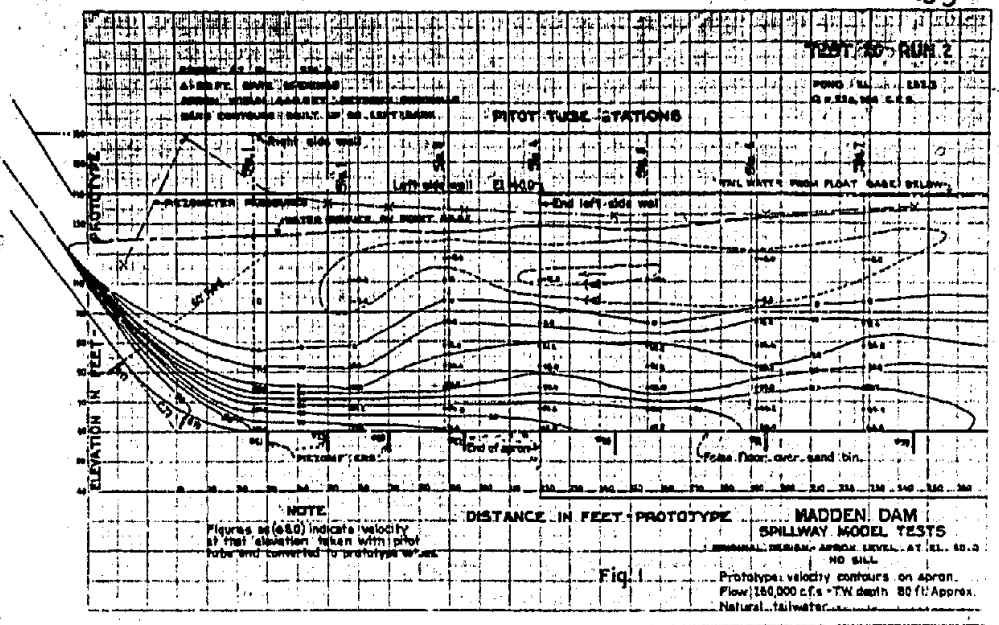


Fig. 1

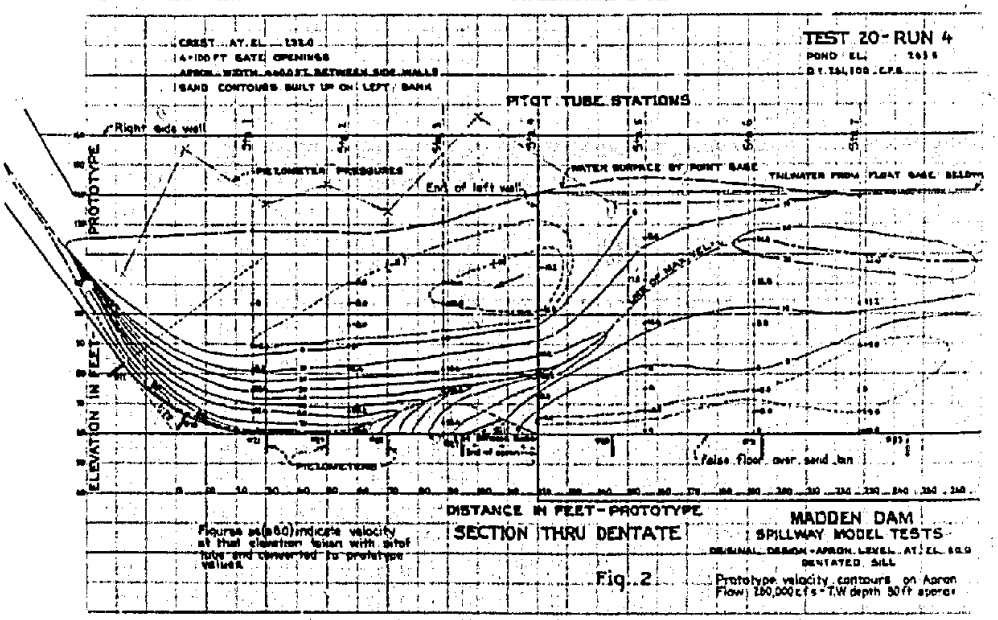


Fig. 2

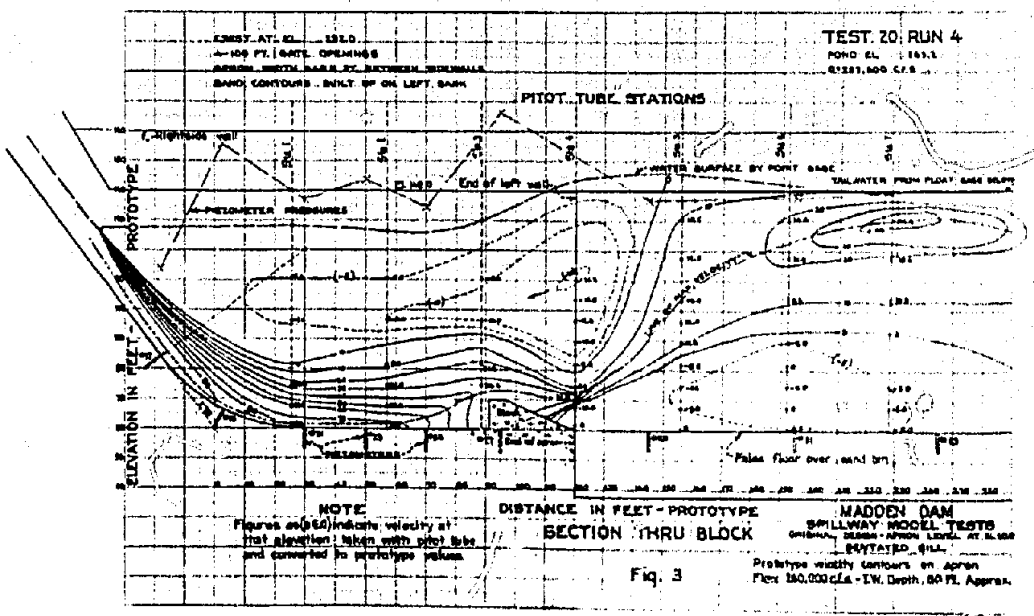


Fig. 3

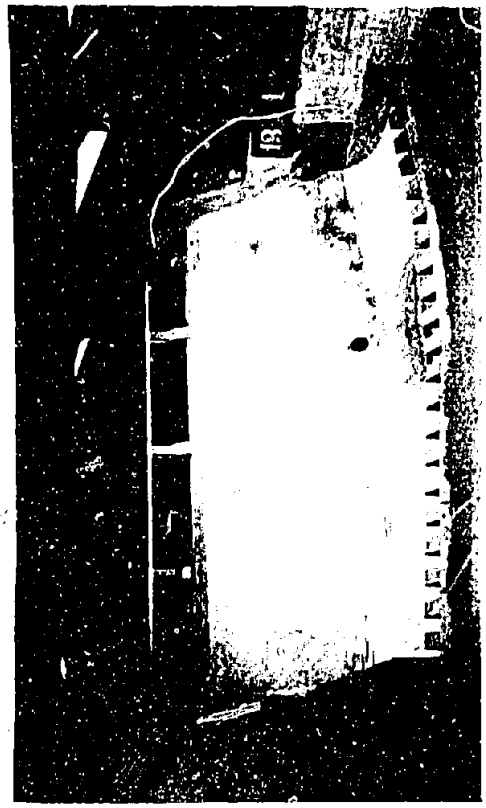


PHOTO 39
RISING APRON SET-UP
EROSION AFTER 1WAY FLOW

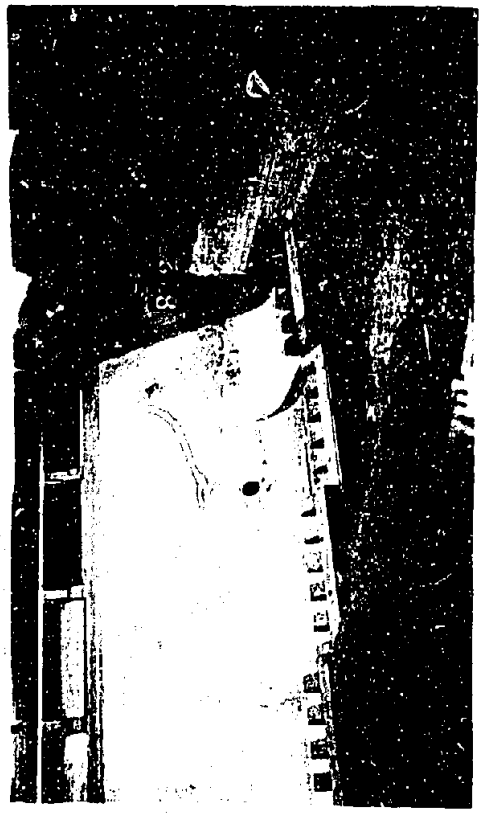


PHOTO 40
EROSION RESULTING
FROM 2-GATE OPERATION

[illegible] BOXES

PHOTO [illegible]
GATES [illegible] SCHARDON
[illegible] 263

RISING APRONS - DENTATED SILL

PLATE 20

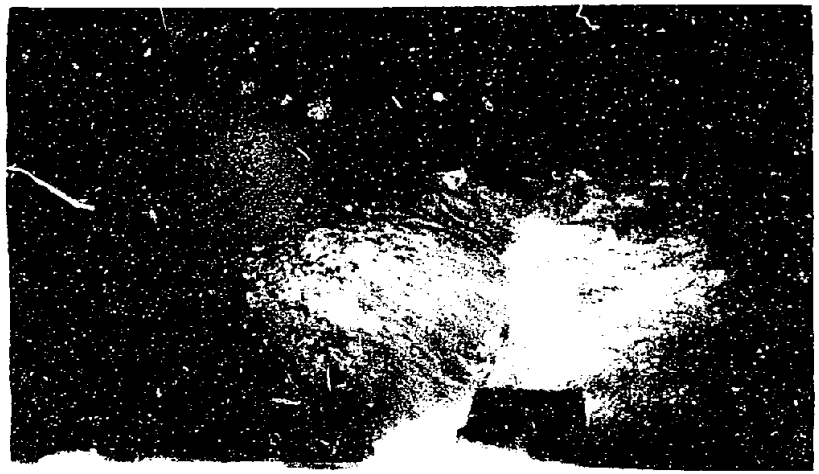


PHOTO 42
WAVE FLOW PATH
DENTATED SILL

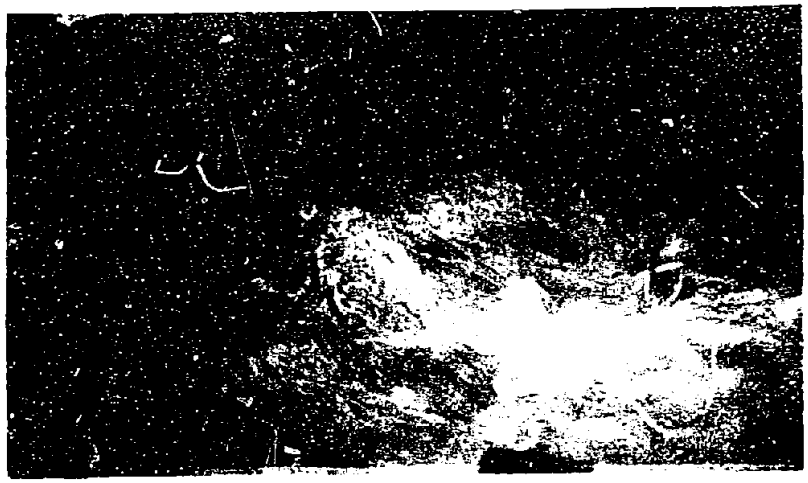


PHOTO 43
WAVE FLOW PATH
PLAIN FLOOR SILL

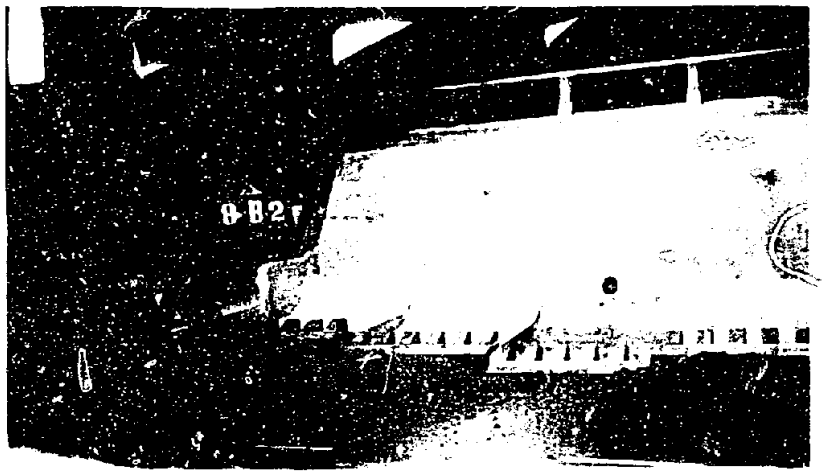


PHOTO 43
EROSION AFTER RUN
DENTATED SILL

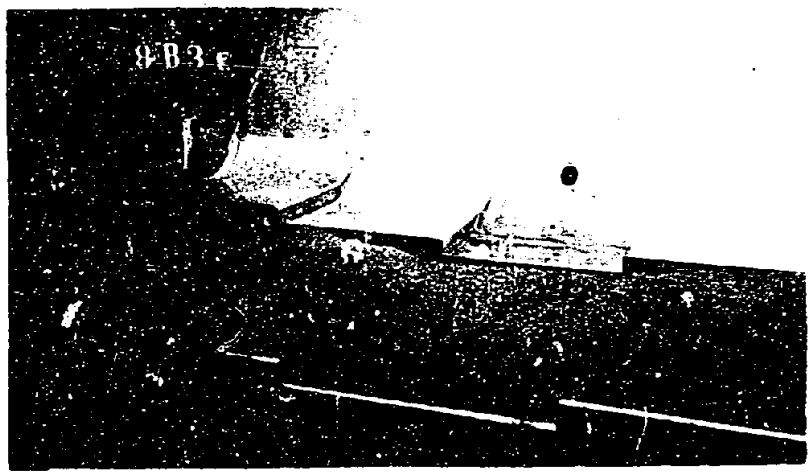


PHOTO 44
EROSION AFTER RUN
PLAIN SILL

The Hydraulic Jump Apron:

Apron level at El. 60. The use of the hydraulic jump was recognized as a most effective means of dissipating the energy of the overflow but it was readily seen that the depths of the natural tailwater over the apron were too great to permit its formation. The apron then must be raised until the natural tailwater depth over it was the same as that required by the jump. In order to obtain some idea of just how much the apron would have to be raised, the tailwater was lowered until a jump formed well back on the apron.

The appearance of the jump is quite distinctive and with a little experience the tailwater can be adjusted by eye to obtain a depth for maximum energy dissipation.

Photo 33, Plate 18, preceding, shows the flow conditions with a tailwater depth of approximately 80 feet, corresponding to the natural depth of the backwater at the damsite. Note the comparatively quiet appearance of the water surface as the overflowing sheet cuts underneath with little or no impediment.

As the tailwater is lowered, however, the overflow jet strikes onto the well of backwater with much agitation and turmoil and consequent rapid loss of velocity through the flow.

Photos on Plate 22 following show the flow appearances. In Photo 46 the tailwater is still too high for the most effective performance of the jump and in Photo 48 the tailwater is just a little too low allowing the jump to sweep off the apron slightly.

In photo 47 the adjustment is just about right, forcing the jump well back against the face of the dam, without drowning it out. This is the condition that it was attempted to attain in all tailwater settings for the various flows of test 2 plotted on Plate 23.

These observations for correct tailwater depths were later checked by measurement with a Pitot tube of the velocities downstream under various tailwater depths, as shown on Plate 24. The Pitot tube remained fixed for each curve-taking and while the velocities indicated are not the average velocities through the section it is thought that the curves show a relative energy dissipation for the different depths of tailwater. Note that there is a decided peak in the curves that gives a definite tailwater depth to obtain a minimum velocity. These peak points are plotted on Plate 23 as test 19 and check closely the adjustments done by eye. Computation of the depths required, from the formula $D_2 = -\frac{D_1}{2}\sqrt{\frac{D_1^2}{4} + \frac{2V_1^2 D_1^2}{g}}$, also show close agreement. D_1 and V_1 were computed taking into account the friction loss determined from tests shown on Plate 85.

Fig. 1 on preceding Plate 19 and the following Plate 25 give the velocity profiles through the flow along the apron. Note the more rapidly diminishing velocity when the tailwater is correct for the jump as in Fig. 2, Plate 25. (Compare velocities at station 7 with Figs. 1 and 3.) It is regretable that the sections were not extended farther downstream for they do not quite include the com-

APRON LEVEL AT EL 69 MAX FLOW OF

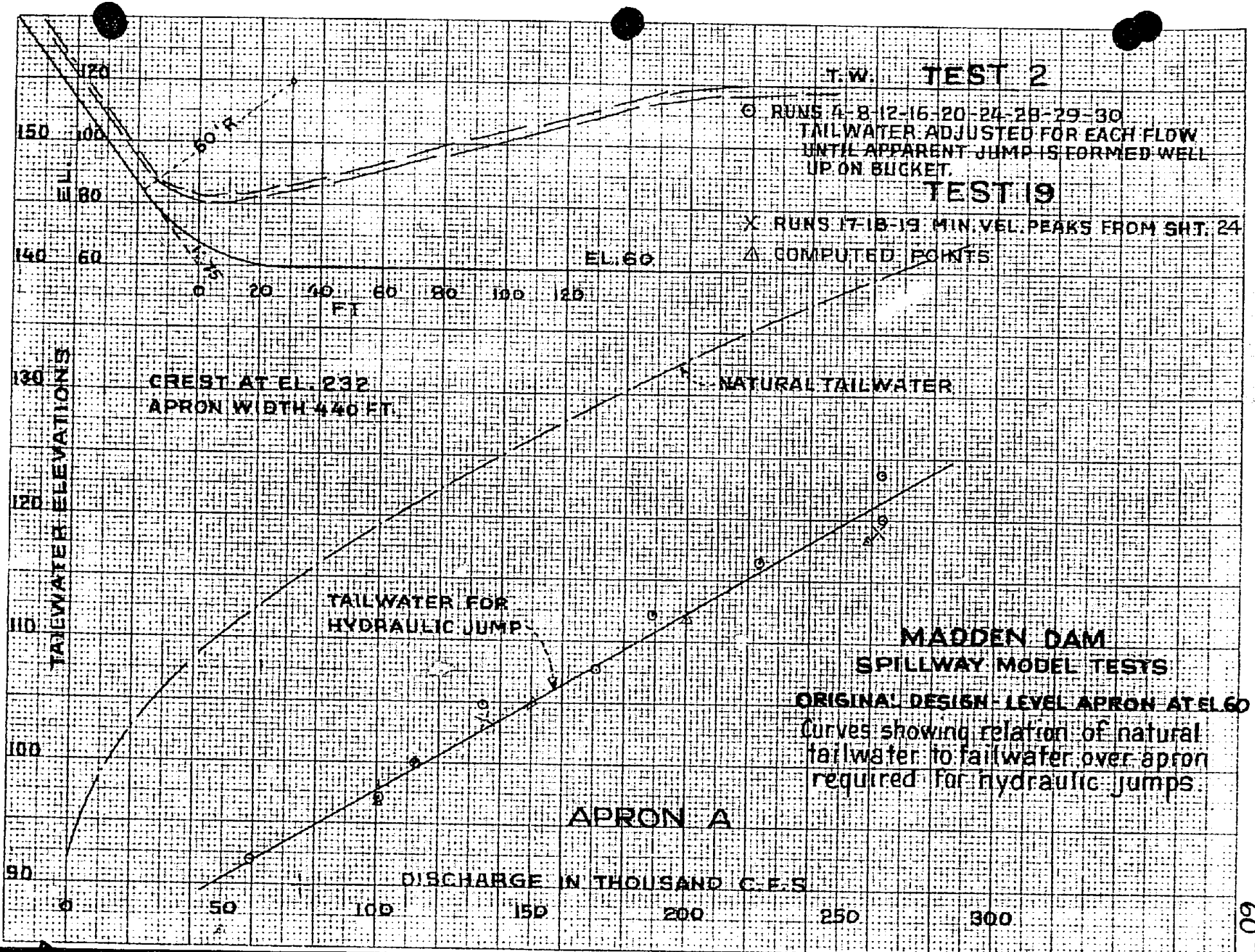
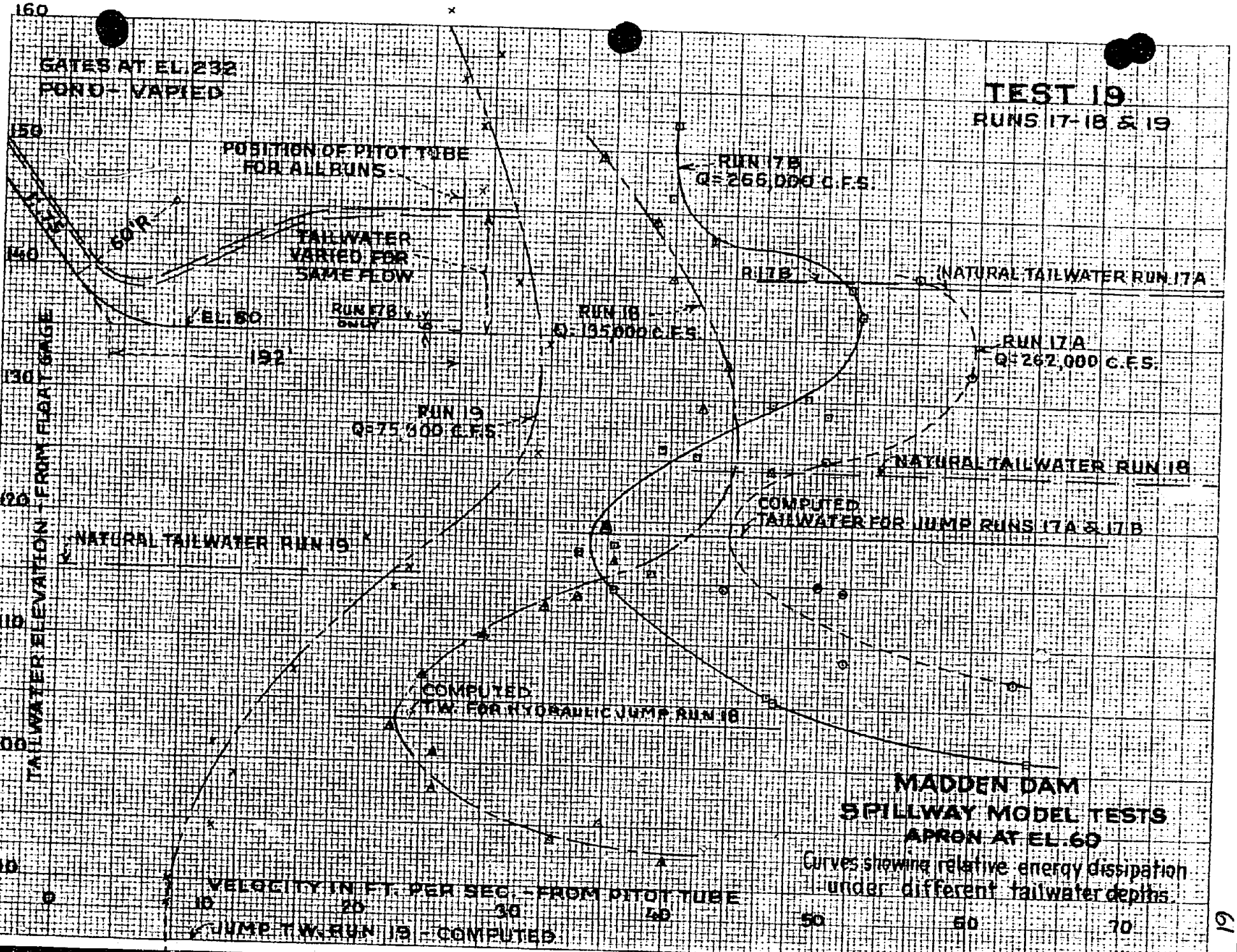

T.W. TEST 2
RUNS 4-8-12-16-20-24-28-29-30
TAILWATER ADJUSTED FOR EACH FLOW
UNTIL APPARENT JUMP IS FORMED WELL
UP ON BUCKET.
TEST 19
RUNS 17-18-19 MIN. VEL. PEAKS FROM SHT. 24
COMPUTED POINTS
EL. 60
CREST AT EL. 232
APRON WIDTH 440 FT.
NATURAL TAILWATER
TAILWATER FOR
HYDRAULIC JUMP
TAILWATER ELEVATIONS
EL.
FT
DISCHARGE IN THOUSAND C.F.S.
APRON A
MADDEN DAM
SPILLWAY MODEL TESTS
ORIGINAL DESIGN - LEVEL APRON AT EL 60
Curves showing relation of natural
tailwater to tailwater over apron
required for hydraulic jumps.

PLATE 23

60



MADDEN DAM
SPILLWAY MODEL TESTS
APRON AT EL. 60
Curves showing relative energy dissipation under different tailwater depths.

PLATE 21

19

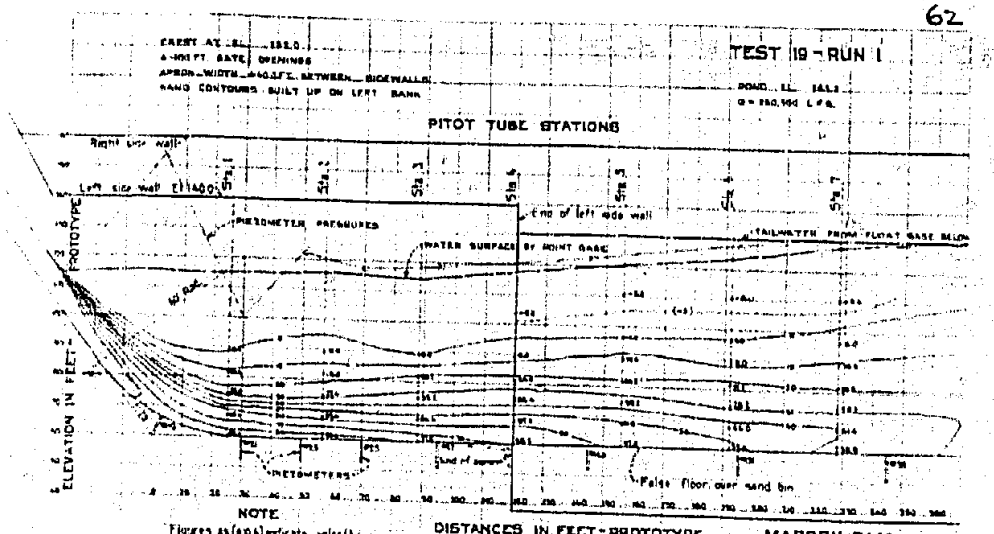


Fig. 1

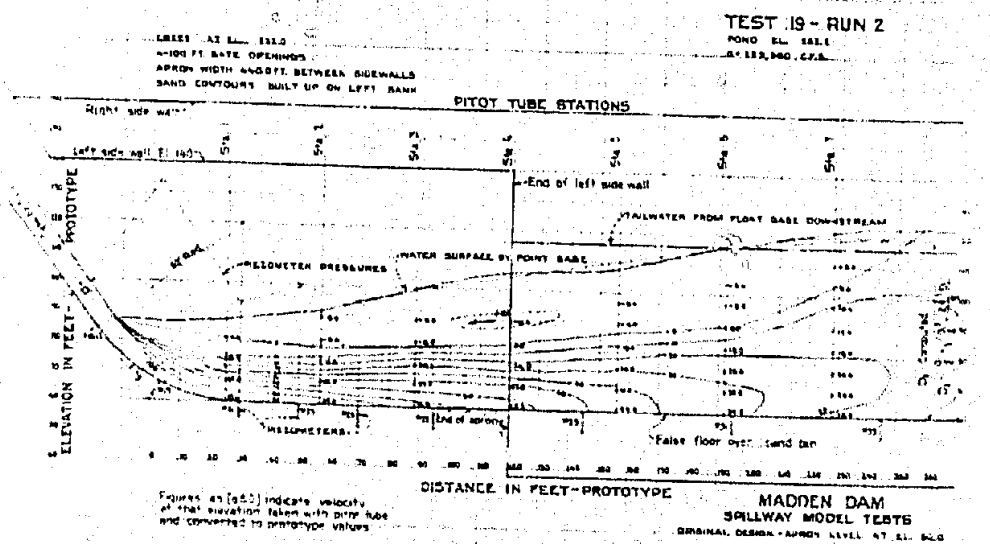


Fig. 2

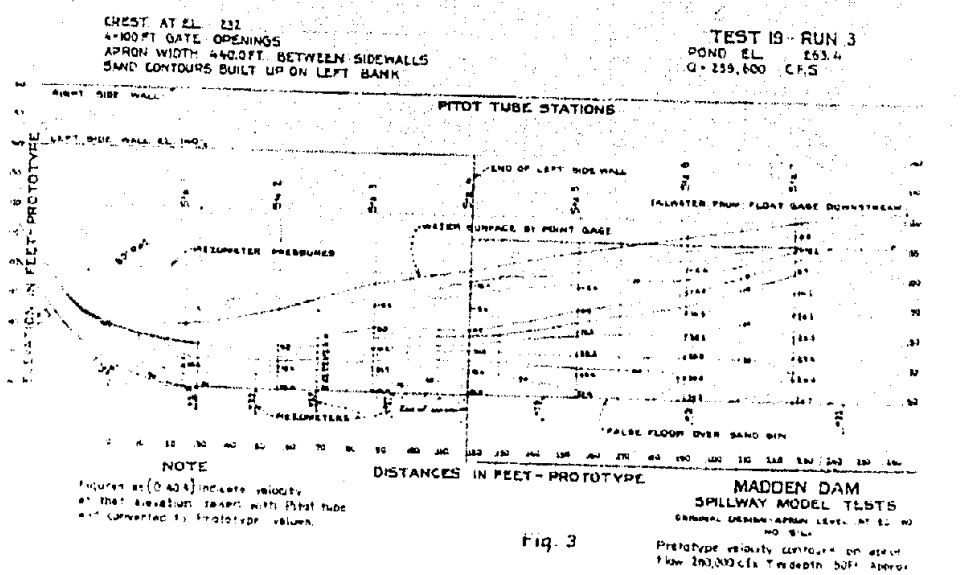


Fig. 3

pleted jump where the velocity comes to practically zero while in the case where the tailwater is too deep for the jump there still remains quite an appreciable velocity for some distance farther downstream. It is seen that the water surface by the point gage rises above that shown by the float gage downstream but where the jump is completed farther downstream the two levels are sensibly the same.

It is seen then that the natural tailwater gives depths, over this apron as originally designed, too deep for much energy dissipation, in fact, the depths cause about the highest velocities possible on the river bed; see Plate 24.

The curve showing tailwater required for the hydraulic jump lies about 20 feet below the natural tailwater curve (Plate 23), practically paralleling it, however, so that it is reasonable to assume that if the apron was raised 20 feet, or to El. 80, depths of tailwater over it would cause the jump to form for almost all stages of flow. This then was the next step.

Apron Level at El. 80:

With the bucket and apron downstream level at El. 80, there was apparently a good hydraulic jump formed, for all stages of flow, by the natural depths of tailwater over the apron.

There was no data taken on this test other than observation notes but the moving-pictures of test 5, run 1, show the behavior of the jump for the entire flood range.

Various lengths of the apron downstream were tried. It seemed even with the apron extended as much as 70 feet beyond the original length that a small sill was necessary to prevent erosion in the sand bin below. However, a plain strip representing about a 2-foot height of sill on which there was no apparent impact from the flow was almost as effective as the 10-foot dentated sill, which the water struck with considerable force as indicated by the boiling above it.

The same was true when the apron was shortened to a length 35 feet beyond the original.

When the small sill was placed at the end of the original apron there was a decided impact on it, which created almost as much boiling as the dentated sill when placed in the same position. The erosion as indicated in the sand bin was slightly less for the dentated sill.

The completed jump, or the point of full recovery where the velocity reaches a minimum, does not occur until a distance of about 150 feet downstream from the end of the original apron, and a sill of much height when placed across the stream will receive full impact of the high velocity along the floor.

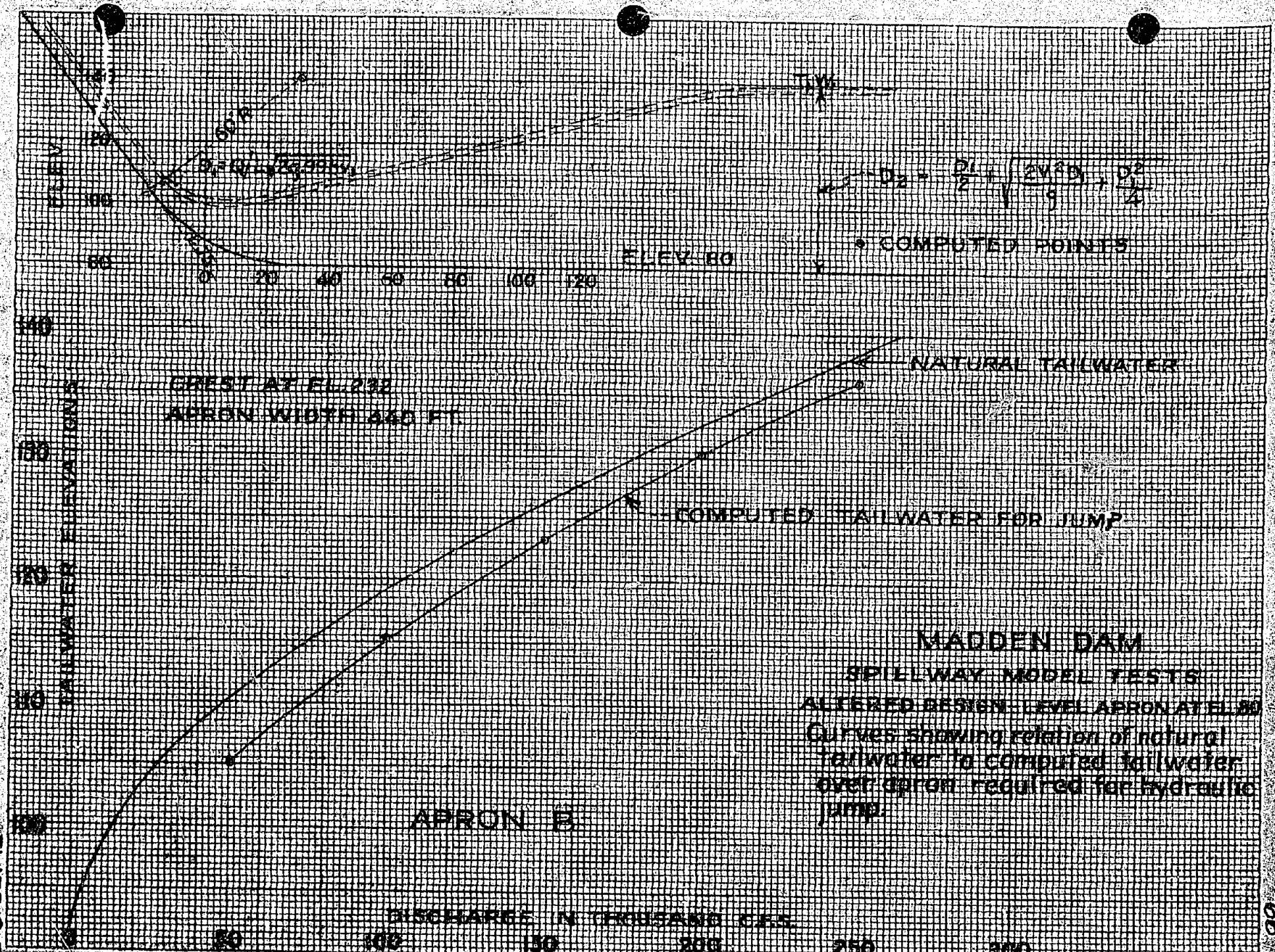
It would seem, then, that a small sill could be used merely as a deflector, of a height sufficient to lift the bottom velocities from the river bed below the apron to a point downstream where they would be retarded naturally in the jump action. The apron length then could be reduced considerably and need only be long enough to cover the range of the violent churning and back-roll at the start of the jump.

A sill 4.5 feet high with its face sloping to the floor 10 feet upstream was tried at the end of the 35-foot extension apron. For a 15-minute run corresponding to the maximum flow there was practically no erosion in the sand bin below the apron, and there was no bubbling whatever that would indicate impact on the sill.

From these very satisfactory results, which were later borne out by further experiments, this type of deflector sill, with the shortened apron, was adopted for later hydraulic jump set-upts.

With the apron level and raised to El. 80, as stated above, there was apparently a good jump formed at all quantities of flow.

This was due to the fact that the curve giving depths required for the jump closely parallels the natural tailwater depth curve. But there would be only one flow, theoretically, where the natural tailwater depth over the apron would give exactly the depth required for the jump even if the apron was raised slightly more, as indicated beneficial by the curves on Plate 26 following. It was thought that if the apron was sloped, the jump could move up or down the slope until the depth required was encountered, and so there would result a good jump formation for each flow, no matter what the variation in the natural tailwater. Also, considerable saving in concrete could be effected by sloping off the apron as the thickness at the end averaged about 20 feet.



MADDEN DAM
SPILLWAY MODEL TESTS
ALTERED DESIGN - LEVEL APRON AT EL. 80
Curves showing relation of natural tailwater to computed tailwater over apron required for hydraulic jump.

PLATE 29

1:6 Sloped Apron Starting at El. 84:

With this set-up, the same series of tests were run where the tailwater was juggled until a depth was found that gave the best hydraulic jump formation well up on the apron. These depths for different flows are plotted in the curve shown on Plate 28 following.

It is seen that this apron is some 6 to 8 feet too low as the required depths are less than those occurring with the natural tailwater over the apron.

With the sloping apron no definite calculation can be made for the depth required for the jump as the point above which the depth is assumed is indeterminate. The theoretical position of the jump on the slope can be computed but it would not be known if this position would start the jump well back against the face of the dam, and yet not so far back as to drown it out. Unless the jump starts well up on the apron, full use is not made of the apron length. The calculated position might start the jump some distance down the slope away from the dam face and so for equal protection this length must be added to the pavement downstream.

It was a case of experimental cut and try and the next step was to raise the elevation of the apron as was indicated from the curves on Plate 27.

Photo 49 on the following Plate 28 shows the jump for the maximum discharge over the spillway. Note that its beginning is well up on the curved bucket so that the greater disturbance of flow takes place over the apron pavement. In the adjustment of the tail-water for the other flows this same condition was obtained.

The sloped-faced sill 4.5 feet high was used at the end of the apron with no evidence of bubbling that would indicate impact on it. That it was effective in raising the velocities from the river bed downstream is evidenced by the lack of serious erosion in the sand bin below; see Photo 50, Plate 28.

Contract with photos on Plate 18 preceding to show difference in surface flow conditions of the two schemes.

In this test, runs were made to obtain the effect of separate gate operation. Photo 51 on following Plate 28 shows an apparent good jump forming on the apron with the flow coming from gates 1 and 2 only. However, there seems to be a concentration set-up in bay 2 as evidenced by the erosion of the sand below; see Photo 52, Plate 28.

This condition was also encountered in the dentated sill set-up; see Plate 20, and emphasizes the danger of opening only one or two gates rather than lowering all four gates simultaneously.

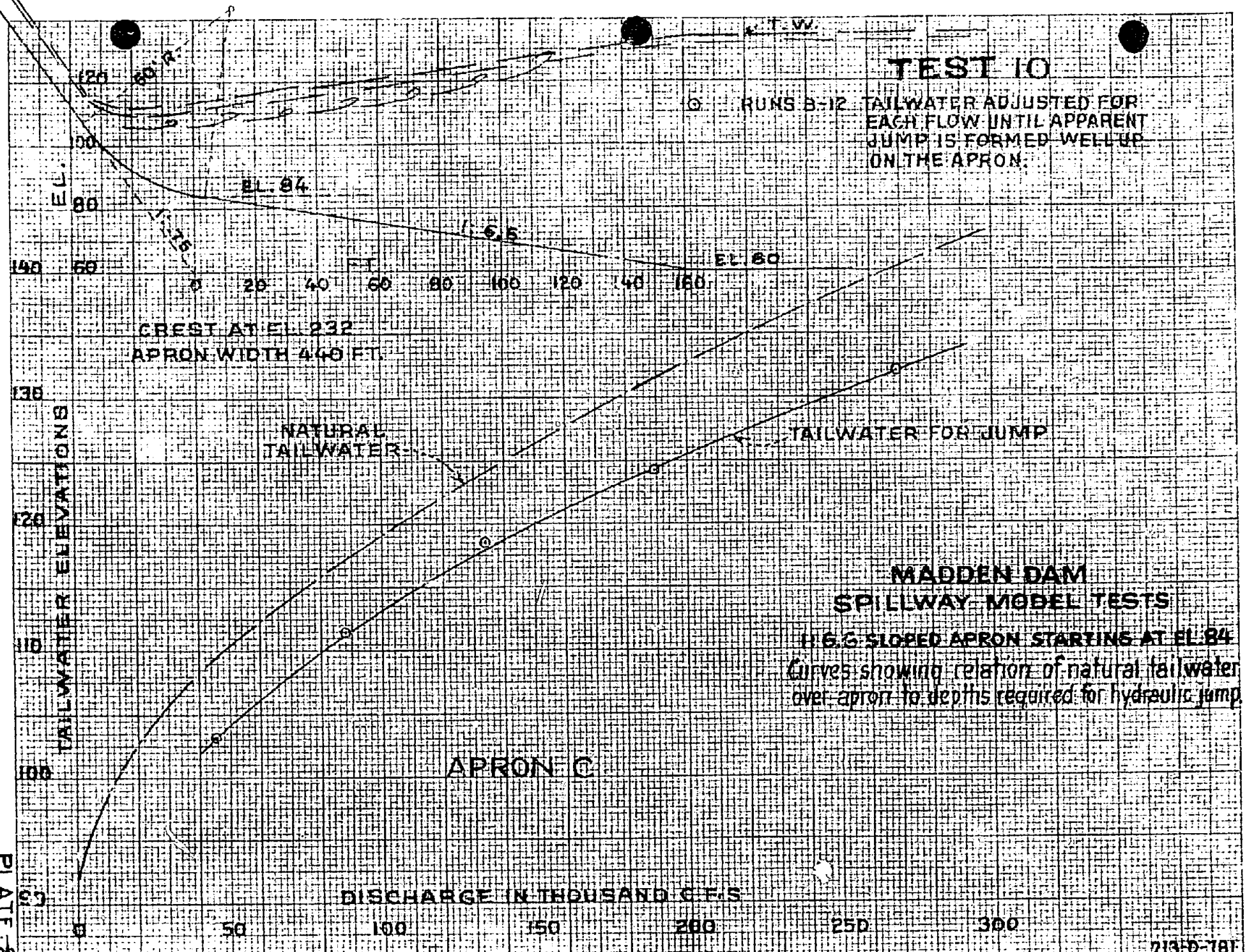

TEST 10
RUNS 8-12 TAILWATER ADJUSTED FOR EACH FLOW UNTIL APPARENT JUMP IS FORMED WELL UP ON THE APRON.
T.W.
EL. 84
1:6.5
EL. 60
CREST AT EL. 232
APRON WIDTH 440 FT.
NATURAL TAILWATER
TAILWATER FOR JUMP
APRON C
TAILWATER ELEVATIONS
EL.
FT.
DISCHARGE IN THOUSAND C.F.S.
MADDEN DAM
SPILLWAY MODEL TESTS
1:6.6 SLOPED APRON STARTING AT EL. 84
Curves showing relation of natural tailwater over apron to depths required for hydraulic jump.
213-D-781

PLATE 69

EROSION AFTER MAY FLOW

EROSION AFTER
ULTIMATE OPERATION

1 6 SLOPED APRON STARTING AT E

1:4 Sloped Apron Starting at El. 90:

The elevation of the apron was raised slightly as was indicated necessary from the above experiment. And again, for various flows, the tailwater was adjusted until the jump formed well up on the apron. These depths are plotted in a curve, shown on Plate 29, which follows very close to the curve of natural tailwater depths over the apron. For the higher flows the curves indicate that the apron should be raised still a little more.

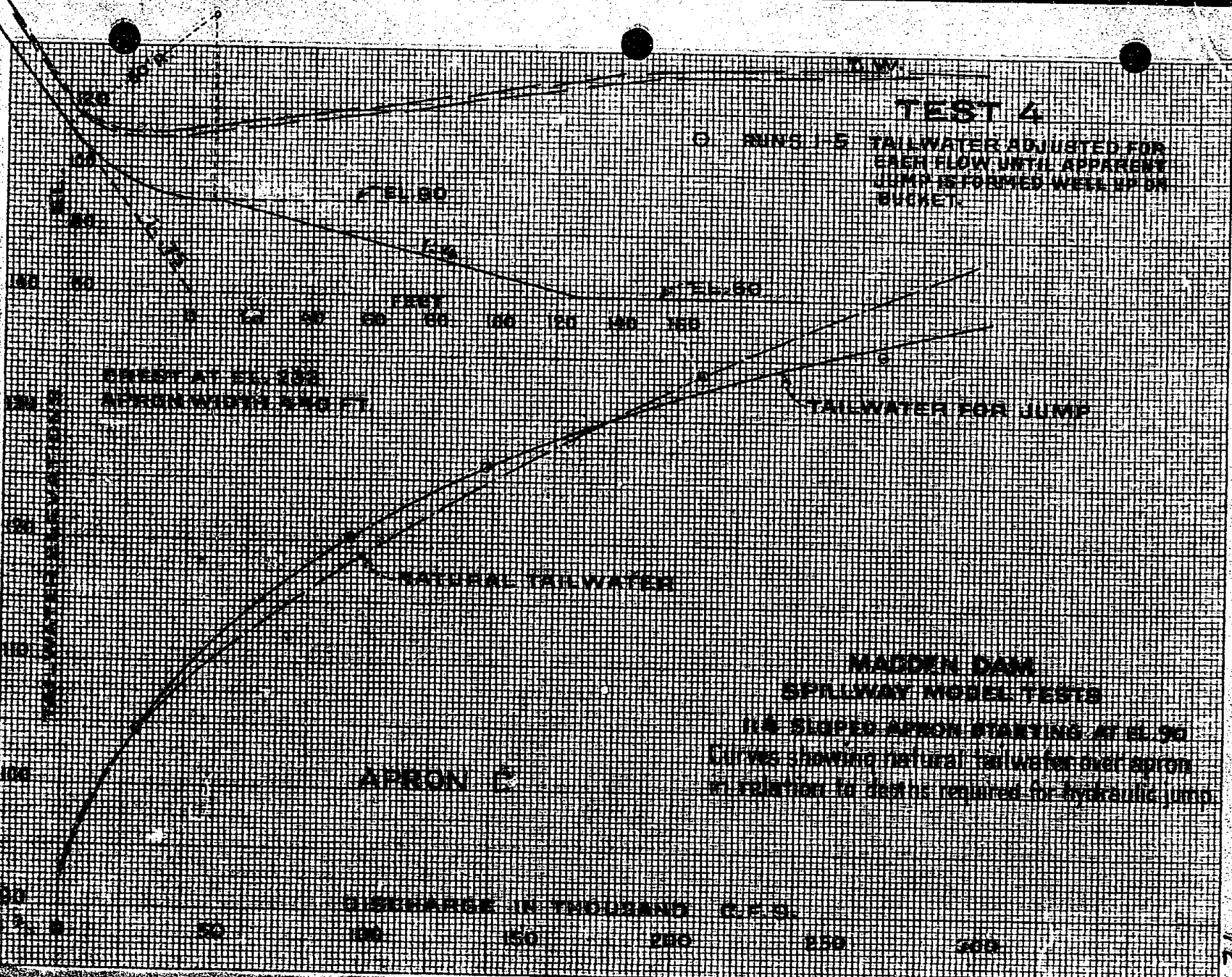

TEST 4
RUNS 1-5 TAILWATER ADJUSTED FOR EACH FLOW UNTIL APPARENT JUMP IS FORMED WELL UP ON BUCKET.
EL 90
EL 50
FEET
CREST AT EL. 362
APRON WIDTH 240 FT.
TAILWATER FOR JUMP
NATURAL TAILWATER
APRON C
DISCHARGE IN THOUSAND C.F.S.
MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRON STARTING AT EL. 90
Curves showing natural tailwater over apron
with relation to depths required for hydraulic jump.

PLATE 29

1:4 Sloped Apron Starting at El. 97.3:

Accordingly, the apron was raised slightly and the bucket curve made tangent to the apron slope and the slope of the dam face. The slope of the apron was retained at 1 to 4. A steeper slope, 1 to 2, was tried, but a satisfactory jump would not form on it. The overflow sheet cut under the tailwater and followed down the apron slope to the bottom with very little loss in velocity.

Another alteration tried was to remove the apron slope entirely, allowing the discharge to leap off the high bucket ledge. Experiments with this set-up will be discussed later under the heading of "The Jump-Off Bucket."

In this test, with the 1:4 sloped apron, the tailwater was adjusted for different settings of flow until it seemed to the eye that a good jump was forming on the apron.

A closer check on the settings that produced a maximum energy loss was obtained when a Pitot tube was placed at the end of the apron and the relative velocities measured under different tailwater depths over the apron. These curves are shown on Plate 30 and Plate 31 following, and the peak points from them have been plotted on Plate 32.

There is a very close agreement in the depths required for the jump to those actually afforded by the tailwater levels, except at the lower flows. This means that at the lower stages the jump will move down the apron some but the velocities off the end are then so low that possibilities of erosion are slight.

The highest possible velocity entering the jump, with a consequent greater tailwater requirement, is obtained when the flow is over the gates from the pond at its maximum elevation.

With the pond held constant at its maximum height, points of least velocity were obtained as the gates were lowered (Plate 33), and show depths slightly higher than those afforded by the tailwater; see Plate 34. The close agreement, however, of these depths, and those obtained from the other condition of flow, to the actual depths that will occur over the apron is accepted as ample proof that the apron is set at the correct elevation.

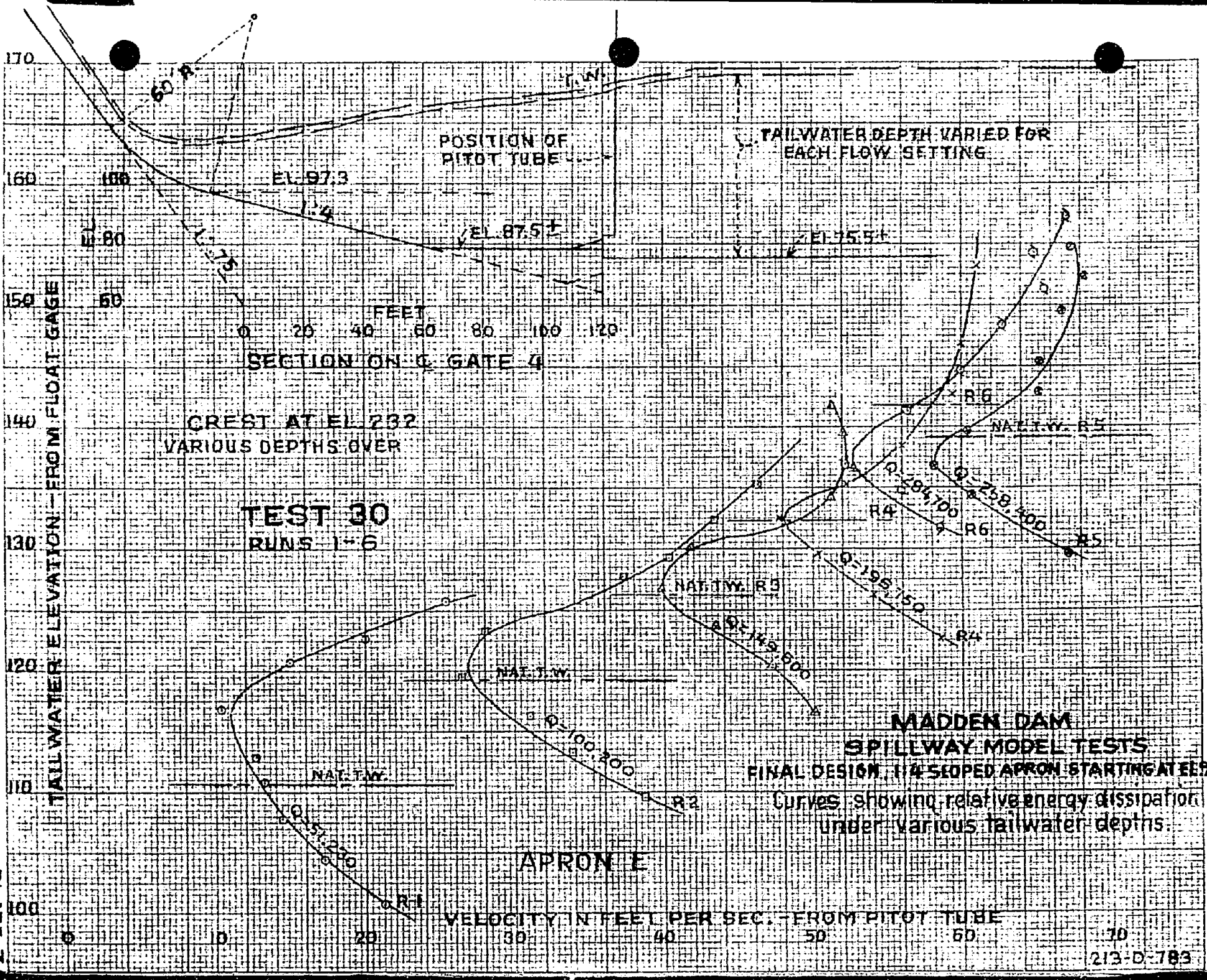

POSITION OF PITOT TUBE
TAILWATER DEPTH VARIED FOR EACH FLOW SETTING
EL. 97.3
EL. 87.5±
EL. 75.5±
1:4
FEET
0 20 40 60 80 100 120
SECTION ON ℄ GATE 4
CREST AT EL. 232
VARIOUS DEPTHS OVER
TEST 30
RUNS 1-6
NAT. T.W. R5
NAT. T.W. R3
NAT. T.W.
NAT. T.W.
R1
R2
R3
R4
R5
R6
APRON E
TAILWATER ELEVATION—FROM FLOAT GAGE
VELOCITY IN FEET PER SEC.—FROM PITOT TUBE
0 10 20 30 40 50 60 70
170 160 150 140 130 120 110 100
MADDEN DAM
SPILLWAY MODEL TESTS
FINAL DESIGN 1:4 SLOPED APRON STARTING AT EL. 97.3
Curves showing relative energy dissipation under various tailwater depths
213-D-783

PLATE 3

75

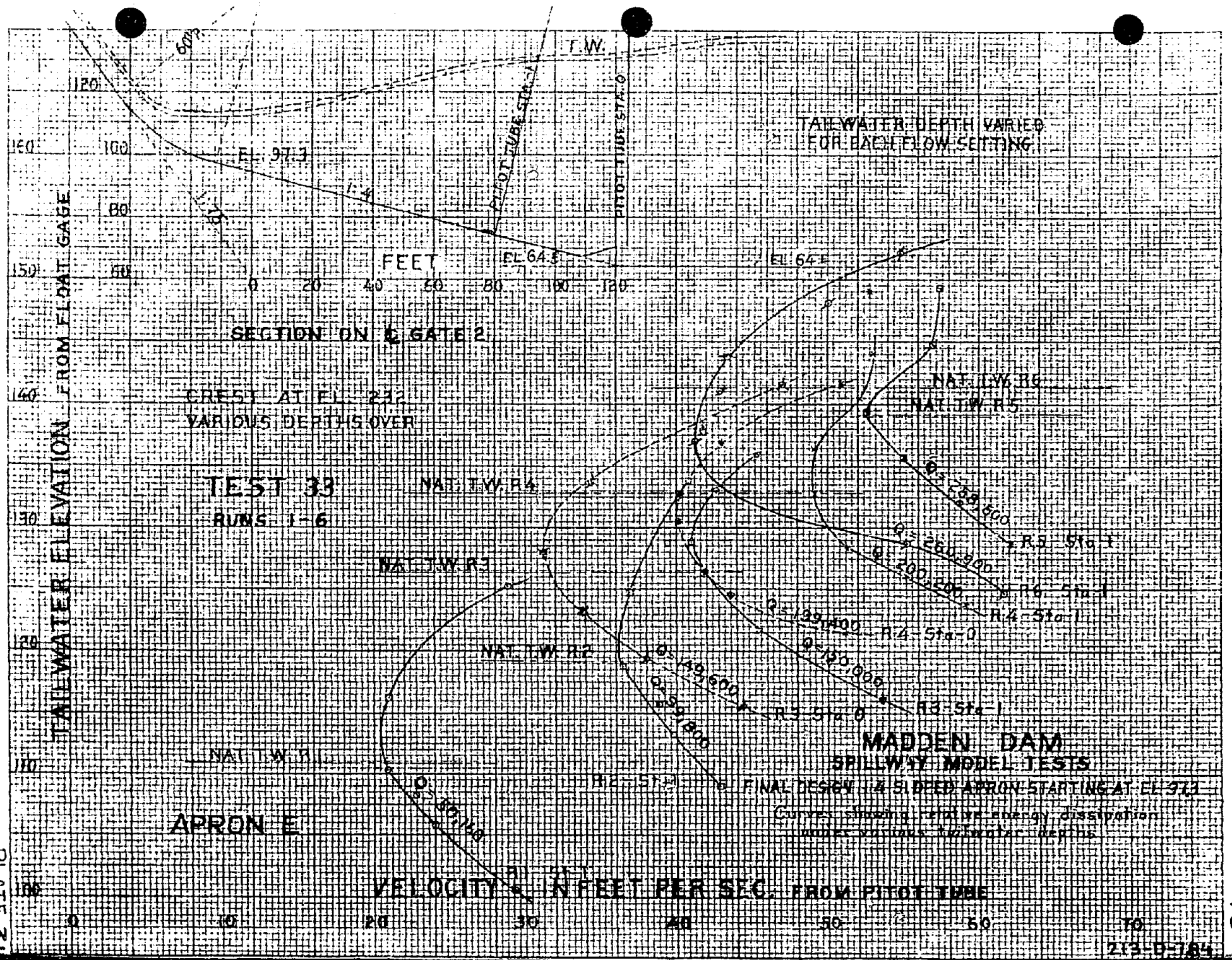

TAILWATER ELEVATION FROM FLOAT GAGE
FEET
SECTION ON ℄ GATE 2
CREST AT EL. 232
VARIOUS DEPTHS OVER
TEST 33
RUNS 1-6
TAILWATER DEPTH VARIED
FOR EACH FLOW SETTING
T.W.
PITOT TUBE STA. 1
PITOT TUBE STA. 0
EL. 97.3
EL. 64.5
NAT. T.W. R6
NAT. T.W. R5
NAT. T.W. R4
NAT. T.W. R3
NAT. T.W. R2
NAT. T.W. R1
R5 - Sta. 1
R6 - Sta. 1
R4 - Sta. 1
R4 - Sta. 0
R3 - Sta. 1
R3 - Sta. 0
R2 - Sta. 1
APRON E
VELOCITY, IN FEET PER SEC. FROM PITOT TUBE
MADDEN DAM
SPILLWAY MODEL TESTS
FINAL DESIGN 1:4 SLOPED APRON STARTING AT EL. 97.3
Curves showing relative energy dissipation under various tailwater depths
213-D-784

PLATE 31

76

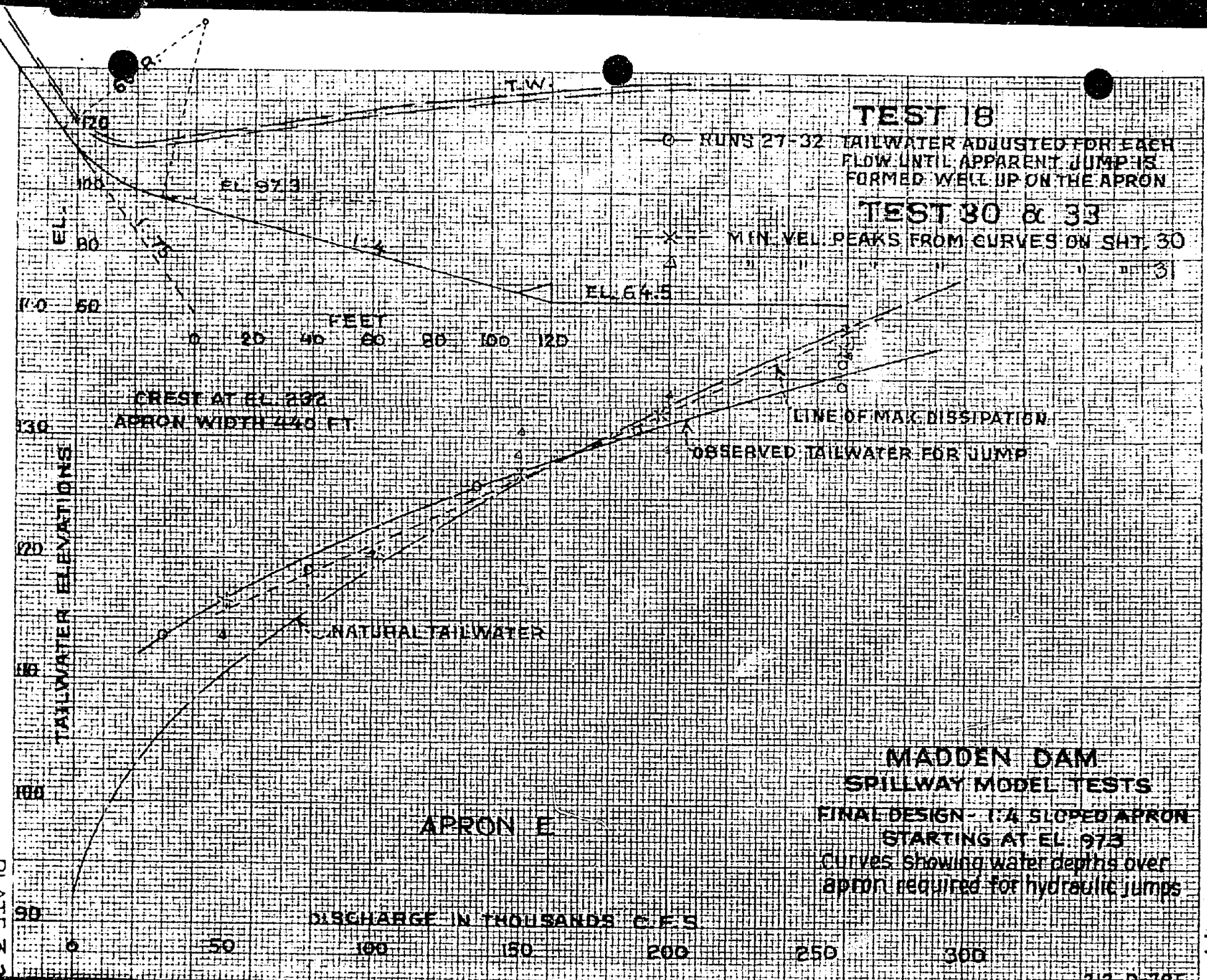
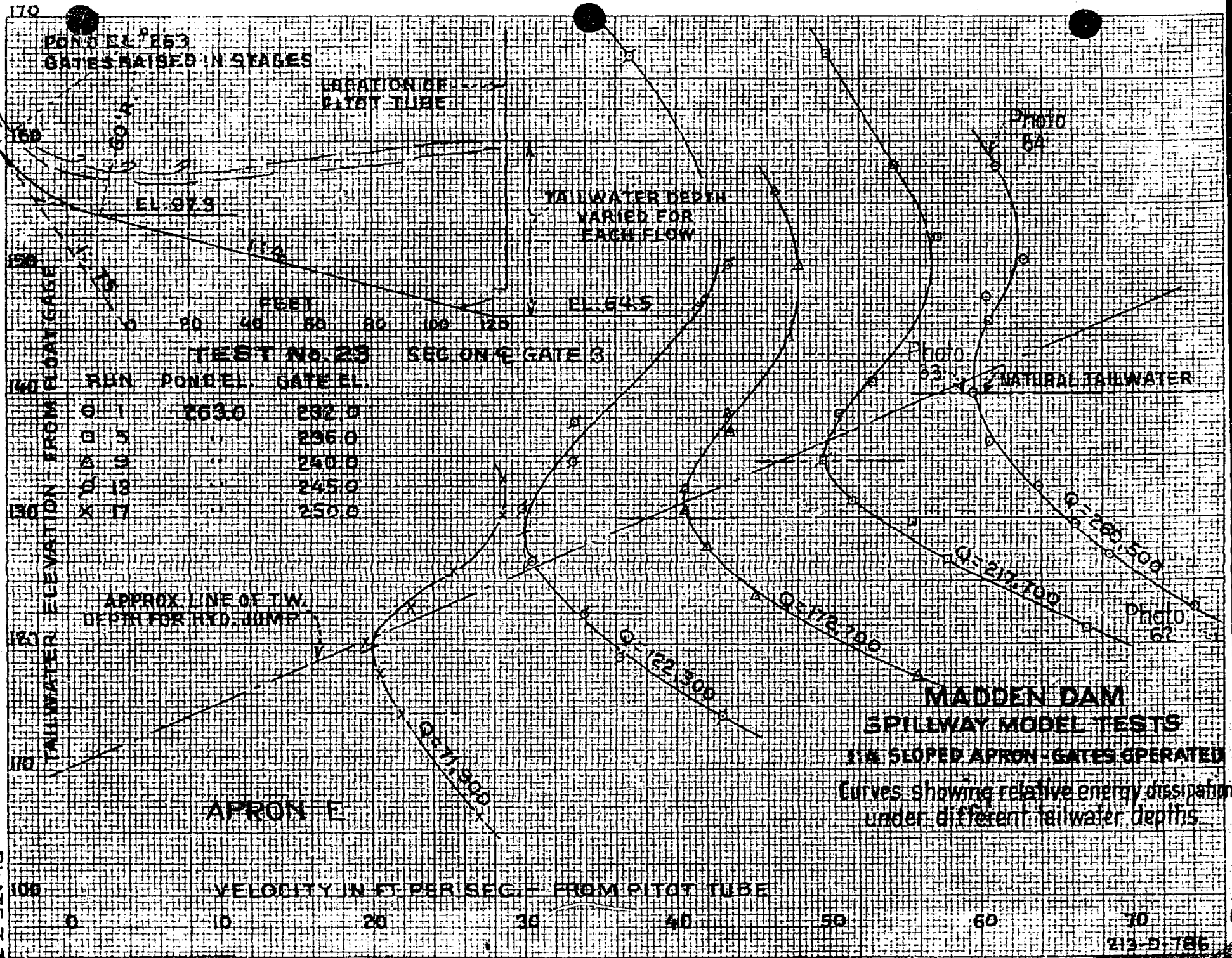

TEST 18
RUNS 27-32 TAILWATER ADJUSTED FOR EACH FLOW UNTIL APPARENT JUMP IS FORMED WELL UP ON THE APRON
TEST 30 & 33
MIN. VEL. PEAKS FROM CURVES ON SHT. 30
T.W.
EL. 973
EL. 64.5
FEET
CREST AT EL. 292
APRON WIDTH 440 FT.
LINE OF MAX. DISSIPATION
OBSERVED TAILWATER FOR JUMP
NATURAL TAILWATER
TAILWATER ELEVATIONS
APRON E
DISCHARGE IN THOUSANDS C.F.S.
MADDEN DAM
SPILLWAY MODEL TESTS
FINAL DESIGN - 1:4 SLOPED APRON STARTING AT EL. 973
Curves showing water depths over apron required for hydraulic jumps
713-D-785

PLATE 32

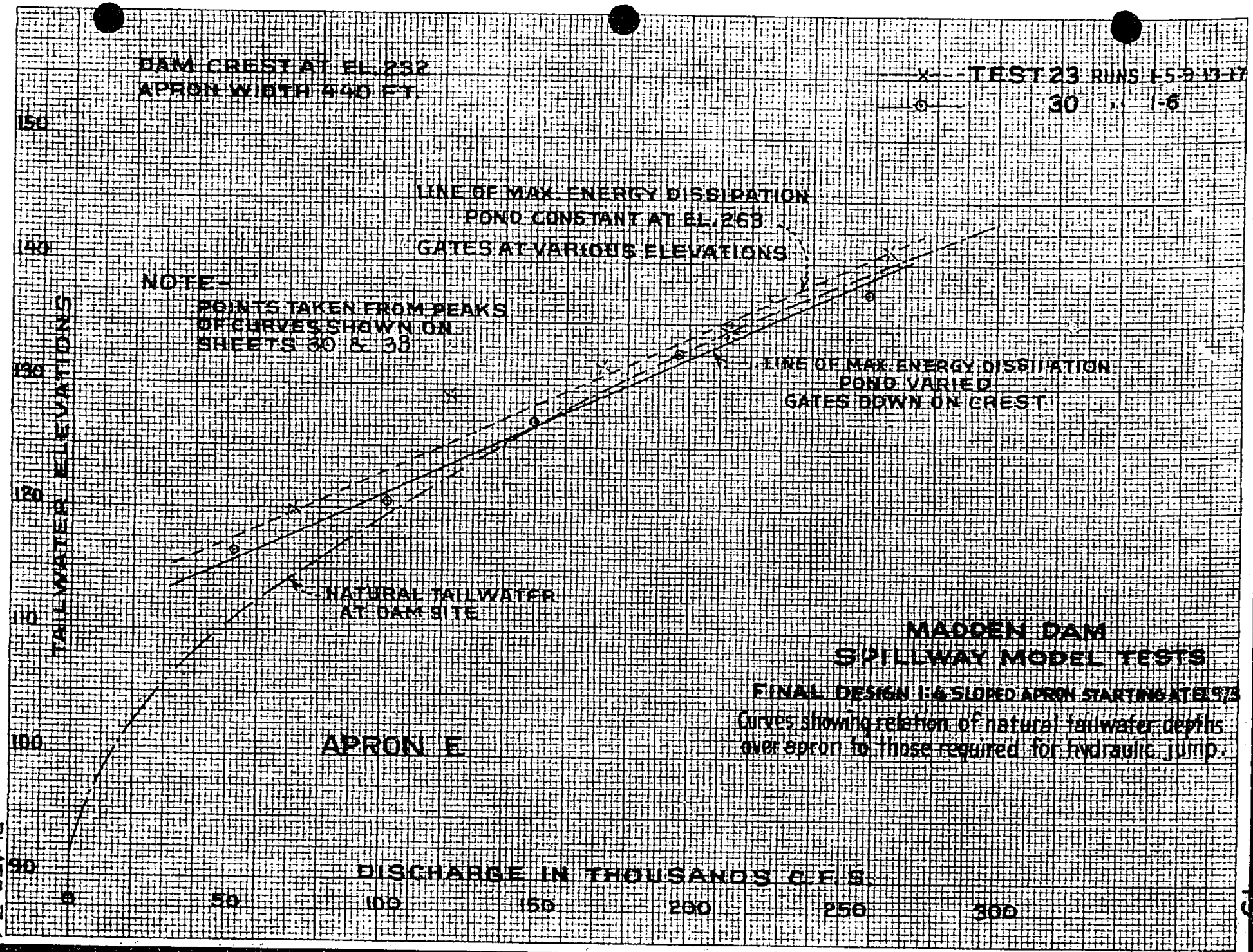


MADDEN DAM
SPILLWAY MODEL TESTS
1:6 SLOPED APRON - GATES OPERATED

Curves showing relative energy dissipation under different tailwater depths

213-D-186

PLATE 33



MADDEN DAM
SPILLWAY MODEL TESTS

FINAL DESIGN 1:4 SLOPED APRON STARTING AT EL. 975

Curves showing relation of natural tailwater depths over apron to those required for hydraulic jump.

PLATE 34

In order to save excavation on the right bank, the apron was gradually raised, in horizontal lines up and downstream, to El. 83 at the right wall. This is about the maximum elevation to which the apron can be raised as seen from test 5, Plate 26. These curves indicate that the correct elevation for a horizontal apron would be 2 to 5 feet above the El. 80 apron tested.

Photo 55 on Plate 35 following shows the erosion after a flow of 260,000 c. f. s., over the raised apron, which when compared to Photo 54 of the erosion from the constant slope apron set-up, indicates that no harm results from raising the apron in this corner. In fact, it seems to be a little better judging from the extent of the erosion. The flow was allowed to continue for the same length of time in each instance.

The standard sand bin adopted, seen in Photo 53, was a compromise between the question of getting a bin below the apron of any size and of losing the effect of the contours by their removal. It was thought that the excessive erosion of the sand in the far corner was due to the interference of the contours that formed a barrier to the flow. The stream, when striking them, would be deflected downward, creating an eddy or roll that would tend to lift the sand out. This seemed to be true as much less erosion resulted

when the contour block was removed from the far corner; see Photo 56. It would then seem that no concentration of flow or any less dissipation of energy was occurring on the right side where the apron was raised.

In the above-mentioned tests there was a 6-foot sloped-face sill used at the end of the apron. It was thought that this height could be lowered, perhaps, to an advantage, as the sill was designed as a deflector rather than a barrier to velocity along the bottom. The theory is that the sill should not be too high so that it deflects the high velocity sheet from the bottom upward across the tailwater thus cutting off the effect of the back pressure necessary to produce the hydraulic jump. It should only be high enough to lift the velocities from the river bed and throw them into the backwater where their retardation takes place in the natural action of the jump. If the sill itself is going to be used to retard the velocities it should be high enough to cover the range of the greater velocities of the jet along the bottom, and consequently its height would vary inversely with the height of the dam, for the higher the dam the thinner the overflow sheet will be at the toe. Another factor, of less importance, is the depth of water that is going to be over the sill. The height of the sill is modified directly by thadepth of the tailwater, for the greater its depth

the thicker the overflow jet will become in passing through it, which calls for a higher sill to cover the range of the thicker jet. A sill or blocks with no tailwater over them is practically useless, as the jet is merely sprayed into the air, almost as high as the original water surface, and falls to the river bed gaining practically its original velocity. There must be tailwater to float it off, so to speak, after the jet has been deflected from the river bed. At that, unless the tailwater is very deep, the high velocity is merely transferred to the surface of the stream and will erode the banks, being in some instances, as great a source of danger as erosion to the stream bed.

Photo 57 on Plate 36 shows the erosion after a run of the maximum flow, with a 4.5-foot sill at the end of the apron instead of the 6-foot sill as used in the test shown in Photo 55. The 4.5-foot sill is apparently just as effective as the higher sill.

The power house wall downstream was raised to El. 140. Note that less erosion resulted in the bin below (Photo 58). This point will be further discussed later under the heading, "The Power House Wall."

Other experiments under this test included extension of the apron with sills in various positions. With the apron extended

20 feet below the sill, a greater erosion resulted in the sand bin. At least, paving below the sill does no good, as the water behind it is practically dead. When the sill was placed at the end of the extension, however, considerably less erosion of the sand resulted. Of course, the greater the length of apron, the less will be the erosion below it, as the present length of apron does not cover the range of the jump action, during the greater floods. With the sill reversed, vertical face turned upstream, the erosion was lessened due to the sill taking the impact of the velocities along the floor. This turned the jet upwards with great bubbling which tends to destroy the jump action and places the dependence on the sill for velocity retardation. At the lower flows there was no impact on the sill noticeable because for jumps of less magnitude the length required for full recovery is less and the high velocities are lost before reaching the end of the apron. This is clearly illustrated in Fig. 1, Plate 38, which shows the velocity contours through the jump for a flow of about 80,000 c. f. s. Although the velocity entering the jump is around 93 feet per second, the highest recorded velocity on the floor below the apron was only 13 feet per second. The sill was removed for this run.

There is little or no loss by friction in the water flowing over the crest, about 99 per cent of the total head being present at the toe to create the velocity entering the jump; see Plate 85. If on the model there is a satisfactory jump performance with the maximum velocity obtainable, there is a factor of safety resulting, as the friction on the prototype may be expected to be greater and consequently a less depth of tailwater actually will be required. To see, however, just how much leeway was possible, the tailwater was lowered in an attempt to sweep the jump off the apron. With the tailwater 5 feet below that afforded by the natural depth at the damsite, there was no appreciable change in the position or action of the jump; see Photo 60, Plate 36, and Fig. 2, Plate 38. At ten feet below normal, the jump has started to move down the slope and the velocities leaving the end of the apron are increased somewhat; see Photo 61, Plate 37, and Fig. 3, Plate 38. The river bed below, however, is still amply protected by the sill which lifts the velocities up and into the jump, that has now moved slightly downstream. When the tailwater is lowered 20 feet below natural; see Photo 62, the start of the jump has moved away from its normal position up on the curve of the bucket, as shown in Photo 63, but yet does not sweep off the apron.

Thus it is seen that there is a considerable factor of safety afforded if there should exist any variance in the actual performance of the prototype from the expected performance as predicted from the model tests. Also, if the tailwater should be lowered by erosion of the gravel deposits in the river bed below, or should depths exist different from those predicted by the rating curve, the effect on the jump performance would be slight.

Photo 64, Plate 37, shows the flow conditions with the tailwater raised 20 feet above the elevation that is expected to exist with the maximum flood. In this case the jump is drowned out, and although the velocities leaving the end of the apron are not greatly increased; see Plate 33, they will persist for a considerably greater distance downstream than if the jump occurred; see Plate 35 preceding. Due to the upward deflection by the sill of the floor velocities there was practically no change in the erosion in the sand bin below the apron, for a wide change in depths of tailwater; see tests under "Stepped Apron" following.

It may be argued that a lower apron could have been used with no attempt being made to obtain the hydraulic jump, as long as the velocities were deflected up from the river bed below the apron. The results would have been far less satisfactory, as there are many disadvantages. In the first place, unless the velocities are

retarded considerably by the jump, there will be impact against a sill of almost any proportions which reverts back to placing dependence on the sill with the danger resulting from any break in it. If the velocities are deflected upward from the river bed, they will not be retarded appreciably without the jump occurring and would fall back to the bottom, causing erosion perhaps at a distance beyond the point of danger to the apron but erosion at any point is considered undesirable. If the sill was made high enough to deflect the velocities to the surface the danger of eroding the river banks is encountered as well as necessitating a heavily reinforced and anchored sill structure.

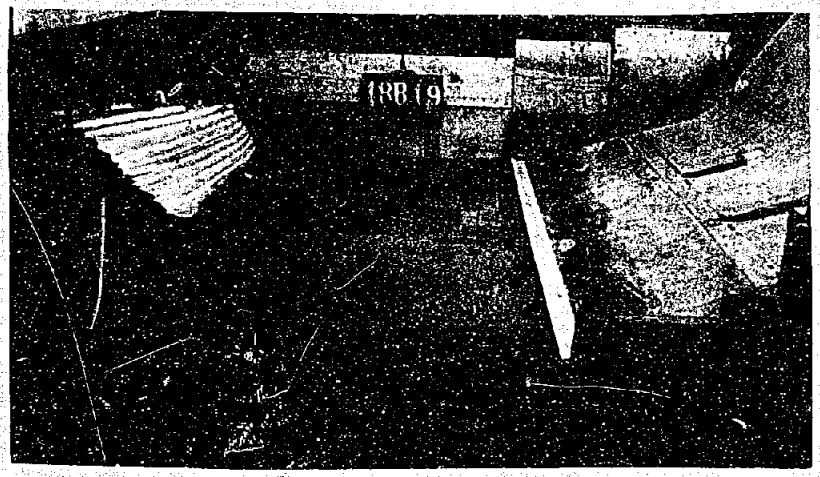


PHOTO 53
CONSTANT SLOPE APRON
SAND BIN BEFORE RUN

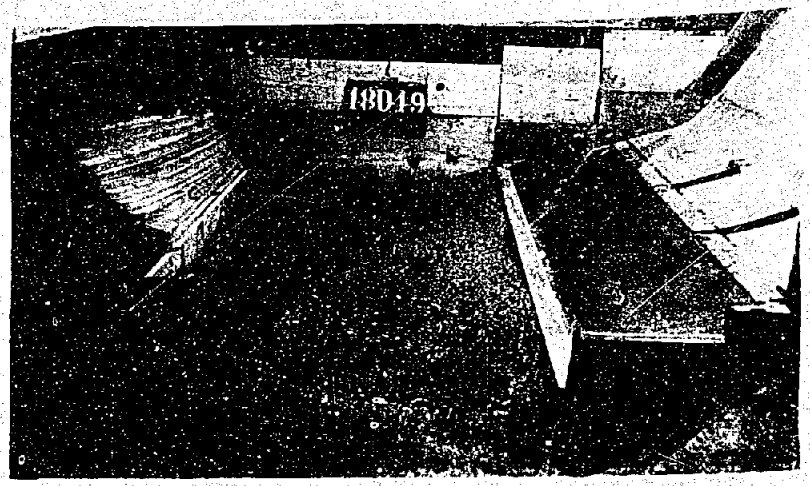


PHOTO 54
EROSION AFTER 280,000
C F S FLOW

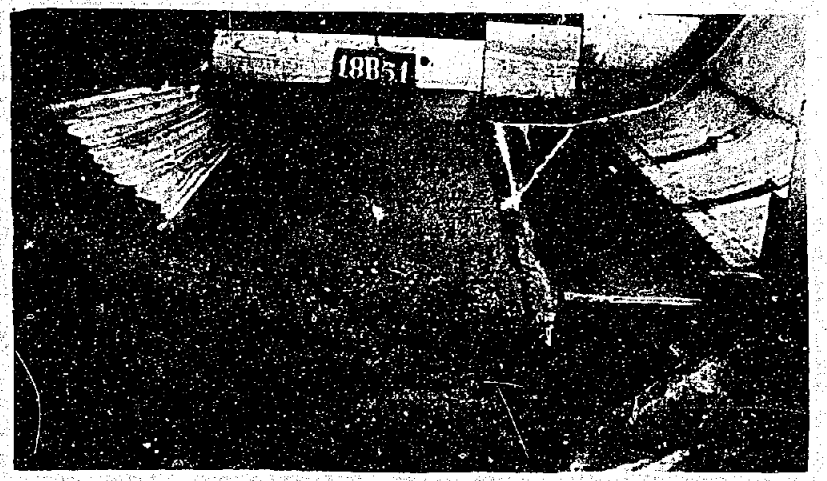


PHOTO 55
APRON RAISED AT FAR SIDE
AFTER 260,000 C.F.S. FLOW

PHOTO 56
CONTOUR BLOCK REMOVED
AFTER 260,000 C.F.S. FLOW

1 4 SLOPED APRON-6 FT. SLOPED FACE SILL

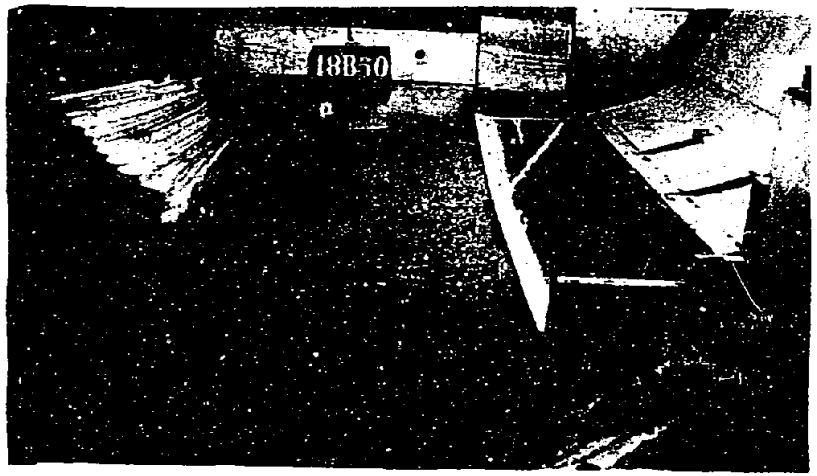


PHOTO 57
WITH 4 5-FOOT SILL
AFTER 260,000 C F S FLOW

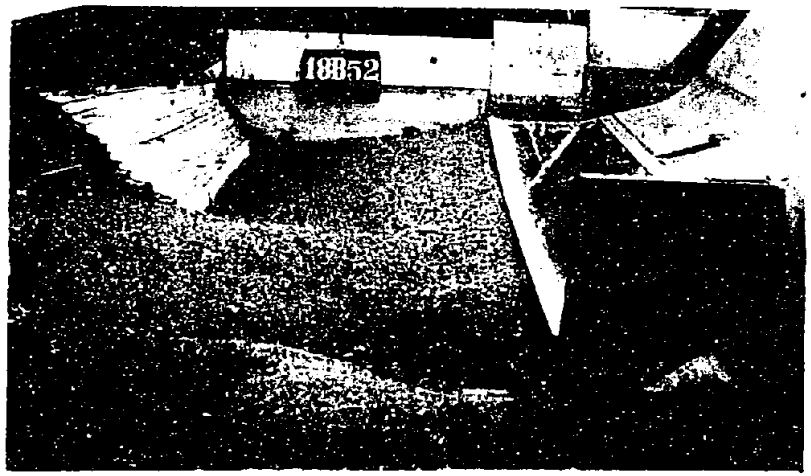


PHOTO 58
P H WALL RAISED TO EL 140
AFTER 260,000 C F S FLOW

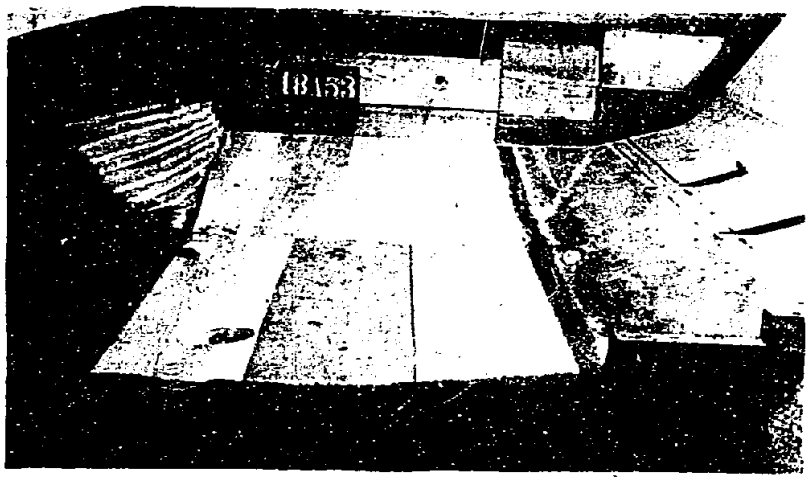


PHOTO 59
SET-UP
PHOTOS 60,61,62,63 & 64

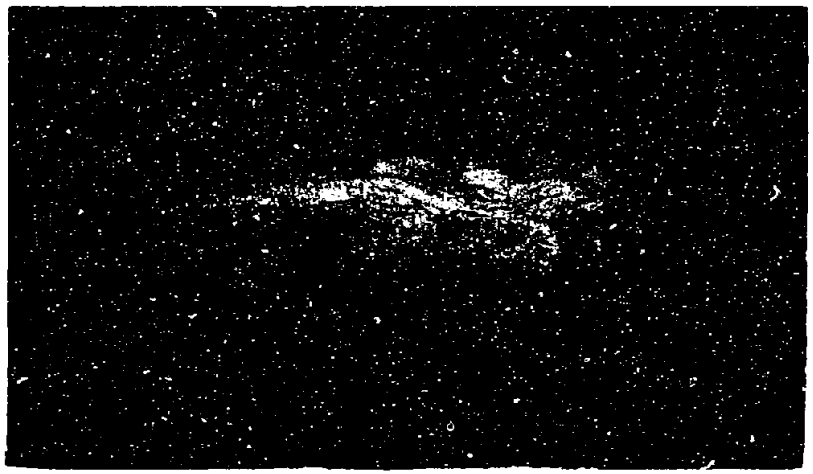


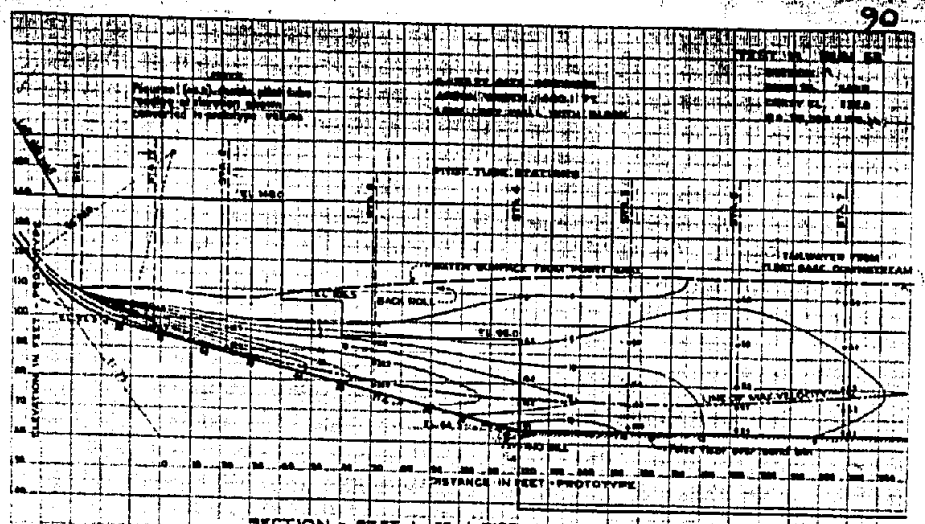
PHOTO 60
FLOW 258,000 C F S
TAILWATER 5 FT BELOW NATURAL

PHOTO 62
FLOW 258 000 C F S
TAILWATER 20FT BELOW NATURAL

PHOTO 64
FLOW 258,000 C F S
TAILWATER 20FT ABOVE NAT

FLOW 258,000 C F S
TAILWATER 10FT BELOW NATURAL

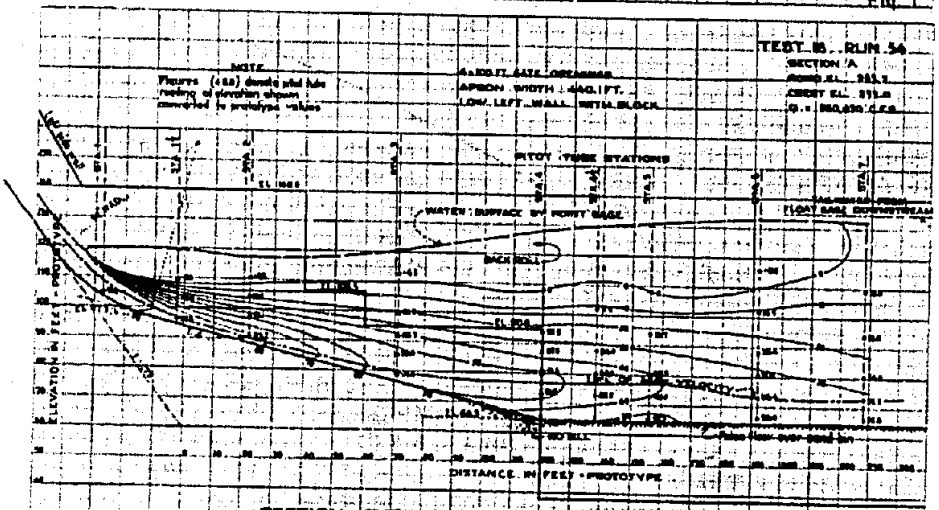
PHOTO 63
FLOW 258,000 C F S
NATURAL TAILWATER



SECTION - 87 FT. L. OF ℄ PIER 3
Q = 60,000 C.F.S. APPROX.

MADDEN DAM
SPILLWAY MODEL TESTS
Prototype velocity contours
Flow approx. 60,000 c.f.s.
Natural tailwater

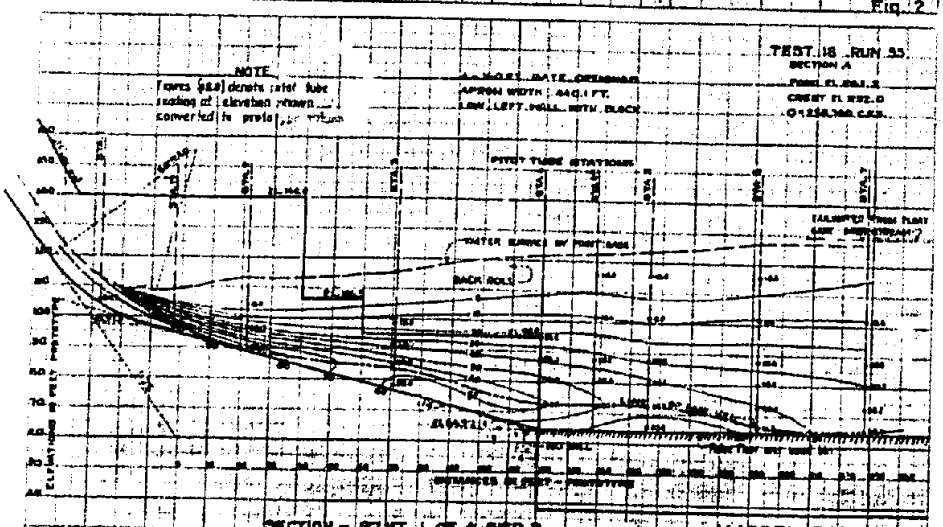
Fig. 1



SECTION - 87 FT. L. OF ℄ PIER 3
Q = 750,000 C.F.S. APPROX.

MADDEN DAM
SPILLWAY MODEL TESTS
Prototype velocity contours
Flow approx. 750,000 c.f.s.
Tailwater 3 ft. below natural

Fig. 2



SECTION - 87 FT. L. OF ℄ PIER 3
Q = 550,000 C.F.S. APPROX.

MADDEN DAM
SPILLWAY MODEL TESTS
Prototype velocity contours
Flow approx. 550,000 C.F.S.
Tailwater 10 ft. below natural

Fig. 3

The Stepped Apron:

It was thought that an advantage might be gained by building the apron in steps which would lessen the floor velocities, break up concentrations and check any tendency the jump might have to move its position up or down the slope, for a light variation in tailwater elevation.

Steps of various sizes were tried, ranging from 2 ft. high and 9 ft. wide, roughly, to 9 ft. high and 37 ft. wide, to ascertain those best adapted to the purpose. Fig. 1, Plate 39, shows sketches of the arrangements tested with data and photographs taken. Figs. 2 and 3 show velocity contours through the flow and are typical for all the set-ups. Very little difference in velocity propensity or distribution was found for any of the variations of the floor construction tested.

The steps checked the velocity only slightly for the maximum flow but seemed to raise the concentration from the floor and spread it through the tailwater to a greater extent then the smooth slope apron. Contrast Fig. 2, Plate 39, with Fig. 2 shown on Plate 81 following.

No concentration existed across the apron as seen from the transverse velocity section, Fig. 3, Plate 39. This was true also for the plain apron; in fact, little or no difference was discernible

in the action of the two aprons except at the lower flows there was perhaps more turbulence and greater energy loss caused by the steps. Also, the jump seemed more stable; that is, it would not move its position for a variation in tailwater depth as readily as on the smooth apron.

Plate 40 shows that the elevation of the apron is approximately correct as the depth of water over it, afforded by the natural river level, causes about the maximum retardation of velocity.

However, for protection of the river bed at the higher flows the turn-up sill at the end of the apron is still necessary. (See photo 66.) A 4.5-ft. sill was used in the runs shown on the following photographs, Plates 41 to 45. This height seemed to stop erosion as effectively as the 6-ft. sill; compare Photo 70 with Photo 67.

The pictures show the relative erosion for various quantities of flow, and it is seen that for the lower stages the sand in the bin is practically undisturbed. The forebay pond was kept constant at its maximum elevation and the gates lowered. This gave the highest velocity possible of the water entering the jump at the toe of the dam and is the extreme case for which the jump has to function. That its performance is satisfactory is seen by the curves

on Plate 33 which shows that the natural tailwater affords depths over the apron to cause the maximum dissipation of energy, and also that there is no indication of erosive velocities on the river bed directly below the apron. A further factor of safety is indicated when no greater erosion results if the tailwater is raised or lowered 12 feet below the normal; see Plates 42 and 43. In the foreground of Photo 73 a great bubbling is noticed which is caused by the flow striking the contour block below the sand bin. This is much less aggravated when the jump occurs, where the velocities are less, but is the cause of much of the erosion in the sand bin in the far right corner. This has been discussed before and was illustrated by showing the erosion with and without the contour block in place; see Plate 35 preceding. As this effect is more or less constant the erosion in the sand bin shows more the relative merits of the set-ups being tested than the extent to which erosion may be expected to occur. It will be noted also that excessive erosion seems to result at the base of the left or power house wall. This was stopped by extending the wall downstream. In the Photos 83 and 84 the wall was extended 60 feet downstream but it was later found that a shorter extension was also very effective.

The pictures shown here of the erosion in the sand bin below the stepped apron set-up differ little from those taken with

the smooth apron installed, insofar as showing the relative amount of erosion for the various quantities of water flowing. (Compare Photo 67, Plate 41 with Photo 54, Plate 35 preceding.) No further views of the sand bin erosion below the smooth apron will be included although in the final design with the apron raised on the right side slightly less erosion resulted. (Note Photo 55, Plate 35.)

The stepped apron has its advantages over the smooth slope, in stabilizing the jump and reducing the velocities somewhat but its construction would call for a large amount of form work and the offsets cause eddies below them that may start cavitation and disintegration of the concrete. The steps although built in staggered lines, could set up a vibration that would be injurious to the structure. They may be used to a greater advantage on installations where the natural tailwater depth over the apron for a given flow does not correspond very closely to that required for the jump, as an aid in preventing the jump from moving its position on the apron for slight variation in tailwater.

The Madden installation is particularly fortunate in having a natural tailwater that furnishes depths over the apron very closely corresponding to those required for the jump; see Plate 34 preceding.

The difficulties that may be encountered in building the apron in steps perhaps overshadow the advantages, so that no strong recommendation is made here for adoption in the design, but it is thought, nevertheless, that their use would cause an improvement in the hydraulic jump performance on the apron.

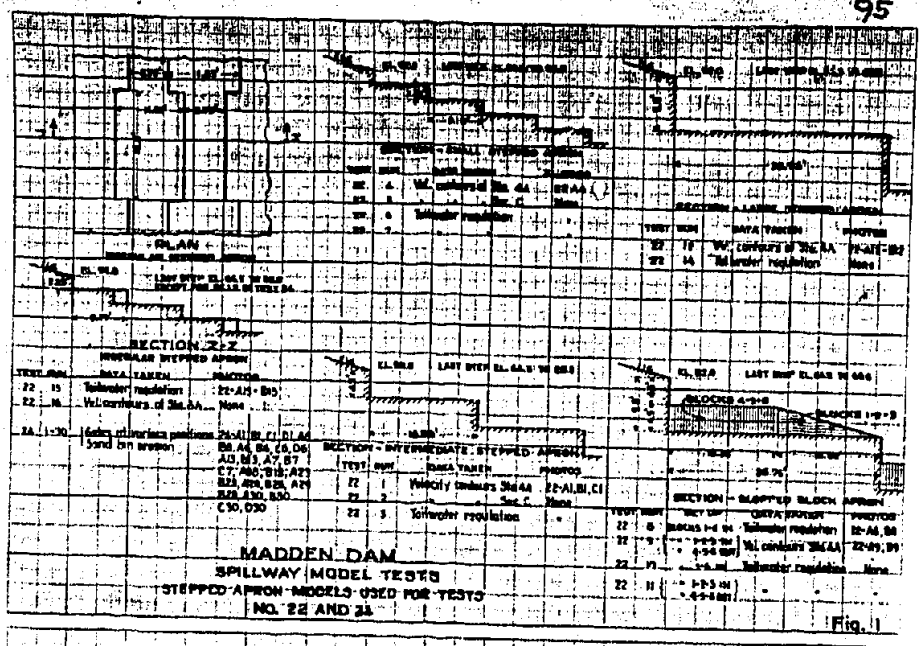


Fig. 1

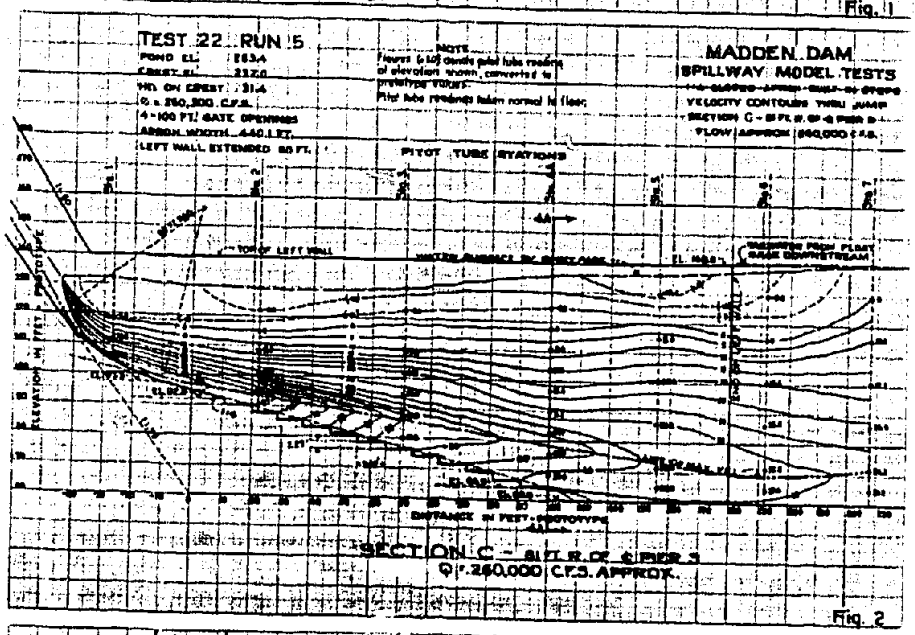


Fig. 2

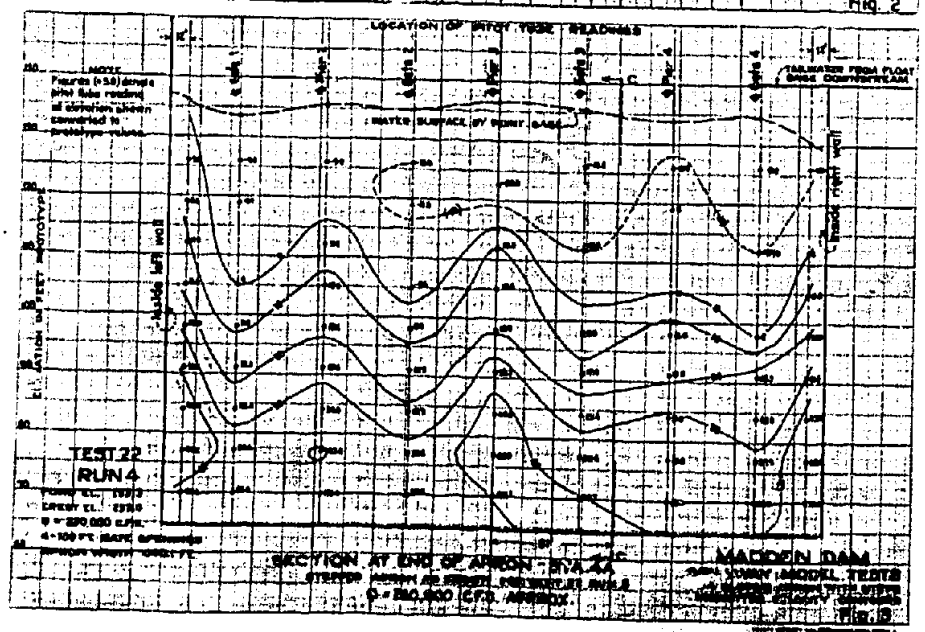


Fig. 3

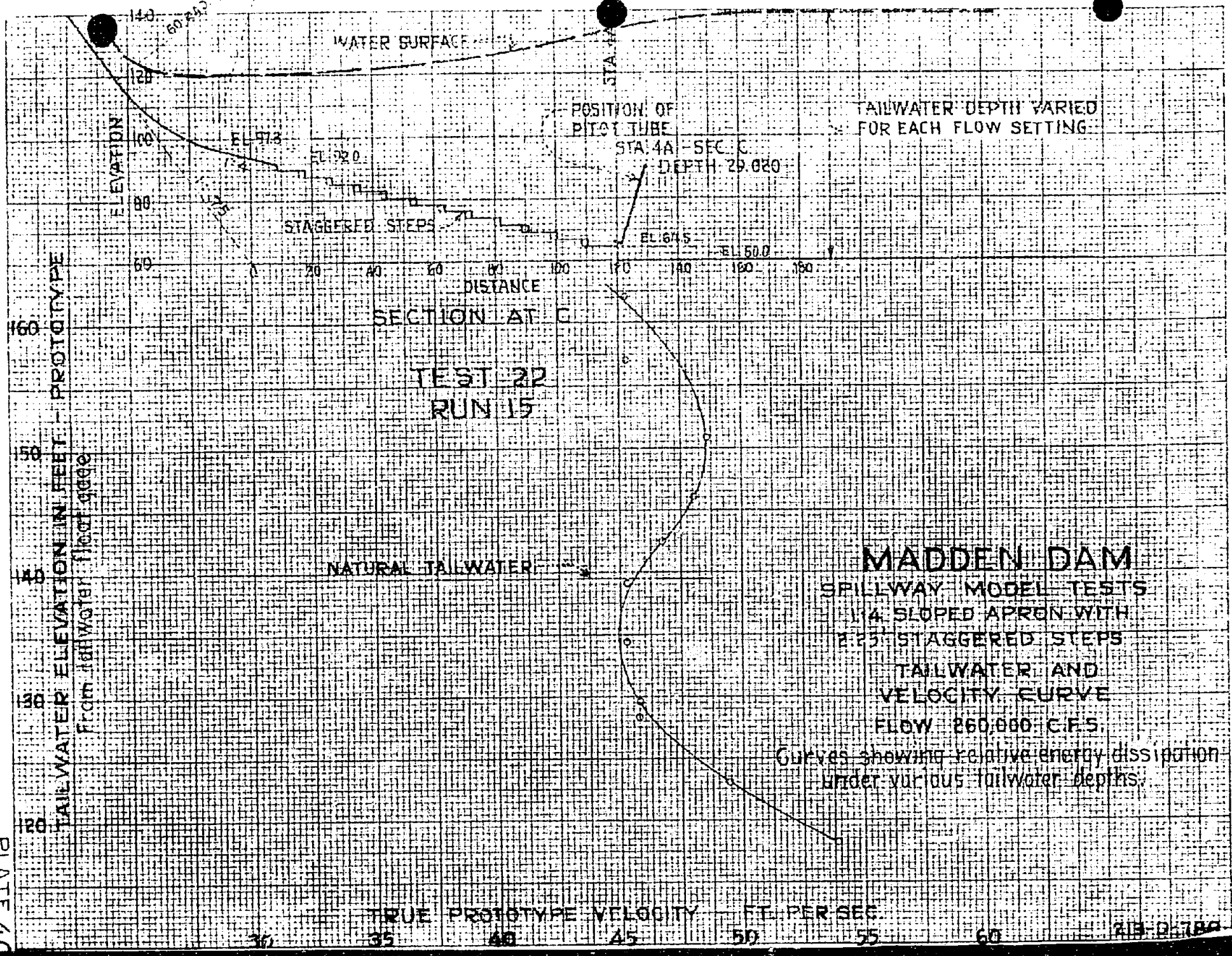

WATER SURFACE
POSITION OF PITOT TUBE
STA. 4A - SEC. C
DEPTH 29.020
TAILWATER DEPTH VARIED FOR EACH FLOW SETTING
EL 97.3
EL 92.0
STAGGERED STEPS
EL 67.5
EL 50.0
ELEVATION
DISTANCE
SECTION AT C
TEST 22
RUN 15
NATURAL TAILWATER
TAILWATER ELEVATION IN FEET - PROTOTYPE
From tailwater floor gage
TRUE PROTOTYPE VELOCITY - FT. PER SEC.
MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRON WITH
2.25' STAGGERED STEPS
TAILWATER AND
VELOCITY CURVE
FLOW 260,000 C.F.S.
Curves showing relative energy dissipation under various tailwater depths.
213-D-789

PLATE 40

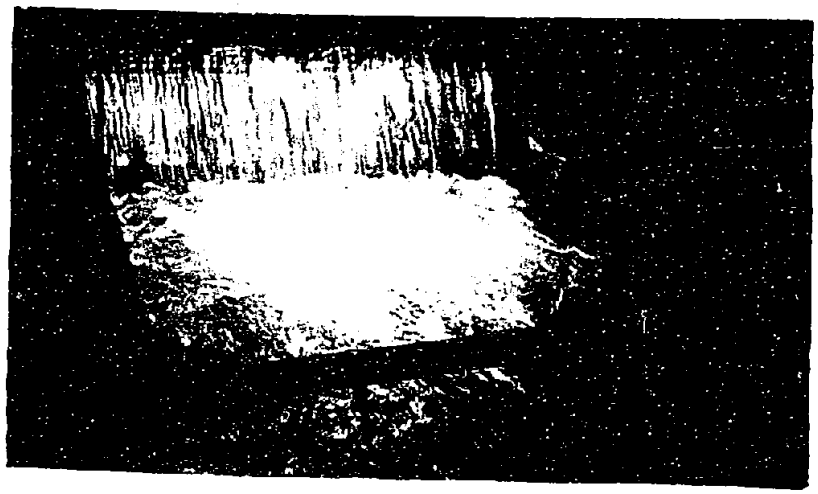


PHOTO 65
WITHOUT SILL
FLOW 257000 CFS

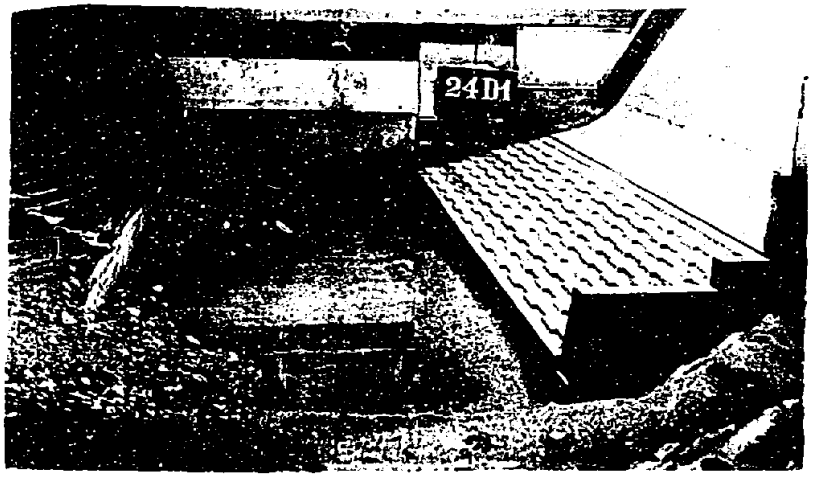


PHOTO 66
WITHOUT SILL
EROSION AFTER MAX FLOW

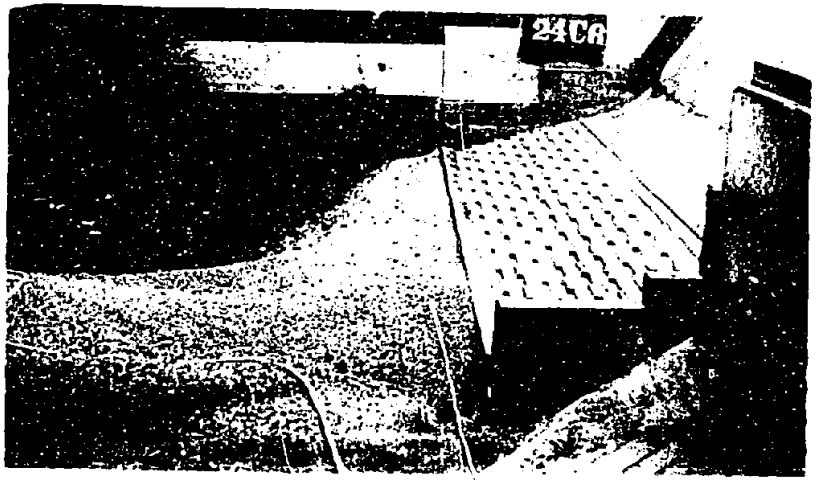


PHOTO 67
WITH SILL AT END OF APRON
EROSION AFTER MAX FLOW

PHOTO 68
WITH SILL AT END OF APRON
AND DEPOSITION IN RIVER BED

STEPPED APRON POND EL 263 GATES AT EL 232

PLATE 4

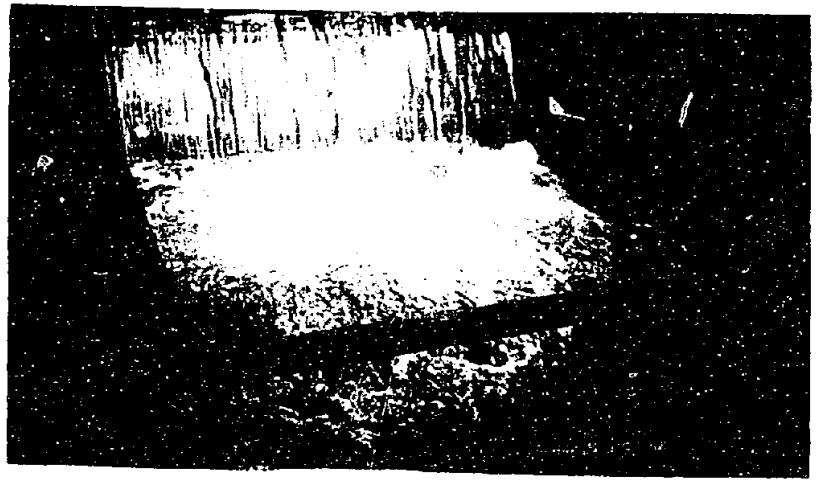


PHOTO 69
POND EL 263-GATES AT EL 232
FLOW 257,000 CFS

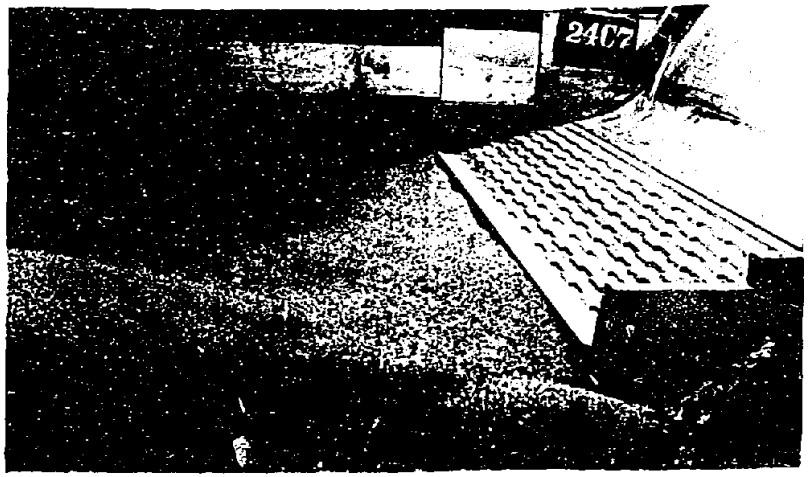


PHOTO 70
EROSION AFTER FLOW OF
257,000 CFS

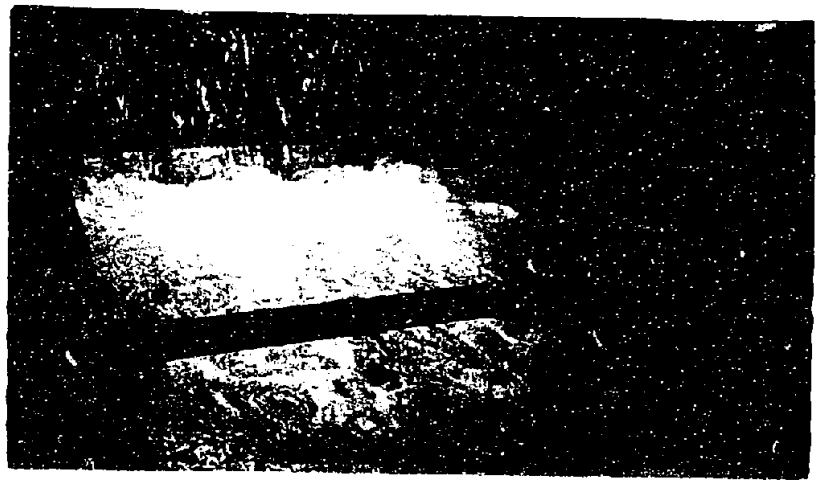


PHOTO 71
FLOW 258,000 CFS
TAILWATER [illegible] BELOW NORMAL

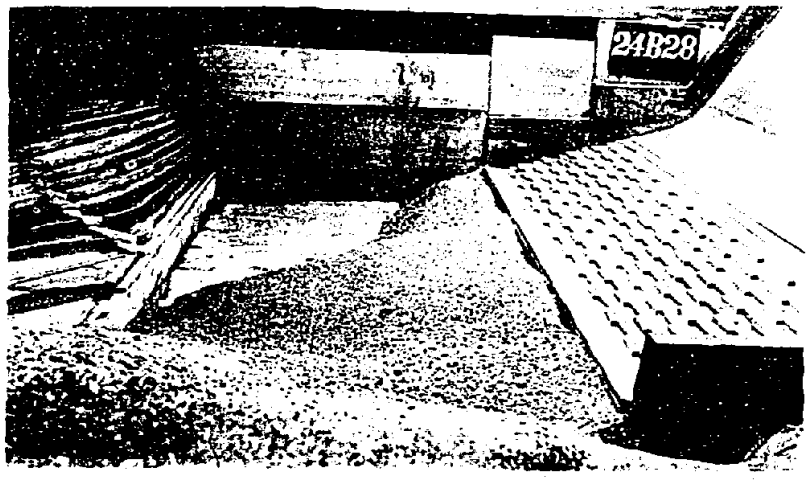


PHOTO 72
EROSION AFTER MAX FLOW
TAILWATER WAS 12 FEET
BELOW NORMAL

STEPPED APRON WITH 4.5 FT SILL

PLATE 42

PHOTO 74
EROSION AFTER MAX FLOW
WATER MASS 2 FEET
ABOVE NORMAL

PHOTO 75
EROSION AFTER 211,000
CFS FLOW

PHOTO 73
FLOW 257,000 CFS
WATER 2 FT ABOVE NORMAL

PHOTO
WATER EL 267 FEET ABOVE
NORMAL 2 FEET

STEPPED APRON-4.5 FOOT SILL-GATE OPERATION

PLATE 43

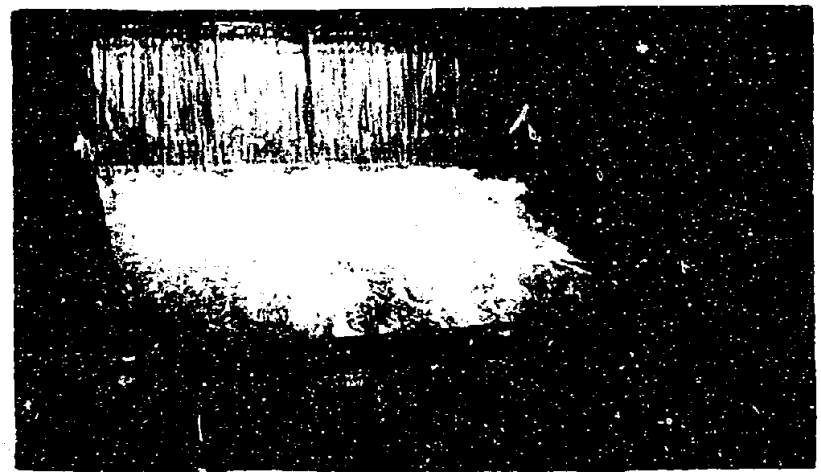


PHOTO 77
POND EL 263-GATES AT EL 240
FLOW 172,000 C F S

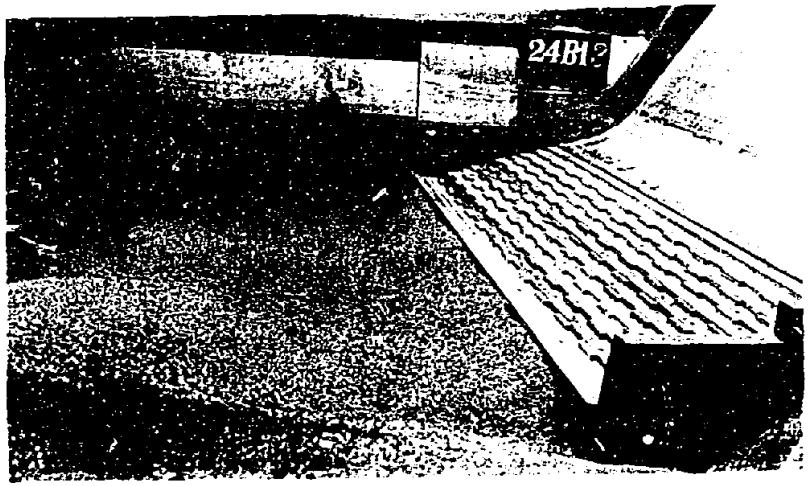


PHOTO 78
EROSION AFTER 172,000
C F S FLOW

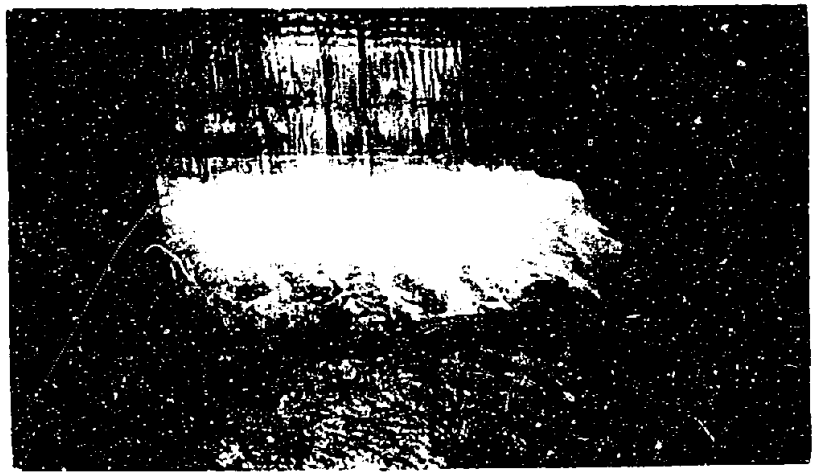


PHOTO 79
POND EL 263-GATES AT EL 245
[illegible] C F S

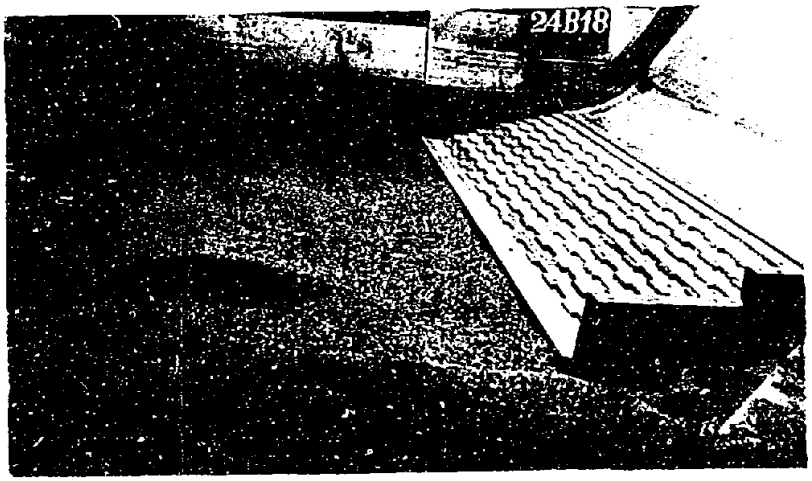


PHOTO 80
EROSION AFTER 119,000
C F S FLOW

STEPPED APRON-4.5 FOOT SILL-GATE OPERATION

PLATE 4[illegible]

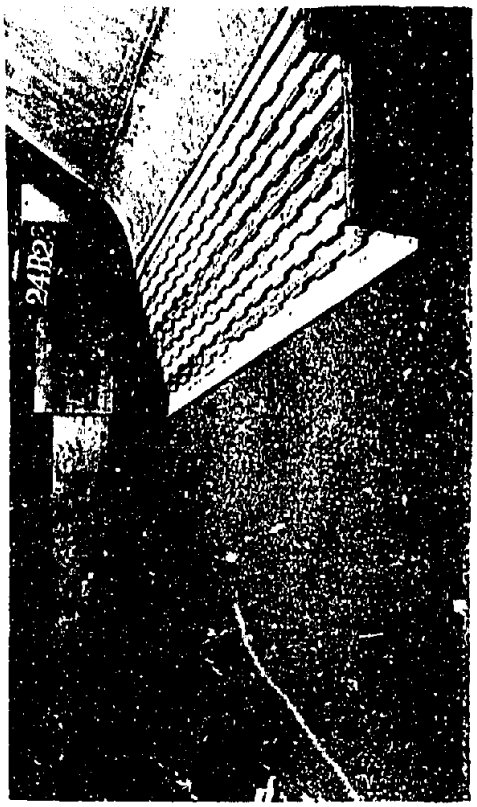


PHOTO 84
EROSION BELOW EXTENDED
POWER HOUSE WALL AFTER

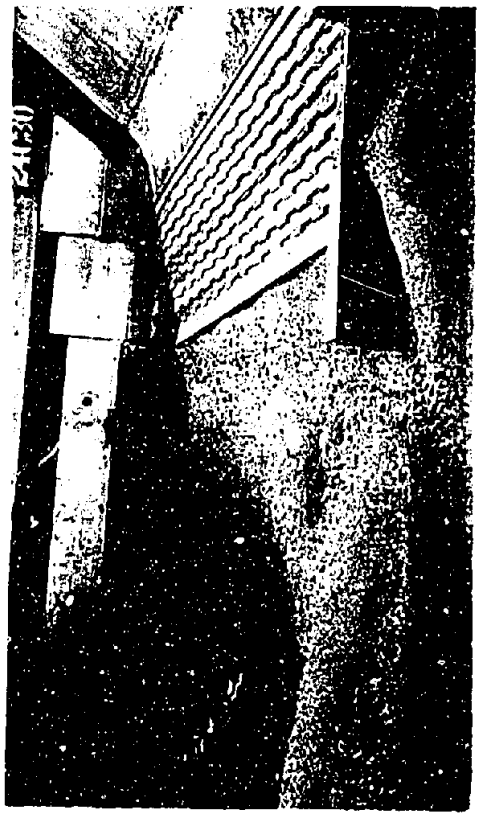


PHOTO 83
WALL EXTENDED 60 FT
DOWNSTREAM AFTER FLOW
OF 25,000 C F S

STEPPED APRON-4.5 FOOT SILL-GATE OPERATION

The Hydraulic Brake Pocket:

The hydraulic brake pocket, another alteration tried on the 1:4 sloped apron was in the form of a channel or pocket cut across the full length just below the start of the slope. In theory, this pocket was supposed to catch a portion of the overflowing sheet of water, deflect it down and around the pocket so that on emerging, at a very slackened velocity, it would unite again with the jet, causing it to thicken and have a resultant velocity much less than originally, or in other words, the pocket was supposed to create a roll or whirl underneath the overflowing sheet that would cause a drag on it and retard its velocity, thus leaving less work for the jump to do and so decrease the velocities at the end of the apron.

However, no satisfactory performances were obtained with the various size pockets tested, dimensions of which are shown on Plate 46 following. The velocity contours shown are typical, little or no change in them being discernible for various alterations of the pocket.

The pocket seemed to do little good but no harm. (Compare with test 30, run 5, Fig. 2, Plate 79, shown later.) In some instances, it might be used to save concrete. In effect, the set-up

would be a jump-off bucket (discussed later) with a supporting apron downstream, lying below the trajectory parabola, that would prevent the pressure of the tailwater from deflecting the sheet upward at the lip of the bucket.

The deflection of the maximum velocity to the surface ((see Plate 46), causing the bubbling as seen in Photo 86, Plate 47, was attributed to the fact that the sill was moved up on the apron leaving a greater distance from its top to the floor, than was present in the other set-ups. This larger space allowed the back-roll below the sill to develop with greater intensity and so deflect the jet upwards.

This would indicate that if erosion did occur below the sill it would not reach a very great depth before the return flow would be increased to an extent capable of deflecting the maximum velocities entirely away from the river bed and perhaps cause a deposition of materiel behind the sill.

This bubbling was noticed in all runs with the sand bin installed below the apron when the level of the sand was lowered to any extent behind the sill. (See Test 18, runs 20, 21 and 24, Plate 65, shown later.) When the sand bin was replaced by a floor or the contours to a level with the end of the apron this bubbling ceased.

In this test with the roll pocket, the tailwater was allowed to sweep off the apron and velocity and depth measurements were made of the overflow jet above and below the pocket.

The following tabulation gives the averages of the measurements taken. The velocities are the average of 18 readings taken with the Pitot tube at nine stations across the apron. The values are quite uniform at the various stations, no particular concentration being noticed.

The depths were taken with a point gage at the different stations and are averaged to the nearest half foot, prototype scale.

It is seen that little or no velocity is lost in the pocket and the jet seems to be unaffected. All of these readings were at the maximum flow and the water in the pocket seemed dead with no whirl apparent. At lower quantities of flow a whirl was developed but little of the velocity seemed to be lost. If the downstream edge of the pocket was raised very much, considerable impact against it developed which was not the desired action. This scheme was tried later, however, with a sill placed below the jump-off bucket.

Run	Pocket	Q C.F.S.	Before Pocket Ave. Vel. Ft. per Sec.	Before Pocket Ave. Depth Ft.	After Pocket Ave. Vel. Ft. per Sec.	After Pocket Ave. Depth Ft.
7	2	260,000	96.7	6.0	96.0	6.5
8	2A	258,500	96.7	6.5	95.6	6.0
13	3	260,000	95.8	6.5	96.4	6.0
15	3A	260,900	96.4	6.0	94.8	6.0
18	3B	260,900	96.3	6.0	96.0	5.5
21	4	256,000	96.0	6.0	91.0	6.5
23	4B	260,500	96.5	6.5	96.5	6.0
Averages		259,600	96.3	6.21	95.2	6.1

At Sta. 1A - Before the pockets.

d (theoretical) = Ave. Q / ave. vel. x width (439.5 ft.)

= 259,600 / 96.3 x 439.5

= 6.13

d (ave. measured) = 6.21

Difference = .08 prototype value

= .08 ÷ 72 = .00111 ft. = 1/64- inches

Diff. in per cent = .08 ÷ 6.21 = .0129 = 1.3%

Vel. (theoretical) = 259,200 / 6.2 x 439.5

= 95.2

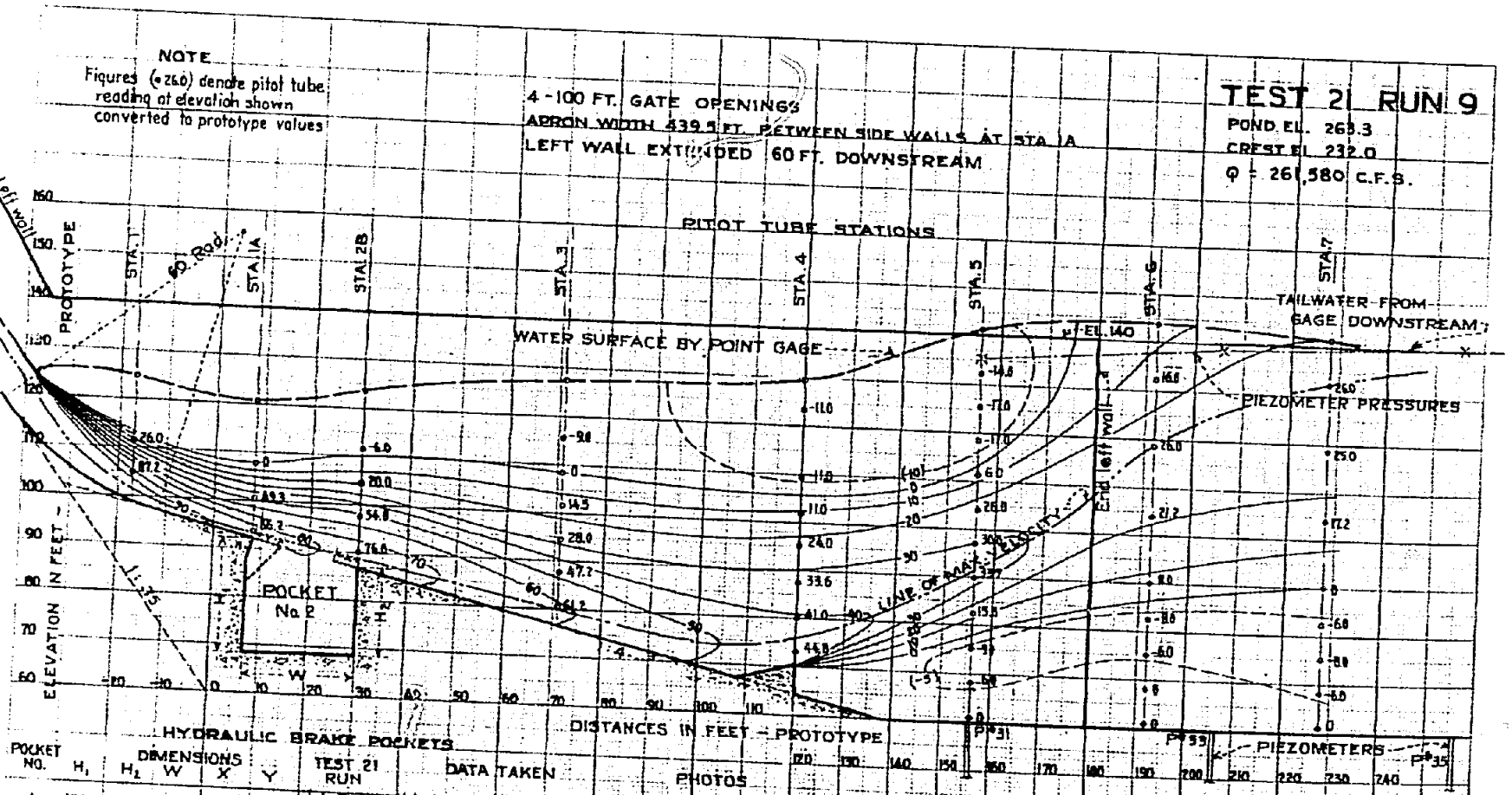
Vel. (ave. measured) = 96.3

Difference = 1.1 ÷ 96.3 = 1.14%

From the above tabulation some interesting data may be obtained and also a check on the accuracy of the testing. Taking an average of the readings before the pocket it is seen that the velocity was measured as 96.3 ft. per sec. and the depth of the jet as 6.21 ft., prototype scale. Computing the depth from the discharge and velocity recorded gives 6.13 ft. or computing the velocity from the observed quantity and depth of flow gives 95.2 ft. per sec. against the measured 96.3 ft. per sec. as indicated by the Pitot tube. This shows a balance, deviating a little over one per cent and is considered as very satisfactory evidence of the accuracy of the apparatus and performance of the testing.

The development of the roll pocket was attempted primarily as a means of saving concrete in the very thick apron which had to be raised considerably to cause the hydraulic jump to form on it. Later this heavy apron was figured in the stability of the dam, greatly increasing the factor of safety against sliding and earthquake stresses.

Hence, further experiments with it were discontinued. However, it is thought that some such pocket could be developed that would be efficient in retarding the velocities of the overflow. It seemed that the pocket should be larger than those tested and have a greater overhang of the upstream edge.



HYDRAULIC BRAKE POCKETS

POCKET NO.	DIMENSIONS H_1	H_2	W	X	Y	TEST 21 RUN	DATA TAKEN	PHOTOS
1	12.7	8.0	18.0	0	0	1-2-4	Velocity contours	21-A1-B1-A2-A4
2	17.4	13.4	22.8	0	0	5	Tailwater regulation	
3	22.2	17.0	27.0	5.0	0	7-8-9	Velocity contours	
3A	22.2	18.5	27.0	5.0	0	10	Tailwater regulation	21-A10
3B	22.2	20.0	27.0	5.0	0	11	"	
4	22.2	15.5	27.0	5.0	0	12-13	Velocity contours	21-A12
4B	22.2	18.5	27.0	5.0	6.0	14	Tailwater regulation	
						15	Velocity contours	
						16	" "	
						21	" "	
						22	Tailwater regulation	
						23	Velocity contours	

MADDEN DAM
SPILLWAY MODEL TESTS
SLOPING APRON WITH ROLL POCKET
Prototype velocity contours on apron
Flow 260,000 c.f.s. approx.
T.W. depth 80 ft. approx.

PLATE 46

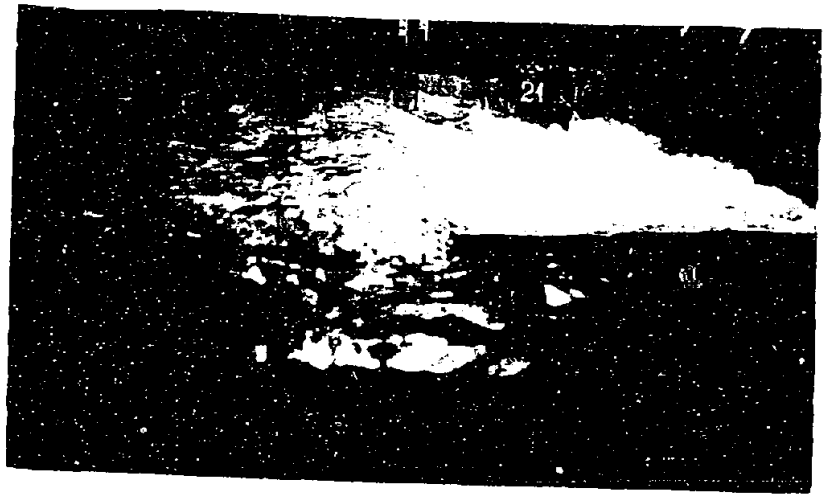


PHOTO 86
MAX FLOW OVER ROLL
POCKET APRON

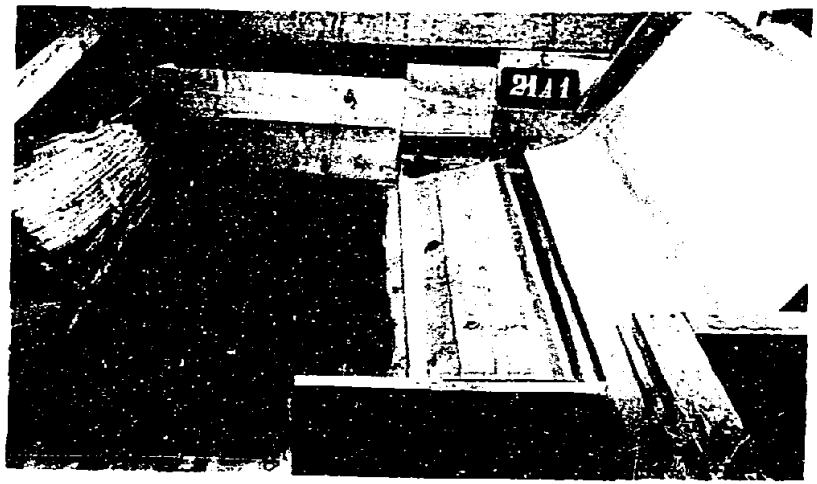


PHOTO 85
ROLL POCKET IN 1:4
SLOPED APRON

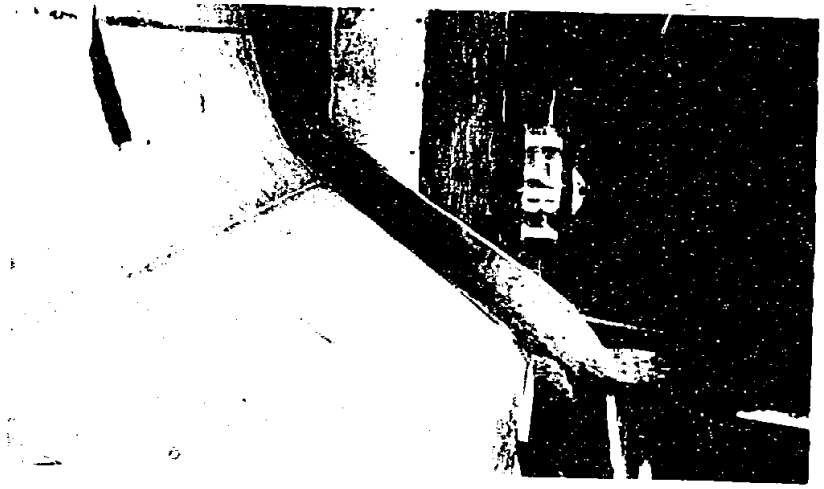


PHOTO 88
PRESSURE PIEZOMETER
OPENINGS ON SIDE WALLS

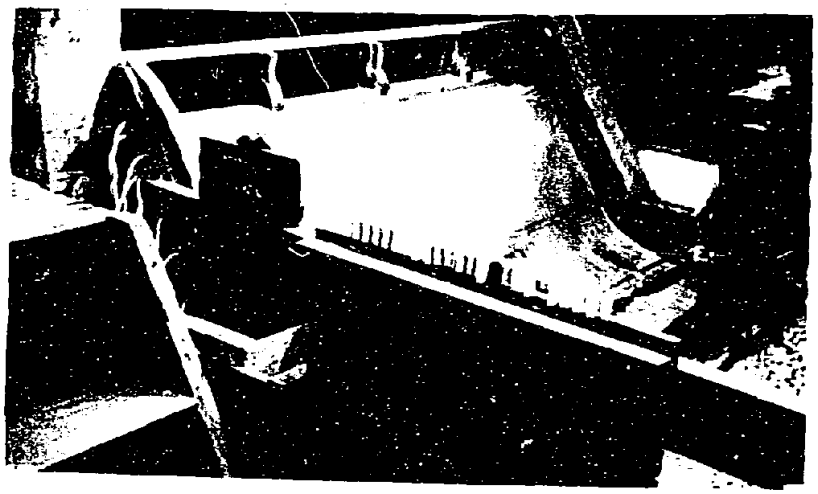


PHOTO 87
SIDE WALL PIEZOMETERS
CONNECTIONS TO GAGE BOARD

The Jump-Off Bucket:

It was early seen that it would be necessary to project the overflowing sheet into the tailwater at an elevation higher than that originally shown at El. 60, in order that the hydraulic jump could function properly and not be drowned out. Preliminary computations showed that if the water entering the jump was at about El. 90, the natural tailwater would furnish depths over this point necessary to form the hydraulic jump.

In an attempt to save concrete in the apron and still secure some sort of jump action to dissipate the energy of the overflow, a set-up was tried with the curved bucket raised to El. 90 with a straight drop off down to the apron at El. 60.

The overflowing sheet struck the wall of tailwater on leaving the ledge of the high bucket and was deflected upward in a spray; see Photos 91 and 92, Plate 48 following. At all stages of discharge the natural tailwater was at an elevation higher than the lip of the bucket, the difference in pressure forcing the jet up and at the same time producing a roll, or backflow upstream, below the ledge of the bucket.

It was found that if the tailwater was raised a certain amount, this spray would be covered to an extent, making a much better operating condition if used in an installation; see Photos 89 and 90, Plate 48.

The points where this condition first occurs as the tailwater is raised have been plotted on Plate 51. The curves show the three conditions of flow. As the tailwater is then lowered from the elevation where the spray is covered, the jet is deflected free into the air and if lowered still further a point is reached where the back pressure is relieved and the jet falls to the floor. These conditions are illustrated in Photos 93 and 94, Plate 49, showing the streams lines through the flow.

If a sill is placed on the floor below a point where the jet strikes, the velocity is killed by impact in the same manner as when the bucket is lowered to the same level as the floor and sill; see Photo 95, Plate 49.

From the lip of the bucket to the floor was filled in with a 1:2 slope in an effort to get the hydraulic jump to function; see Photo 97, Plate 50. The slope in this case, however, was too steep to obtain the proper jump action. At lower discharges the overflow jet followed the slope down and struck the sill below with great force, apparently losing little velocity in passing through the tailwater. The sill served effectively up until near maximum flow when the jet leaped the sill and continued downstream causing erosion of the sand deposits in the river bed below.

When the sill was placed up on the slope the jet struck it at all flows up to and including the maximum; see Photo 98, Plate 50.

It was found that a 1:4 slope was about as steep a floor that the jet would follow and give anything like the hydraulic jump action in retarding the velocity of flow.

It may be seen from the curves on Plate 51 that the jump-off bucket at El. 90 was too high for the natural tailwater to furnish a depth over the lip that would cover the spraying jet, which is the condition of best operation.

The bucket then was lowered in one test to El. 80, the apron remaining at El. 60 and which set-up gave very satisfactory performance at all stages of flow; that is, the jet was deflected upward and the spray subdued by the covering of tailwater.

The action of the jump-off bucket is very effective in preventing erosion below the apron; in fact, the back-roll established under the lip of the bucket will pick up sand from the river bed below and deposit it against the face of the bucket. This happened in the test with the bucket at El. 80 and the sand deposition was so great in a few minutes as to form a sloped bank away from the ledge of the bucket which apparently relieved the upward pressure at that point and the jet shot down to the floor sweeping out all the sand in the bin below; see Photo 96, Plate 49.

Due to the sensitiveness and instability of performance of the jump-off bucket scheme no further studies were made of this set-up.

However, an arrangement that may have proven feasible with further investigation would be a jump-off bucket at El. 86 with a straight face down to the apron pavement 100 ft. long at El. 60 at the end of which a 10-ft. dentated sill would be placed.

With the bucket at El. 86, or lowered 4 ft. from elevation of the bucket used in test 3, shown on Plate 51, the indications are that the natural tailwater would furnish correct depths to cover the spraying jet and give the condition desired.

The sill would afford protection if the jet should happen to cut down to the floor instead of being deflected upward and would also act to prevent this by forming a barrier to the movement of sand and gravel upstream by the back-roll.

The Dentated Bucket:

With the idea of projecting the overflow jet at various elevations and "diffusing" the energy in the tailwater, the high bucket was dentated at intervals as shown in Photos 99 and 100, Plate 50. Little retardation was accomplished with either set-ups as the sand in the bin below the apron was washed out entirely almost at the start of a run with maximum flow over the spillway. It seemed that the jets fell to the bottom quickly with little loss and swept off the end of the pavement at high velocities. Nothing very promising was evidenced with this set-up so that further investigations were dropped.

However, it seems that if the dentates or openings between the blocks were made smaller there would be a back-roll or current upstream, set-up by the flow from the high buckets, which would be opposed by the flow through the dentates at the floor level and a resultant dissipation of energy would take place by impact of the two currents.

Incidentally, model tests with this type of bucket located at the toe of a dam discharging into a side channel trough gave very satisfactory results in quieting the tailwater. The set-up was tried in the model of the Hoover Dam side channel spillways, tests on which were conducted by the writer along with those on the Madden Dam at the laboratory in Fort Collins.

In the side channel set-up the back-roll is confined and the downstream side of the trough also acts as a deflector to turn the flow back upstream so that it is much stronger in offering opposition to the flow coming down the face of the dam through the dentates.

The success of this operation where the channel downstream is open, as in Madden, is problematical.

The ruggedness of operation and the adaptability to the conditions at Madden gave preference to the use of the hydraulic jump method so that investigations of other devices for dissipating the energy of the overflow were dropped.

PHOTO 89
SPRAY COVERED BY TAILWATER
BEST OPERATING CONDITION

PHOTO 90
SPRAY COVERED BY TAILWATER
LOOKING DOWN FROM BRIDGE

PHOTO 91
JET SPRAYED UPWARD

PHOTO 92
JET SPRAYED UPWARD
LOOKING DOWN FROM BRIDGE

JUMP-OFF BUCKET AT EL.90

PLATE 48

PHOTO 93
JET SPRAYED UPWARD
STREAM LINES THROUGH FLOW

PHOTO 94
JET DEFLECTED TO FLOOR
TREAM LINES THROUGH FLOW

PHOTO 95
JET DEFLECTED TO FLOOR
AGAINST SILL - STREAM
LINES THROUGH FLOW

PHOTO 96
JUMP-OFF BUCKET AT EL. 80
SAND DEPOSITION AGAINST
FACE AFTER FLOW

JUMP-OFF BUCKET AT EL. 87 AND EL. 80

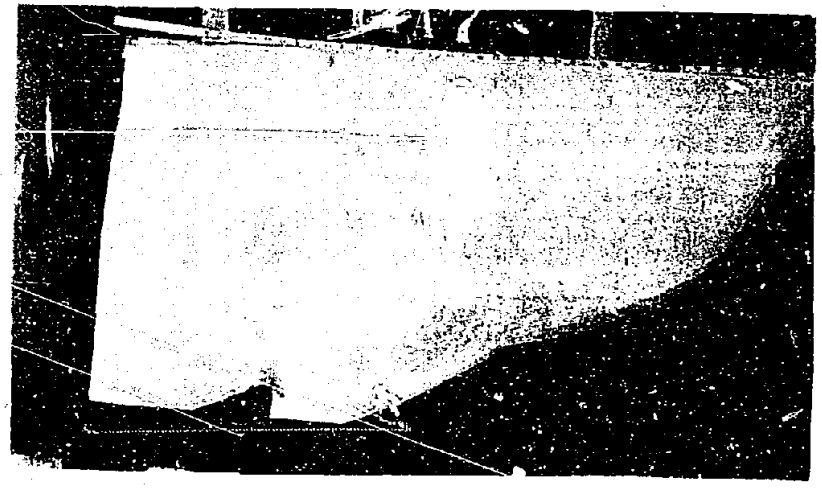


PHOTO 97
1:2 SLOPE - SILL BELOW
STREAM LINES THROUGH FLOW

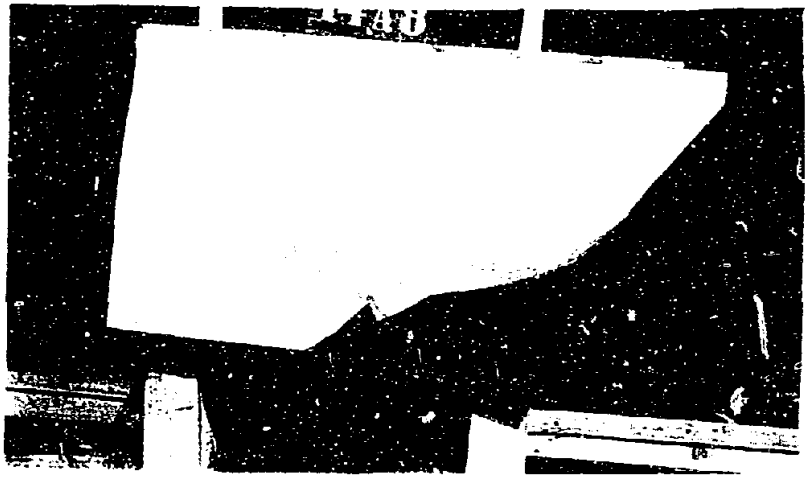


PHOTO 98
1:2 SLOPE - SILL ON SLOPE
STREAM LINES THROUGH FLOW

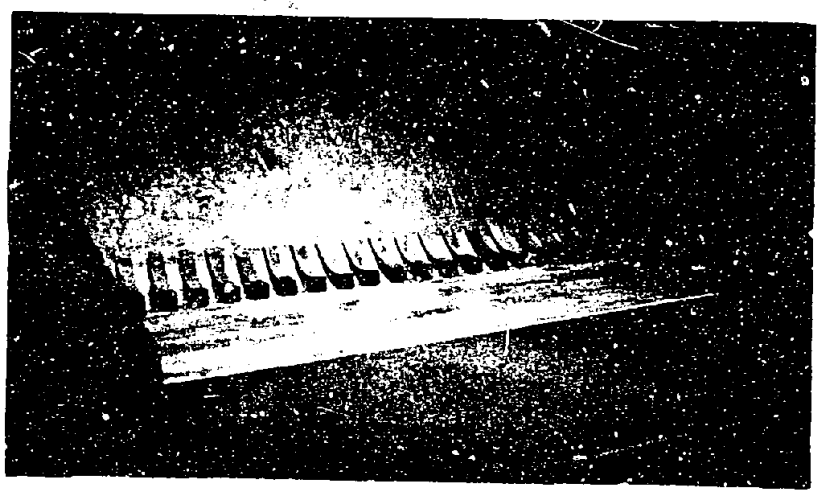


PHOTO 99
APRON AT EL. 80
DENTATES AT EL 90
SAND BIN BEFORE FLOW

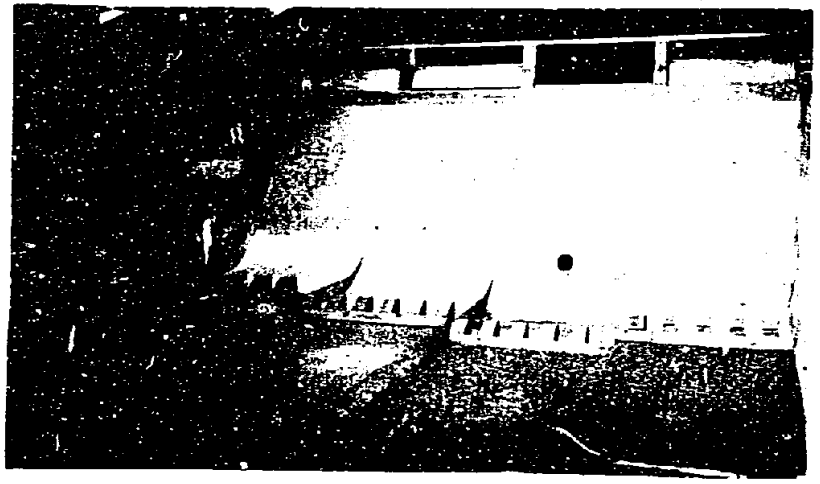


PHOTO 100
APRON AT EL. 80
DENTATES AT ELS. 90 & 100
SAND BIN EROSION AFTER FLOW

1:2 SLOPED APRON AND DENTATED BUCKETS

PLATE 50

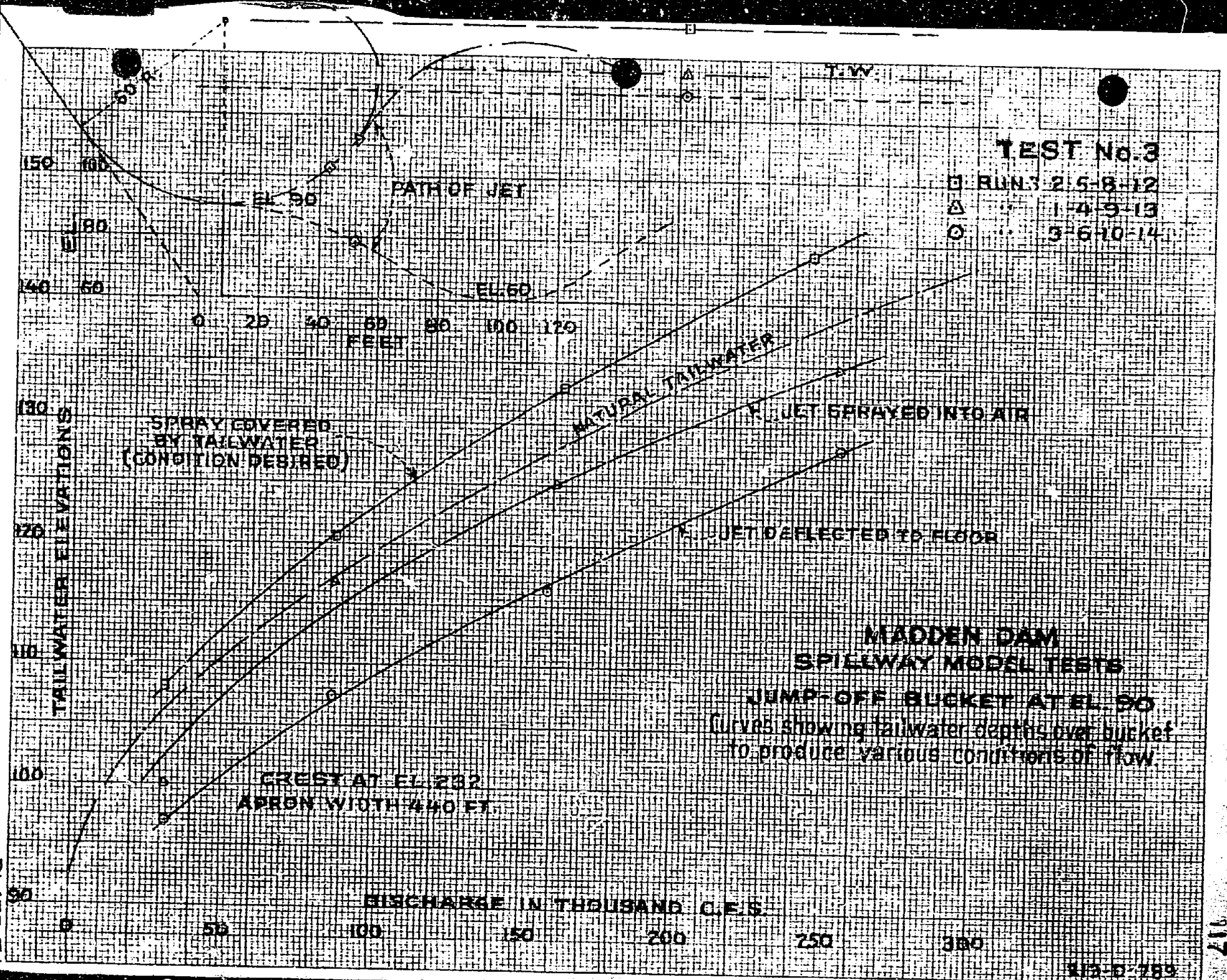
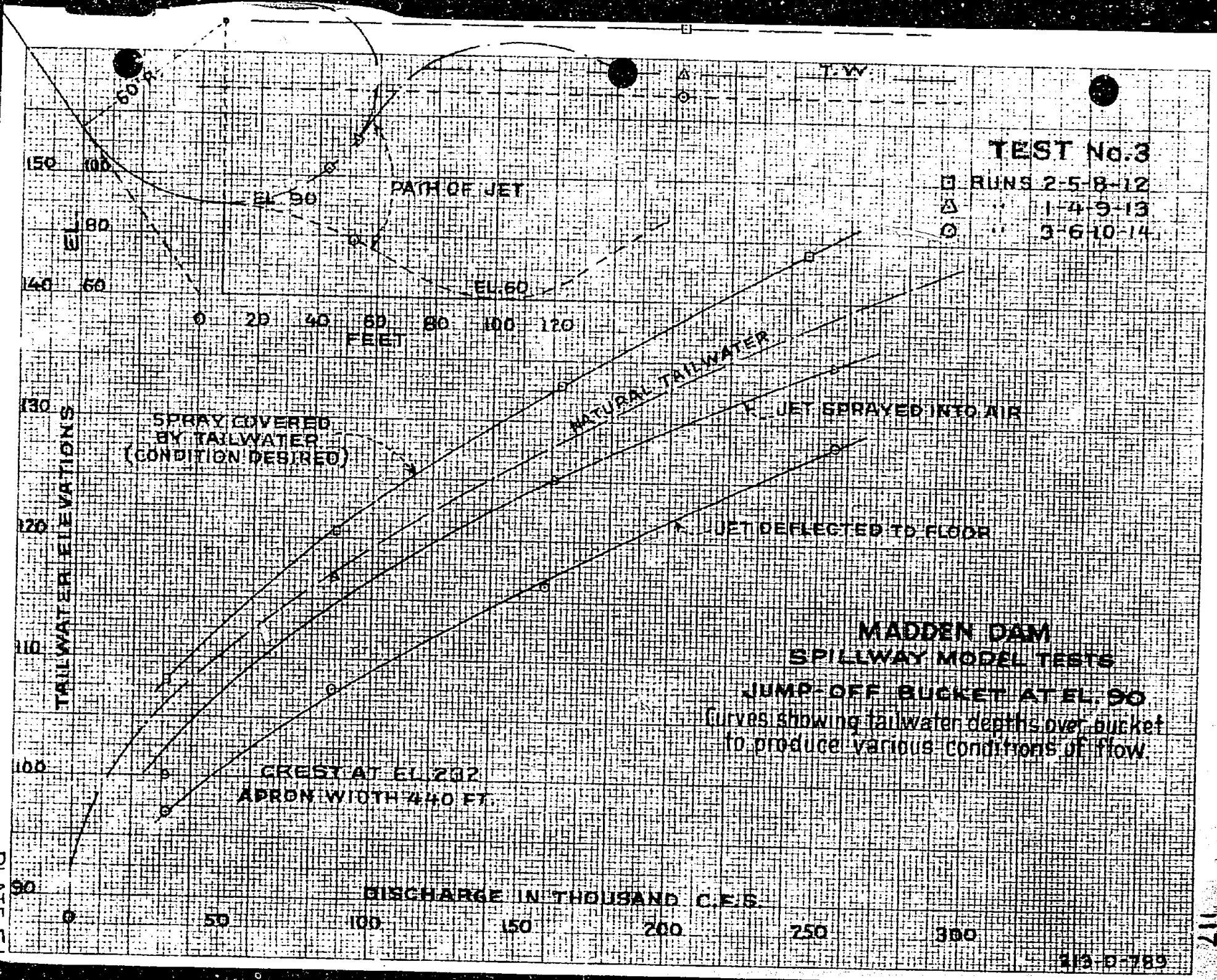


PLATE 51

XIS-D-789



MADDEN DAM
SPILLWAY MODEL TESTS
JUMP-OFF BUCKET AT EL. 90
Curves showing tailwater depths over bucket to produce various conditions of flow.

213-D-789

PLATE 51

Drum Gate Discharge and Operation:

Models of the drum gates were installed in the crest and coefficients of discharge and nappes and pressures were taken with the gates at different positions under various pond elevations; see Plate 52 following. A good view of the gate installation can be obtained from Photo 52, Plate 28, and Photo 106, Plate 64.

Taking points from Plate 52 and plotting coefficients against position of gates, it is seen that maximum efficiency is obtained when the crests of the gates are near El. 244 (see Plate 53), or for a constant head over, when the angle of the gate chord is from 45 to 50 degrees with the vertical; see Plate 54. It was known that sloping the face of a sharp-crested weir would increase the apparent coefficient but it was hard at first to reconcile a higher value than the ogee crest showed.

In order to check the influence of the curved face of the drum gates, sharp-crested flat gates were substituted and similar runs made; see Plate 55. These curves were obtained with pier contraction suppressed, so comparing with like curves of the drum gates' installation (Plate 56), it was found that there was very little difference, except, of course, that with the gates in a horizontal position the ogee crest showed a decided better coefficient than the flat crest; see Plate 57.

There is no doubt of some influence caused by the curved face of the gate and even the flat gate is affected by its installation

on top of the curved dam crest, but it seems that the variation in coefficient when the gates are set at an angle is caused by the assumption of the head acting on the crest.

In the tests the head was measured, in the customary manner, from the highest point on the gate to the water surface upstream where the velocity of approach was negligible.

At some position of the gate near its midpoint of travel, or at about a 45-degree angle, the top of the gate is probably close to the maximum rise of the under-nappe of the overflow and the head as measured then more closely approaches the theoretical head over the perfect crest.

When the gate is raised further, the top lies below the maximum rise of the under-nappe and if the head is assumed to be from the top of the gate a less coefficient results.

If the gate is lowered from this mid-position, the under-nappe tends to leap the tip end of the curved gate and to rise higher than the actual high point of the gate's surface. If the head then is assumed from the high point of the curved surface of the gate to forebay level, the coefficient will be lower.

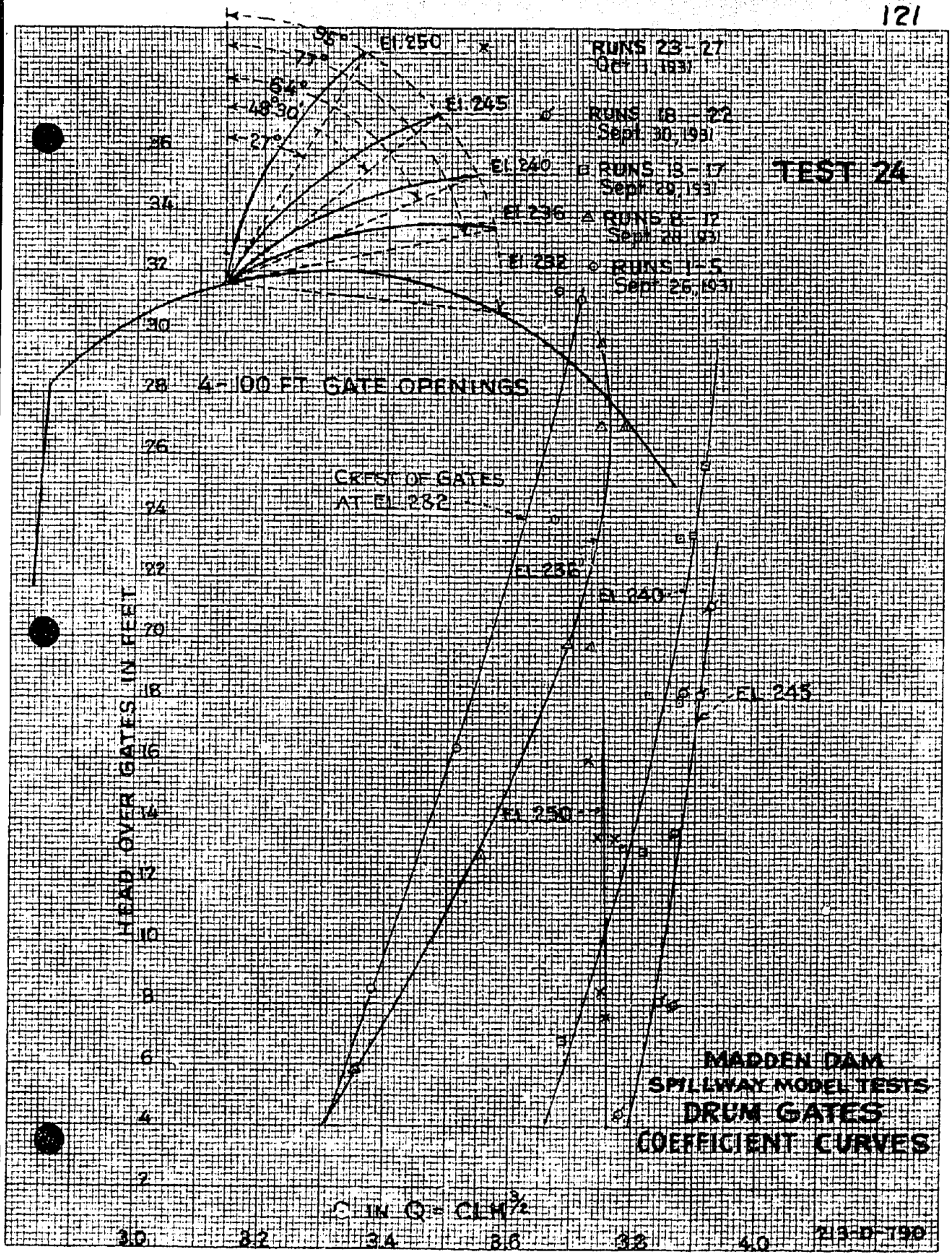
In other words, if the head is taken as the distance from the point of highest rise of the under-nappe to the forebay water surface, the coefficient will be the same no matter what the slope of the gate. With the gate about its mid-position the head, as was measured, more nearly corresponds to this distance and the highest coefficient is obtained.

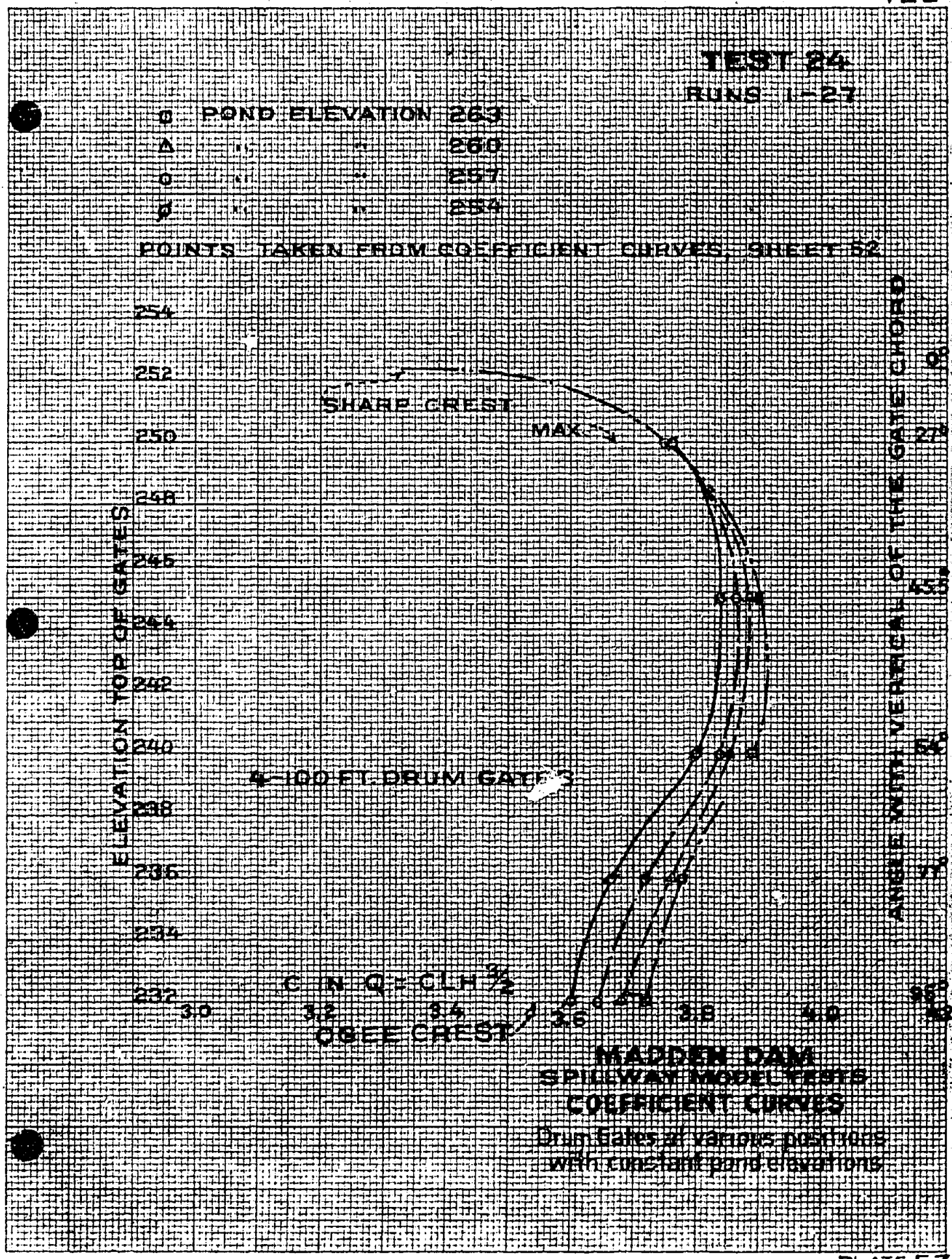
Comparing Plates 52 and 56, it is seen that the loss by pier contraction is small. The greatest loss seems to occur when the gates are in their maximum raised position, but here the measured heads and quantities of flow involved were so small that too much weight should not be given to their values.

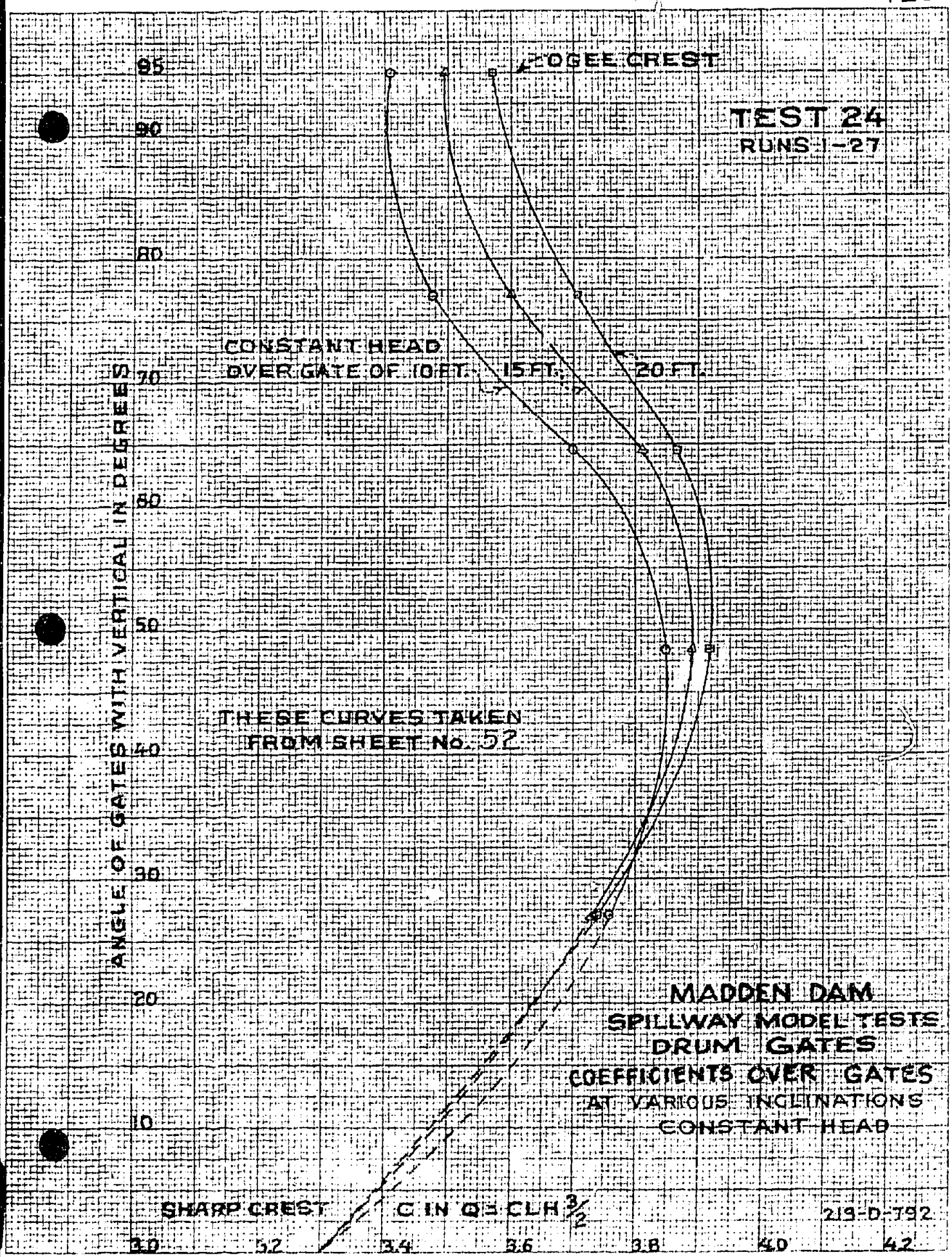
Plate 58 shows some coefficients taken at a later date from the preceding plates and after the drum gates had been removed and the crest filled in solid with a smooth block. There is a very close agreement between the curves taken at a wide interval apart and after various alterations to the model and is considered a good indication of the accuracy and consistency of the testing.

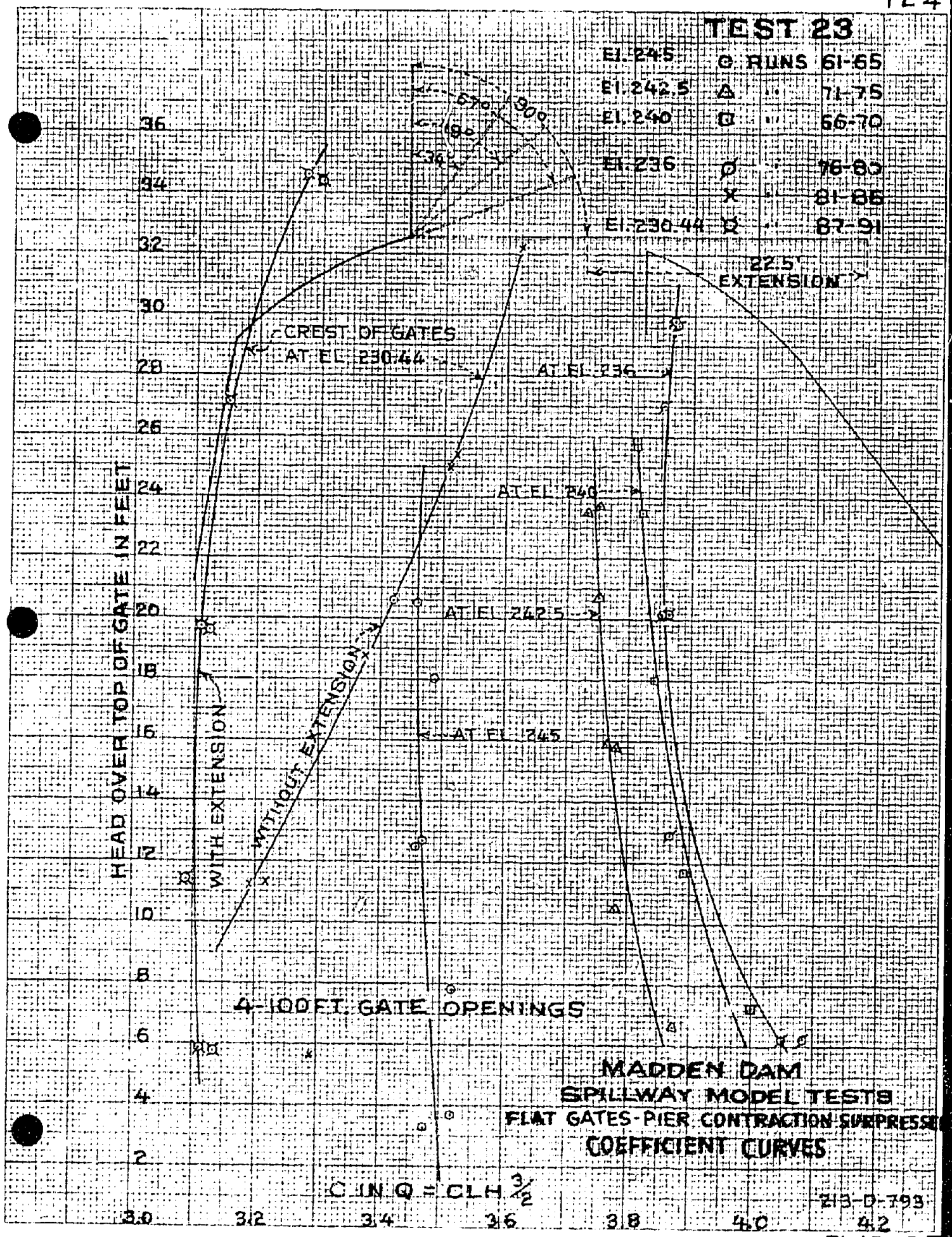
The assumed capacity of the spillway as used for design may be verified. The curves show a coefficient near 3.71 for 31 ft. over the crest, or a pond elevation of 263, which gives a discharge of 256,000 c. f. s. and this together with the 30,000 c. f. s. taken as the discharge of the sluices and needle valves obtains a discharge in excess of the required 280,000 c. f. s. flow.

However, the indications are that the flow over the top of the gates when in the maximum raised position will be 70,000 c. f. s. with the pond at its highest elevation and that it will have to recede to El. 260 before the discharge can be throttled to 50,000 c. f. s. It may be desired to throttle the discharge after the peak of a flood has passed to protect shipping in canal, as traffic is interrupted when the flow exceeds 50,000 c. f. s. at the point where the river and canal join.


TEST 24
RUNS 23-27
Oct 1, 1931
RUNS 18-22
Sept 30, 1931
RUNS 13-17
Sept 29, 1931
RUNS 8-12
Sept 28, 1931
RUNS 1-5
Sept 26, 1931
El. 250
El. 245
El. 240
El. 236
El. 232
96°
77°
64°
48°30'
27°
4-100 FT GATE OPENINGS
CREST OF GATES
AT EL 232
El. 236
El. 240
El. 245
El. 250
HEAD OVER GATES IN FEET
C IN Q = CLH^3/2
MADDEN DAM
SPILLWAY MODEL TESTS
DRUM GATES
COEFFICIENT CURVES
213-D-790

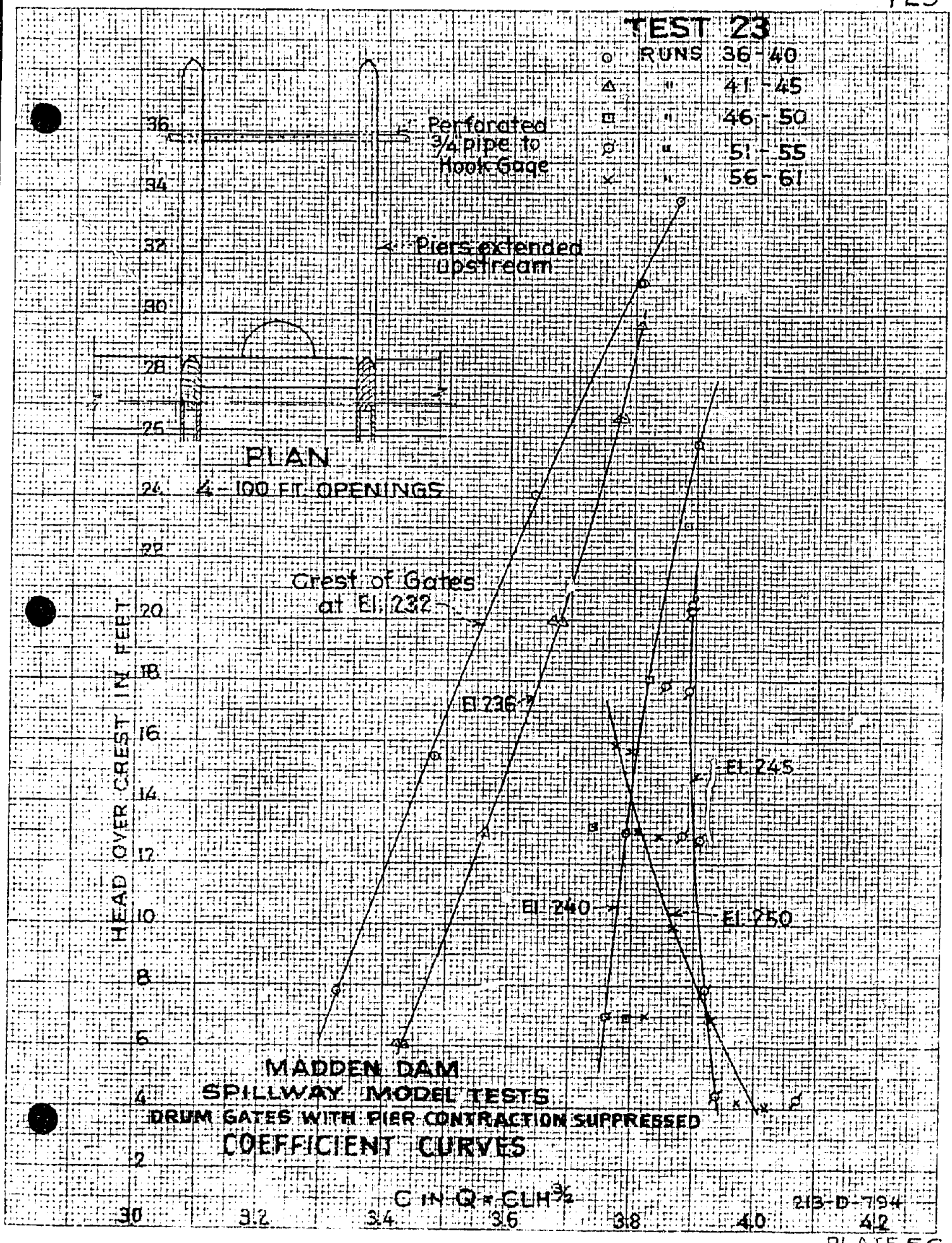

TEST 24
RUNS 1-27
POND ELEVATION 263
" " 260
" " 257
" " 254
POINTS TAKEN FROM COEFFICIENT CURVES, SHEET 52
ELEVATION TOP OF GATES
254
252
250
248
246
244
242
240
238
236
234
232
SHARP CREST
MAX
4-100 FT. DRUM GATES
C IN Q = CLH 3/2
3.0
3.2
3.4
3.6
3.8
4.0
OGEE CREST
ANGLE WITH VERTICAL OF THE GATE CHORD
9°
27°
45°
64°
77°
90°
MADDEN DAM
SPILLWAY MODEL TESTS
COEFFICIENT CURVES
Drum Gates at various positions
with constant pond elevations

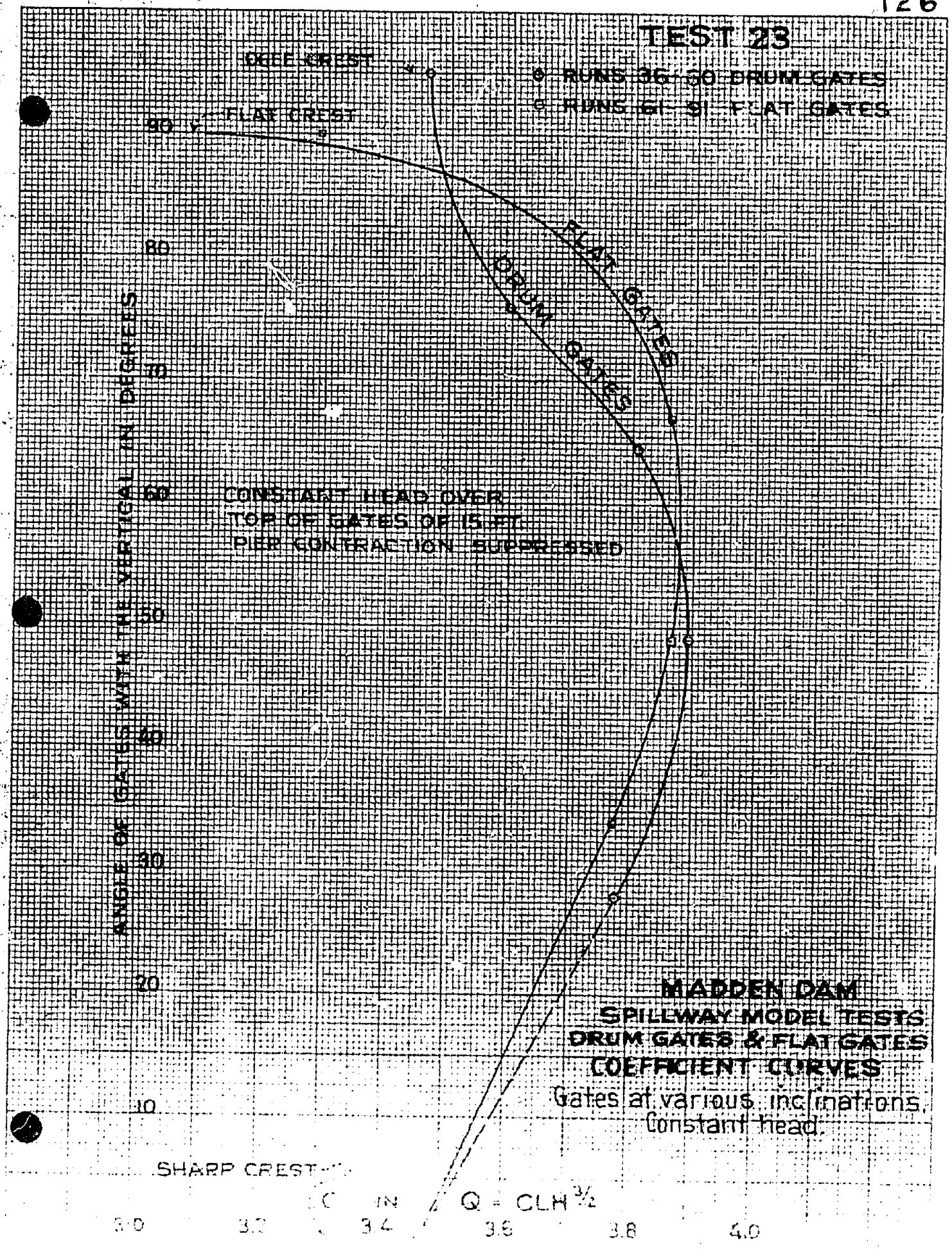

OGEE CREST
TEST 24
RUNS 1-27
CONSTANT HEAD
OVER GATE OF 10 FT.
15 FT.
20 FT.
THESE CURVES TAKEN
FROM SHEET No. 52
ANGLE OF GATES WITH VERTICAL IN DEGREES
95
90
80
70
60
50
40
30
20
10
SHARP CREST
C IN Q = CLH 3/2
3.0
3.2
3.4
3.6
3.8
4.0
4.2
MADDEN DAM
SPILLWAY MODEL TESTS
DRUM GATES
COEFFICIENTS OVER GATES
AT VARIOUS INCLINATIONS
CONSTANT HEAD
213-D-792

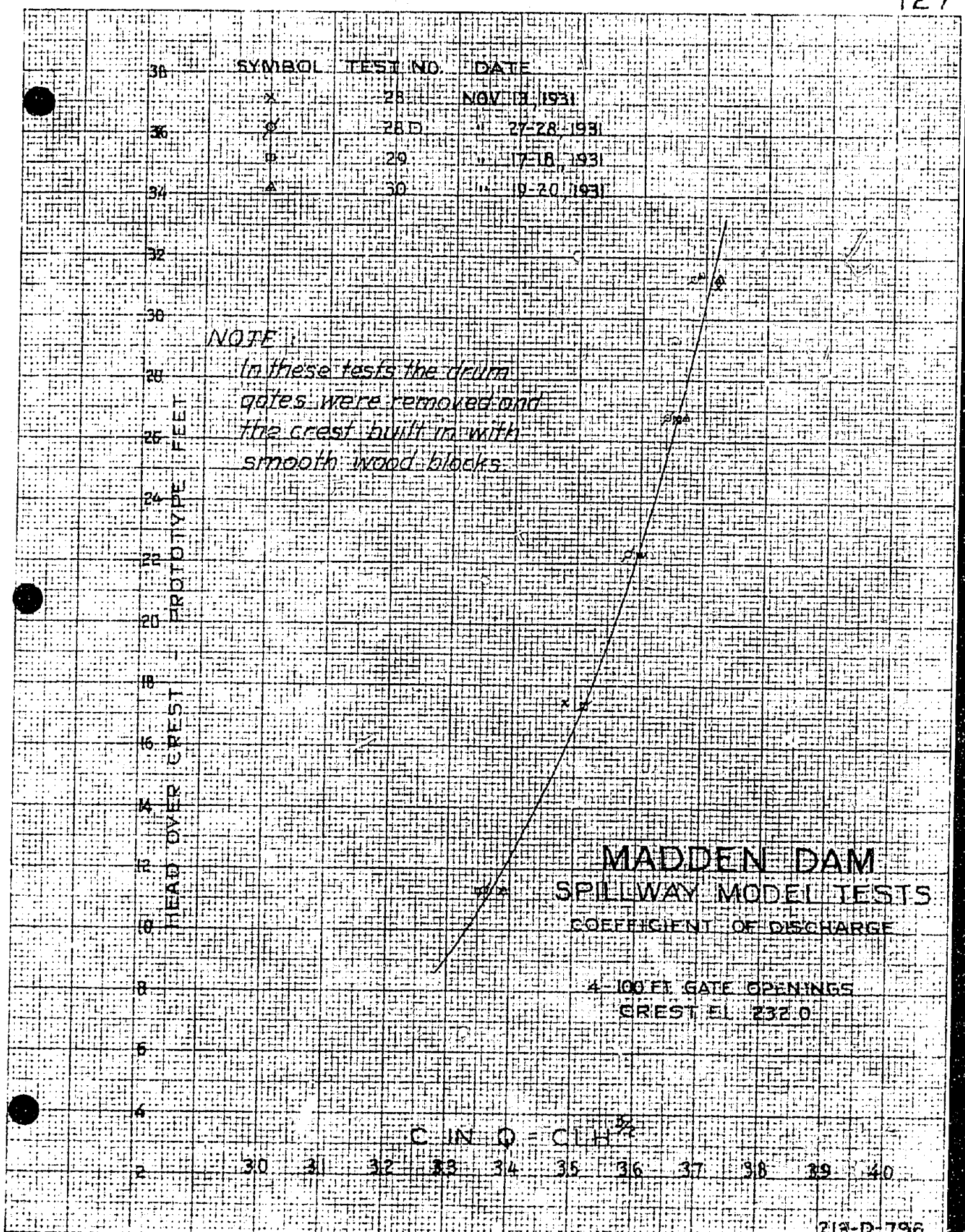


MADDEN DAM
SPILLWAY MODEL TESTS
FLAT GATES-PIER CONTRACTION SUPPRESSED
COEFFICIENT CURVES

213-D-793


TEST 23
RUNS 36-40
" 41-45
" 46-50
" 51-55
" 56-61
Perforated 3/4 pipe to Hook Gage
Piers extended upstream
PLAN
4-100 FT. OPENINGS
Crest of Gates at El. 232
El 236
El 240
El 245
El 250
HEAD OVER CREST IN FEET
C IN Q = CLH^3/2
MADDEN DAM
SPILLWAY MODEL TESTS
DRUM GATES WITH PIER CONTRACTION SUPPRESSED
COEFFICIENT CURVES
213-D-794


TEST 23
OGEE CREST
FLAT CREST
RUNS 36-50 DRUM GATES
RUNS 61-91 FLAT GATES
DRUM GATES
FLAT GATES
CONSTANT HEAD OVER
TOP OF GATES OF 15 FT.
PIER CONTRACTION SUPPRESSED
ANGLE OF GATES WITH THE VERTICAL IN DEGREES
90
80
70
60
50
40
30
20
10
SHARP CREST
C IN Q = CLH^3/2
3.0
3.2
3.4
3.6
3.8
4.0
MADDEN DAM
SPILLWAY MODEL TESTS
DRUM GATES & FLAT GATES
COEFFICIENT CURVES
Gates at various inclinations.
Constant head.



213-D-796

PLATE 58

Nappes and Pressures on Crest:

The upper water surface or nappe was measured at various depths over the crest and for different gate settings. (See Photo 29, Plate 16, for point of gage apparatus.) Pressures at points along the crest were obtained from piezometer openings connected to glass water gage tubes. These are recorded on Plate 59 following.

It was desired to locate the intake to the float, controlling the movement of the drum gates, at a point about the location of piezometer #3 shown on Plate 59.

This location was found undesirable as it may be seen from the pressure curves that as the gate is lowered, due to the increased velocity and centrifugal force of the overflow, less and less pressure is recorded at piezometer #3, although the forebay level remains the same. This would necessitate moving the intake to some point in the bulkhead section far enough away from the spillway to be out of the influence of the draw-down.

Plate 60 shows nappes and pressures after the drum gates were removed and replaced by smooth blocks, in which additional piezometers were placed.

It is desired to locate piezometers in similar positions on the prototype structure to afford a check against model results and the curves are included as a record for comparison.

A comparison of the model results with the theoretical computation of the distribution of pressures on the crest is shown on Plate 61 following. No test was made at exactly the same head as assumed in the computations but the one shown corresponds in water surface elevation of the overflow nappe although the head upstream is higher resulting from a lower coefficient of discharge being obtained in the experiments. It is seen that there is a close agreement in the vertical pressure on top of the crest but the experiments indicate a slightly greater vacuum by extending farther downstream on the back of the dam. The main difference seems to lie in the position of the upper and lower nappes.

It might be advantageous to vent this vacuum by a channel cut across the entire length of the crest and just below of the piers, into which there would be openings at intervals from an air header conduit. (Sketch shown in Fig. 2, Plate 60.) This would give positive aeration to the entire length of the overflowing sheet and prevent the probable making and breaking of the vacuum due to spasmodic aeration from the ends at the piers.

However, it has been maintained but never proven, to the writer's knowledge, that this vacuum on the downstream side of the crest increases the coefficient of discharge. It would then, perhaps, be unwise to install the vent until this point was checked by

model tests. It was planned to make this experiment, among several others, on the Madden model, but due to such a full program of testing at the laboratory it was never done.

It would seem desirable, nevertheless, to more positively aerate the outside ends of the drum gates by placing a vent down through the end walls, opening just below the lip of the gate. The only opening afforded at the ends where the training walls join the piers is a two-foot offset that on the model seems to fill up and prevent air reaching the under-nappe.

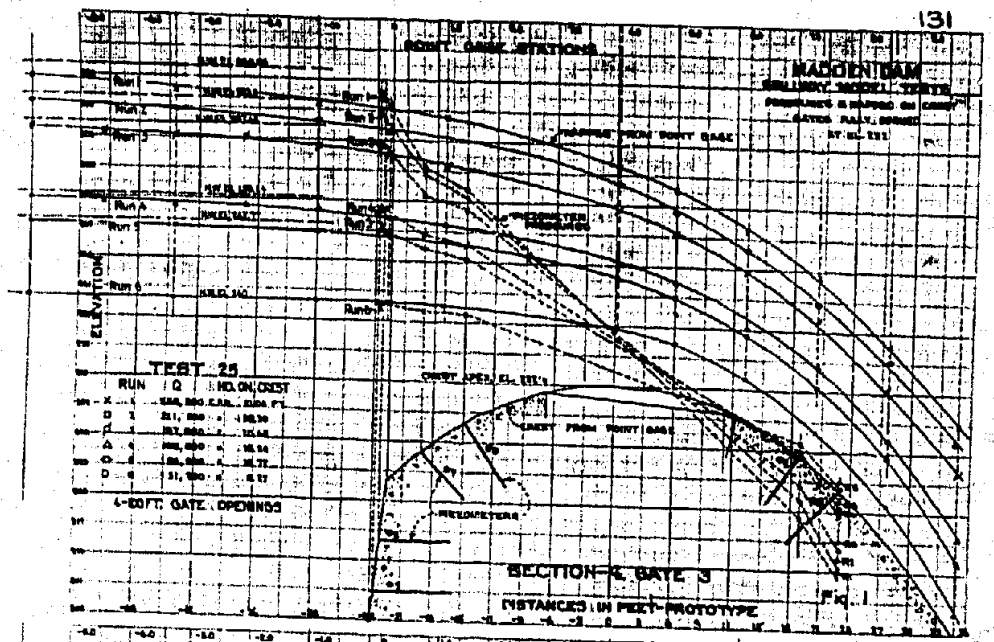


Fig. 1

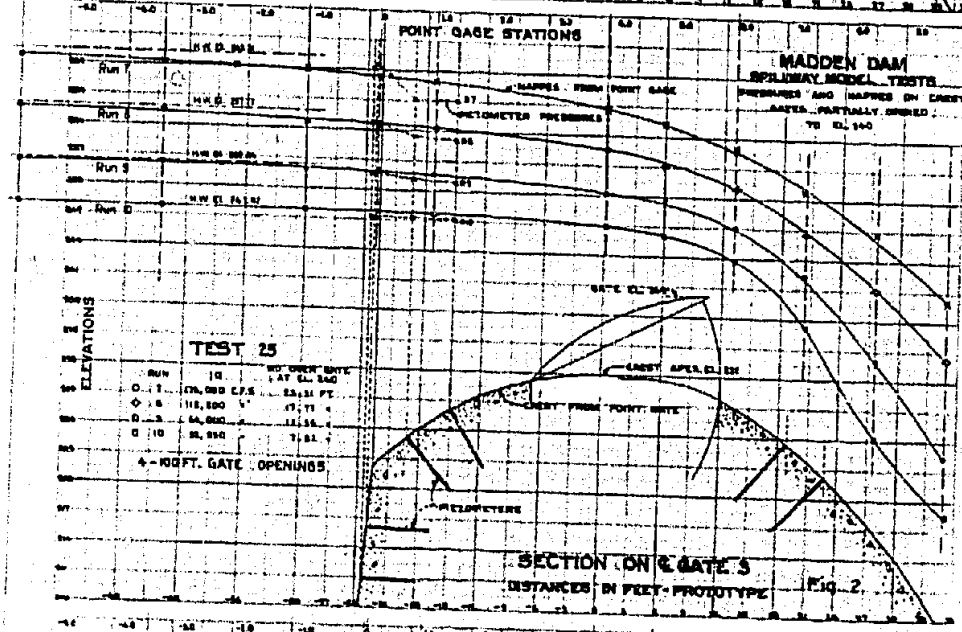


Fig. 2

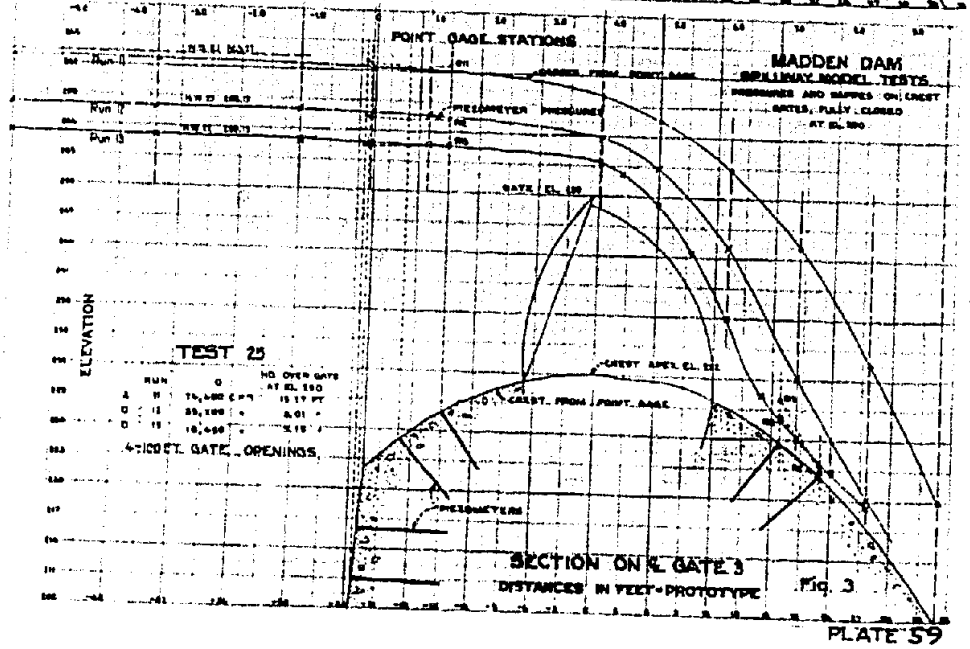


Fig. 3

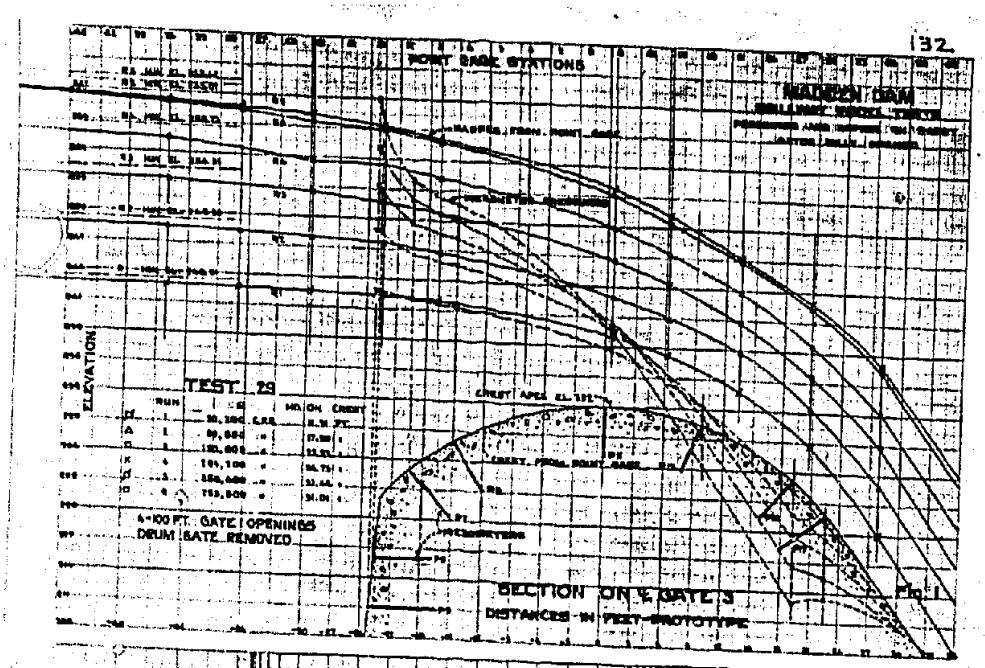


Fig. 1

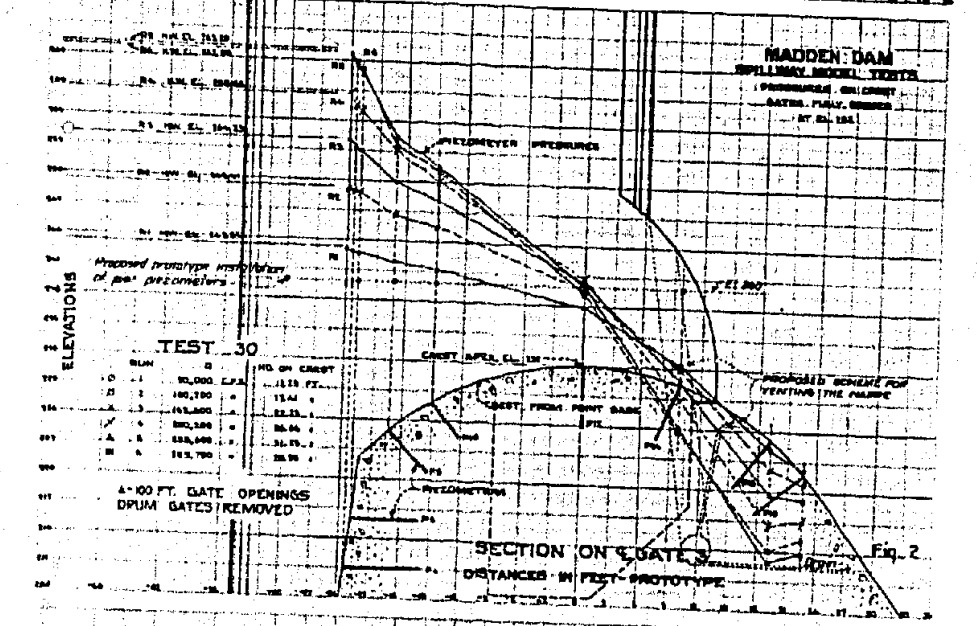


Fig. 2

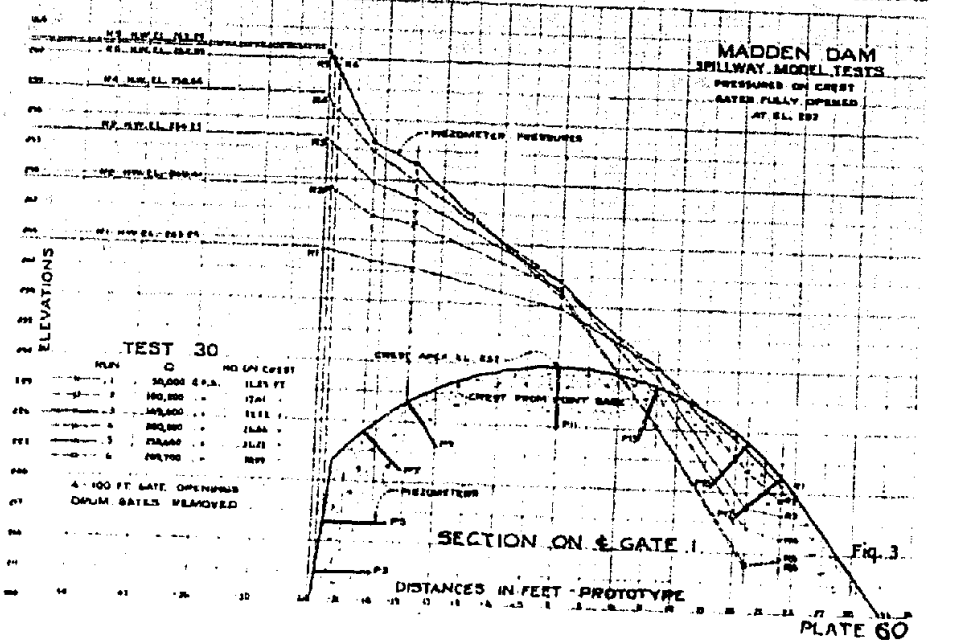
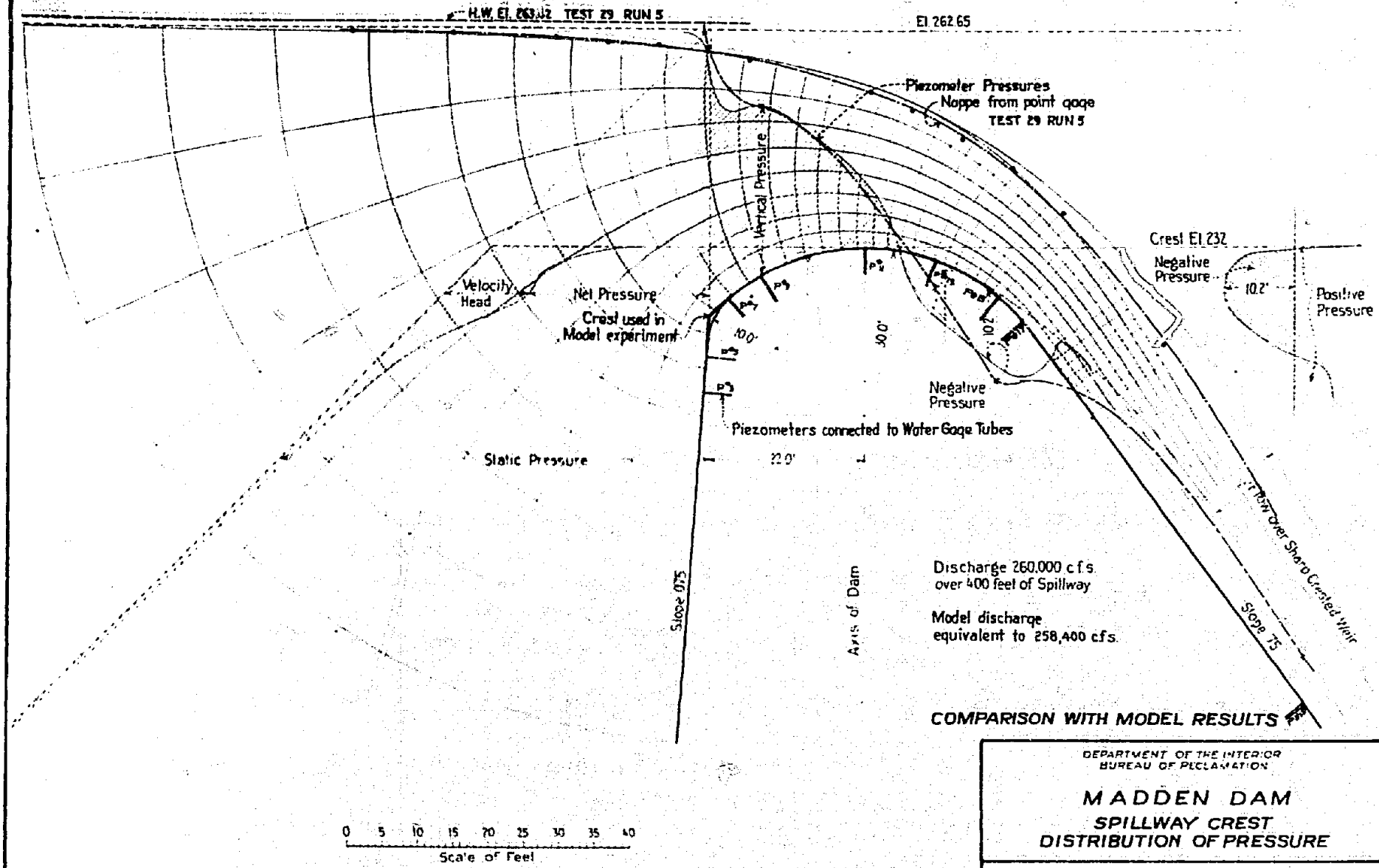


Fig. 3



COMPARISON WITH MODEL RESULTS

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

MADDEN DAM

SPILLWAY CREST DISTRIBUTION OF PRESSURE

DRAWN C.R.	SUBMITTED	
TRACED C.B.O.	RECOMMENDED	
CHECKED S.R.	APPROVED	
	DENVER, COLORADO MARCH 12 1931	213-D-127

PLATE 61

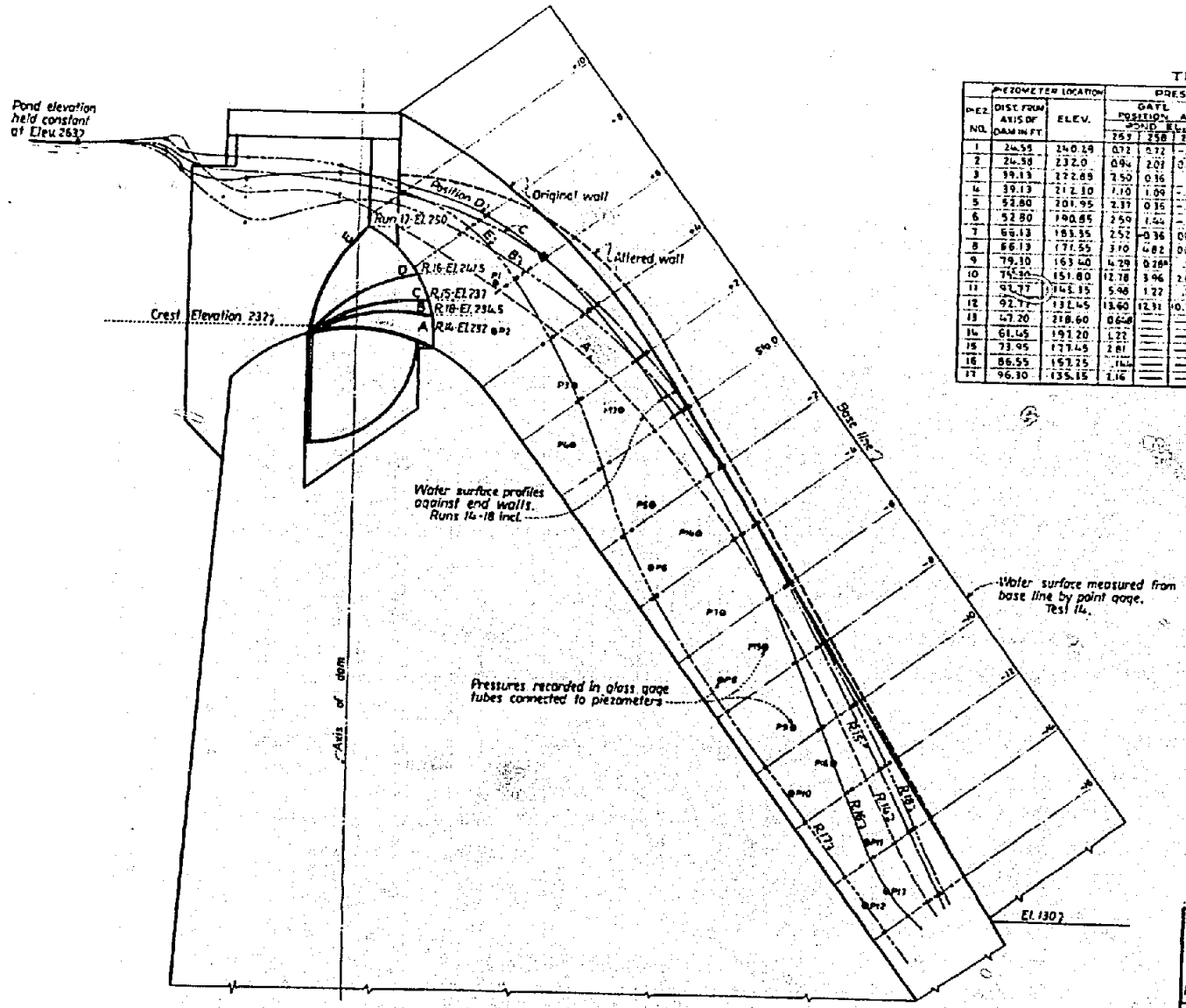
Nappes and Pressures Against Side Walls:

In order to check the height and stability of the end walls of the spillway the nappes and pressures against them were measured for various positions of the gates. The maximum possible case was taken with the forebay at its highest elevation and the gate's opening. The nappes shown on Plate 62 indicate the highest rise of the water surface against the wall, which rise, due to surface tension, is probably exaggerated on the model but if used for design the outlines taken are on the safe side.

The surfaces were measured on both sidewalls and showed such close agreement that the curves were drawn through an average of the points. On Photo 87, Plate 47, preceding, is shown the straight edge, mounted above the wall from which the surfaces were measured by point gage.

It is seen from the drawing (Plate 62) that the original wall barely contained the jet when the gate was raised slightly above the crest. The wall height was raised in the final design to overcome this, and also the point of take-off from the back of the pier was lowered as the original was unnecessarily high to contain the overflow.

Pressures were measured by piezometer openings through the wall at various points, flush and smooth with the surface; see Photo 88 on preceding Plate 47. Connection to a water column in glass gage tubes was made with a rubber hose. The tubes were filled with water to the elevation of the piezometer opening and the average rise above that point, recorded during the run, and multiplied by the scale ratio of 72, was taken as the prototype pressure against the wall at that point. These values have been tabulated on Plate 62. It will be noted that very little pressure is exerted against the walls, except at the very bottom where the overflow jet has spread enough to strike the sides. At this lower point the walls are sufficiently backed up, by the power house substructure on one side and the bank on the other so that no reinforcement or additional thickness was necessary to give the walls ample strength.



TEST 17

PIEZ NO.	PIEZOMETER LOCATION: DIST. FROM AXIS OF DAM IN FT	PIEZOMETER LOCATION: ELEV.	PRESSURE IN FEET OF WATER: GATE POSITION A, POND EL. 263	GATE POSITION A, POND EL. 258	GATE POSITION A, POND EL. 253	GATE POSITION B, POND EL. 263	GATE POSITION C, POND EL. 263	GATE POSITION D, POND EL. 263	GATE POSITION E, POND EL. 263
1	24.55	240.29	0.72	0.72	-.288	3.13	2.89	1.44	1.22
2	24.58	232.0	0.94	2.01	0.72	2.45	1.44	0.72	1.585
3	39.13	222.89	2.50	0.36	0.72	2.04	1.95	0.864	----
4	39.13	212.10	1.10	1.09	-.576	1.66	1.44	1.225	1.73
5	52.80	201.95	2.37	0.36	-.288	2.26	3.24	1.872	0.000
6	52.80	190.85	2.59	1.44	-.216	2.34	2.30	2.09	0.000
7	66.13	183.35	2.52	-0.36	0.00	2.33	2.66	2.88	----
8	66.13	171.55	3.10	4.82	0.00	2.77	2.95	3.10	0.072
9	79.30	163.40	4.29	0.288	.288	2.84	3.10	2.88	----
10	79.30	151.80	12.78	3.96	2.66	1.80	1.76	8.64	0.144
11	93.77	144.15	5.98	1.22	.216	3.10	7.86	0.144	0.000
12	93.77	132.45	13.60	12.31	10.37	28.10	29.30	7.85	3.72
13	47.20	218.60	0.648	----	----	0.81	0.504	0.216	----
14	61.45	191.20	1.22	----	----	1.945	0.72	1.22	----
15	73.95	177.45	2.81	----	----	2.45	2.66	----	0.030
16	86.55	163.25	1.44	----	----	1.51	2.16	-0.576	0.000
17	96.30	135.15	2.16	----	----	12.24	9.36	-0.576	0.000

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
THE PANAMA CANAL
MADDEN DAM PROJECT

MADDEN DAM SPILLWAY MODEL
WATER SURFACE PROFILES AND PIEZOMETER PRESSURES AGAINST RIGHT WALL

DRAWN W.F.B. SUBMITTED
TRACED C.P.W. RECOMMENDED
CHECKED APPROVED
213-D-353 DENVER, COLO. JAN. 8, 1942

136

PLATE 62

Simultaneous Gate Operation:

The photographs on the preceding plates show the flow over the gates at different settings, also the jump performance on the apron under these conditions. The overflow jet strikes the back of the dam as the gates are lowered as seen by the water line in the photos.

Photo No.	Plate No.	Pond	Gates at	Flow, C. F. S.
69	42	El. 263	El. 232	257,000
75	43	El. 263	El. 236	211,000
77	44	El. 263	El. 240	172,000
79	44	El. 263	El. 245	119,000
81	45	El. 263	El. 250	70,000

It is seen that a balanced effective jump occurs when the gates are moved together and this method of simultaneous operation should be strictly adhered to on the prototype.

Separate Gate Operation:

When one or two gates are operated separately it is seen (Plate 10) that the tailwater pours into the jump from the sides interfering with its performance and producing a concentration of flow. (See resultant bed erosion in Photo 52, Plate 28.)

This condition is not peculiar to the sloped "hydraulic jump apron," but occurs even with the dentated sill installed at the end of the apron; see Photo 40, Plate 20. These photographs show clearly the danger of separate gate operation.

Sluice Openings and Deflectors:

Tests were made to ascertain the effect of the sluices discharging into the jump. Fig. 1, Plate 65, shows the velocity contours through the jump at a section between the sluice opening and pier. Fig. 2 is taken through the opening with flow from the sluice as well as over the spillway. A comparison shows no particular concentration or change in the flow lines through the jump. In the above tests the sluice openings were located on the center line of the gates as laid out in the original design.

There seemed to be a concentration or piling up behind the piers as the flow from the two gates met down the back of the dam. It was thought that if the sluices could be moved to the center line of the piers, the deflectors over the openings would tend to break up this concentration. To simulate the effect, a false pier was placed above the sluice opening on center line of gate 2 and velocity contours taken below; see Fig. 3, Plate 65. Comparing with Fig. 1, taken away from the sluice opening, there is little change seen and no interference with the jump performance. The same test was made on sluice No. 3 and little difference noted; see Fig. 1, Plate 66. Photo 101, Plate 63, shows the false pier set-up.

The deflector over the sluice opening, as originally drawn up, was of too flat a radius to turn the water completely out of the

passage. It is seen over the nearest sluiceway in Photo 102, Plate 63. The far sluiceway has a deflector of a sharper radius and lifts the water sheet over the passageway, perhaps to a greater extent than is required. A compromise between the two gave a deflector which was satisfactory and which was finally adopted. Its operation may be seen in Photos 103 and 104 on Plate 63 following. (Note the rib lines in Photo 103 caused by the meeting of the jets behind the piers.) The deflectors seemed to effectively flatten out this roll. Because of this advantage and some structural reasons, the sluices were finally shifted to the center line of the piers.

Dimensions of other types of deflectors tested are given in Fig. 3, Plate 66. Little difference was noted in the flow when the tailwater was raised to its normal elevation no matter what type of deflector was used. (Compare Fig. 3, Plate 65, and Fig. 1, Plate 66.)

It is conceivable that the high velocity overflow sheet past the sluice exits would create a vacuum in them. To check this, a pipe was placed in the passage and connected to a water U-tube. The arrangement is shown on Fig. 3, Plate 66, and the pressures are recorded on Plate 67.

With the condition of no backwater on the apron, as illustrated in Photos 103 and 104, it is seen that a vacuum is set up in the passage at all stages of flow. The cause of the vacuum decreasing

after the peak point, is probably due to aeration at the deflector lip which the increased flow tends to leap.

However, under the actual operating conditions; that is, with the natural tailwater over the apron, it is seen that no vacuum exists in the passage. This flow on the apron is illustrated in Photos 6, 8, 10, 14 and 16. Plate 85 shows the water surface profiles over the apron.

The curve shown on Plate 67 does not necessarily give the actual pressures just at the sluice exit because the piezometer pipe was not extended past the bends in the passageway of the model. Consequently, there was an air-cushion between it and the actual tailwater which would cause the manometer readings to give a pressure somewhat less than actual.

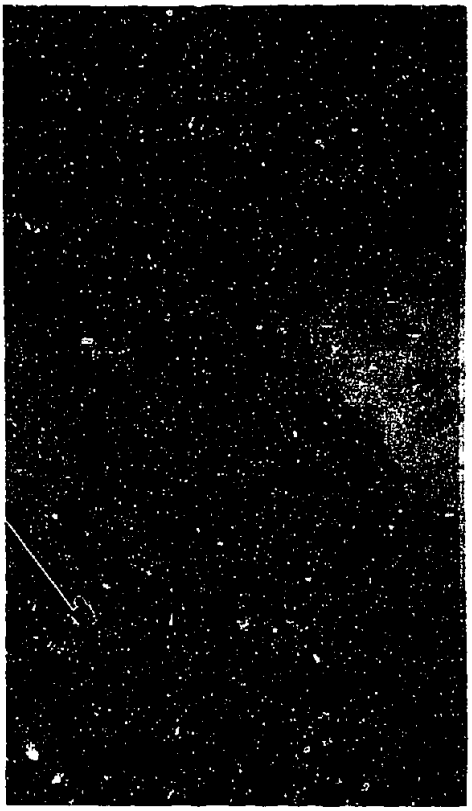
The test showed, at any rate, that there was no vacuum existing to necessitate an increase in the air vent capacity.

Sluice and needle valve operation is shown on Plate 64, Photo 105. The tailwater is adjusted at its natural elevation as shown by the rating curve for this flow. Photo 106 shows the erosion of the gravel deposits caused by the flow. There was no dangerous erosion noted below the apron but the indications were that the draft tube tailrace would be pretty well cleaned of sand by the flow from the needle valves. Fig. 2, Plate 66, gives velocity contours through the flow from a sluiceway.

A sand bin was located below the apron in this series of tests and in some of the curves on Plates 65 and 66 it is seen that velocity readings seem to be taken below the sand profiles. The sand profiles were taken by point gage after a 15-minute period of flow. The flow was brought up again and the Pitot tube measurements taken. In some instances further erosion of the sand may have occurred or the point of the tube may have been pushed down into the sand unknowingly.

The flow and sand erosion for these runs are shown in Photos 107 and 108 and are typical for all of them. The excessive erosion in the right corner of the bin was due to interference of the contour blocks, as was previously mentioned. Also, it was noticed that in all runs with the sand bin below the apron, there was more bubbling below the sill due, probably, to a greater back-roll being formed behind the sill, that deflects the high velocities upward.

Notice the quiet water below the power house when the high wall is installed.



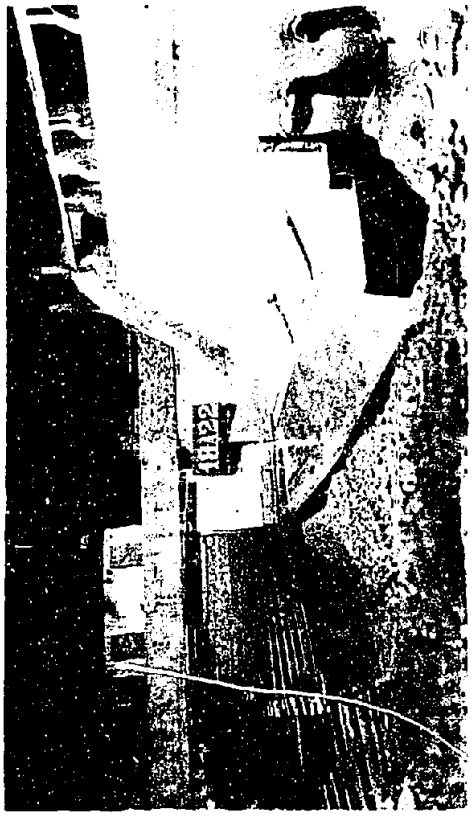
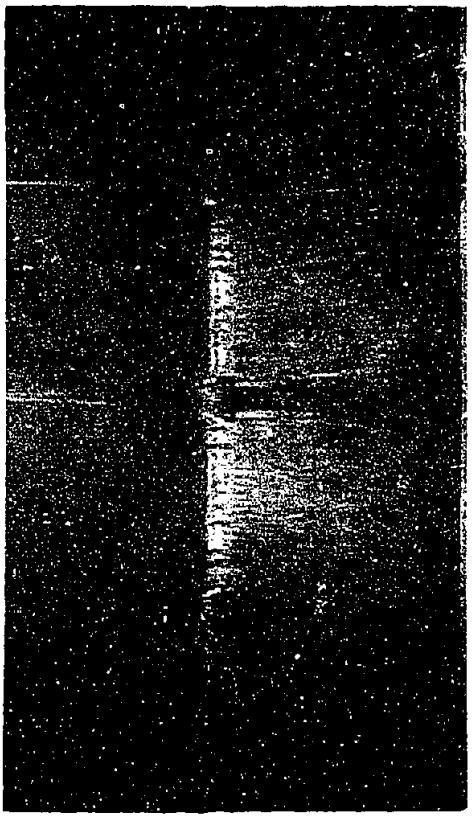
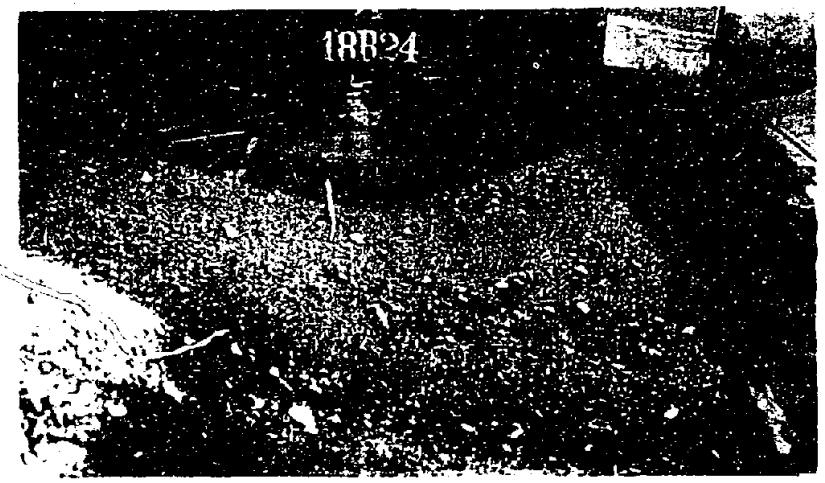


PHOTO 101
SLUICE DEFLECTORS OVER
NOS 2 & 3 - FALSE PIERS



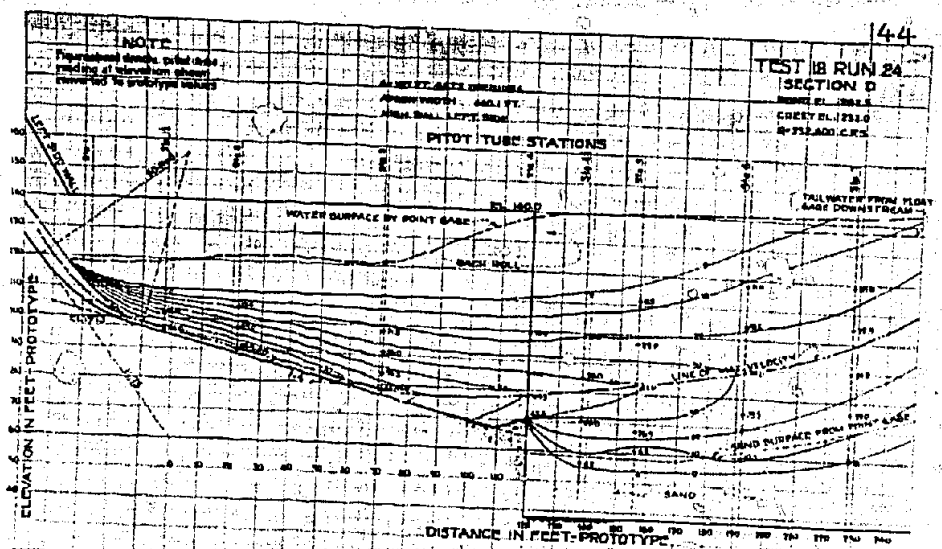
SLUICE DEFLECTORS-JUMP SWEPT OFF APRON

PHOTO 105
EROSION AFTER FLOW
FROM SLUICES AND
NEEDLE VALVES

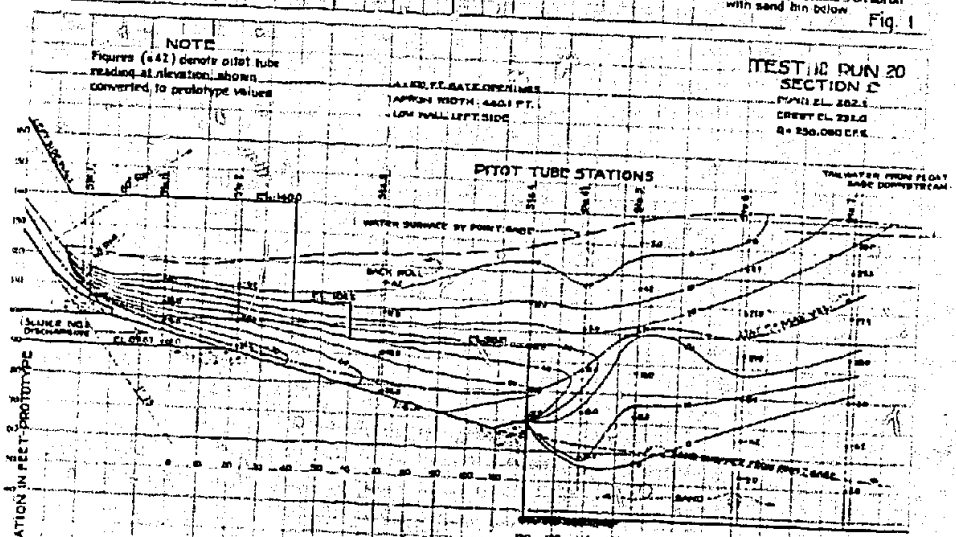


AND EROSION AFTER
FLOW

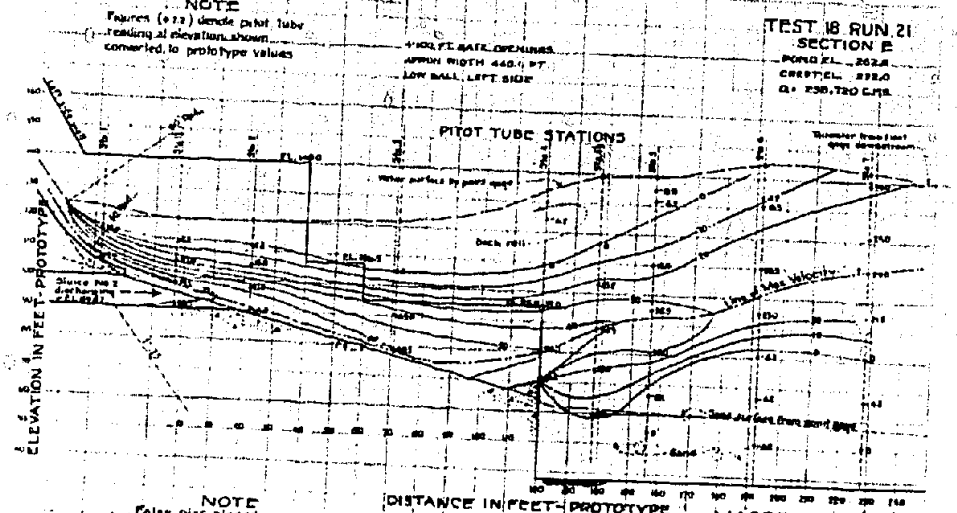
SLUICE OPERATION



MADDEN DAM
SPILLWAY MODEL TESTS
SECTION D'EW SLUICE AND PIER
Prototype velocity contours on apron with sand bin below
Fig. 1



MADDEN DAM
SPILLWAY MODEL TESTS
SECTION THRU SLUICE OPENING ON ℄ GATE
Prototype velocity contours on apron with sand bin below
Fig. 2

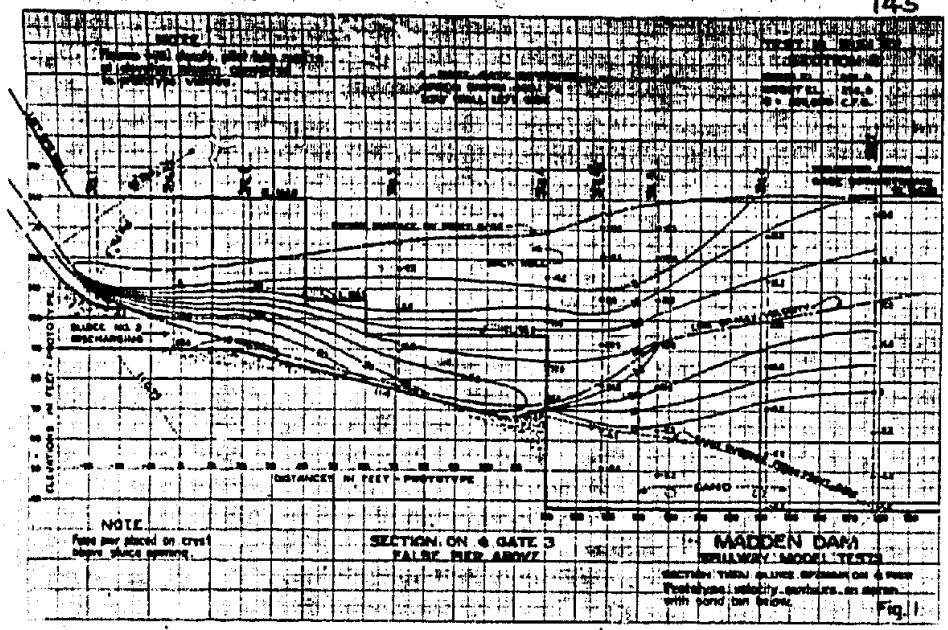


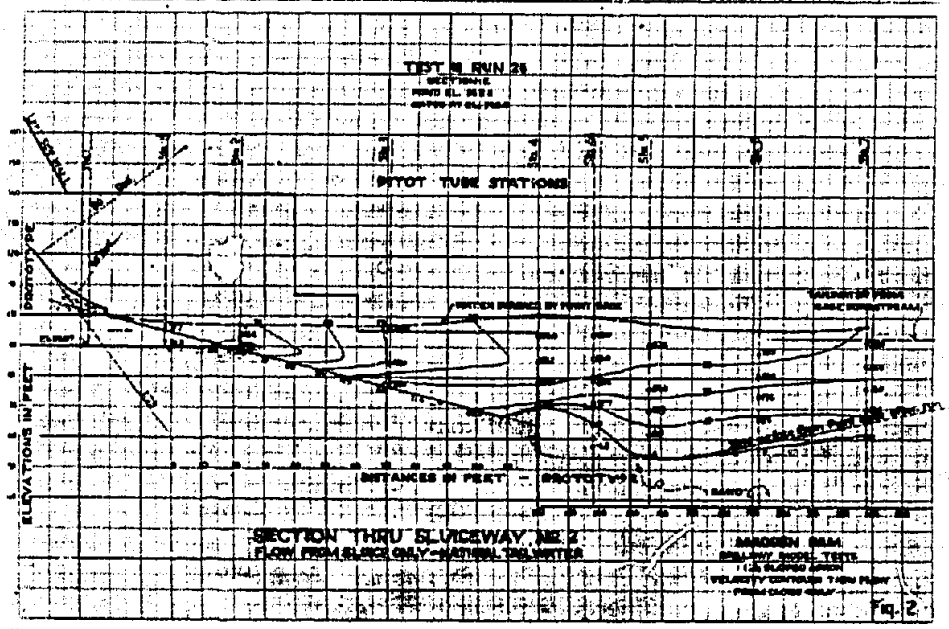
NOTE
False pier placed on crest above sluice opening

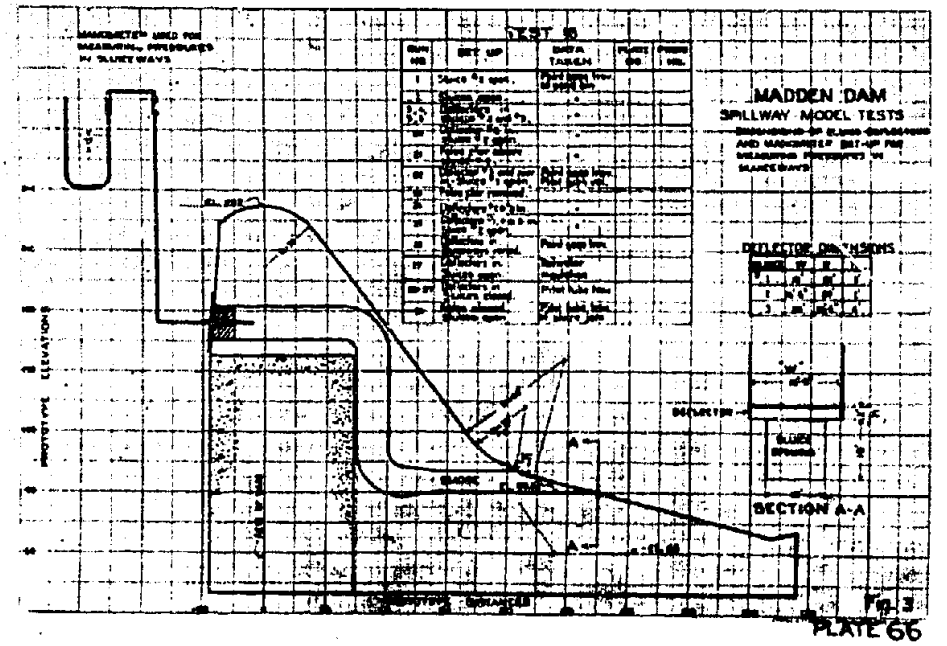
SECTION ON ℄ GATE 2
FALSE PIER ABOVE

MADDEN DAM
SPILLWAY MODEL TESTS
SECTION THRU SLUICE OPENING ON ℄ PIER
Prototype velocity contours on apron with sand bin below
Fig. 3

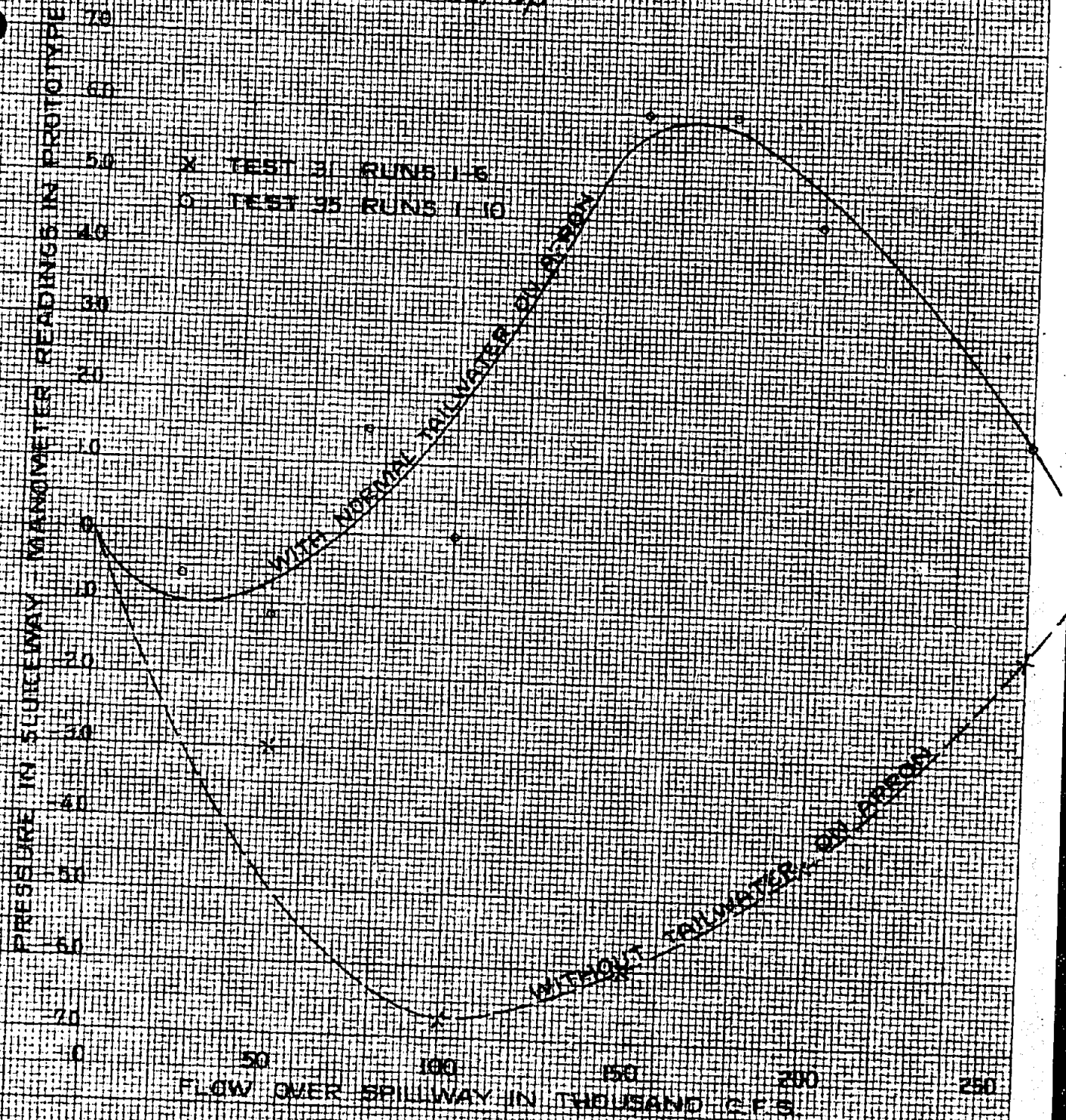
PLATE 65


ELEVATIONS IN FEET - PROTOTYPE
DISTANCES IN FEET - PROTOTYPE
SLUICE NO. 2 DISCHARGING
NOTE
SECTION ON ℄ GATE 3
FALSE PIER ABOVE
MADDEN DAM
SPILLWAY MODEL TESTS
Fig. 1


TEST 9 RUN 25
PITOT TUBE STATIONS
ELEVATIONS IN FEET
PROTOTYPE
DISTANCES IN FEET - PROTOTYPE
SECTION THRU SLUICEWAY NO. 2
FLOW FROM SLUICE ONLY - NATURAL TAILWATER
MADDEN DAM
SPILLWAY MODEL TESTS
Fig. 2


TEST 10
MADDEN DAM
SPILLWAY MODEL TESTS
DEFLECTOR DIMENSIONS
SECTION A-A
ELEVATIONS
Fig. 3

See Fig. 3 Plate 66
for Manometer Set-up



MADDEN DAM

SPILLWAY MODEL TESTS

RELATIVE PRESSURES IN SLUICEWAYS
FOR VARIOUS FLOWS OVER SPILLWAY
WITH AND WITHOUT BACKWATER ON APRON

Right Bank Excavation:

On the right side, the bank swings in across the spillway apron in the path of the flow from the last gate. On the drawing of the original layout (Plate 17), this hillside was left in and faced with a concrete pavement. The model tests showed that the slope of the hillside was too steep for the water to "ride" and consequently the flow slipped down into a concentrated jet at the base of the slope. It was tried to flatten this slope but very little success was had in attempting to make the water flow in a very uniform sheet along any sloped side. A warped surface probably could have been worked out that would carry its share of the discharge but, at that, there was no provision for retarding its velocity. A sill placed along the slope was not very satisfactory as the tailwater over it became less and less up the slope and the jet was sprayed up into the air.

It was decided to cut this bank away where it protruded across the apron but it was still necessary to work out some transition back to the hillside below the apron. Photo 109, Plate 68, following, shows one form of excavation tried, in which the face was cut away on an up-slope downstream. At the higher discharges, the stream struck this slope with considerable velocity, causing a bubbling as seen in Photo 110.

This necessitated cutting away some more of the bank, the

best form of excavation found being a warped surface as shown in Photo 20, Plate 9. No impact as evidenced by bubbling was noticed with this set-up, even at the maximum flow. (See Photo 18, Plate 9, and other photographs of the final model layout.)

Velocities along the curve of the hillside were measured at the maximum flow and have been plotted in Fig. 1, Plate 69. The readings shown on this drawing were taken with the point of the Pitot tube turned parallel to the line of the contours so as to obtain the maximum velocity following the curve of the hillside. The curve in plan of the contours has been developed out flat on the drawing and the velocities projected. (See Plate 13 for plan of the hillside excavation.)

It is seen that at maximum discharge this bank then will be subjected to an average velocity of around 35 ft. per sec. over practically its entire surface but just at the end of the apron the velocities will exceed 50 ft. per sec.

One remedy is to cut the bank away farther allowing a greater mass of water below the apron to retard the velocity, but just at the end of the present side wall, the walls of the hillside would still be subjected to the high velocity, which might tend to undercut the concrete side wall and paving.

An extension of the side wall and paving downstream to cover the range of this high velocity would be a great addition to

the factor of safety at a comparatively small additional cost. It is suggested, also, that a short cutoff wall into the bank at the end of the side wall and paving would aid in preventing a back-cutting. If the bank facing were extended 30 ft. downstream it could be sloped off, as shown on Plate 69, to be symmetrical with the P. H. wall on the opposite side apron extension.

Apron Extension:

An extension of the apron and sill downstream this 30 ft. would protect the river bed from these high velocities and would in itself check the velocities considerably. This has been discussed previously under "Apron Extension," page 82, where it was found that the erosion in the sand bin was lessened considerably by an extension downstream of only 20 ft. (See Photos 111 and 112, Plate 68, following.) Plates 96 and 97 give sand contours in bin taken by point gage after flow for equal periods with and without extension.

The concrete for this extension of the apron and side wall could be taken from the main apron by flattening its slope slightly with no detriment to the performance of the hydraulic jump but, in fact, with an improvement in the flow conditions. It was estimated roughly that if the slope of the apron was flattened to 1:4.3, which would lower the point of tangency at the bucket curve a little over 2.0 ft. to about El. 94, enough concrete would be cut away to re-

place in an extension of the apron for 30 ft. downstream.

This exact set-up was not tested on the model but other tests indicate that this would be entirely feasible and beneficial.

Plate 70, following, gives curves showing the depths required for the hydraulic jump for two 1:4 sloped aprons, one apron about 2.5 feet below the other in elevation. These curves were developed similarly for each apron by adjusting the tailwater until a good hydraulic jump was formed well up on the curve of the bucket. The curves practically coincide, showing little change in flow conditions if the apron is lowered only 2.5 ft. At the smaller discharges the lower apron is better as the tailwater depths over it, required for the jump, more nearly approach those furnished by the natural tailwater; see Plate 29 and Plate 32.

This may also be seen on Plate 34, where the curves indicate that at the lower stages of flow the natural tailwater does not furnish sufficient depth to force the jump well up on the apron. It may be supposed that, since the tailwater depth is fixed by the rating curve, if the apron is lowered the depth over it is increased and more nearly approaches that required for the jump. Lowering the apron would benefit at the higher stages of flood, also, as it is seen from Plate 34 that under the maximum conditions of operation a slightly greater depth is required than that afforded by the normal tailwater. Also, a little factor of safety, against the jump sweeping off the apron, is obtained, should the tailwater rating curve lie below that

predicted or should the sand deposits wash out of the river bed below and lower the tailwater elevation from that expected.

The extension of the apron and right side wall downstream 30 ft. from the present design, will add greatly to the amount of protection afforded the river bed and banks. Its value would far exceed that of any other comparatively small alteration that could be made to the flood control design. Little, if any, additional cost would result as the concrete for this extension can be furnished by lowering the slope of the main apron only two feet at its upper end. It would be beneficial to the hydraulic jump performance to lower the apron at its upper end and to flatten the slope, especially for the lower stages of flow which will, too, occur more frequently. The additional length will bring the apron to the end of the power house wall and make the design more consistent.

For the above reasons it is herewith strongly recommended that this alteration be made, somewhat along the lines as shown on Plate 71 following.

PHOTO 152
FIRST RIGHT BANK
EXCAVATION

PHOTO 153
FLOW SHOWING

PHOTO 154
EXTENSION
ACROSS SAND BAR

RIGHT BANK EXCAVATION AND APRON EXTENSION

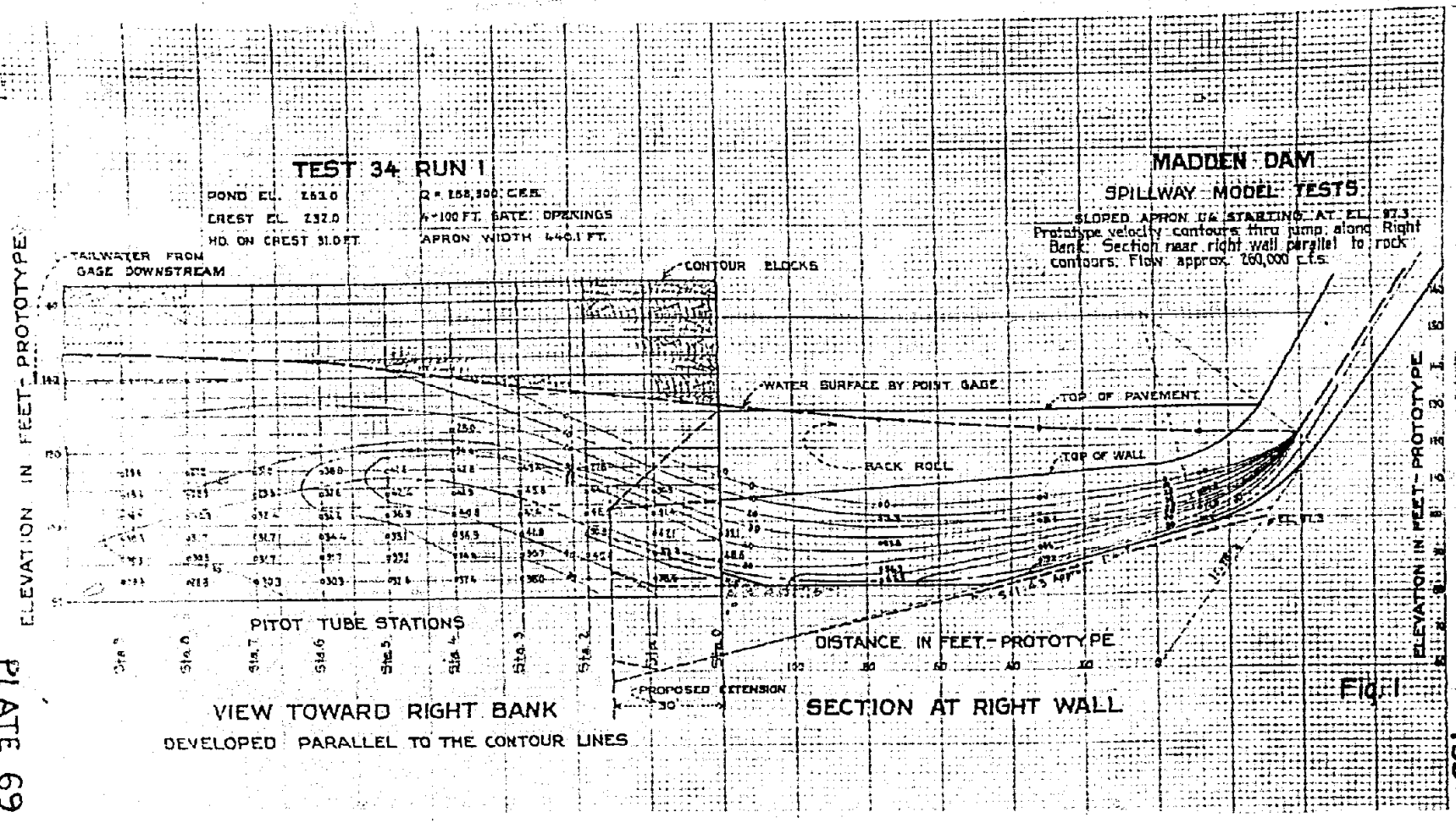
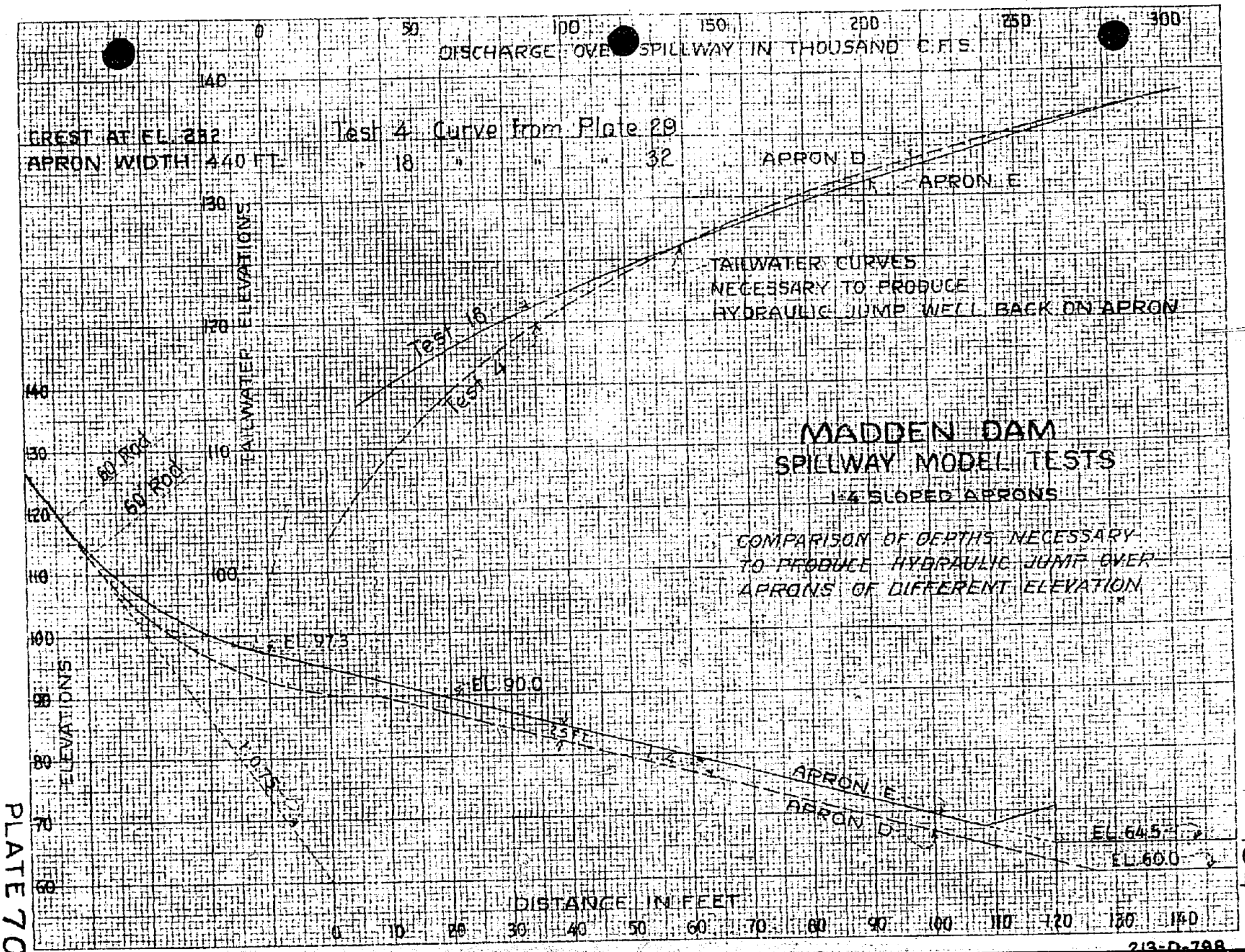

TEST 34 RUN 1
POND EL. 261.0
CREST EL. 232.0
HD. ON CREST 31.0 FT.
Q = 158,300 C.F.S.
4 - 100 FT. GATE OPENINGS
APRON WIDTH 440.1 FT.
MADDEN DAM
SPILLWAY MODEL TESTS
SLOPED APRON 0% STARTING AT EL. 97.3
Prototype velocity contours thru jump along Right Bank. Section near right wall parallel to rock contours. Flow approx. 260,000 c.f.s.
TAILWATER FROM GAGE DOWNSTREAM
CONTOUR BLOCKS
WATER SURFACE BY POINT GAGE
TOP OF PAVEMENT
TOP OF WALL
BACK ROLL
ELEVATION IN FEET - PROTOTYPE
PITOT TUBE STATIONS
DISTANCE IN FEET - PROTOTYPE
PROPOSED EXTENSION
30'
VIEW TOWARD RIGHT BANK
DEVELOPED PARALLEL TO THE CONTOUR LINES
SECTION AT RIGHT WALL
Fig. 1

PLATE 69



MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRONS

COMPARISON OF DEPTHS NECESSARY TO PRODUCE HYDRAULIC JUMP OVER APRONS OF DIFFERENT ELEVATION

213-D-798

PLATE 70

154

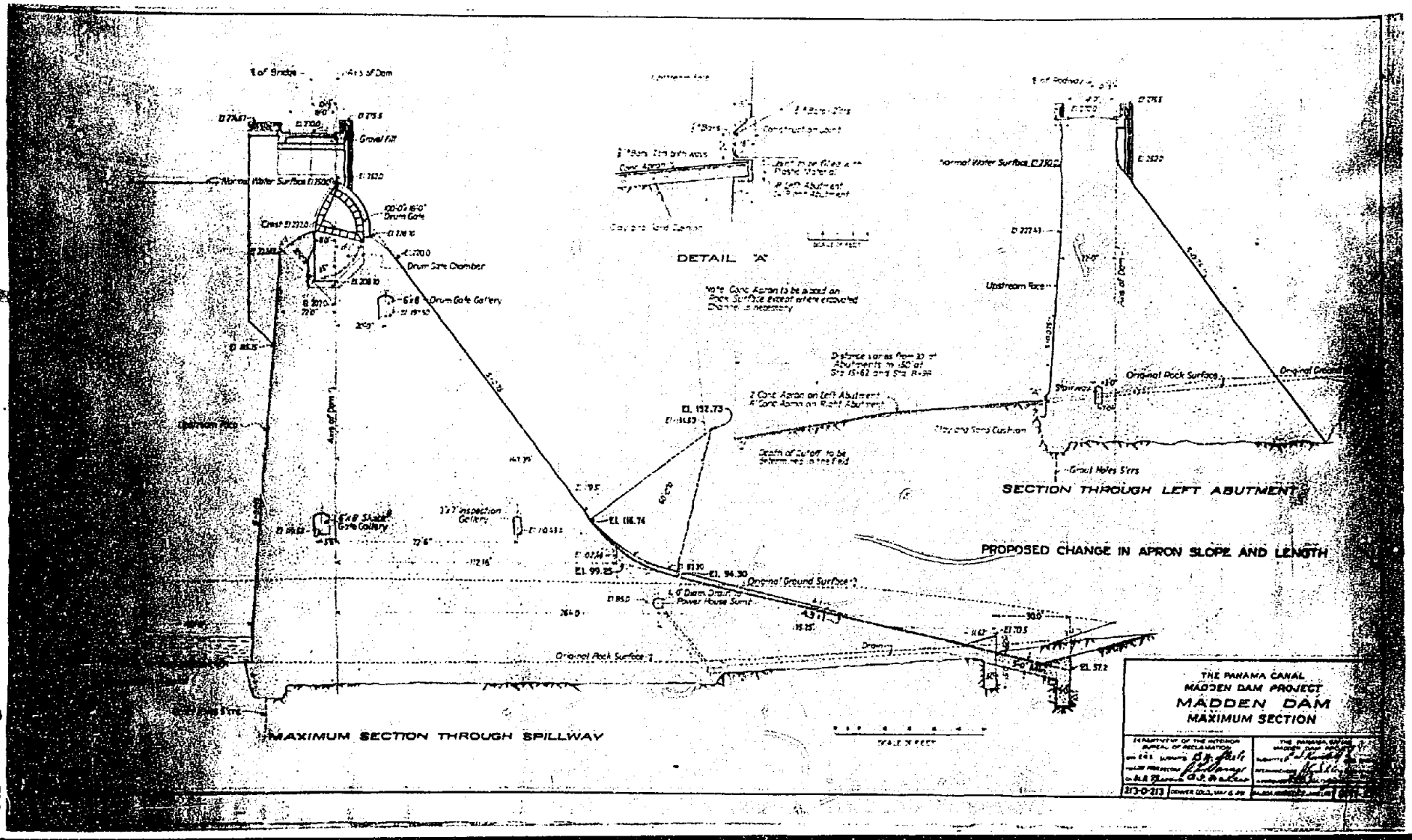

MAXIMUM SECTION THROUGH SPILLWAY
DETAIL "A"
SECTION THROUGH LEFT ABUTMENT
PROPOSED CHANGE IN APRON SLOPE AND LENGTH
Normal Water Surface
Gravel Fill
Drum Gate
Drum Gate Chamber
Drum Gate Gallery
Upstream Face
Axis of Dam
Original Rock Surface
Original Ground Surface
Power House Sump
Grout Holes 5'crs
Stairway
Inspection Gallery
Slide Gate Gallery
Note: Cont. Apron to be based on Rock Surface Except where excavated Depth of Cutoff to be determined in the field
Distance varies from 30' at Abutments to 150' at Sta 15+62 and Sta 16+38
2' Cont. Apron on Left Abutment
4' Cont. Apron on Right Abutment
Plug and Sump Cushion
EL. 132.73
EL. 116.74
EL. 99.25
EL. 94.30
EL. 70.5
EL. 57.2
THE PANAMA CANAL
MADDEN DAM PROJECT
MADDEN DAM
MAXIMUM SECTION
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
THE PANAMA CANAL
313-D-213

PLATE 71

The Power House Wall:

The river bed just below the damsite makes a rather abrupt turn and naturally forms a reverse flow or eddy along the left bank. This eddy is accentuated by the overpour of the water from the tail-race, which is at a higher level, into the jump on the apron. The reverse flow tends to deposit sand and gravel in the draft tube openings and due to its violent surging and pulsating causes variations in the tailwater level that would be detrimental to speed regulation of the turbines. Also, the overpour into the hydraulic jump on the spillway apron creates an unbalanced condition resulting in less velocity retardation.

The remedy is to build the division wall downstream from the power house to a distance where the recovery in the jump has been completed, at which point the water surface levels off at a constant elevation, and to build the wall high enough to prevent overpour. The tests showed that the wall should be extended at least sixty feet downstream from its present length to completely eliminate the whirl for a flow corresponding to the maximum flood of 280,000 c. f. s. At this stage the tailwater is at El. 141.3 and the wall height should be brought up to that point to prevent any overflow.

Of course, the nearer the height and length of the wall

are brought to these maximum dimensions, the more protection is gained against the effects of the whirl. The wall, as finally recommended, was the least size that would afford sufficient protection at the highest flood stage. Its height at El. 128 is sufficient to contain the jump; that is, the actual water surface in the jump does not greatly exceed El. 128 until a point beyond the end of the apron; see Fig. 2, Plate 79. The overpour from the wall at El. 128 is then on top of the jump and does not sweep into it as is the case when the wall is lower at El. 95. The length as extended 30 ft. below the apron does not entirely stop the eddy at maximum discharge but is much more effective than the shorter wall. Absolute elimination of the whirl and back-flow against the power house is attained with the final design of wall for flows up to 160,000 c. f. s., which flood has a probable frequency of occurrence of once in 160 years. This is illustrated on Plate 72, following, which shows flows of about 150,000 c. f. s. with both the original and final design of wall installed. (Note the mirror-like surface of the power house tailrace when overflow is prevented by the high wall, Photo 114.) Any backflow movement of the water picks up sand and deposits it in the draft tube openings; see Photo 115. The pipe seen projecting from the river bed was a support for the Pitot tube track and offered no obstruction to the flow of water. In the picture there also may be seen the

point gage apparatus set in place for measuring the nappes over the crest. With the entire elimination of the backflow, no sand is deposited in the tailrace; see Photo 116. The eddy and overpour into the spillway are quite violent at the maximum discharge when the low wall is installed; see Photo 117. This is overcome to a great extent by raising the wall to El. 128 but there is still some overflow of the wall as the tailwater is at about El. 141; see Photo 118. Even this small backflow tends to leave sand in the tailraces; see Photo 120. The overflow is checked considerably, however, resulting in a very quiet tailrace, as seen by comparison of the two close-up views; see Photos 121 and 122, Plate 74.

The sand deposition as seen in Photo 120 gives an idea of what may be expected to result to the earth overburden when a flood of large proportions occurs. The flow was allowed to run for two hours in this test which represents a period of about 17 hours on the prototype. Note that the rock contours below the apron and along the right bank are swept clean of their overburden and the channel is established along this side. The channel on the left bank is caused by the reverse flow or eddy. Note that its extent is less when no overflow of the power house wall occurs; see Photo 116. The rope seen in the photographs is stretched at El. 92, no flow water level.

Photos 123 and 124 illustrate the flow when the power house wall was removed entirely. The aggravated whirl is clearly seen, as well as the excessive deposition of sand in the draft tube exits. Note the extended channel on the left bank caused by the reverse flow.

This reverse flow could be retarded by building a jetty or dike out into the stream from the left bank and would aid considerably even if a high power house is used as there is a natural tendency for an eddy here due to the curve in the river bed.

Other views of flow conditions and sand deposition when the high wall is used may be seen in photographs of the final design on Plates 5 to 10, also Photo 58, Plate 36, and Photo 107, Plate 64.

The value of the wall extension, even at a low elevation, is seen in Photos 83 and 84, Plate 45.

Plate 75 following shows the energy and hydraulic gradients through the jump on the apron for various flows with both the high and low wall installed. There is little difference discernible but the sand erosion test showed slightly less erosion resulting when the high wall was used. Contrast Photo 55, Plate 35, and Photo 58, Plate 36.

PHOTO 113
LOW WALL
FLOW 150,000 C.F.S.

PHOTO 114
HIGH WALL
FLOW 150,000 C.F.S.

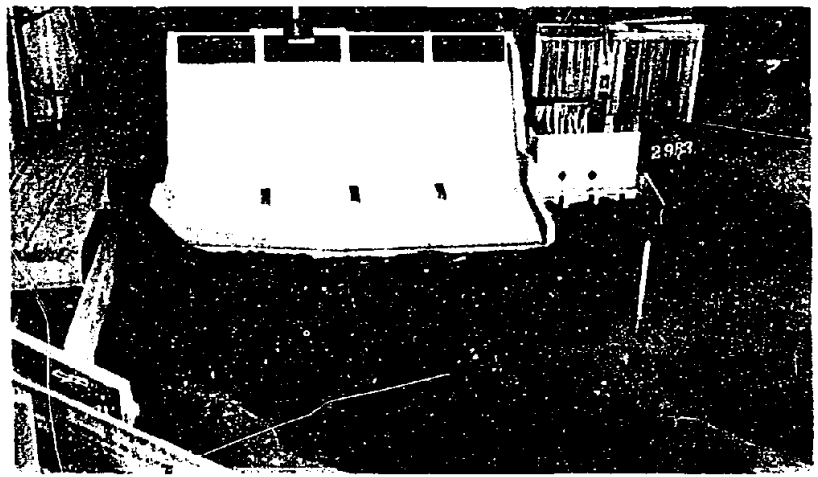


PHOTO 115
LOW WALL
SAND DEPOSITION AFTER
50,000 C.F.S. FLOW

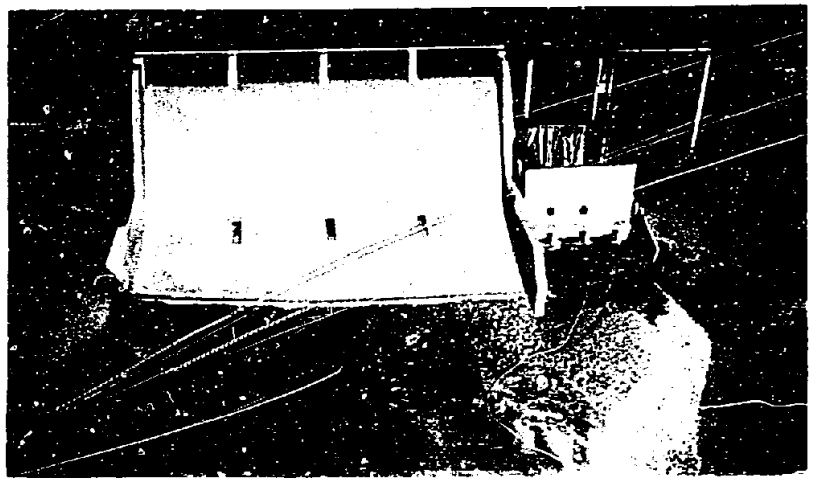
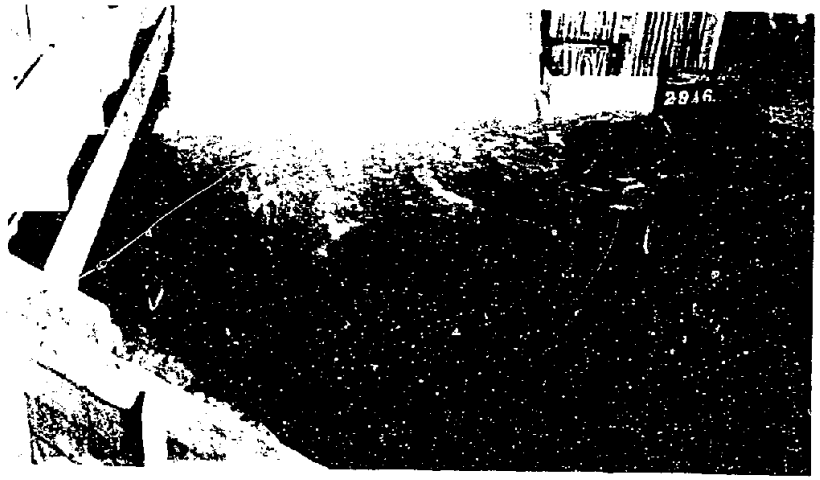


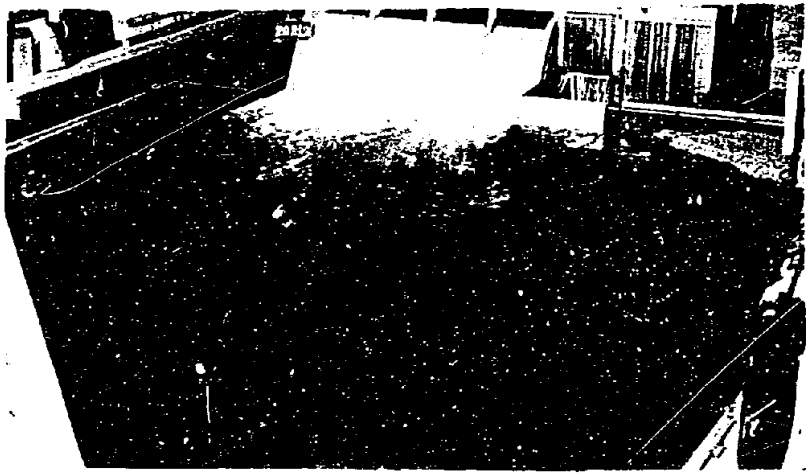
PHOTO 116
HIGH WALL
SAND DEPOSITION AFTER
150,000 C.F.S. FLOW

EFFECT OF HIGH AND LOW POWER HOUSE WALLS

PLATE 72



LOW WALL
FLOW 280,000 C F S
SLUICES & NEEDLE VALVES OPEN



HIGH WALL
FLOW 280,000 C F S
SLUICES & NEEDLE VALVES OPEN

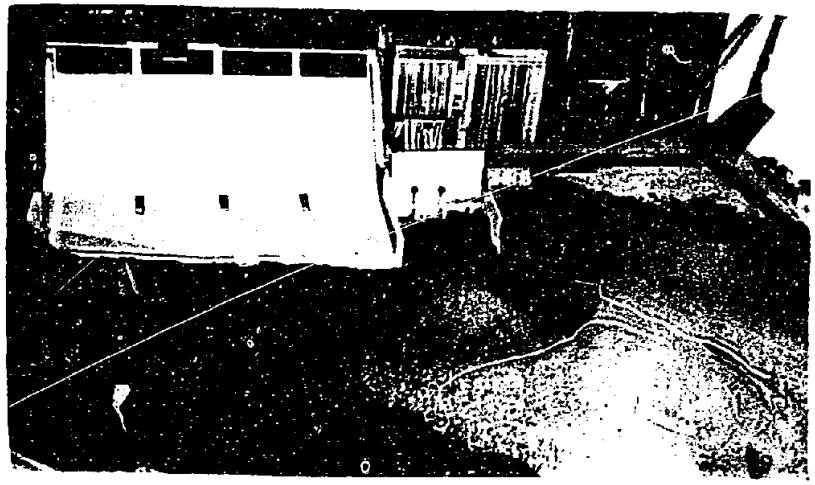


PHOTO 119
LOW WALL
SAND DEPOSITION AFTER
280,000 C F S FLOW

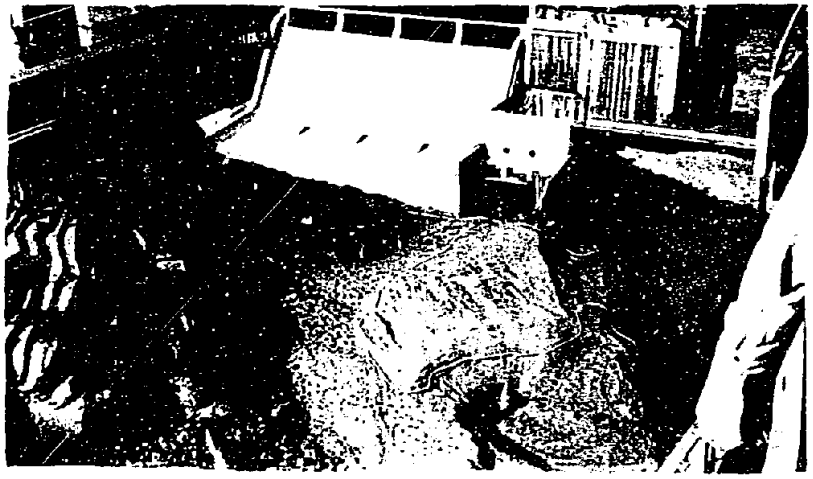


PHOTO 120
HIGH WALL
SAND DEPOSITION AFTER
280,000 C F S FLOW

FIGS AND LOW P H WALL S-E ROSION AFTER WATER FLOOD

PLATE

PHOTO 121
LOW WALL
FLOW 257,000 C.F.S.
SLUICES & NEEDLE VALVES CLOSED

PHOTO 122
HIGH WALL
FLOW 257,000 C.F.S.
SLUICES & NEEDLE VALVES CLOSED

PHOTO 123
NO WALL
FLOW 280,000 C.F.S.
SLUICES & NEEDLE VALVES OPEN

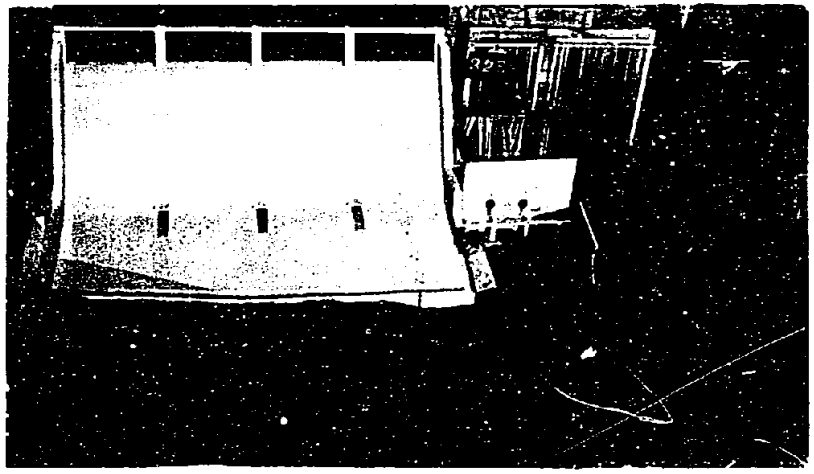


PHOTO 124
NO WALL
FLOW 280,000 C.F.S.
SLUICES & NEEDLE VALVES CLOSED

HIGH LOW AND NO POWER HOUSE WALL

PLATE 74

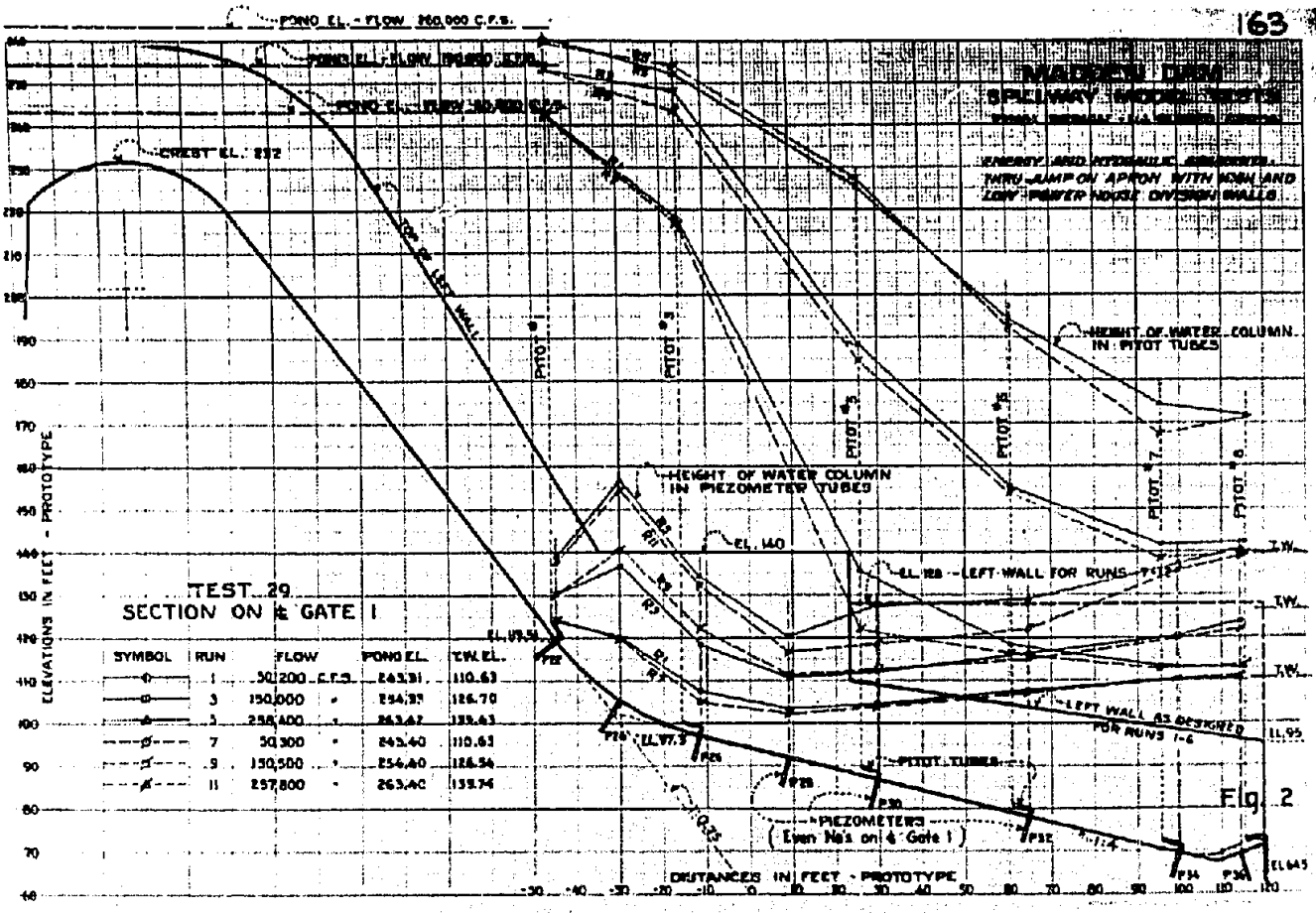

MADDEN DAM
SPILLWAY MODEL TESTS
ENERGY AND HYDRAULIC GRADIENTS
THRU JUMP ON APRON WITH HIGH AND
LOW POWER HOUSE DIVISION WALLS
POND EL. - FLOW 260,000 C.F.S.
POND EL. - FLOW 150,000 C.F.S.
POND EL. - FLOW 50,000 C.F.S.
CREST EL. 232
TOP OF LEFT WALL
PITOT #1
PITOT #3
PITOT #5
PITOT #7
PITOT #8
HEIGHT OF WATER COLUMN IN PITOT TUBES
HEIGHT OF WATER COLUMN IN PIEZOMETER TUBES
EL. 140
EL. 128 - LEFT WALL FOR RUNS 7-12
LEFT WALL AS DESIGNED FOR RUNS 1-6
PITOT TUBES
PIEZOMETERS
(Even No's on ℄ Gate 1)
T.W.
EL. 95
EL. 119.51
EL. 97.5
EL. 84.5
TEST 29
SECTION ON ℄ GATE 1
SYMBOL | RUN | FLOW | POND EL. | T.W. EL.
1 | 50,200 C.F.S. | 243.31 | 110.63
3 | 150,000 " | 254.33 | 126.70
5 | 250,400 " | 263.42 | 139.43
7 | 50,300 " | 243.40 | 110.63
9 | 150,500 " | 254.40 | 126.34
11 | 257,800 " | 263.40 | 139.34
ELEVATIONS IN FEET - PROTOTYPE
DISTANCES IN FEET - PROTOTYPE

Fig. 2

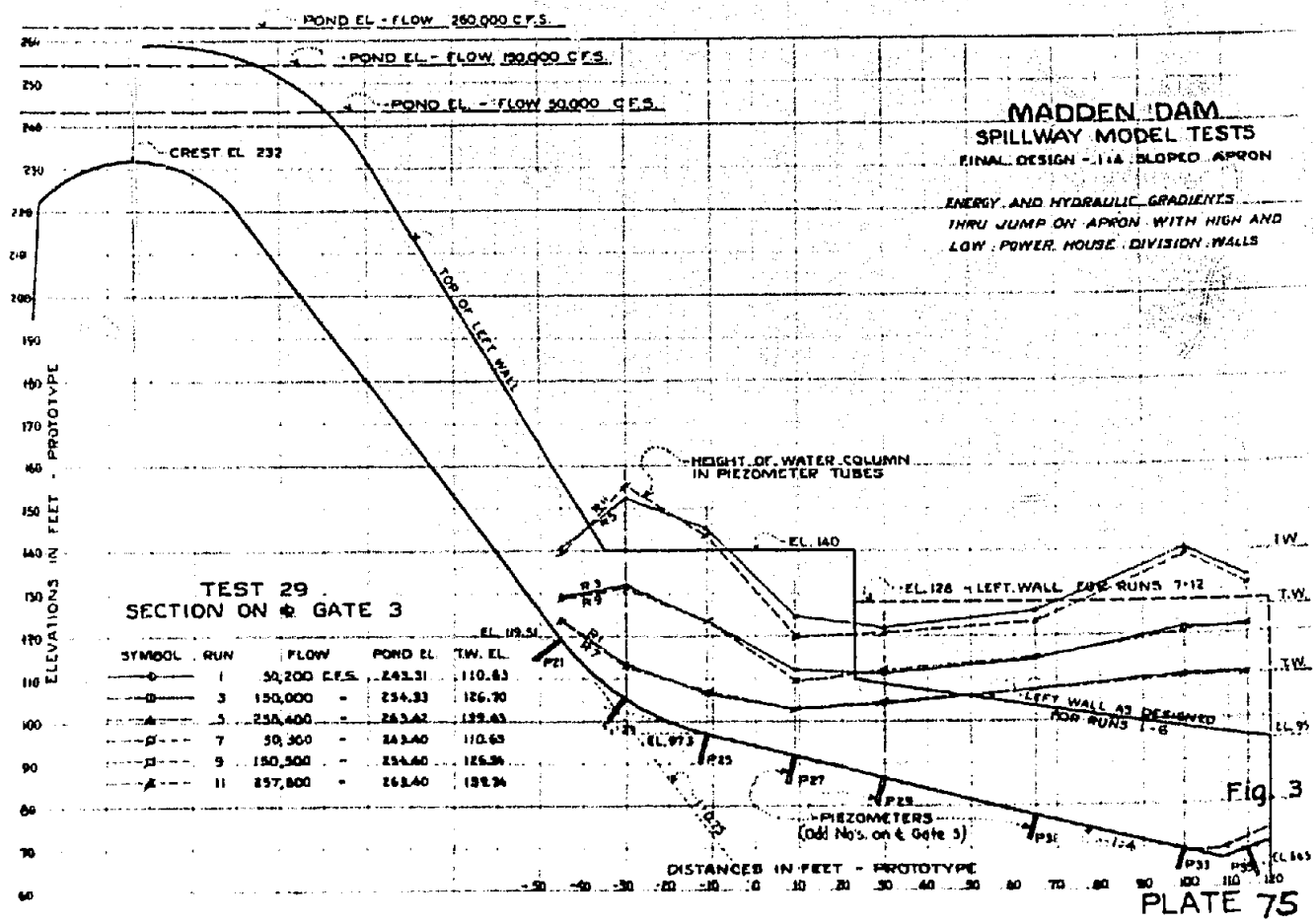

POND EL. - FLOW 260,000 C.F.S.
POND EL. - FLOW 150,000 C.F.S.
POND EL. - FLOW 50,000 C.F.S.
MADDEN DAM
SPILLWAY MODEL TESTS
FINAL DESIGN - 1:4 SLOPED APRON
ENERGY AND HYDRAULIC GRADIENTS
THRU JUMP ON APRON WITH HIGH AND
LOW POWER HOUSE DIVISION WALLS
CREST EL. 232
TOP OF LEFT WALL
HEIGHT OF WATER COLUMN IN PIEZOMETER TUBES
EL. 140
EL. 128 - LEFT WALL FOR RUNS 7-12
LEFT WALL AS DESIGNED FOR RUNS 1-6
T.W.
EL. 95
EL. 119.51
EL. 97.5
EL. 84.5
PIEZOMETERS
(Odd No's. on ℄ Gate 3)
TEST 29
SECTION ON ℄ GATE 3
SYMBOL | RUN | FLOW | POND EL. | T.W. EL.
1 | 50,200 C.F.S. | 243.31 | 110.63
3 | 150,000 " | 254.33 | 126.70
5 | 250,400 " | 263.42 | 139.43
7 | 50,300 " | 243.40 | 110.63
9 | 150,500 " | 254.40 | 126.34
11 | 257,800 " | 263.40 | 139.34
ELEVATIONS IN FEET - PROTOTYPE
DISTANCES IN FEET - PROTOTYPE

Fig. 3

The Final Design:

The final design of the model then included the high power house wall at El. 128 with extension 30 ft. beyond the present length of apron, its downstream face being sloped off 1 to 1; see Photo 126, Plate 76. Other design features, altered from the original layout, which were developed from the models are: the one to four sloped apron starting at El. 97.3, the 6-ft. sloped face sill at the end of the apron, the location of sluiceways on center line of piers, the shape of deflectors over the sluice exits, the height and shape of the end training walls of the spillway, the right bank excavation and apron rise on that side and the correct location of the intake to the float controlling the drum gate operation. Other points that were checked were the capacity of the spillway, coefficients of discharge being determined for the drum gates in all positions with various flows over them; the amount and effect of the vacuum on the crest, the proper aeration of the drum gates from the piers, the elevation of the sluice outlets with respect to the effect of discharging into the hydraulic jump on the apron and erosion below the apron, the possibility of vacuum in sluiceway due to spillway overflow past the exit, and the necessary height of the pavement on the right bank was checked. The proper operation of the spillway gates was also determined and the erosion of the sand and gravel deposits in the river bed pictured after a large flood over the spillway.

Photographs of flow conditions during various stages of an increasing flood have been previously given on Plates 5 and 10. Cross sections through the flow on the apron at these various stages are shown on the following plates: 83, 84, 85, 86, 87 and 88. They give the velocity contours and water surface profiles through the jump and the pressures resulting on the apron and bucket. Lines of high velocity through the jump may be traced in Photos 115 and 116 on Plate 83A which picture stream lines etched on thin iron plates that were first covered with wet paint and immersed in the stream.

The rather complete data included here of a large number of flow stages is presented as proof of satisfactory performance of the hydraulic jump during the whole range of a flood discharge. Also, if properly interpreted, there may be obtained much valuable data for use of the hydraulic jump in other structures.

The writer hopes, in the near future, to find time to attempt an analysis of some of the data and present it in a form from which the information can be applied more directly to the design of other installations.

In an attempt to obtain a mathematical check on the hydraulic jump performance on the apron, tests were made to determine the D_1 and V_1 or the depth and velocity, respectively, entering the jump.

The measurements taken of the velocity with the Pitot tube and the depth by point gage at the toe of the dam for various flows are shown on Plates 83 and 84. They were complicated somewhat by the influence of the sluice openings with the deflectors above but an average of values shows a remarkable check on the theoretical $Q = A\ V$ formula. This is considered a good indication of the accuracy of the testing methods and apparatus. Note that the distribution of velocities through the depth and width of the overflow jet are quite regular without noticeable concentration at any particular point. This satisfies one of the first prerequisites of a good jump performance.

Although the depth and velocity entering the jump may be determined with a fair degree of accuracy there are no acceptable formulas developed, to the writer's knowledge, which would allow a theoretical computation, with any assurance, of the depth of tailwater required to form a satisfactory hydraulic jump on the sloping apron.

The tests showed that on a level apron the depth downstream necessary to produce the jump checks very closely to that computed

from the momentum formula, (see Fig. 2, Plate 25) but on a sloped apron the datum from which to measure D_2 is indeterminate.

Consequently, the sloped apron had to be located, by a series of experiments, at a certain elevation so that with the given tailwater at any flow, the depth was sufficient to start the jump as far as possible back against the face of the dam.

The depth necessary to form the jump was taken as the depth at which the least velocities were recorded on the floor at the end of the apron; see Plates 30, 31 and 33.

This depth over the apron, as finally located, corresponded very closely to the depth furnished by the natural tailwater for all conditions of flow (see Plate 34), thus assuring the highest possible retardation of the velocities leaving the apron.

This scheme of taking the depth necessary to produce the jump as the depth under which the floor velocities would be the least is substantiated when used on the level apron set-up by comparison with momentum formula which will give close coincident depths; see Plate 24.

Plate 85, following, is a condensation of some of the curves previously included, to show the relative conditions of flow over the whole range of required operation. The nappes over the crest and profiles of the surface of the overflow jet, with no tailwater over the apron, were taken by point gage measurements. The velocity is computed

from the head as shown by the difference in elevation to which the water rose in the Pitot tubes and the elevation of the water surface on the apron. The Pitot tubes were fixed in the apron and connected to glass gage tubes by a rubber hose. The water columns in the glass tubes fluctuated quite a bit and the readings plotted were the maximum rise during the run, consequently, the velocities obtained are not necessarily the average through the cross section, but will serve to show the relative values from one point to another. To reconcile the depths as measured by the point gage with the $Q = A V$ formula it was necessary to assume a coefficient of 0.975 for the velocity as indicated by the fixed Pitot tubes which was a ratio of the average values through the jet as found by the test shown on Plates 83 and 84 to the maximum values obtained here. (Velocities drawn on Plates 83 and 84 were measured with the movable Pitot tube, which had a coefficient of 0.975.)

However, the values as measured by the fixed Pitot tubes were the highest value of the velocity obtained and to be on the safe side were used in computing the consequent depths required to cause the hydraulic jump. These depths or D_2 for various flows have been plotted, on Plate 85, parallel to the slope of the floor and, as customary, where the lines cross the tailwater elevation, projected upstream, a locus of the theoretical position of the jump was obtained. It is perhaps significant that these computed positions lie fairly close together near the mid-point of the apron. Another line

is drawn through the points where the theoretical depths cross the actual water surfaces showing the locus of the positions where the computed depths necessary for the jump are actually encountered on the apron.

It is seen that the jump starts well back against the face of the dam at all stages of flow so that the entire length of the apron is utilized in protecting the river bed from the high velocities and turmoil most evident at the beginning of the jump.

The head loss in friction by flow over the model was taken as the distance from the maximum pond surface to the energy gradient as shown by the height to which the water rose in the Pitot tube gage glasses; that is, with no tailwater over the apron.

The friction loss has been plotted as per cent of total head for various depths of flow over the crest and at different stations along the apron.

It is seen that the loss is very small, about one per cent of the total head, down to the point where the flow enters the jump. This gives a factor of safety in predicting the performance of the prototype where the loss in energy by friction may be expected to greatly exceed that shown for flow over the practically friction-loss model.

The relative loss in energy along the floor caused by the jump may be taken as the difference between the energy gradients without and with the tailwater over the apron. The loss is greater as the flow decreases. Or, taking the mean velocity to be repre-

sented by the difference between the tailwater surface and the height of the water column in the Pitot tubes, it is seen that the loss is much more rapid along the apron for the lower flows. At the higher discharges the full recovery in the jump does not take place until some distance downstream so that the velocities leaving the apron are still rather high. Consequently, the greater the length of the apron, to the point where the jump is complete, the more protection is given to the river bed.

As a resume, it may be stated again that it was found that the apron could be shortened to cover less than the distance needed to complete the jump by turning up the high velocities that persist along the floor by placing a small sloped face sill at the end of the apron. This sill need only be high enough to project the velocities clear of the river bed below and allow them to continue downstream considerably underneath the surface of the water until finally retarded by the static pressure in the natural course of the hydraulic jump action; compare Plates 79 and 81.

The sill finally adopted was 6 ft. high with a sloped face down to the apron although one 4.5 feet high seemed just as effective in preventing erosion in the river bed below.

If the sill is made too large it will project the high velocities from the floor of the apron to the surface of the stream when their rate of retardation will be much less and the river banks will be subjected to erosive forces that would prove objectionable.

The higher sill would also be subjected to greater impact, thus increasing the possibilities of a break, which, however small, would endanger the whole structure.

This was considered as the chief objection to the use of the dentated sill layout, as originally planned, and led to the investigation of the sloped apron, which was finally adopted.

The hydraulic jump forms on this slope at all stages of flood, and the dissipation of the tremendous energy of the falling sheet takes place by the natural action of impact of water against water.

Acknowledgment is made to Mr. W. F. Bingham, Junior Engineer for the Bureau of Reclamation, for his services in plotting the test data and making the curves and drawings included in this report.

I would also like to express here my appreciation of the fine cooperation and assistance given me by the bureau engineers and the whole personnel employed at the hydraulic laboratory in performing these tests.

Respectfully submitted,

R. R. Randolph, Jr., Asst. Engr.,
The Panama Canal.

Denver, Colorado,
February 1, 1932.

PHOTO 125
PLAN OF FINAL DESIGN MODEL

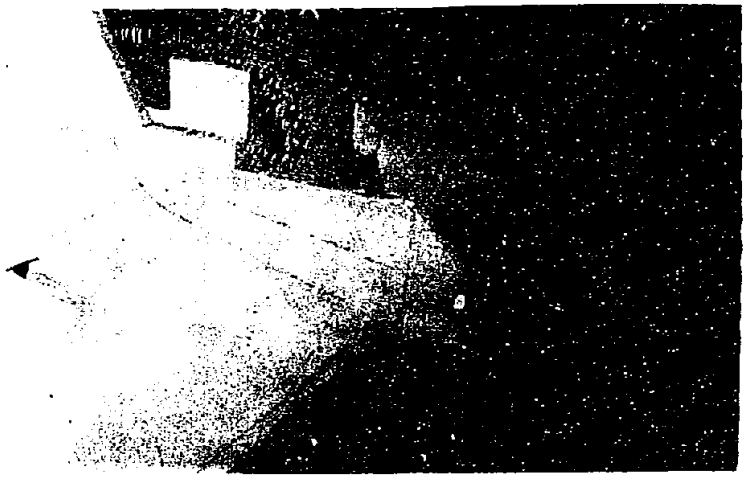


PHOTO 126
FINAL DESIGN MODEL
LOOKING TOWARD POWER HOUSE

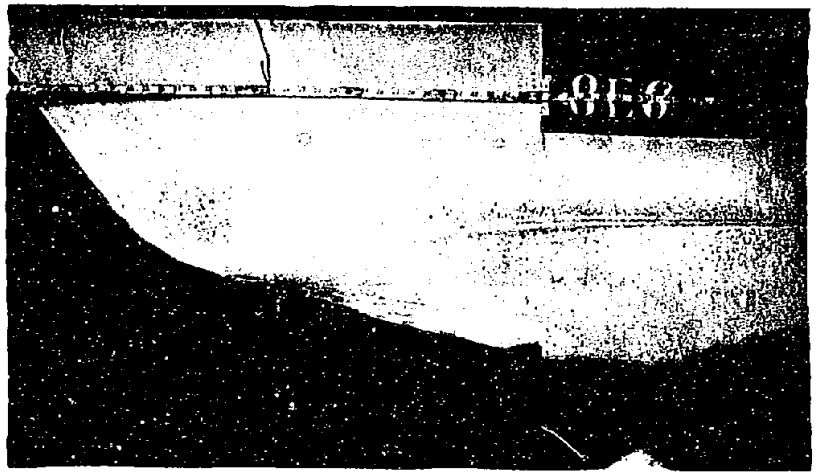


PHOTO 127
STREAM LINES THROUGH JUMP
FLOW 76,000 C.F.S.

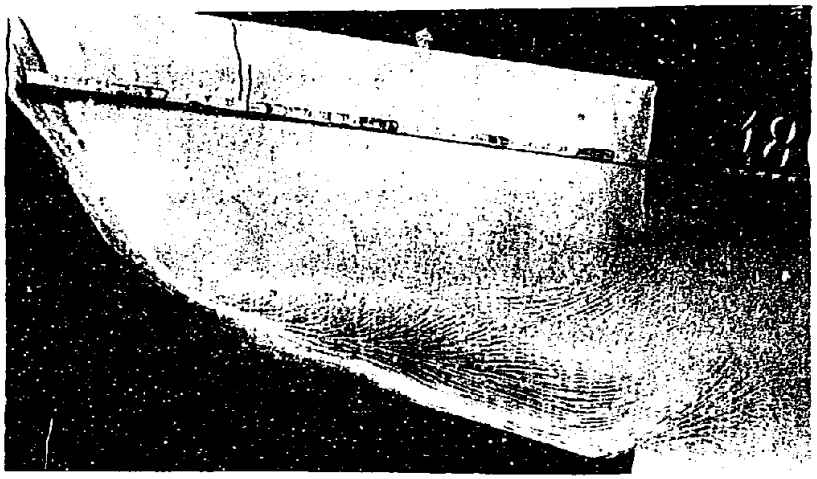


PHOTO 128
STREAM LINES THROUGH JUMP
FLOW 258,000 C.F.S.

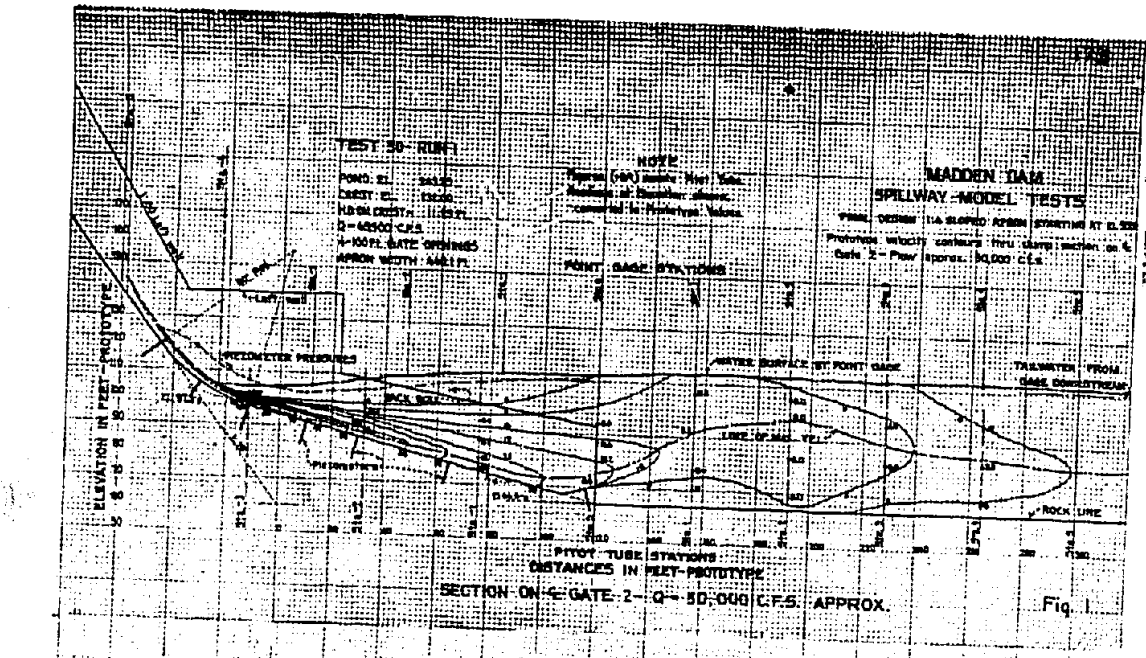


Fig. 1

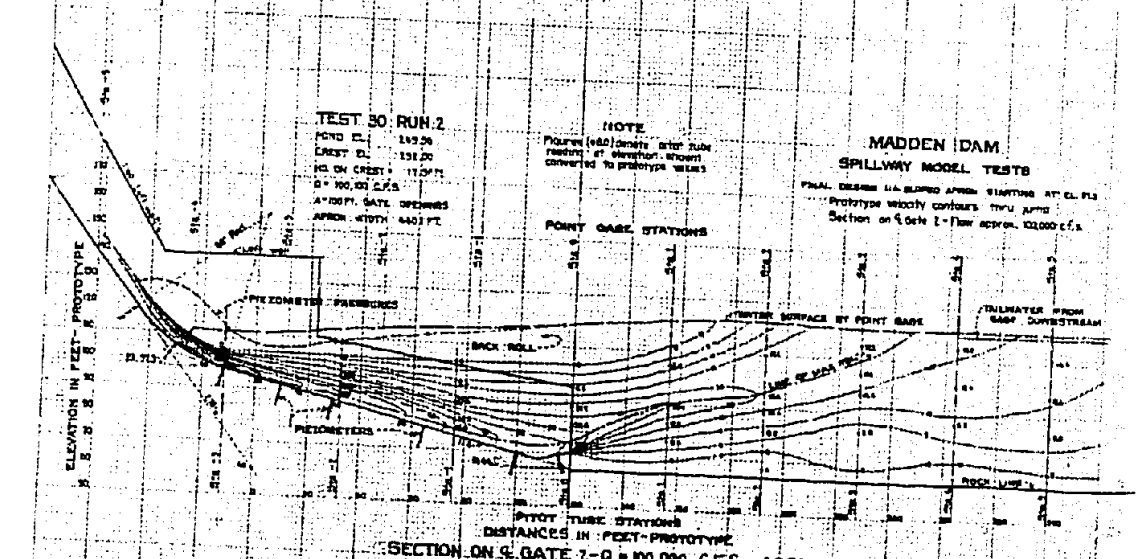


Fig. 2

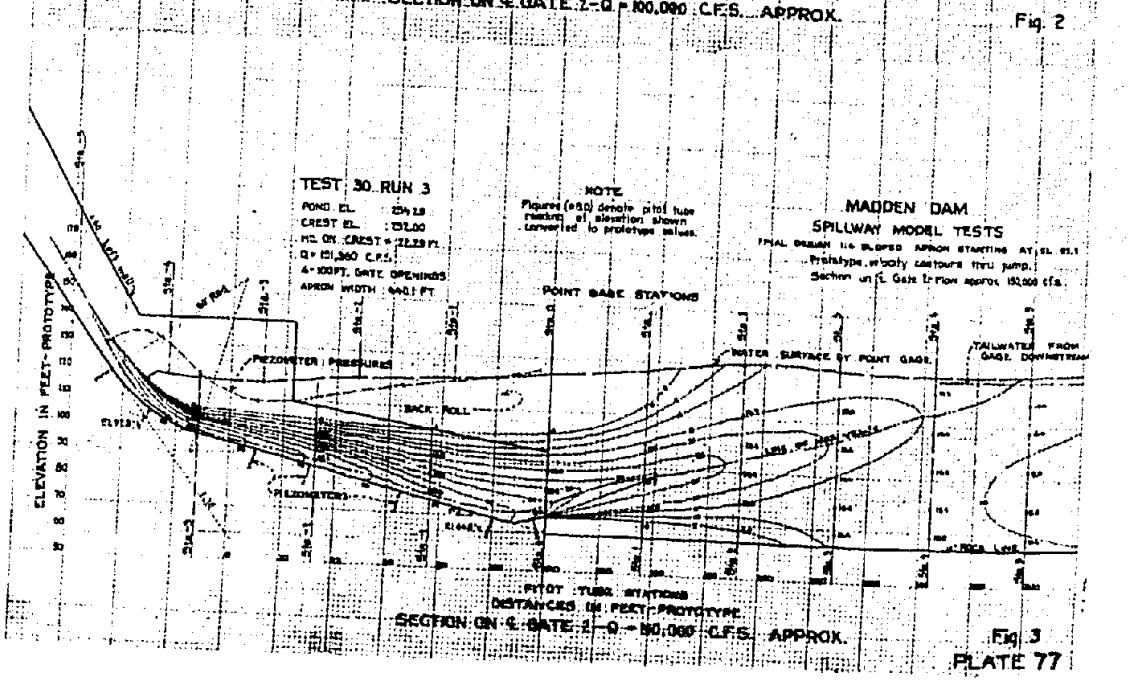


Fig. 3

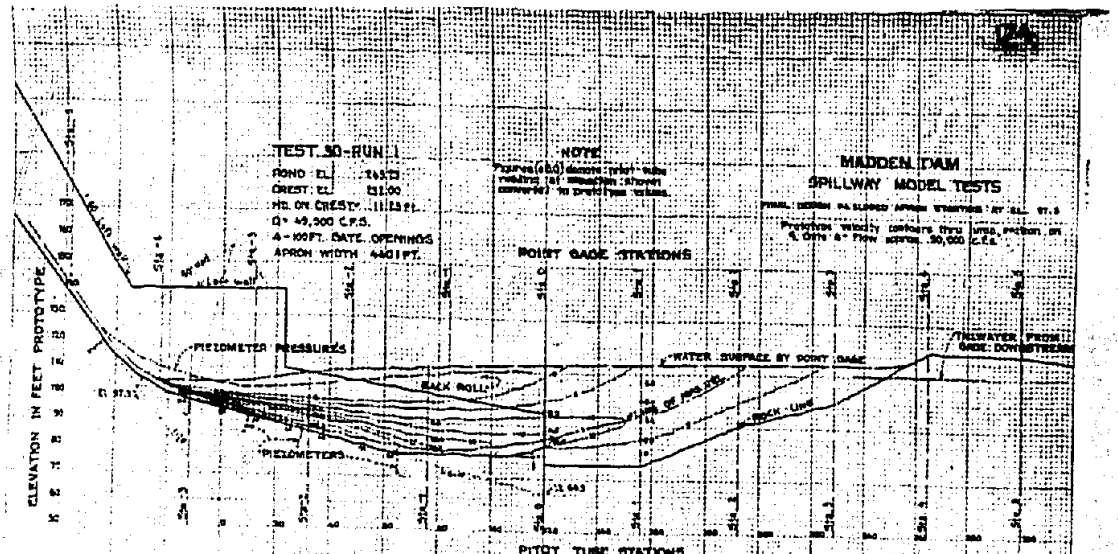


Fig. 1

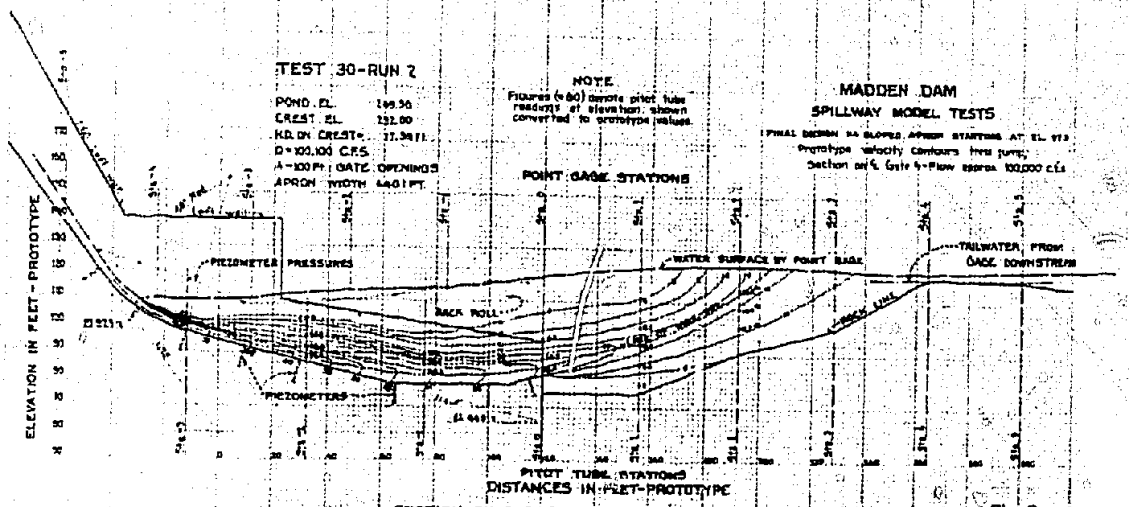


Fig. 2

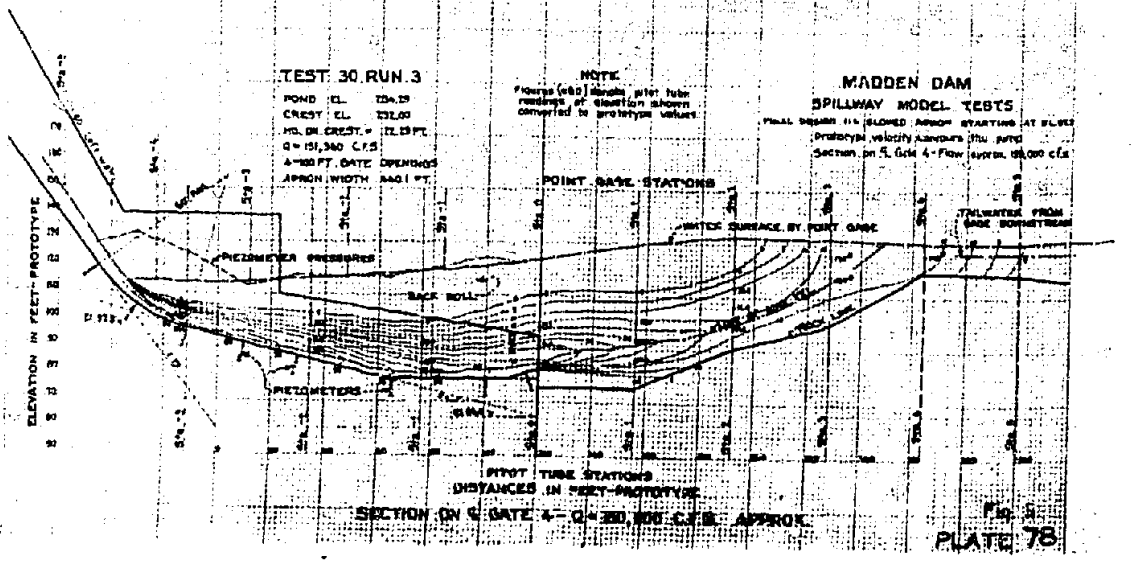


Fig. 3

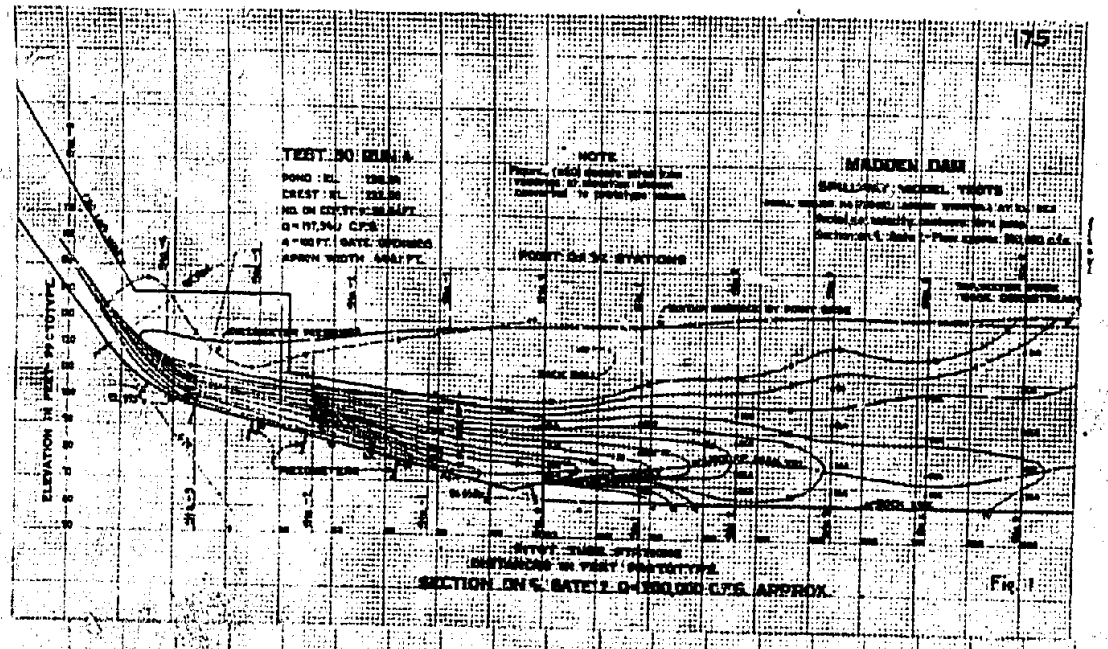


Fig. 1

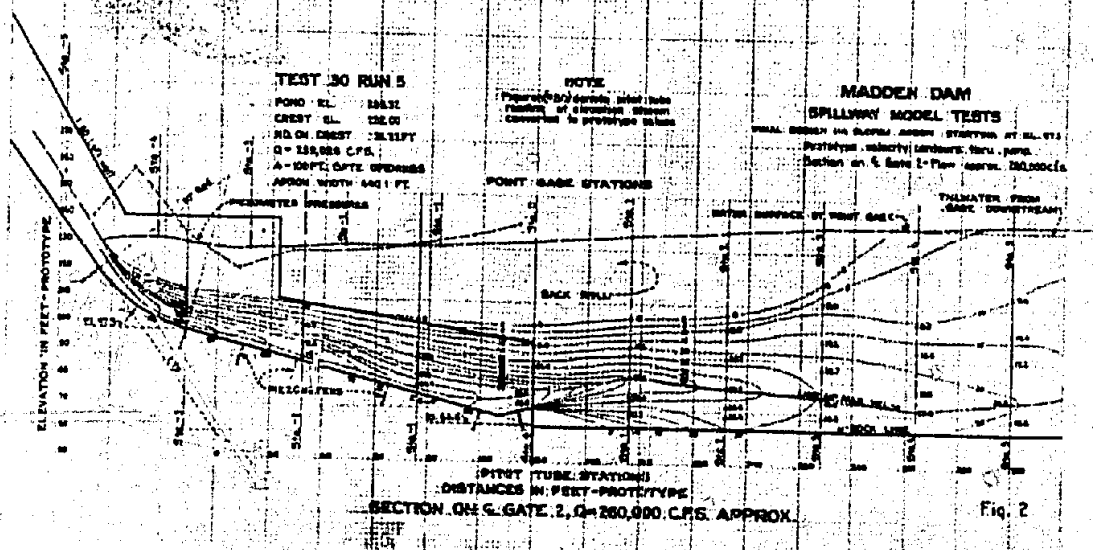


Fig. 2

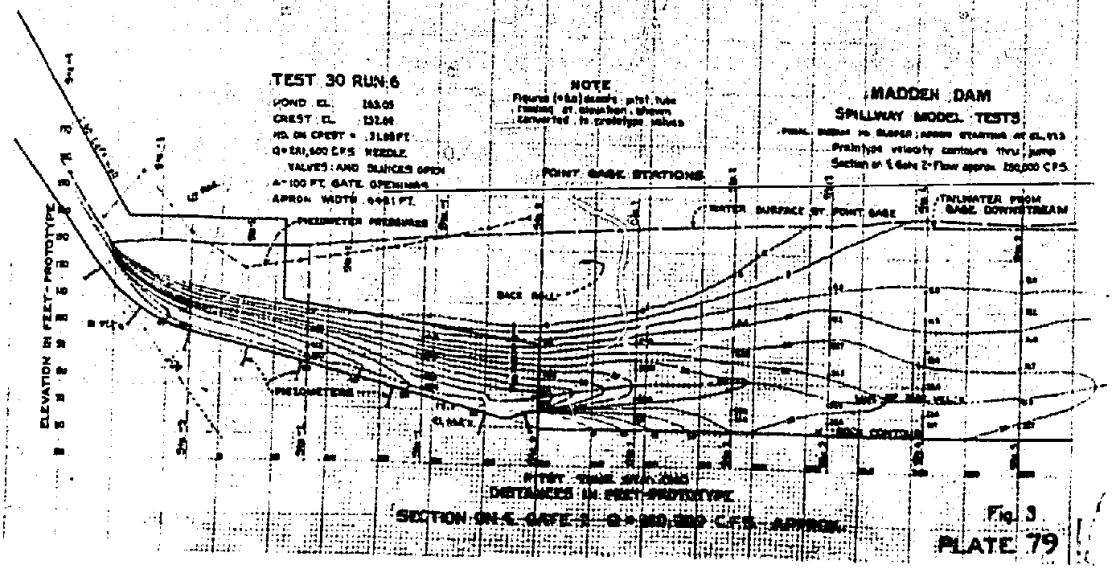


Fig. 3

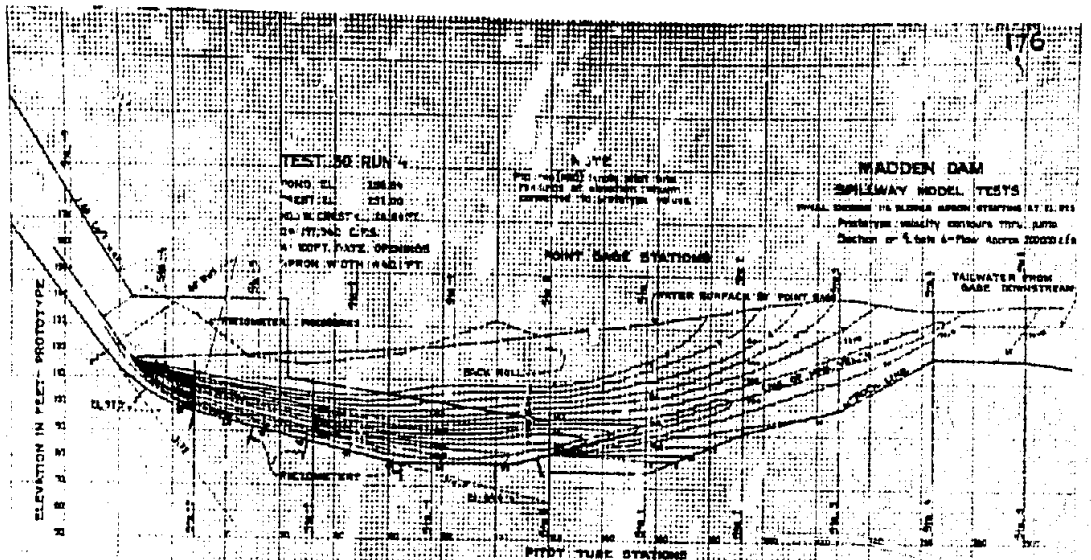


Fig. 1

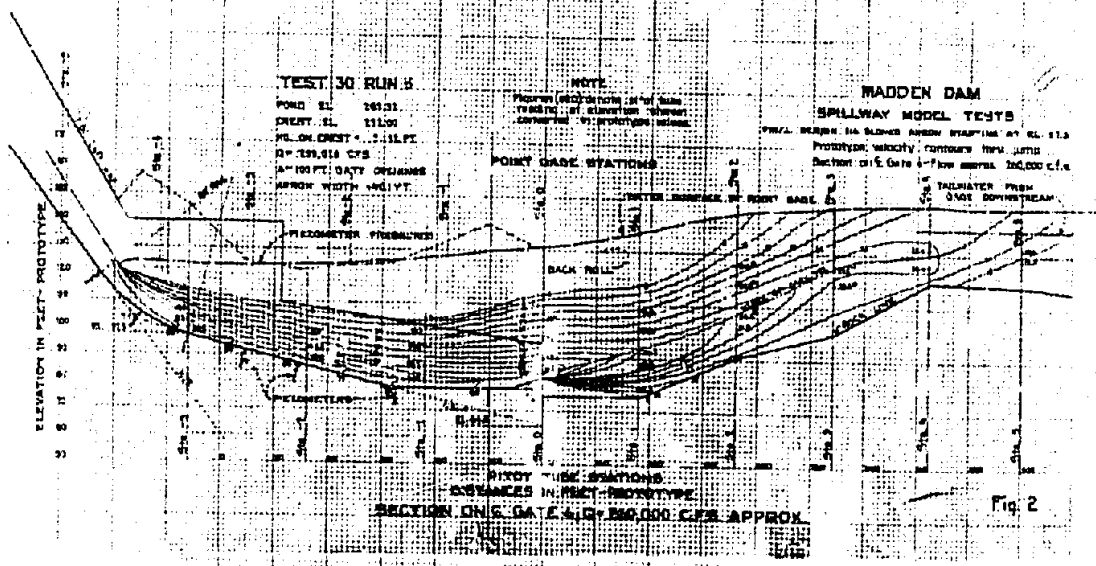


Fig. 2

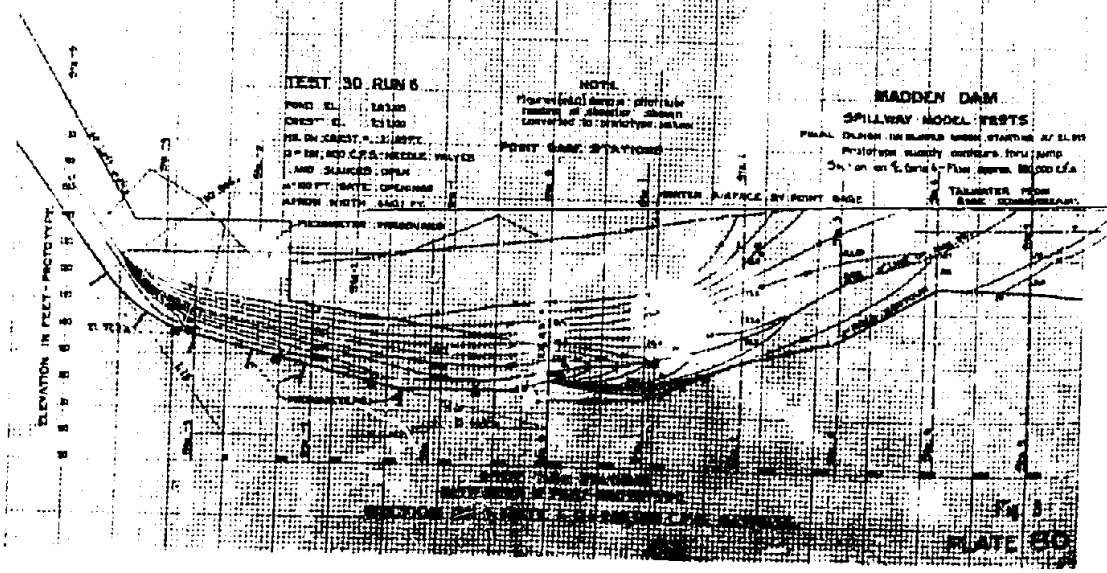


Fig. 3

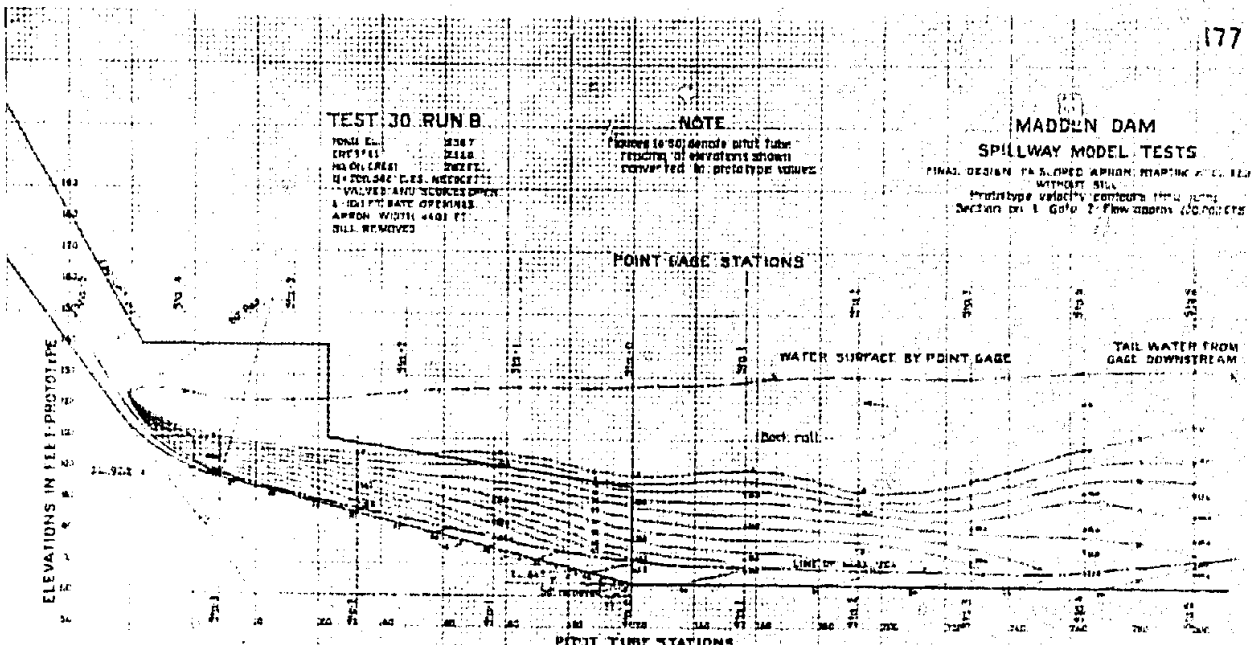

TEST 30 RUN 8
NOTE
MADDEN DAM
SPILLWAY MODEL TESTS
POINT GAGE STATIONS
WATER SURFACE BY POINT GAGE
TAIL WATER FROM GAGE DOWNSTREAM
ELEVATIONS IN FEET - PROTOTYPE
PITOT TUBE STATIONS
DISTANCES IN FEET - PROTOTYPE
SECTION ON ℄ GATE 2 - Q - 200,000 C.F.S.

Fig. 1

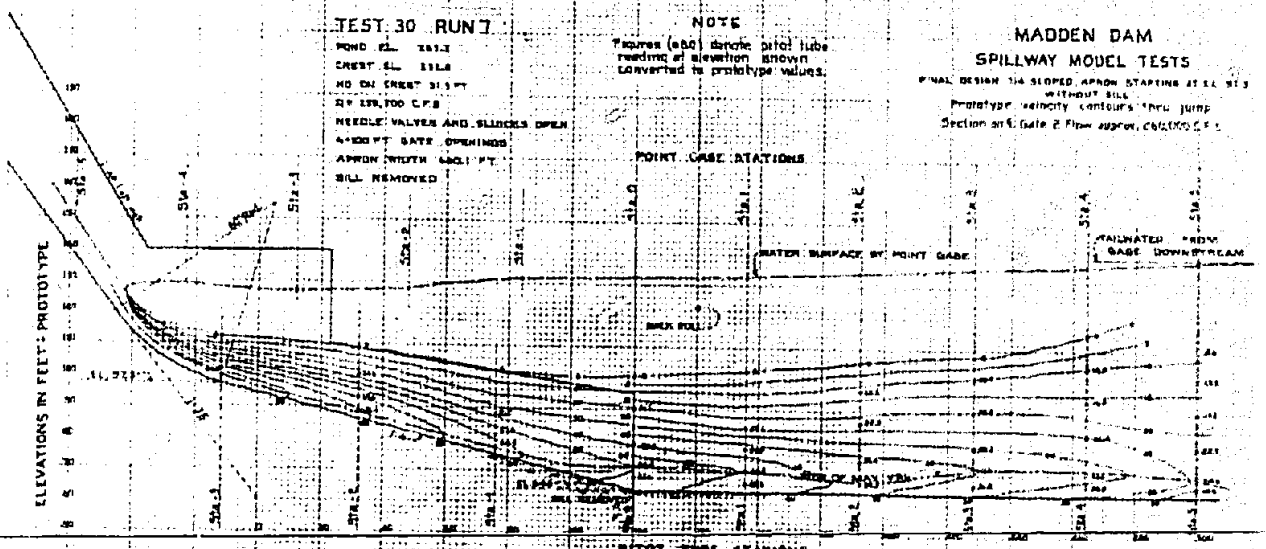

TEST 30 RUN 7
NOTE
MADDEN DAM
SPILLWAY MODEL TESTS
POINT GAGE STATIONS
WATER SURFACE BY POINT GAGE
TAILWATER FROM GAGE DOWNSTREAM
ELEVATIONS IN FEET - PROTOTYPE
PITOT TUBE STATIONS
DISTANCES IN FEET PROTOTYPE
SECTION ON ℄ GATE 2 - Q = 260,000 C.F.S. APPROX.

Fig. 2

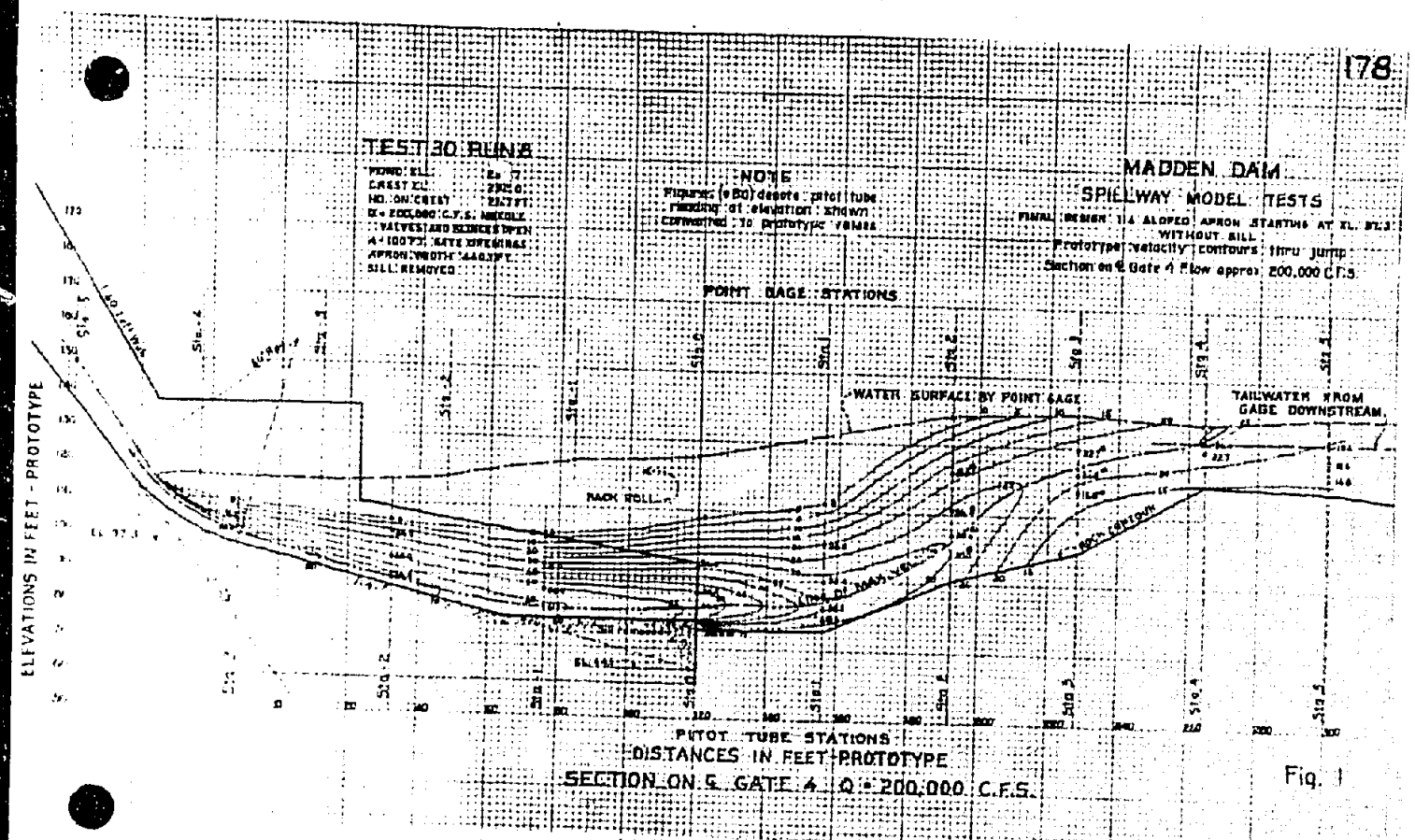

TEST 30 RUN 8
NOTE
Figures (+80) denote pitot tube reading at elevation shown converted to prototype values
MADDEN DAM
SPILLWAY MODEL TESTS
FINAL DESIGN 1:4 SLOPED APRON STARTING AT EL. 97.5 WITHOUT SILL
Prototype velocity contours thru jump
Section on ℄ Gate 4 Flow approx. 200,000 C.F.S.
POINT GAGE STATIONS
WATER SURFACE BY POINT GAGE
TAILWATER FROM GAGE DOWNSTREAM
ROCK ROLL
ROCK CONTOUR
ELEVATIONS IN FEET - PROTOTYPE
PITOT TUBE STATIONS
DISTANCES IN FEET PROTOTYPE
SECTION ON ℄ GATE 4 Q = 200,000 C.F.S.

Fig. 1

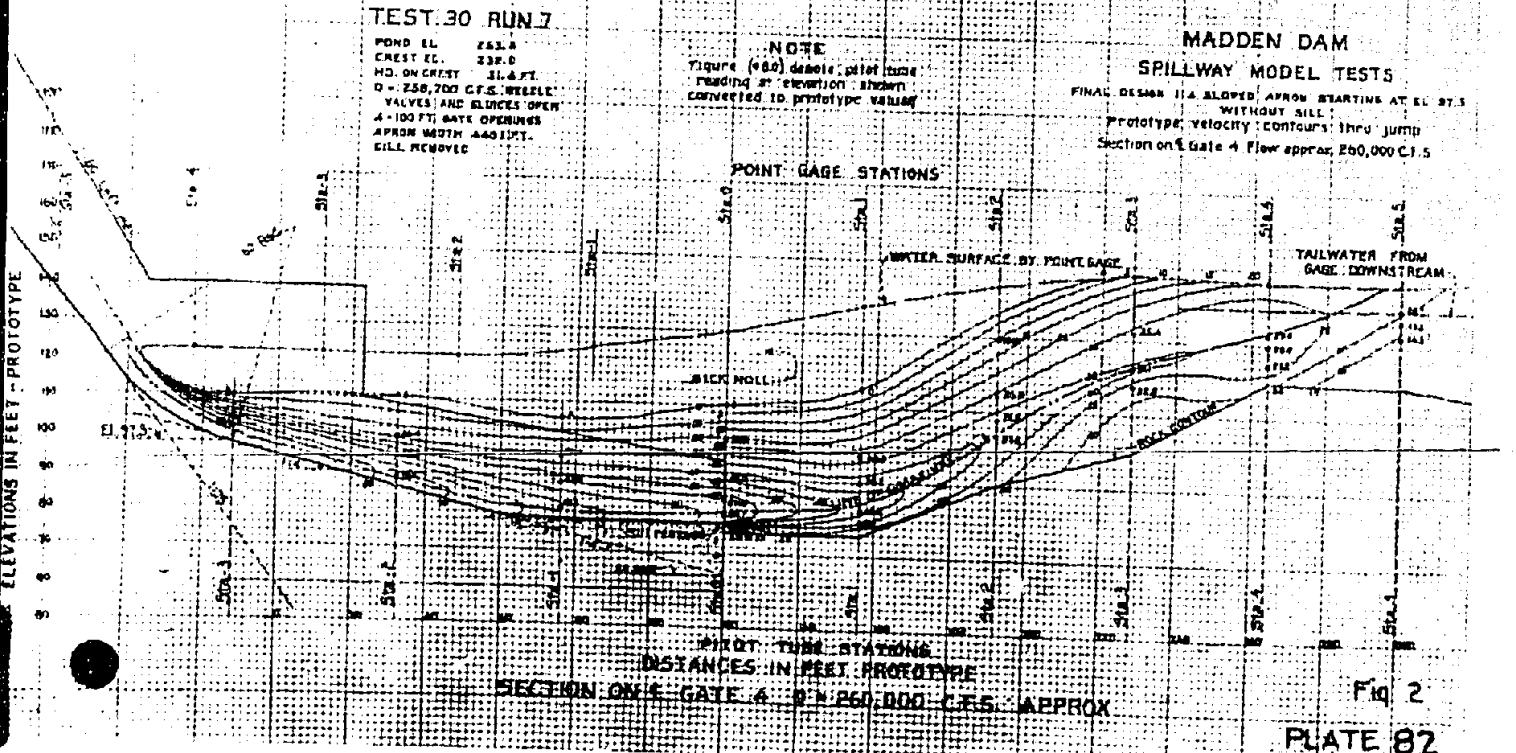
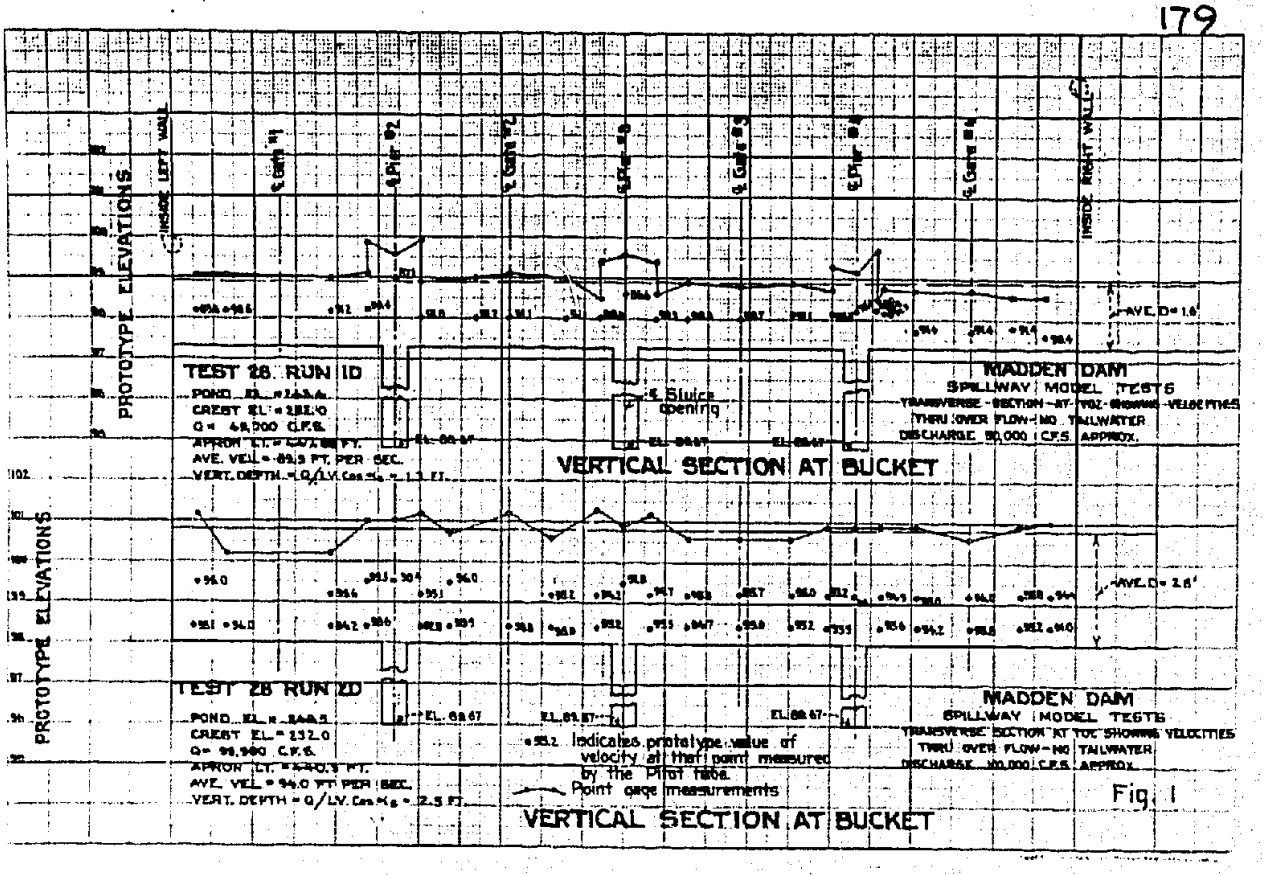
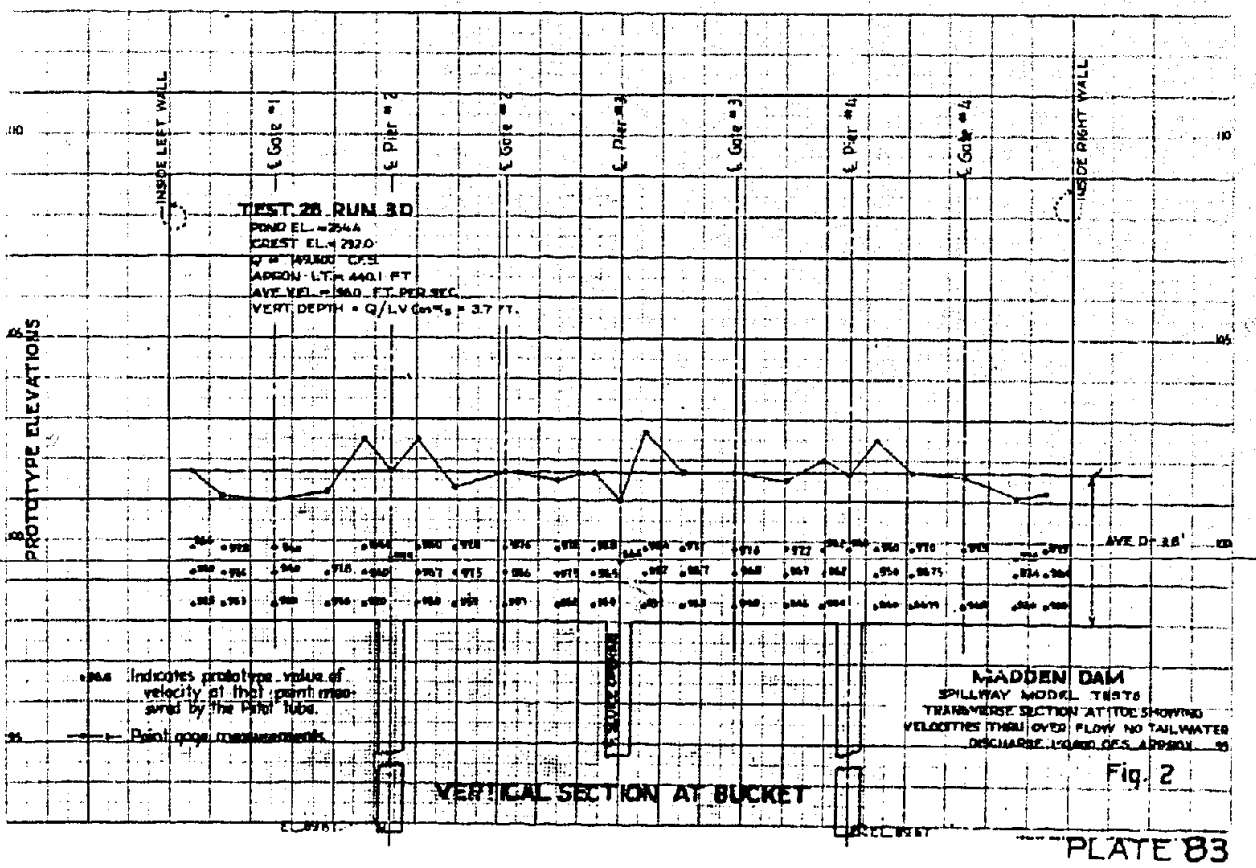
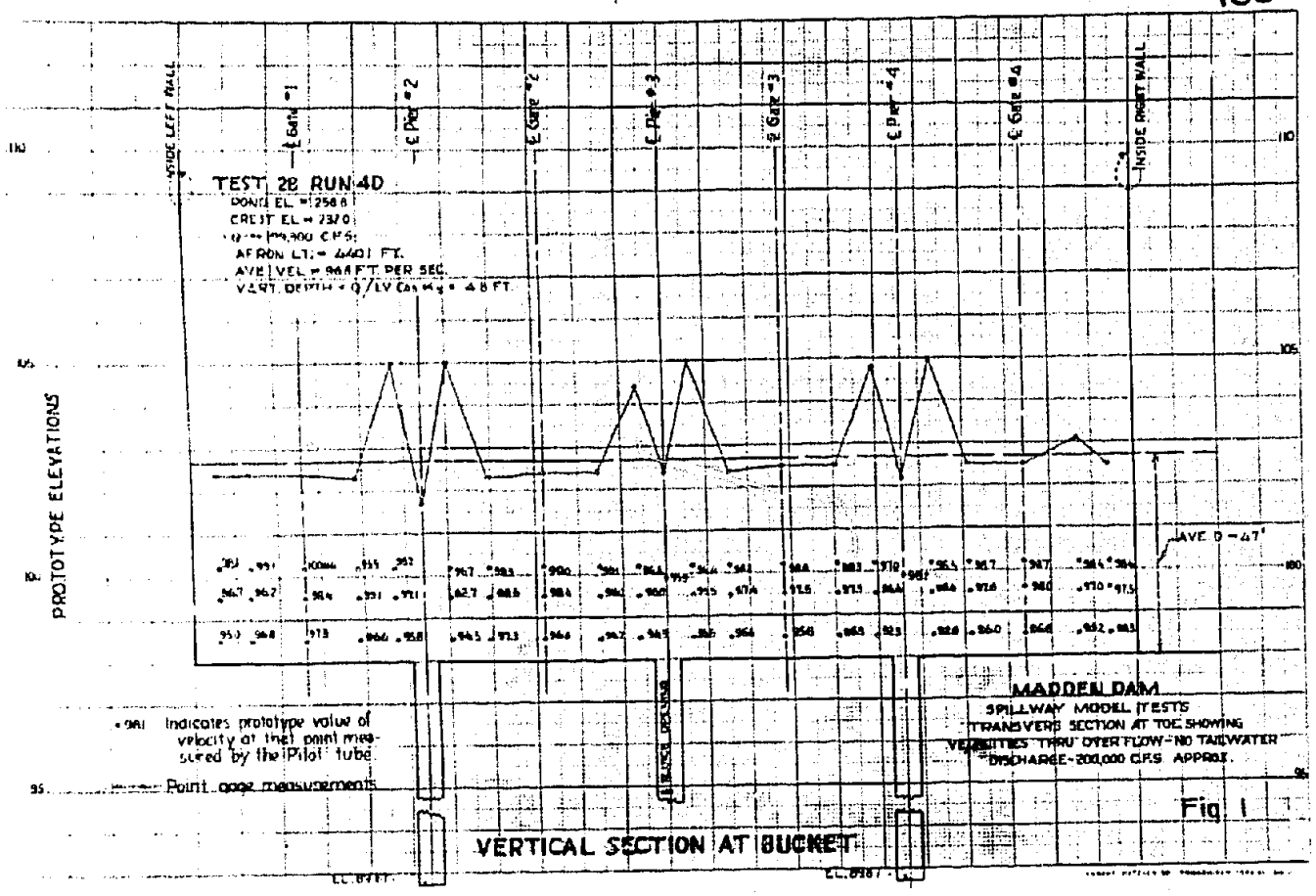

TEST 30 RUN 7
POND EL. 253.8
CREST EL. 232.0
HD. ON CREST 31.8 FT.
Q = 258,700 C.F.S. WHEELE VALVES AND SLUICES OPEN
4 - 100 FT. GATE OPENINGS
APRON WIDTH 440 FT.
SILL REMOVED
NOTE
Figure (+80) denote pitot tube reading at elevation shown converted to prototype values
MADDEN DAM
SPILLWAY MODEL TESTS
FINAL DESIGN 1:4 SLOPED APRON STARTING AT EL. 97.5 WITHOUT SILL
Prototype velocity contours thru jump
Section on ℄ Gate 4 Flow approx. 260,000 C.F.S.
POINT GAGE STATIONS
WATER SURFACE BY POINT GAGE
TAILWATER FROM GAGE DOWNSTREAM
ROCK ROLL
ROCK CONTOUR
ELEVATIONS IN FEET - PROTOTYPE
PITOT TUBE STATIONS
DISTANCES IN FEET PROTOTYPE
SECTION ON ℄ GATE 4 Q = 260,000 C.F.S. APPROX

Fig 2


PROTOTYPE ELEVATIONS
INSIDE LEFT WALL
℄ Gate #1
℄ Pier #2
℄ Gate #2
℄ Pier #3
℄ Gate #3
℄ Pier #4
℄ Gate #4
INSIDE RIGHT WALL
AVE. D = 1.6'
TEST 28 RUN 1D
POND EL. = 263.6
CREST EL. = 232.0
Q = 48,700 C.F.S.
APRON EL. = 207.88 FT.
AVE. VEL. = 89.3 FT. PER SEC.
VERT. DEPTH = Q/LV. Cos ϕs = 1.3 FT.
℄ Sluice opening
EL. 88.67
MADDEN DAM
SPILLWAY MODEL TESTS
TRANSVERSE SECTION AT TOE SHOWING VELOCITIES
THRU OVER FLOW - NO TAILWATER
DISCHARGE 50,000 C.F.S. APPROX.
VERTICAL SECTION AT BUCKET
PROTOTYPE ELEVATIONS
AVE. D = 2.8'
TEST 28 RUN 2D
POND EL. = 266.5
CREST EL. = 232.0
Q = 98,960 C.F.S.
APRON EL. = 190.5 FT.
AVE. VEL. = 94.0 FT. PER SEC.
VERT. DEPTH = Q/LV. Cos ϕs = 2.5 FT.
EL. 88.67
EL. 89.67
EL. 88.67
95.2 Indicates prototype value of velocity at that point measured by the Pitot tube.
Point gage measurements
MADDEN DAM
SPILLWAY MODEL TESTS
TRANSVERSE SECTION AT TOE SHOWING VELOCITIES
THRU OVER FLOW - NO TAILWATER
DISCHARGE 50,000 C.F.S. APPROX.
Fig. 1
VERTICAL SECTION AT BUCKET


INSIDE LEFT WALL
℄ Gate #1
℄ Pier #2
℄ Gate #2
℄ Pier #3
℄ Gate #3
℄ Pier #4
℄ Gate #4
INSIDE RIGHT WALL
TEST 28 RUN 3D
POND EL. = 254.4
CREST EL. = 232.0
Q = 149,800 C.F.S.
APRON EL. = 440.1 FT
AVE. VEL. = 94.0 FT. PER SEC.
VERT. DEPTH = Q/LV Cos ϕs = 3.7 FT.
PROTOTYPE ELEVATIONS
AVE. D = 3.8'
95.6 Indicates prototype value of velocity at that point measured by the Pitot tube.
Point gage measurements
MADDEN DAM
SPILLWAY MODEL TESTS
TRANSVERSE SECTION AT TOE SHOWING VELOCITIES THRU OVER FLOW NO TAILWATER
DISCHARGE 149,800 C.F.S. APPROX.
Fig. 2
VERTICAL SECTION AT BUCKET
EL. 89.67
EL. 89.67


TEST 28 RUN 4D
POND EL. = 1258.8
CREST EL. = 1232.0
Q = 199,900 CFS
APRON LT. = 440.1 FT.
AVE. VEL. = 96.8 FT. PER SEC.
VERT. DEPTH = Q/LV Cos θ = 4.8 FT.
INSIDE LEFT WALL
INSIDE RIGHT WALL
℄ Gate #1
℄ Pier #2
℄ Gate #2
℄ Pier #3
℄ Gate #3
℄ Pier #4
℄ Gate #4
PROTOTYPE ELEVATIONS
AVE D = 4.7'
MADDEN DAM
SPILLWAY MODEL TESTS
TRANSVERS SECTION AT TOE SHOWING
VELOCITIES THRU OVERFLOW - NO TAILWATER
DISCHARGE - 200,000 CFS APPROX.
Indicates prototype value of velocity at that point measured by the Pitot tube
Point gage measurements
Fig. 1
VERTICAL SECTION AT BUCKET

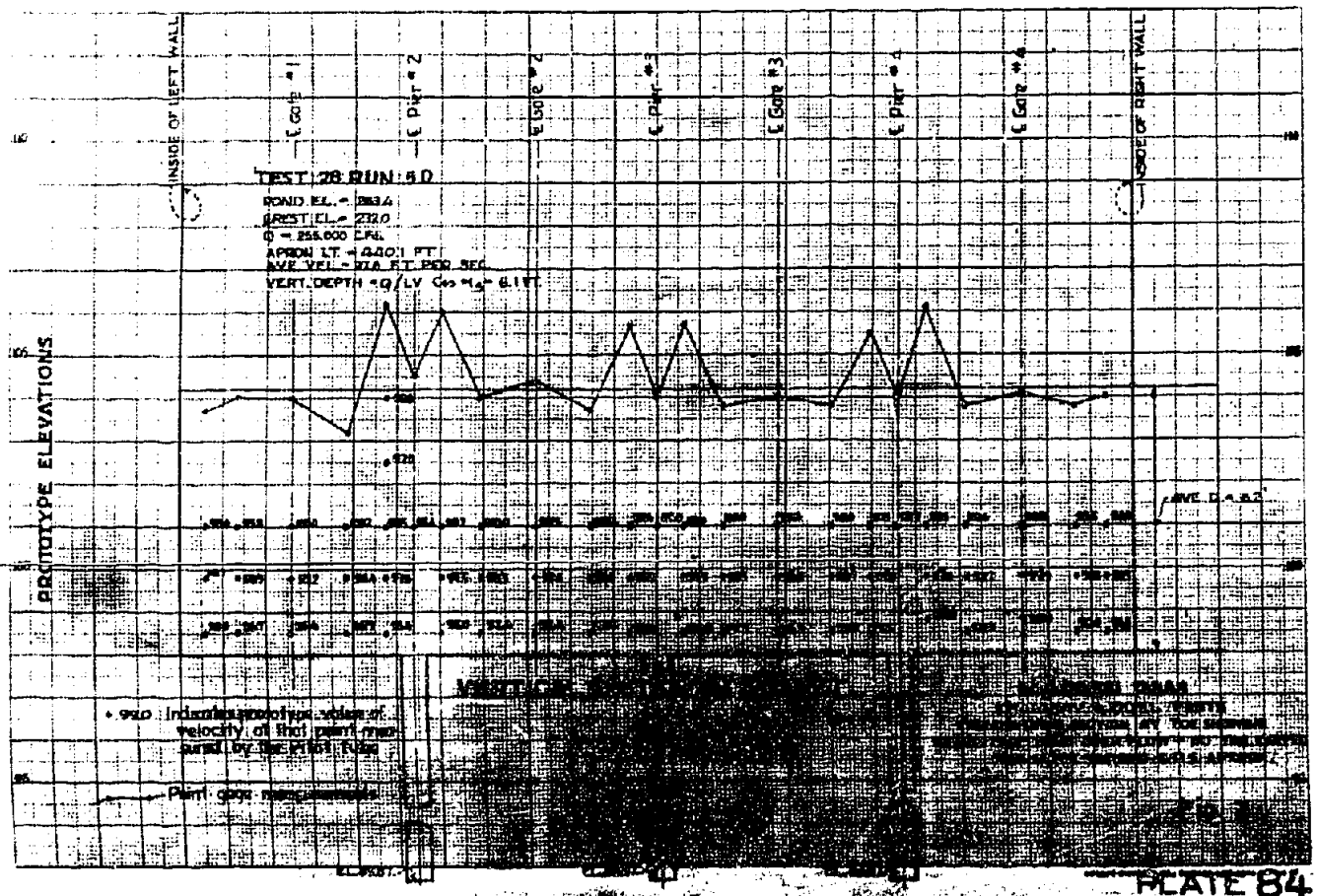

TEST 28 RUN 5D
POND EL. = 1263.4
CREST EL. = 1232.0
Q = 255,000 CFS
APRON LT. = 440.1 FT.
AVE. VEL. = 97.8 FT. PER SEC.
VERT. DEPTH = Q/LV Cos θ = 6.1 FT.
INSIDE OF LEFT WALL
INSIDE OF RIGHT WALL
℄ Gate #1
℄ Pier #2
℄ Gate #2
℄ Pier #3
℄ Gate #3
℄ Pier #4
℄ Gate #4
PROTOTYPE ELEVATIONS
AVE D = 5.2'
Indicates prototype value of velocity at that point measured by the Pitot Tube
Point gage measurements

PLATE 84

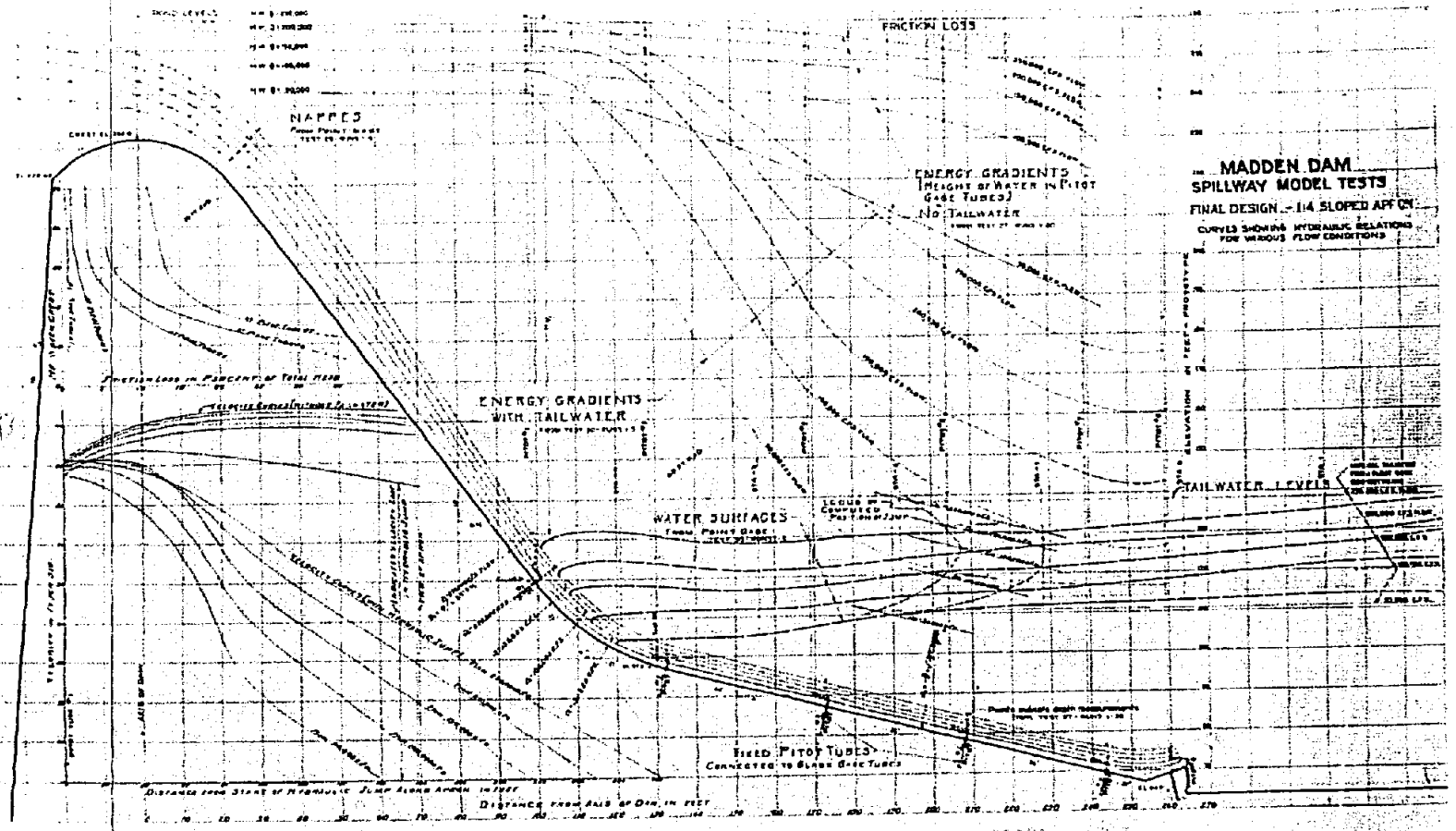
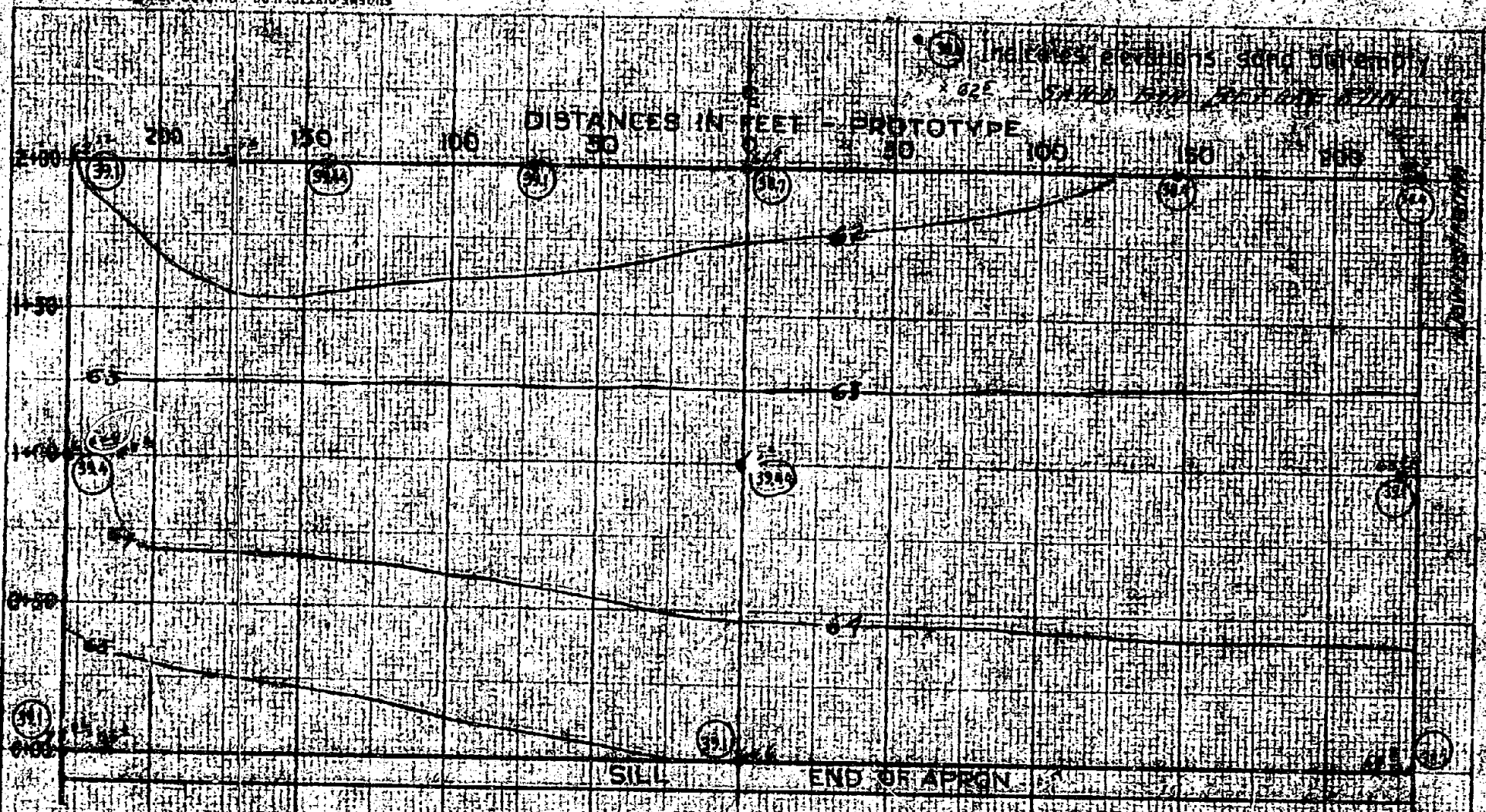
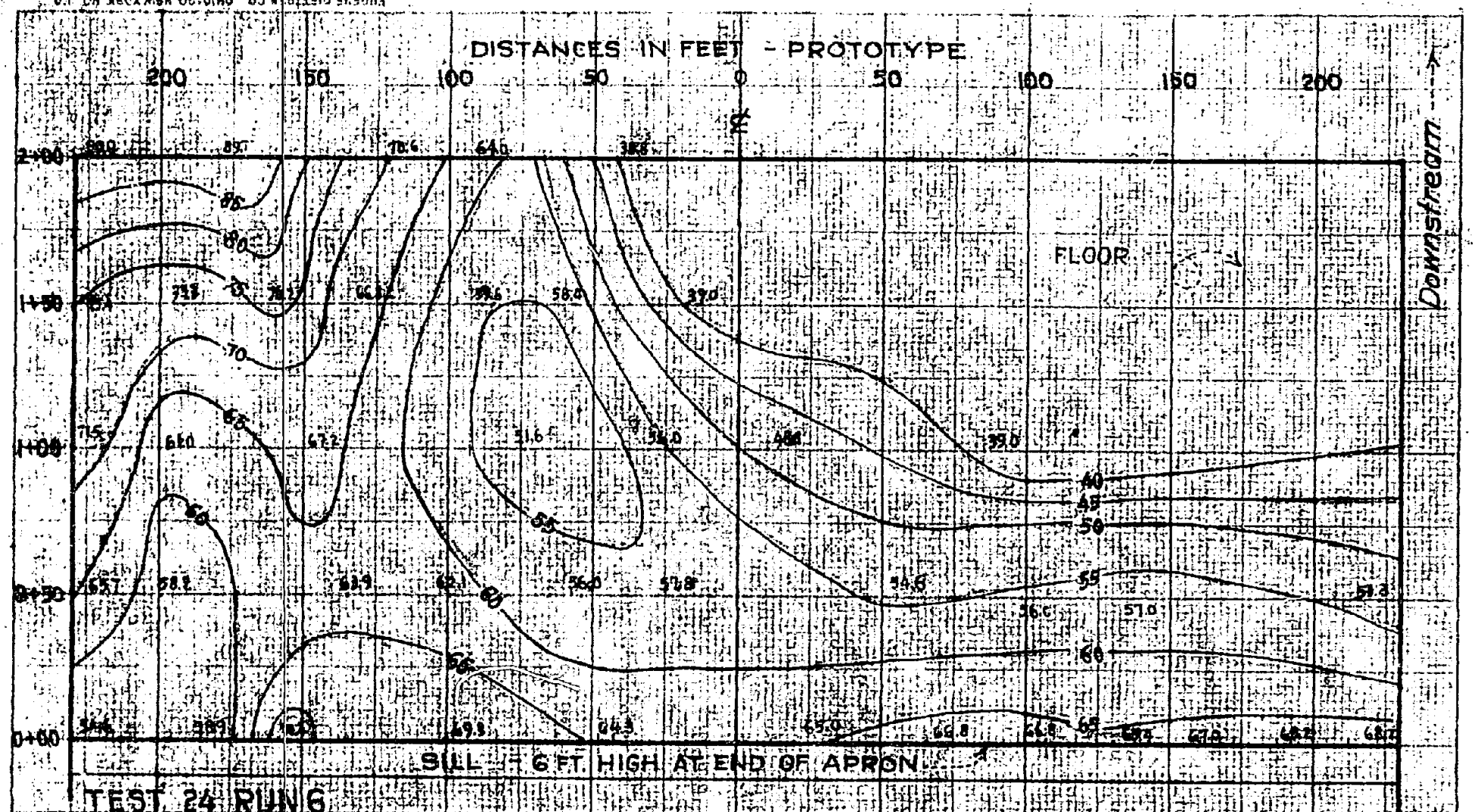

MADDEN DAM
SPILLWAY MODEL TESTS
FINAL DESIGN - 1:4 SLOPED APRON
CURVES SHOWING HYDRAULIC RELATIONS FOR VARIOUS FLOW CONDITIONS
FRICTION LOSS
NAPPES
ENERGY GRADIENTS
(HEIGHT OF WATER IN PITOT GAGE TUBES)
NO TAILWATER
ENERGY GRADIENTS
WITH TAILWATER
WATER SURFACES
TAILWATER LEVELS
FIXED PITOT TUBES
CONNECTED TO BLOCK GAGE TUBES
DISTANCE FROM AXIS OF DAM, IN FEET

PLATE 85



MADDEN DAM
SPILLWAY MODEL TESTS
2.25 x 9.2 STAGGERED STEPPED APRON
SAND BIN EMPTY
SAND CONTOURS BEFORE RUN
AS USED IN TEST 24

PLATE 25A



TEST 24 RUN 6

POND EL. 1263.40

GATES EL. 232.00

Q = 258,700 C.F.S.

T.W. EL. 199.47

58.2 indicates elevations taken with point gage after flow for a 15 min. period.

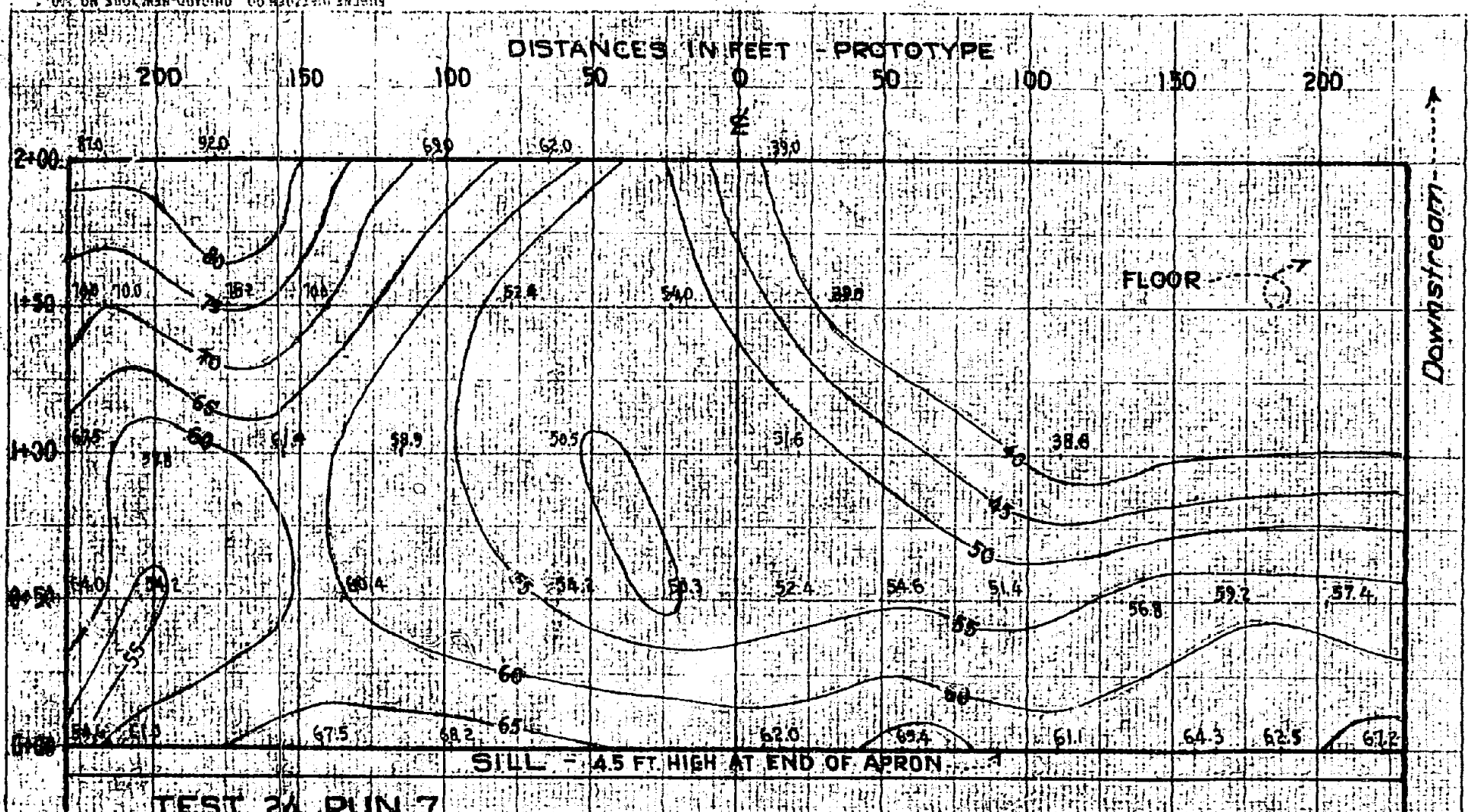
MADDEN DAM

SPILLWAY MODEL TESTS

2.25 x 9.2' STAGGERED STEPPED APRON

EROSION CONTOURS IN SAND BIN AFTER MAX FLOW. 6 FT. SLOPED FACE SILL

PLATE 86

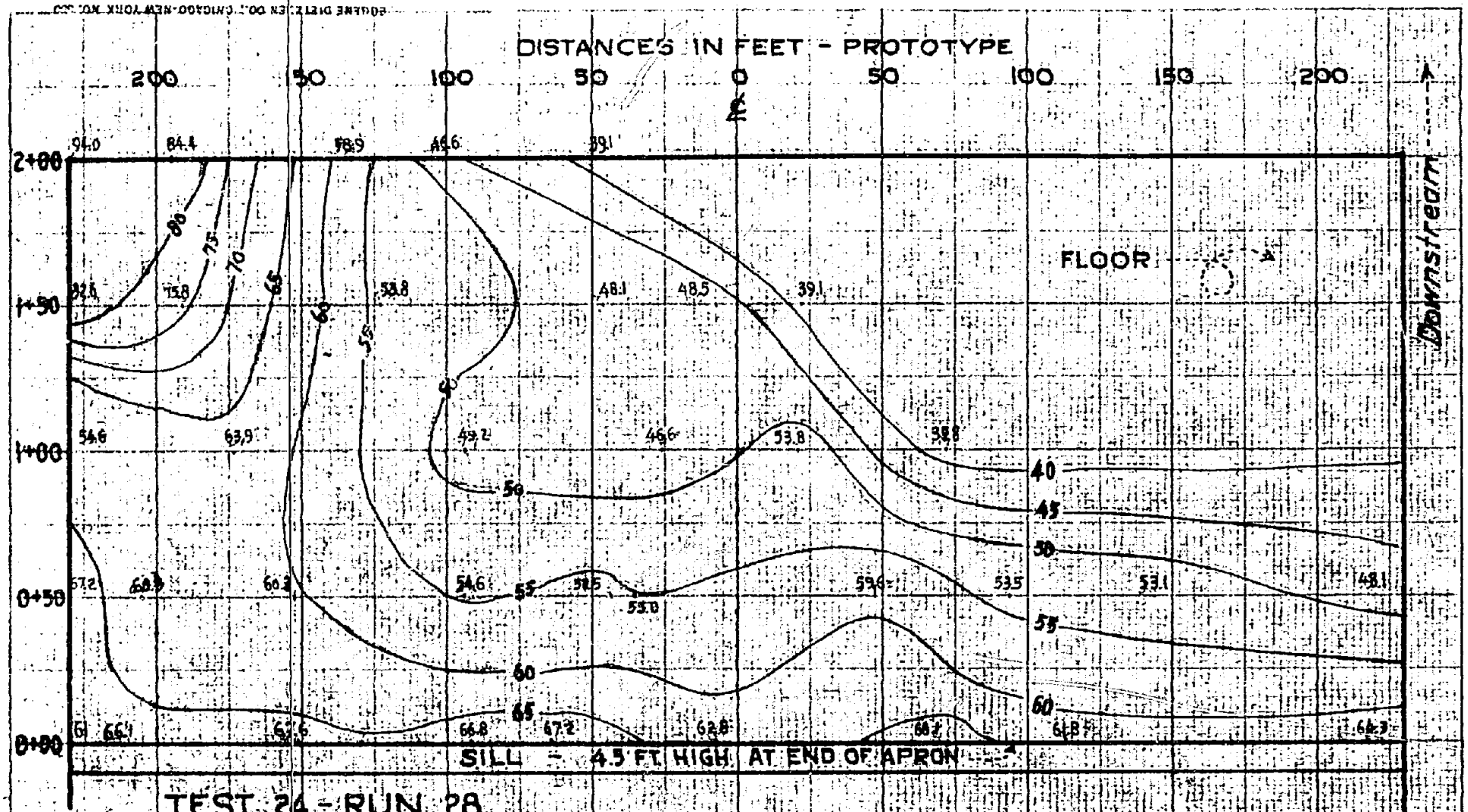


TEST 24 RUN 7
POND EL. 263.09
GATES EL. 232.0
Q = 257,400 C.F.S.
T.W. EL. 139.58

54.2 Indicates elevations taken with point gage after flow for a 15 min. period

MADDEN DAM
SPILLWAY MODEL TESTS
2.25' X 9.2' STAGGERED STEPPED APRON
4.5 FT. SLOPED FACE SILL
EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW

PLATE 87



TEST 24 - RUN 28

POND EL. 263.1
GATES EL. 232.0
Q = 257,620 C.F.S.
T.W. EL. 127.26 - 12 FT. BELOW NATURAL

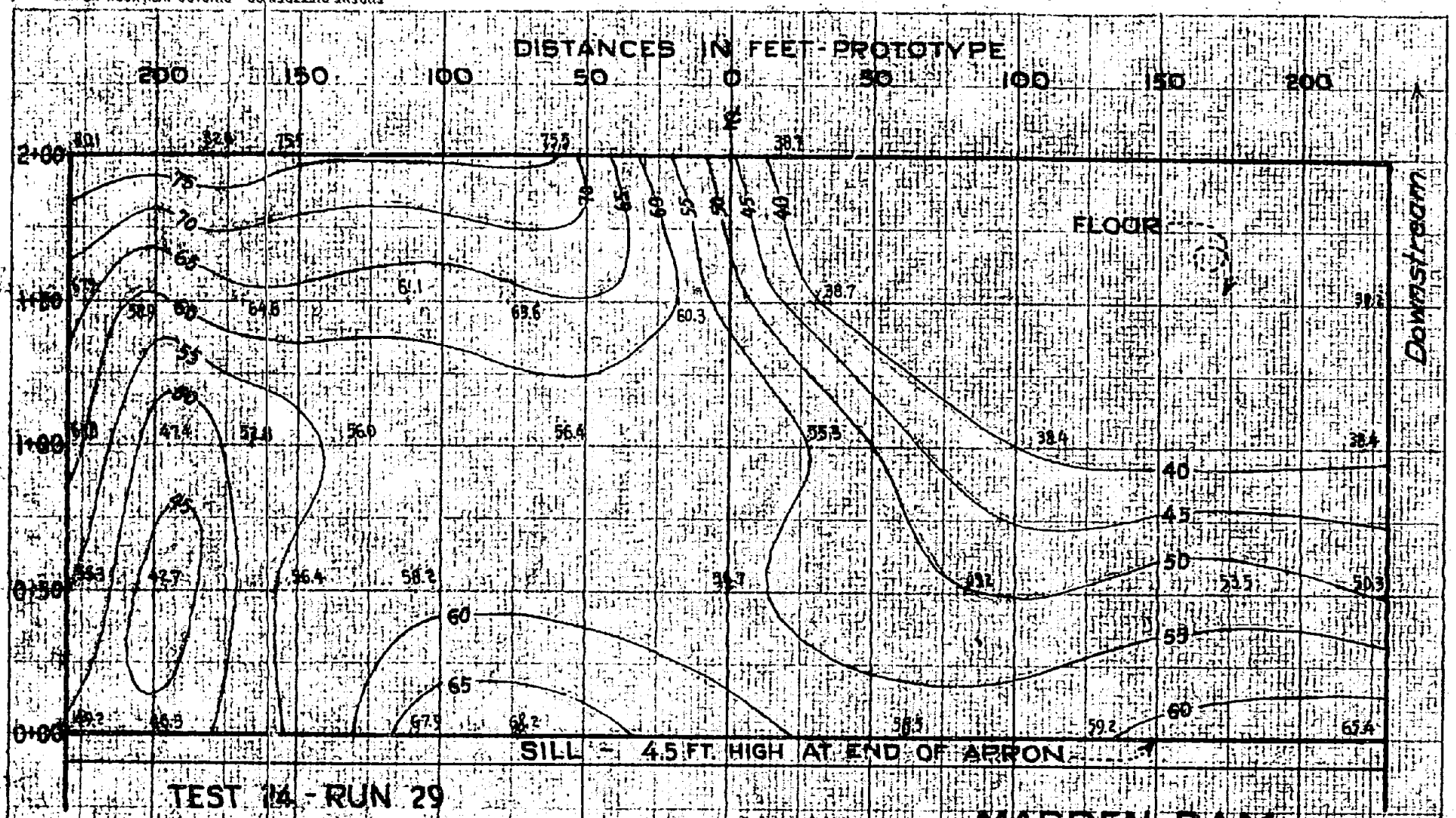
60.3 Indicates elevations taken with point gage after flow for a 15 min. period.

MADDEN DAM
SPILLWAY MODEL TESTS

2.25' x 9.2' STAGGERED STEPPED APRON
4.5 FT. SILL - T.W. 12 FT. BELOW NATURAL
EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW.

PLATE 88

EUGENE DIETZGEN CO., CHICAGO-NEW YORK NO. 350



TEST 14 - RUN 29

POND EL. 263.11
GATES EL. 232.0
Q = 258,260 CFS
T.W. EL. 151.56 - 12 FT. ABOVE NATURAL

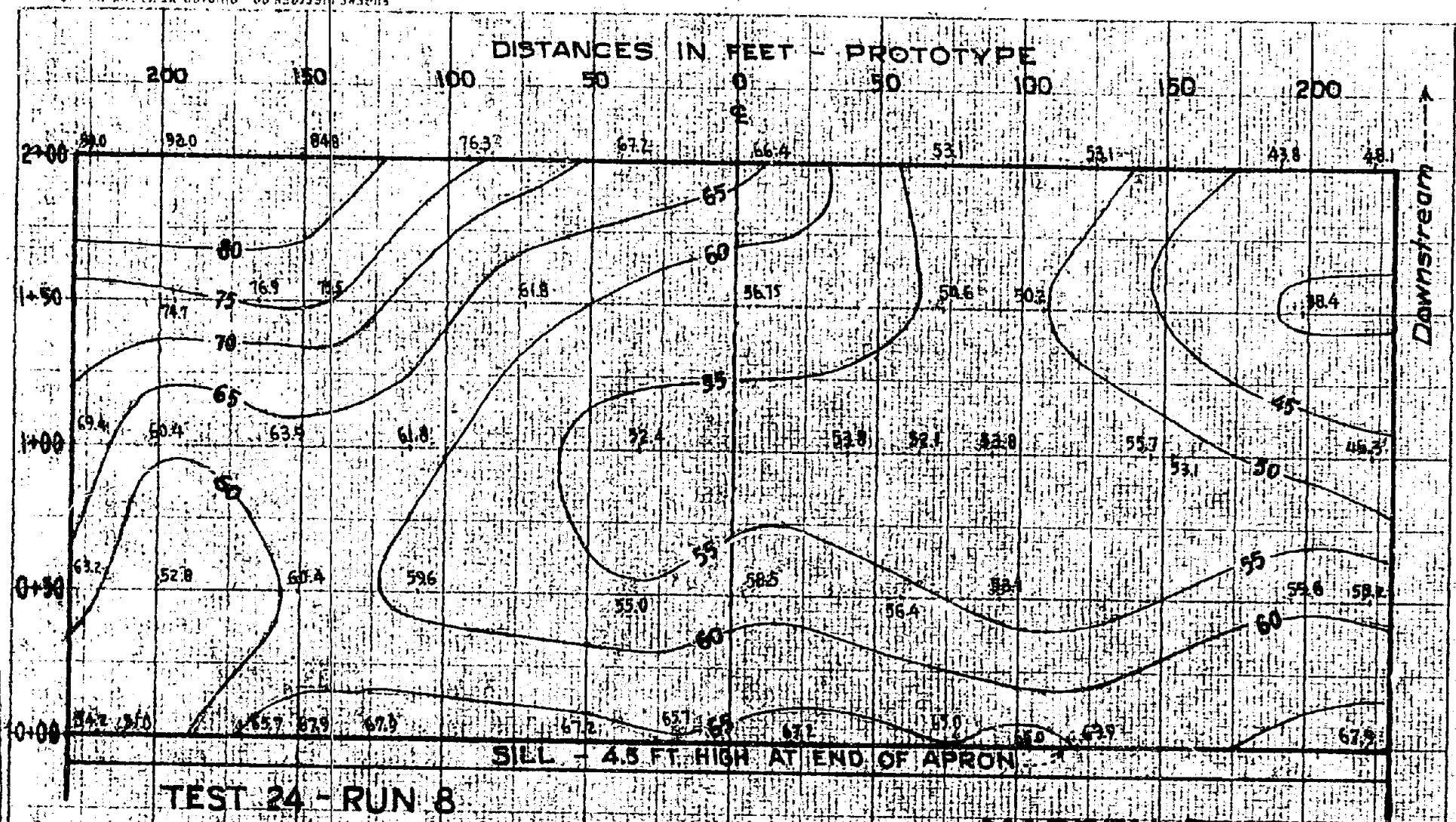
42.7 Indicates elevations taken with point gage after flow for a 15 min. period.

MADDEN DAM
SPILLWAY MODEL TESTS

2.25 x 9.2 STAGGERED STEPPED APRON
4.5 FT. SILL - T.W. 12 FT. ABOVE NATURAL
EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW.

PLATE 89

EUGENE DIETZGEN CO., CHICAGO-NEW YORK No. 359



TEST 24 - RUN 8

POND EL. 262.86
GATES EL. 236.0
Q = 209,000 C.F.S.
T. W. EL. 195.48

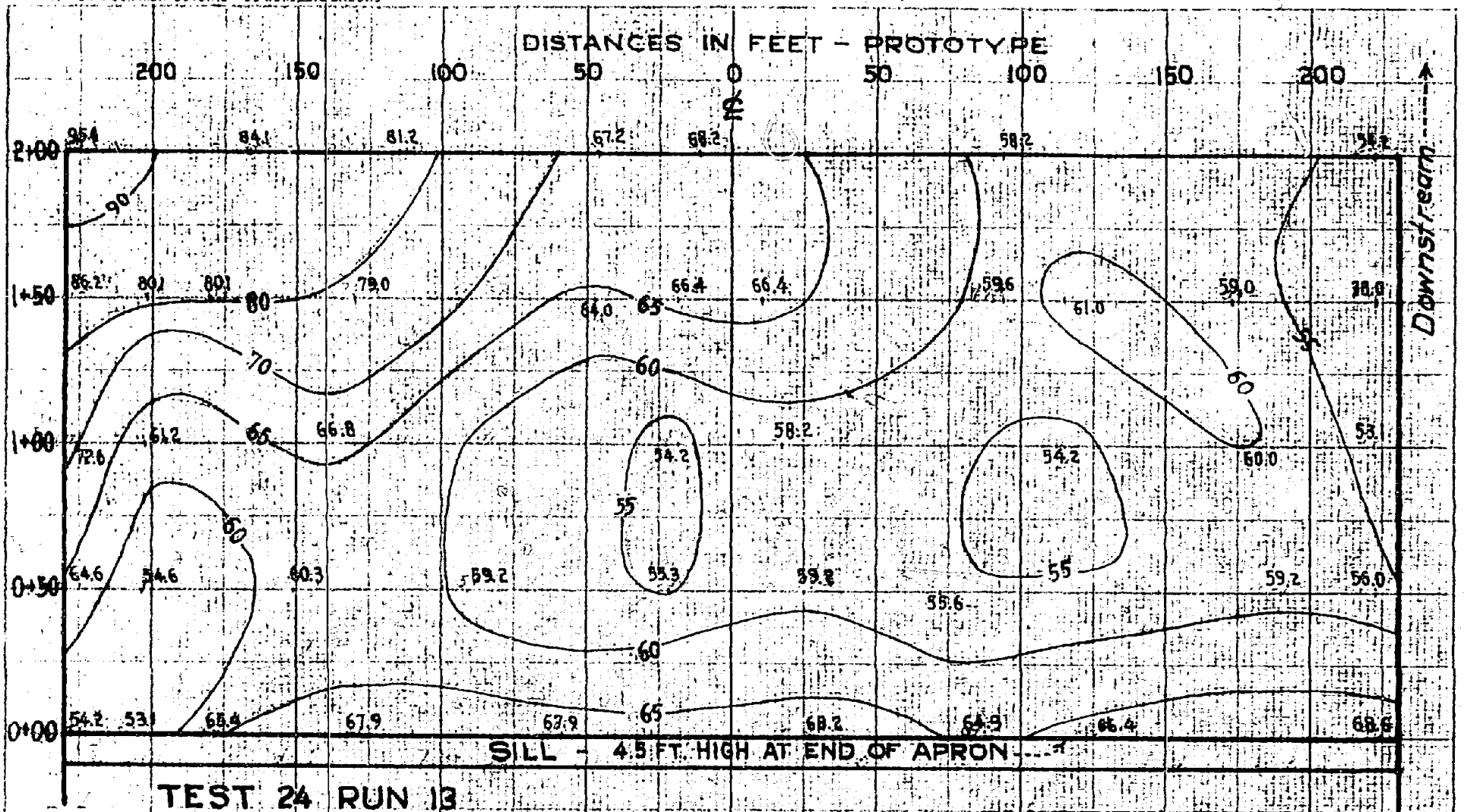
52.8 Indicates elevations taken with point gage after flow for a 15 min. period.

MADDEN DAM
SPILLWAY MODEL TESTS

2.25 x 9.2 STAGGERED STEPPED APRON
4.5 FT. SILL

EROSION CONTOURS IN SAND BIN AFTER FLOW OF 211,000 C.F.S.

PLATE 90



TEST 24 RUN 13

POND EL. 263.44
GATES EL. 240.0
Q = 176,880 C.F.S.
T.W. EL. 130.25

54.6 Indicates elevations taken with point gage after flow for a 15 min. period.

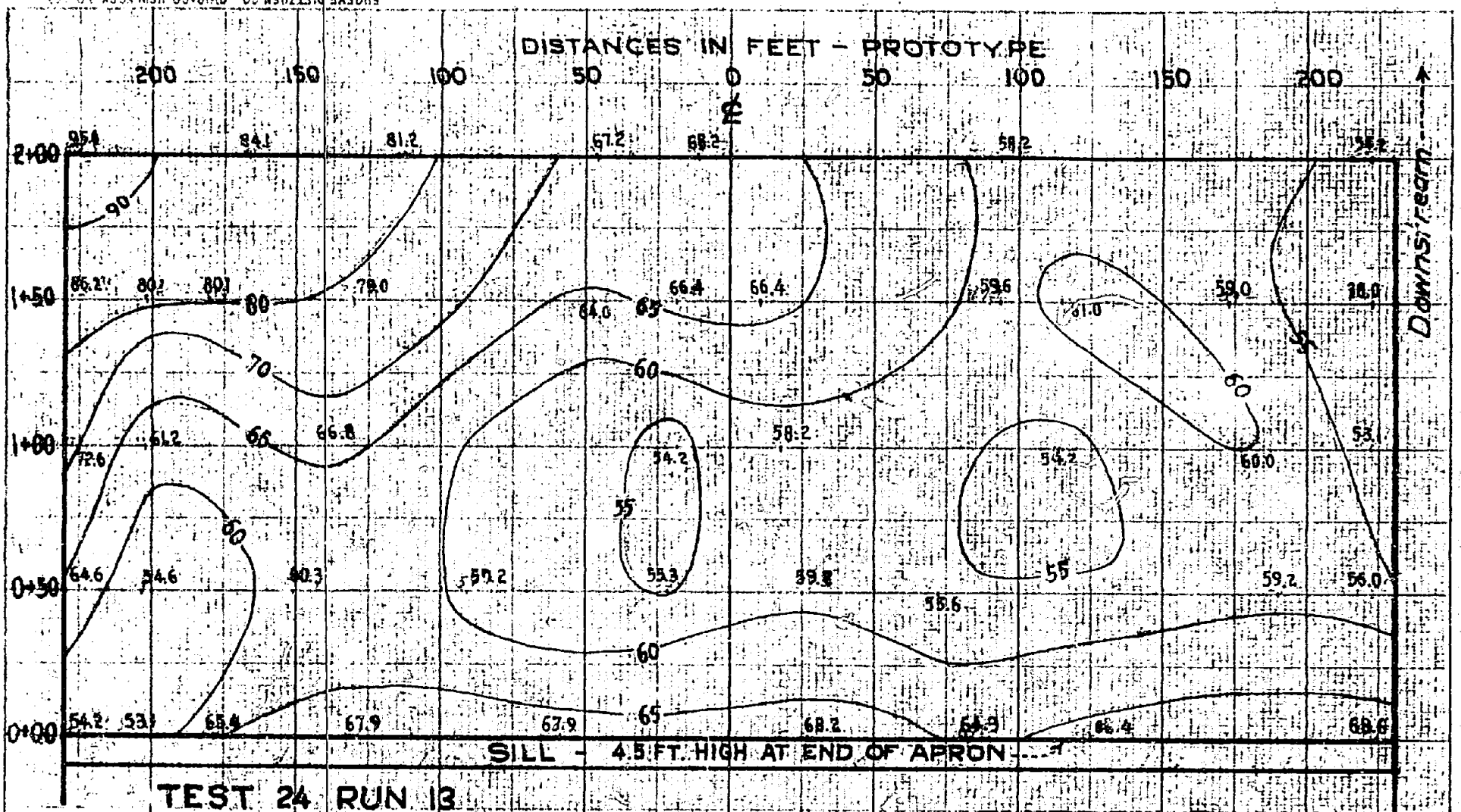
MADDEN DAM
SPILLWAY MODEL TESTS

2.25' x 9.2' STAGGERED STEPPED APRON
4.5' SILL

EROSION CONTOURS IN SAND BIN AFTER FLOW OF 172,000 C.F.S.

PLATE 91

188



TEST 24 RUN 13

POND EL. 263.44
GATES EL. 240.0
Q = 176,880 C.F.S.
T.W. EL. 130.25

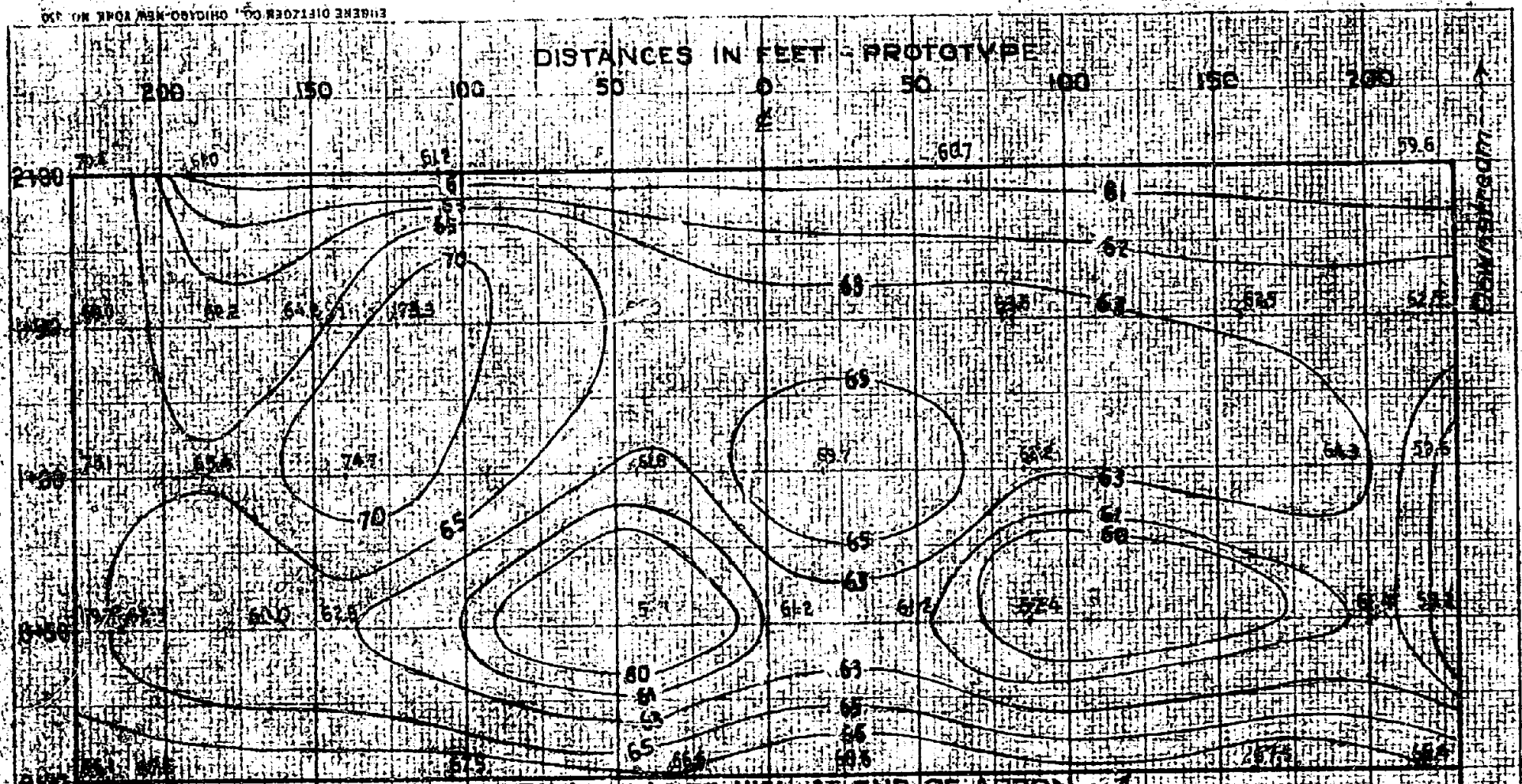
54.6, Indicates elevations taken with point gage after flow for a 15min. period.

MADDEN DAM
SPILLWAY MODEL TESTS

2.25' x 9.2' STAGGERED STEPPED APRON
4.5' SILL
EROSION CONTOURS IN SAND BIN
AFTER FLOW OF 172,000 C.F.S.

PLATE 91

188

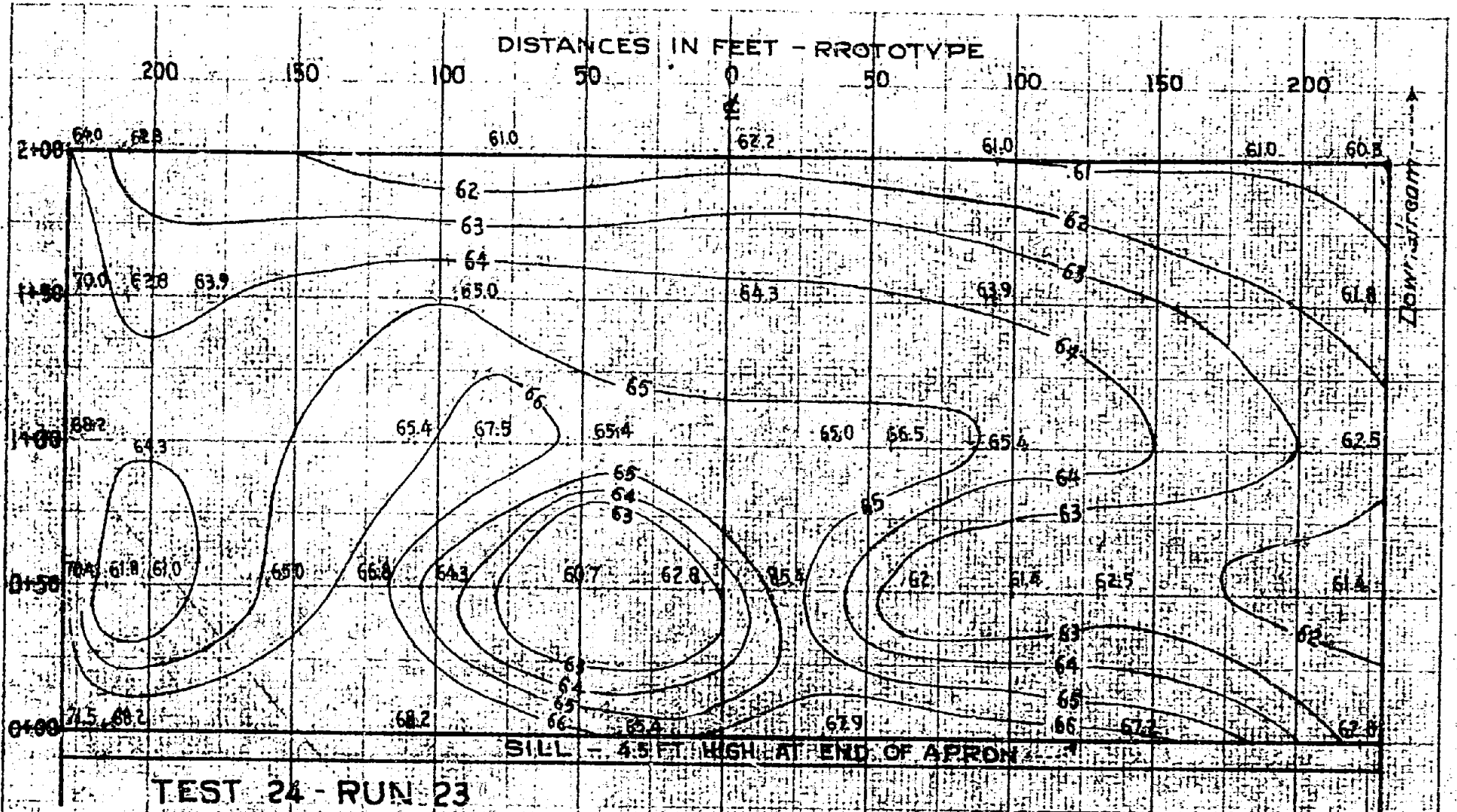


TEST 24 - RUN 18

POND EL. 262.99
GATES EL. 245.0
Q = 120,360 CFS
T.W. EL. 122.23

62.5 indicates elevations taken with point gage after flow for a 15 min. period

MADDEN DAM
SPILLWAY MODEL TESTS
2.25 & 9.2 STAGGERED STEPPED APRON
4.5 FT SILL
EROSION CONTOURS IN SAND BIN
AFTER FLOW OF 119,000 CFS



TEST 24 - RUN 23
POND EL. 263.18
GATES EL. 250.0
Q = 74,360 C.F.S.
T.W. EL. 114.99

61.0 indicates elevations taken with point gage after flow for 15 min. period

MADDEN DAM
SPILLWAY MODEL TESTS
2.25 x 9.2 STAGGERED STEPPED APRON
4.5 FT. SILL
EROSION CONTOURS IN SAND BIN
AFTER FLOW OF 70,000 C.F.S.

PLATE 93

190

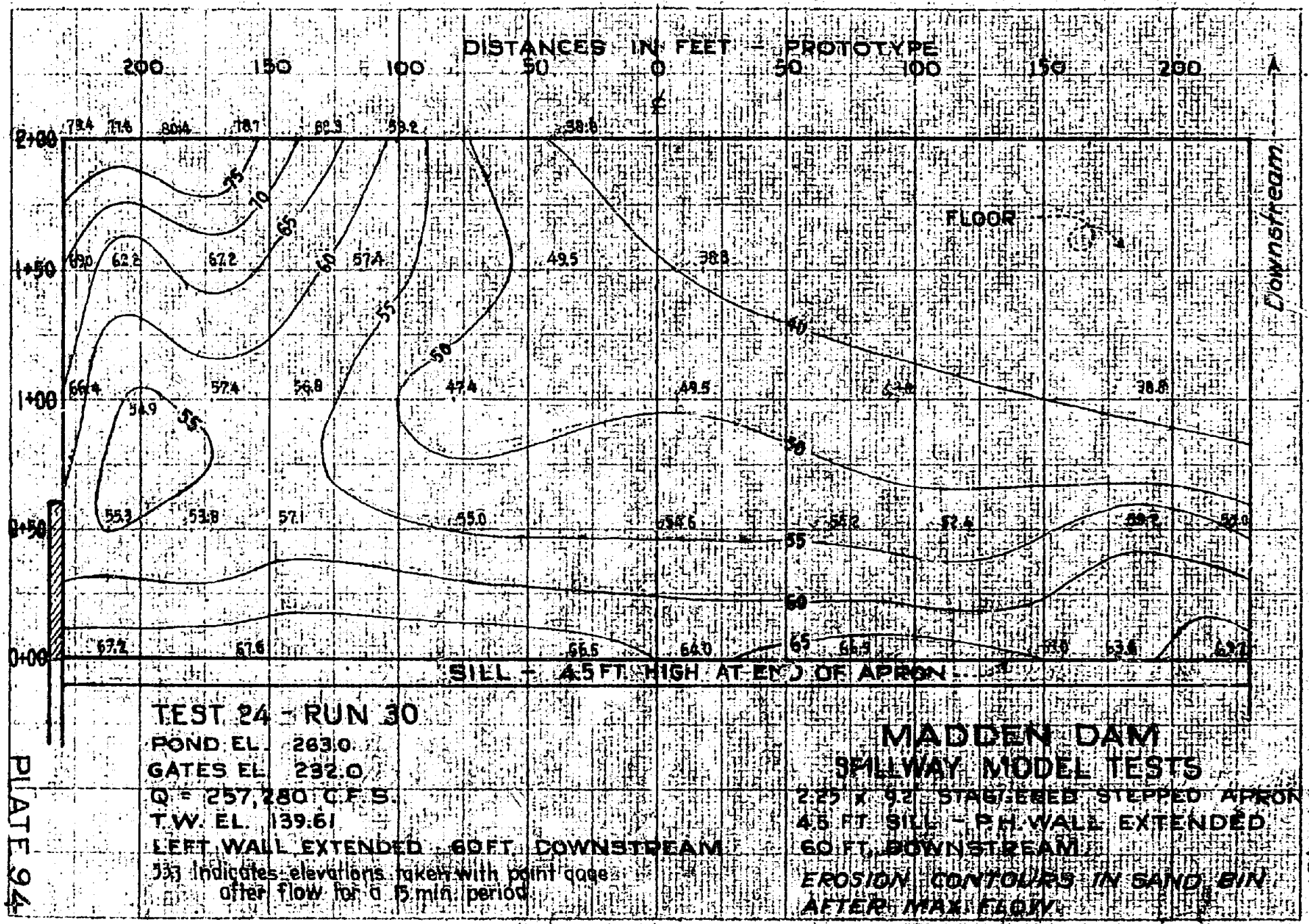
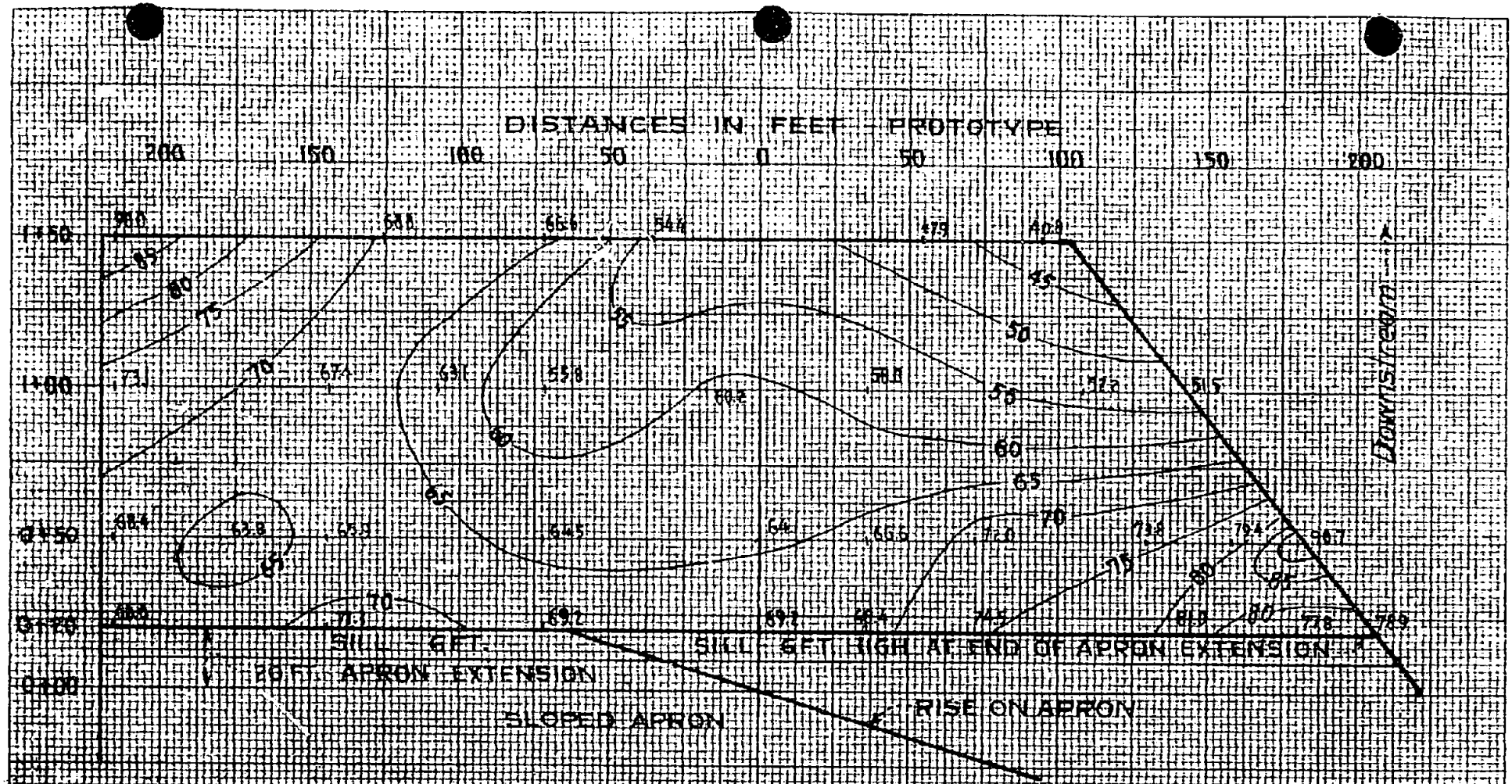


PLATE 94

EUGENE DIETZGEN CO., CHICAGO-NEW YORK NO. 130



TEST 18 RUN 13

POND EL. 262.89
GATES EL. 262.0
Q = 260,100 C.F.S.
TW EL. 240.40

El. includes elevation used with point gage after flow for a 15min. period.

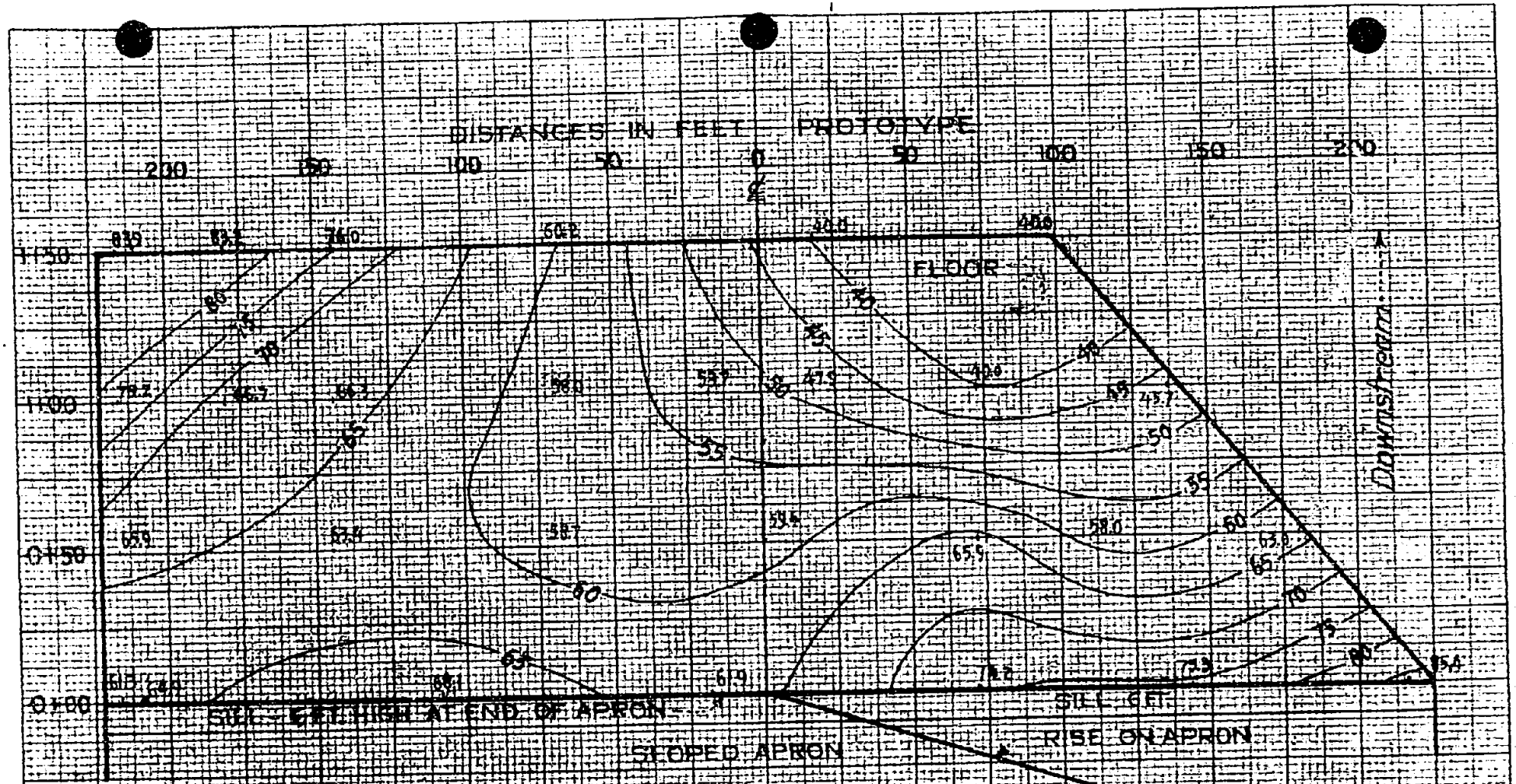
MADDEN DAM

SPILLWAY MODEL TESTS

1:4 SLOPED APRON WITH RISE ON RIGHT SIDE
APRON EXTENDED 20 FT. 6FT. SILL AT END

EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW

PLATE 95



MADDEN DAM
SPILLWAY MODEL TESTS

1:4 SLOPED APRON WITH RISE ON RIGHT SIDE
6 FT SILL AT END

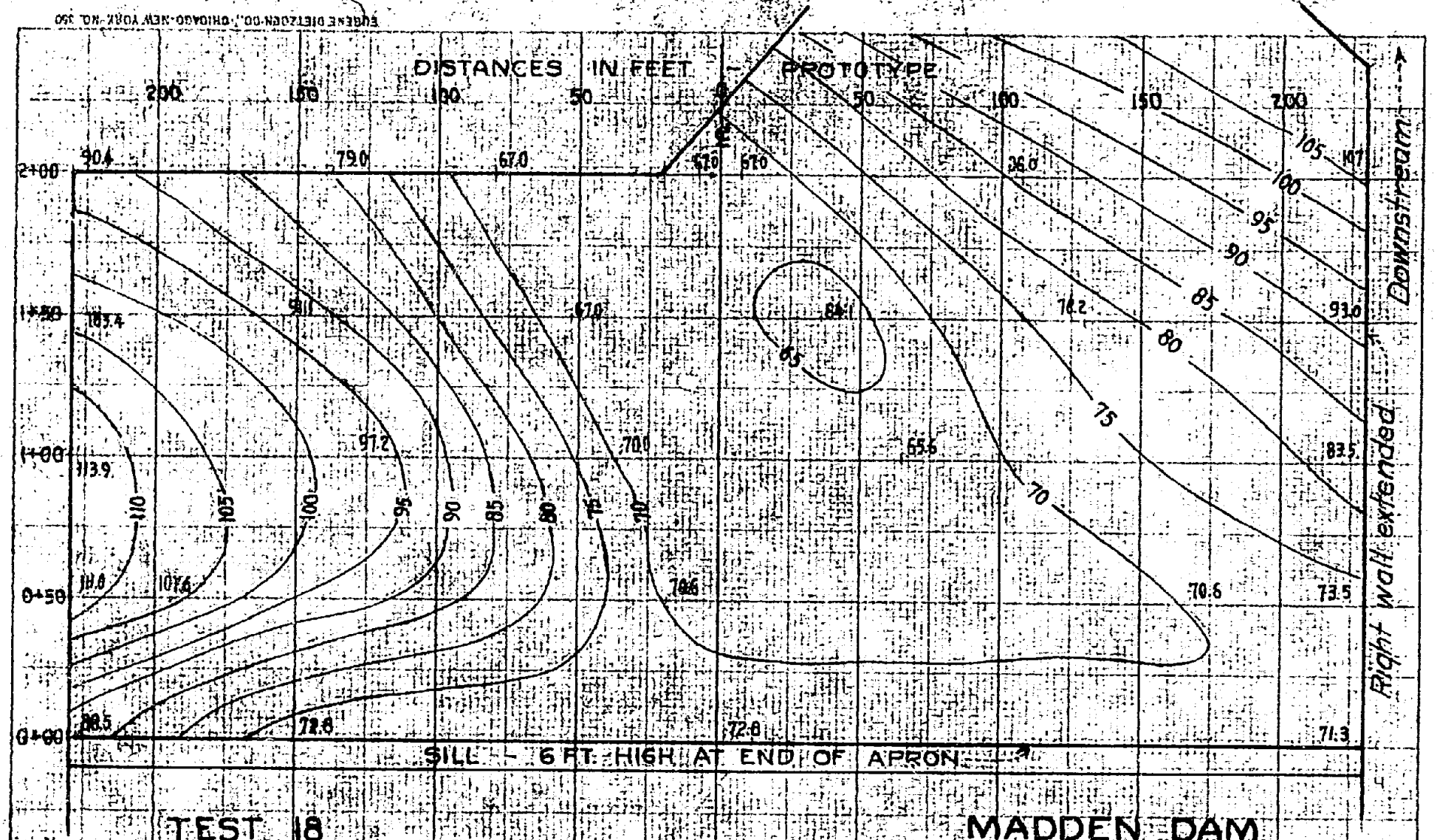
EROSION CONTOURS IN SAND BIN
AFTER MAX. FLOW

TEST 18 RUN 19

POND EL 263.36
GATES EL 232.0
Q = 463,300 C.F.S.
T.W. EL 141.68

62.4 indicates elevation taken with point gage after flow for a 15 min. period

PLATE 96



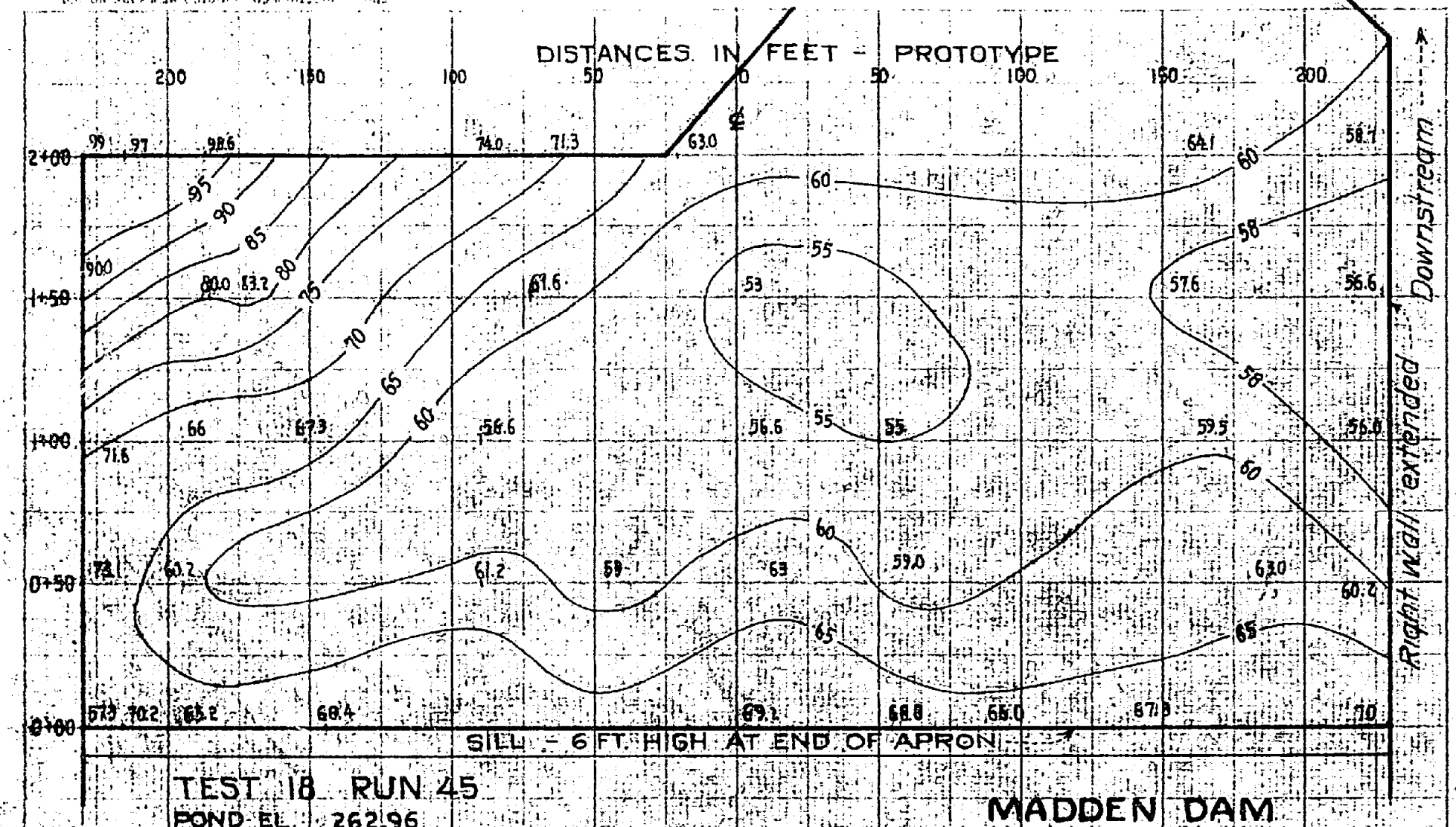
TEST 18
RUN 45

107.6 Indicates elevations taken with point gage before run

MADDEN DAM
SPILLWAY MODEL TESTS

1:4 SLOPED APRON - CONTINUOUS
6 FT. SLOPED SILL
CONTOUR BLOCK BELOW BIN REMOVED
CONTOURS IN SAND BIN BEFORE RUN

PLATE 97



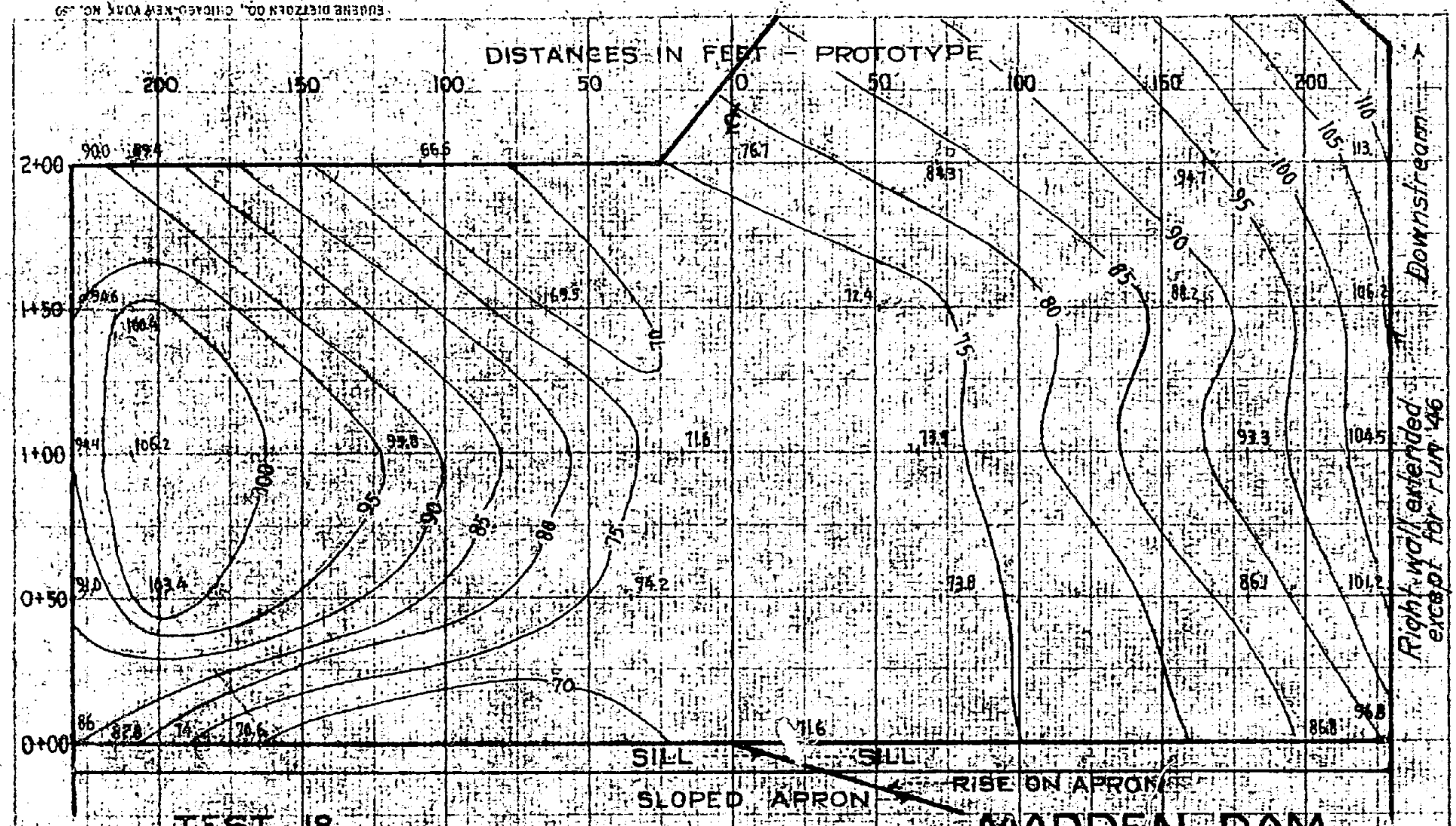
TEST 18 RUN 45
POND EL. 262.96
GATES EL. 232.0
Q = 261,180 C.F.S.
T.W. EL. 141.77

60.2 Indicates elevations taken with point gage after flow for a 15 min. period.

MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRON - CONTINUOUS
6 FT. SLOPED SILL
CONTOUR BLOCK BELOW BIN REMOVED

EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW.

PLATE 98



MADDEN DAM
SPILLWAY MODEL TESTS

1:4 SLOPED APRON WITH RISE ON RT. SIDE
CONTOUR BLOCK BELOW BIN REMOVED

CONTOURS IN SAND BIN BEFORE RUN

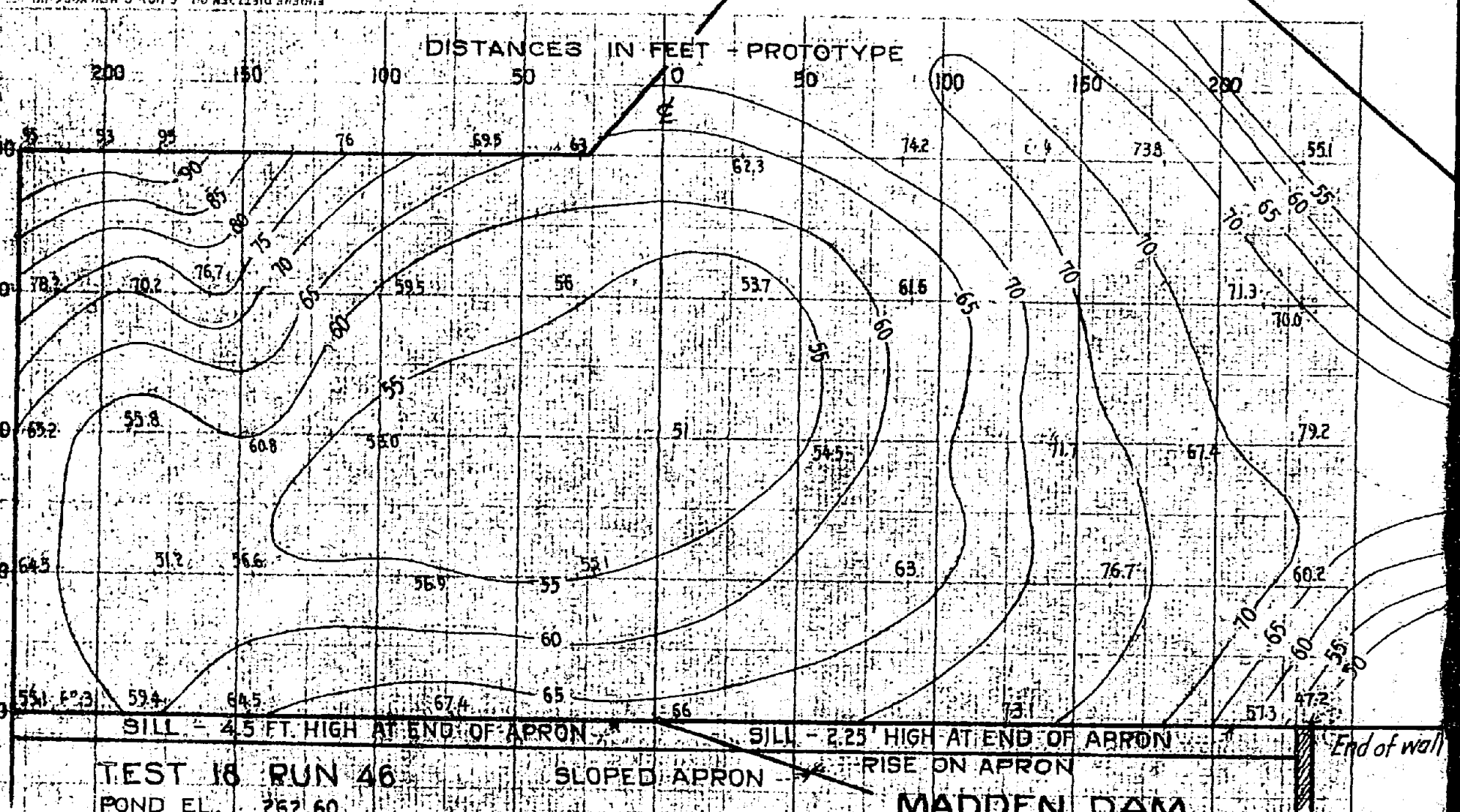
TEST 18
RUNS 46-47-48

163.4 Indicates elevations taken with point gage before run

PLATE 99

196

EUGENE DIETZGEN CO., CHICAGO-NEW YORK NO. 340



TEST 18 RUN 46

POND EL. 262.60

GATES EL. 232.0

Q = 257,900 C.F.S.

T.W. EL. 130.42

51.2 Indicates elevations taken with point gage after flow for a 15 min period.

MADDEN DAM
SPILLWAY MODEL TESTS

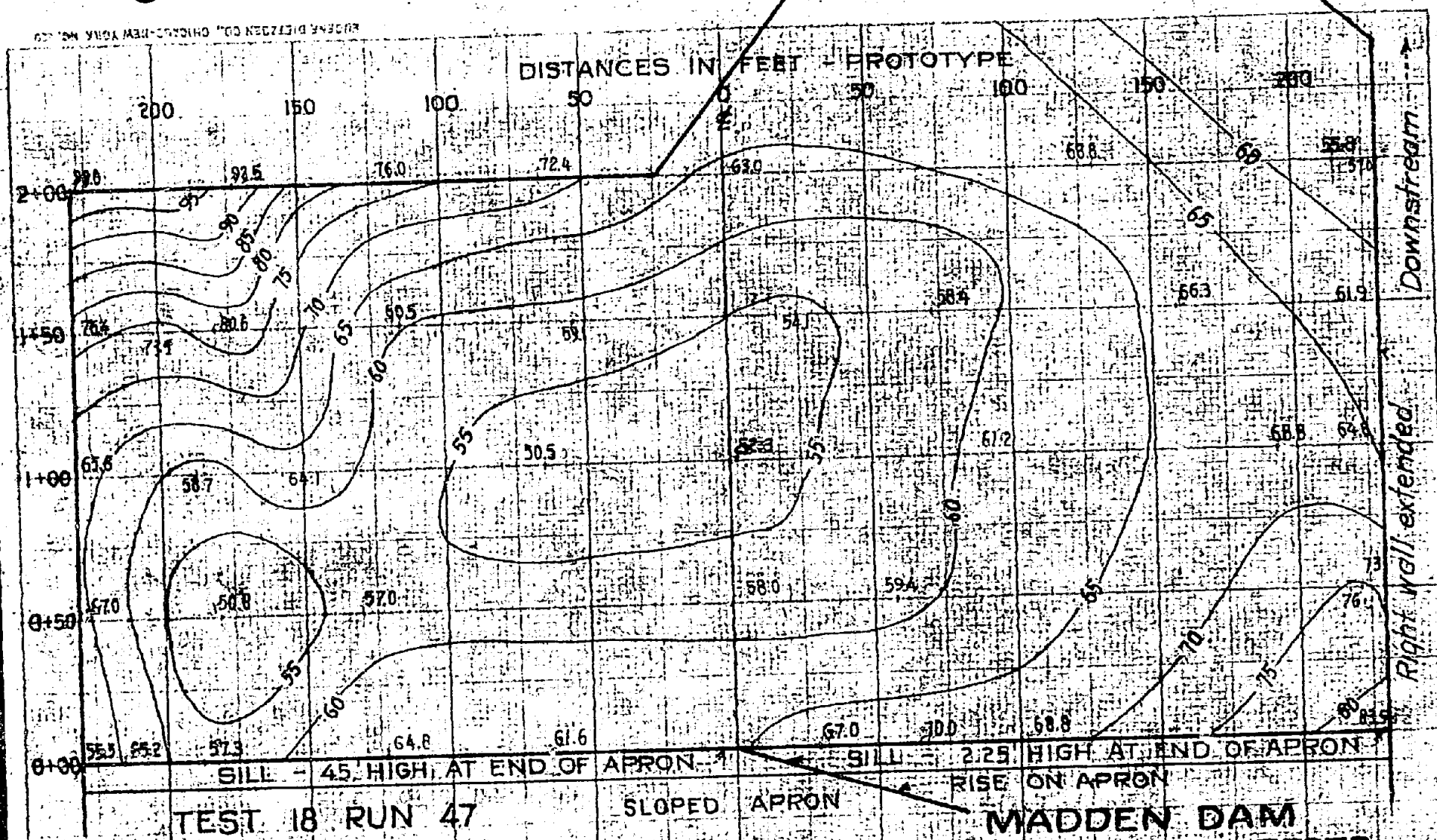
1:4 SLOPED APRON WITH RISE ON RT. SIDE

4.5 SILL ON SLOPE - 2.25 SILL ON RISE

CONTOUR BLOCK BELOW BIN REMOVED

EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW

PLATE 100



TEST 18 RUN 47

POND EL 262.74
GATES EL. 232.0
Q = 259,424 C.F.S.
T.W. EL. 141.79

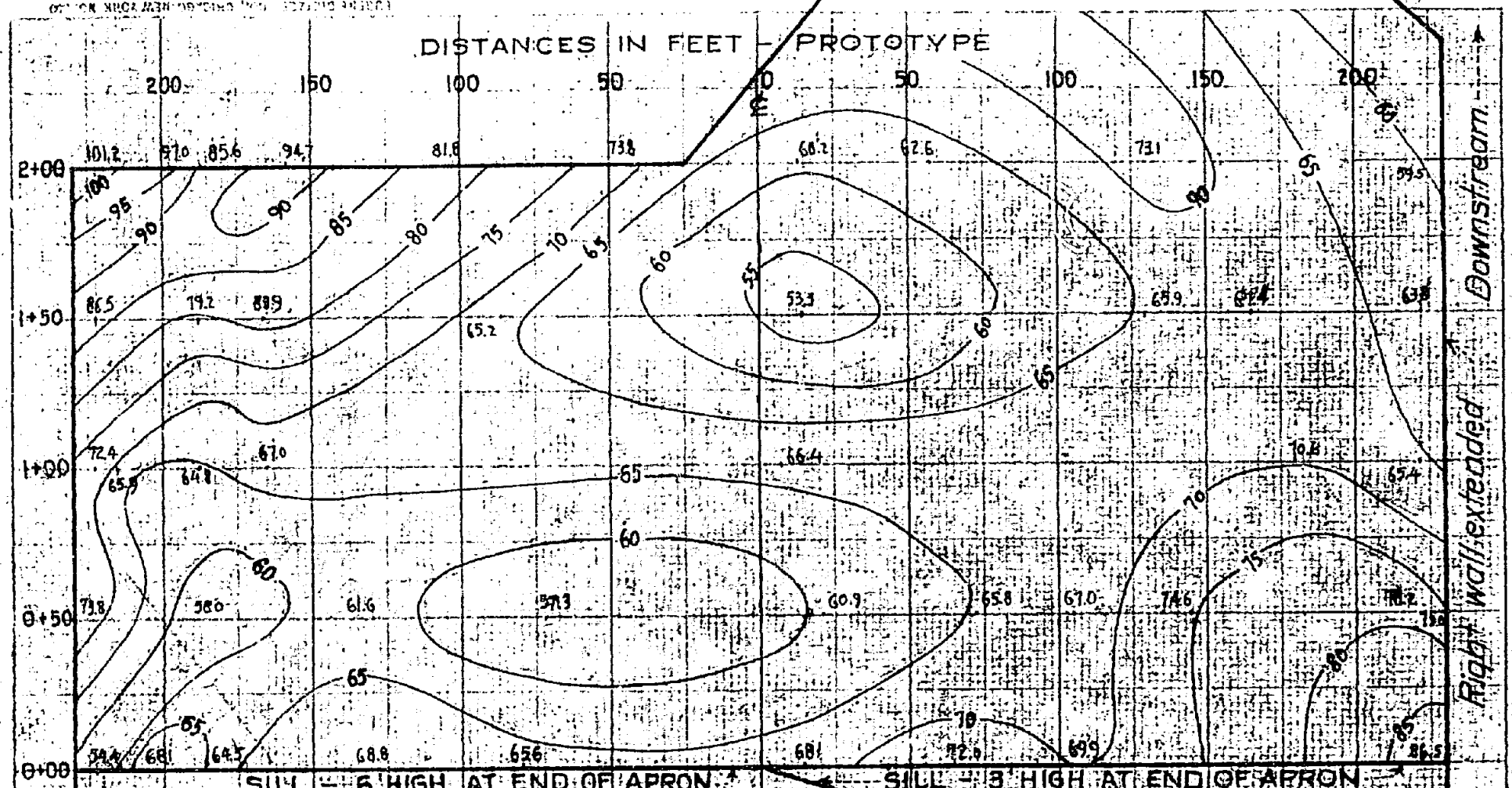
50.8 Indicates elevations taken with point gage after flow for a 15 min. period.

MADDEN DAM
SPILLWAY MODEL TESTS

1:4 SLOPED APRON WITH RISE ON RT. SIDE
4.5' SILL ON SLOPE - 2.25' SILL ON RISE
CONTOUR BLOCK BELOW BIN REMOVED
EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW

PLATE 101

198



TEST 18 RUN 48

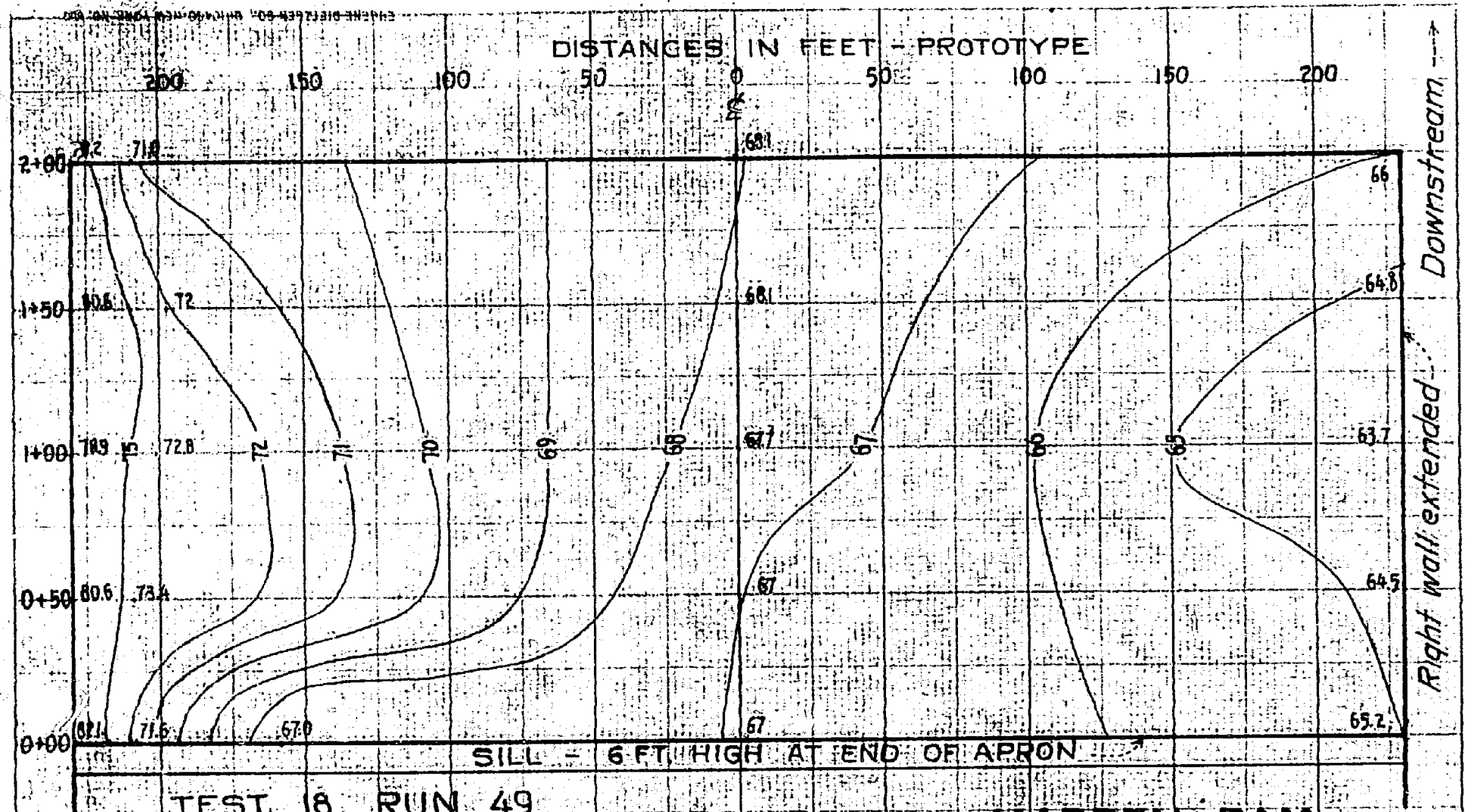
POND EL. 262.8
GATES EL. 232.0
Q = 254,800 C.F.S.
T.W. EL. 141.5

58.0 Indicates elevations taken with point gage after flow for a 15 min. period.

MADDEN DAM
SPILLWAY MODEL TESTS

174 SLOPED APRON WITH RISE ON RT. SIDE
6' SILL ON SLOPE - 3' SILL ON RISE
CONTOUR BLOCK BELOW BIN REMOVED
EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW)

PLATE 102



TEST 18 RUN 49

73.4 Indicates elevations taken with point gage before run.

MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRON - CONTINUOUS
6 FT. SLOPED SILL

CONTOURS IN SAND BIN BEFORE RUN

PLATE 103

200

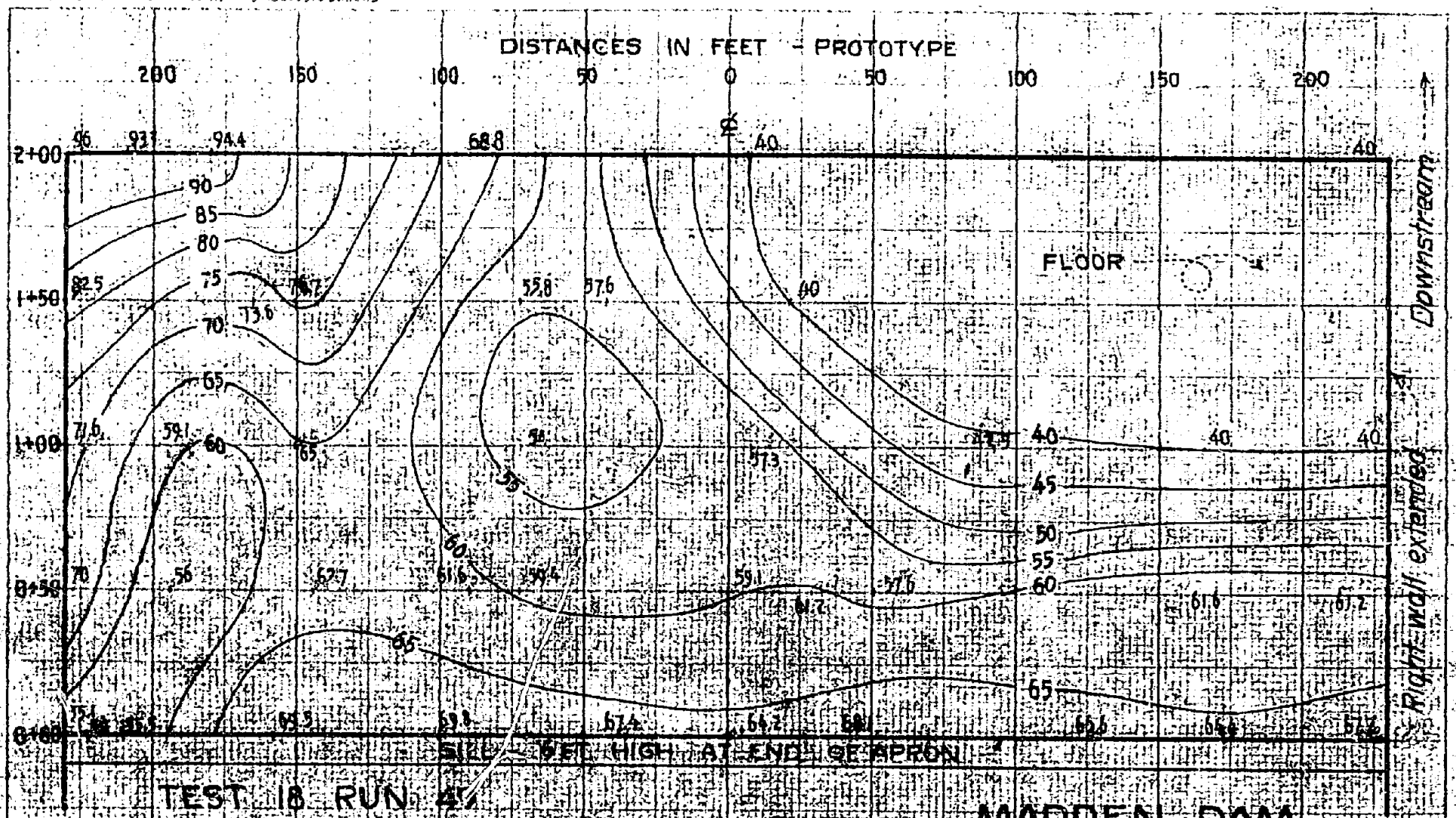

DISTANCES IN FEET - PROTOTYPE
200
150
100
50
0
50
100
150
200
2+00
1+50
1+00
0+50
0+00
℄
FLOOR
Downstream
Right wall extended
SILL - 6 FT. HIGH AT END OF APRON
TEST 18 RUN 47
POND EL. 282.74
GATES EL. 282.0
Q = 258,800 C.F.S.
T.W. EL. 141.91
67.7 indicates elevations taken with point gage after flow for a 15 min. period.
MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRON - CONTINUOUS
6 FT. SLOPED SILL
EROSION CONTOURS IN SAND BED
AFTER MAX. FLOW.

PLATE 104

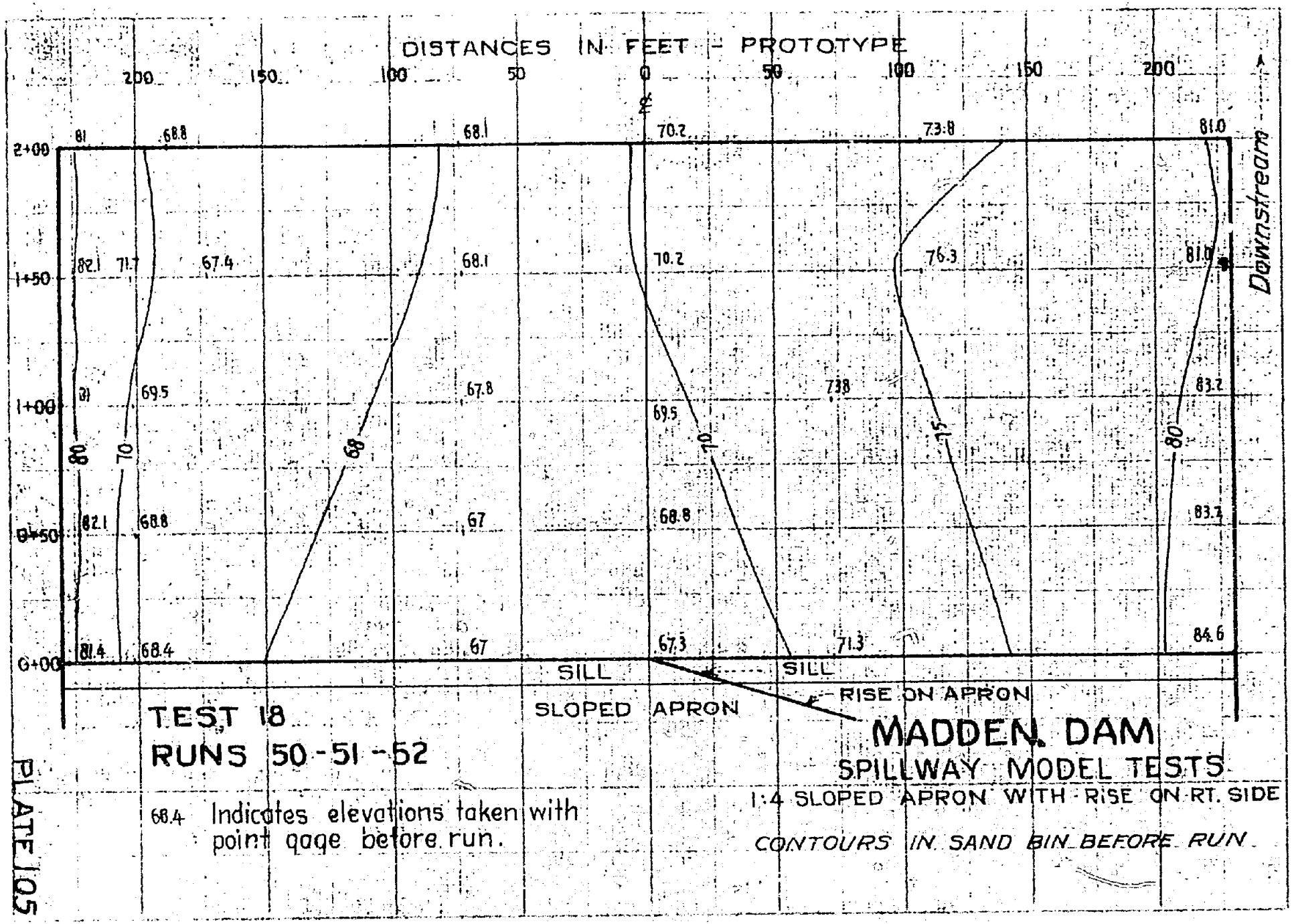
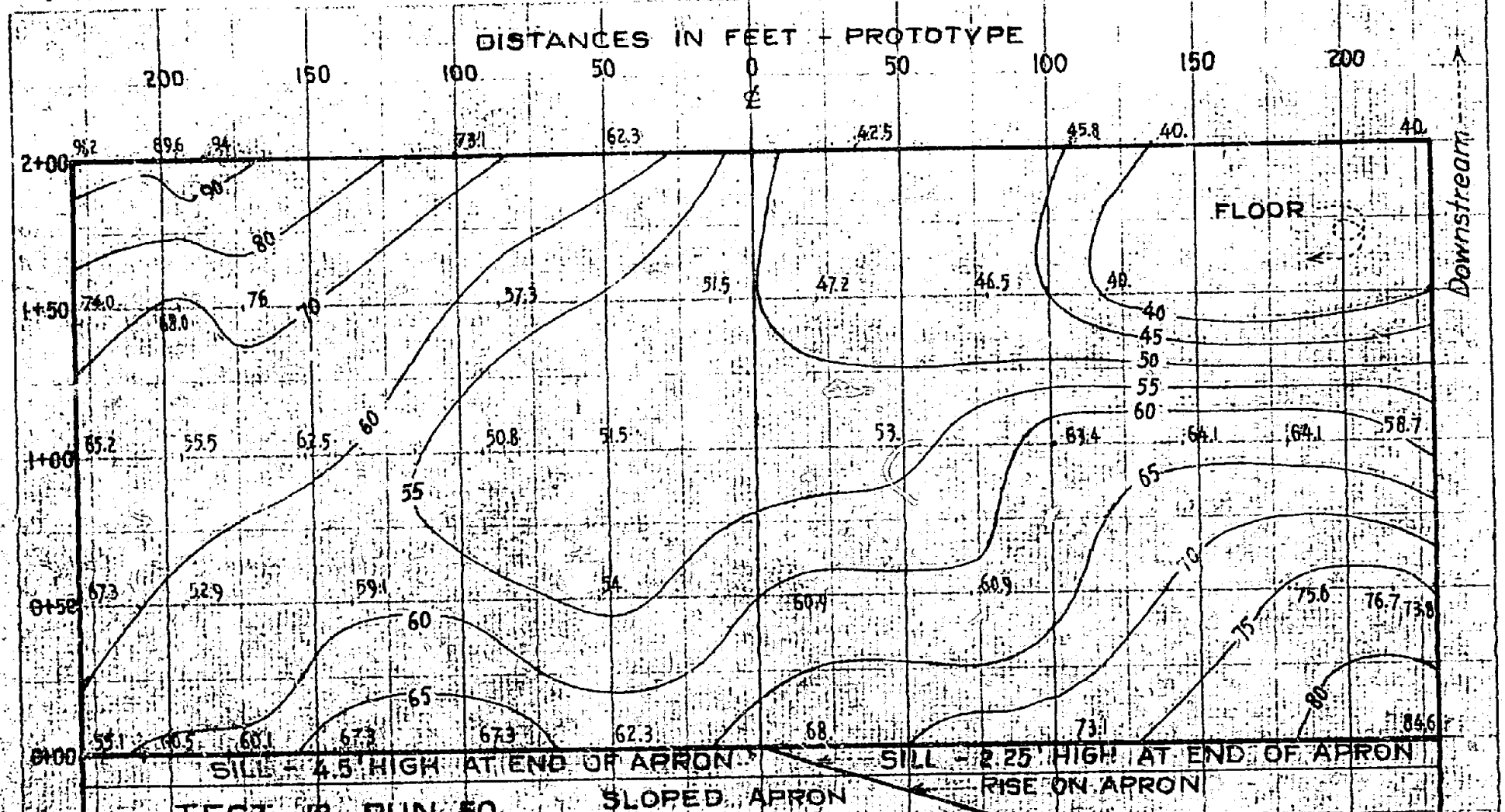

DISTANCES IN FEET - PROTOTYPE
200
150
100
50
0
50
100
150
200
℄
Downstream
2+00
1+50
1+00
0+50
0+00
81
68.8
68.1
70.2
73.8
81.0
82.1
71.7
67.4
68.1
70.2
76.3
81.0
81
69.5
67.8
73.8
83.2
69.5
82.1
68.8
67
68.8
83.2
81.4
68.4
67
67.3
71.3
84.6
80
70
68
70
75
80
SILL
SILL
SLOPED APRON
RISE ON APRON
TEST 18
RUNS 50-51-52
68.4 Indicates elevations taken with point gage before run.
MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRON WITH RISE ON RT. SIDE
CONTOURS IN SAND BIN BEFORE RUN

PLATE 105



TEST 18 RUN 50
POND EL. 262.82
GATES EL. 232.0
Q = 259,690 C.F.S.
T.W. EL. 141.92

52.9 Indicates elevations taken with point gage after flow for a 15 min. period

MADDEN DAM
SPILLWAY MODEL TESTS

1:4 SLOPED APRON WITH RISE ON RT. SIDE
4.5' SILL ON SLOPE - 2.25' SILL ON RISE

EROSION CONTOURS IN SAND BIN AFTER MAX. FLOW

PLATE 10

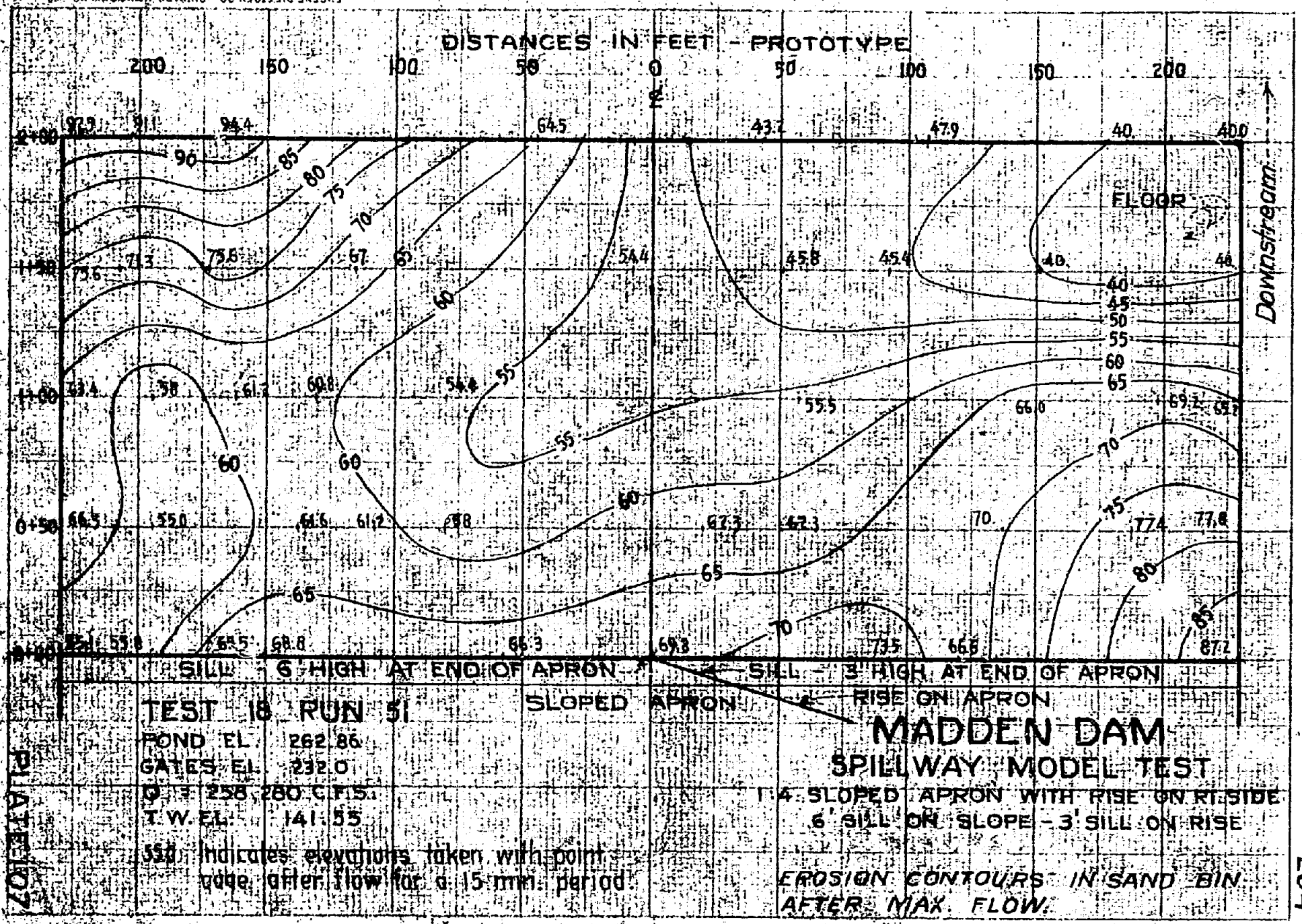
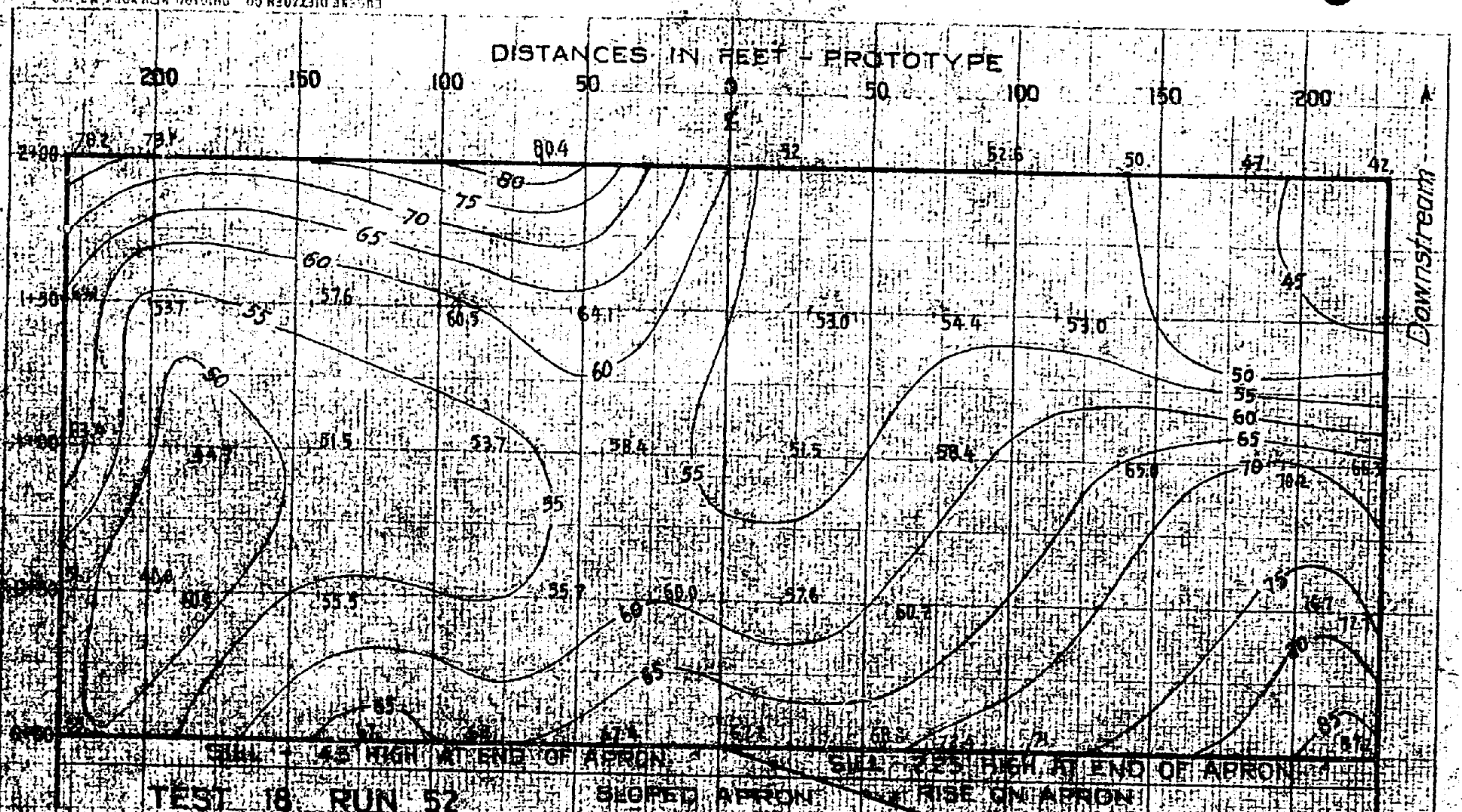


PLATE 107

204



TEST 18 RUN 52
POND EL 252.96
GATES EL 232.0
Q = 250,480 C.F.S.
T.W. EL 142.04
44.7 indicates elevations taken with point gage
Q was flow for a 12 min. period

MADDEN DAM
SPILLWAY MODEL TESTS
1:4 SLOPED APRON WITH RISE ON RI SIDE
4.5' SILL ON SLOPE 2.25' SILL ON RISE
EROSION CONTOURS IN SAND BED
AFTER MAX. FLOW