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HYDRAULIC LABORATORY REPORT NO. HYD-2.3

Technical Memorandum No. 347

HYDRAULIC MODEL EXPERIMENTS FOR THE DESIGN OF THE BOULDER DAM
BOOK 3
QUANTITATIVE RESULTS ON THE FLOW IN THE SPILLWAY CHANNELS AND SPILLWAY TUNNELS

By

E. W. LANE, RESEARCH ENGINEER

Denver, Colorado
Sept. 7, 1933

UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: HYDRAULIC MODEL EXPERIMENTS FOR -

THE DESIGN OF THE BOULDER DAM

BOOK 3

QUANTITATIVE RESULTS ON THE FLOW IN THE SPILLWAY CHANNELS AND SPILLWAY TUNNELS

By E. W. LANE, RESEARCH ENGINEER

Under direction of

J. L. SAVAGE, CHIEF DESIGNING ENGINEER

TECHNICAL MEMORANDUM No. 347

Denver, Colorado

Sept. 7, 1933

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Denver, Colorado, Sept. 7, 1933.

MEMORANDUM TO CHIEF DESIGNING ENGINEER

By E. W. Lane, Research Engineer

Subject: Hydraulic Model Experiments for the Design of the Boulder Dam - Book 3 - Quantitative Results of the Flow in Spillway Channels and Spillway Tunnels.

When the design of the side channel spillways for the Boulder Dam was undertaken, there was much uncertainty regarding many points which it was necessary to consider in making plans for the spillways. As a result of the model tests most of these uncertainties have been removed and consequently future designs can be made with much greater confidence and exactness. In order to convey to those who may be confronted with side channel spillway problems in the future, the information gained by these extensive model tests, the results are presented in considerable detail in these reports. The previous reports deal largely with those results which were subject to analysis by visual observation. This report deals with the problems of the spillway channel and tunnel largely from a quantitative standpoint.

Some of the questions which arose in the preparations of the spillway designs were: (1) Can the theory developed for flow in the side channel spillway be relied upon? (2) How far can the tests on small models be relied upon to show quantitatively how very large

spillways will perform? (3) What air content must be assumed for the water in the channel? (4) What shape is necessary in the transition floor in order that a vacuum may not form beneath the flowing water? (5) Will the water pass around the vertical and horizontal bends without undesirable effects? Many of these questions will arise in connection with any spillway design and therefore the answers to them will be valuable to designers of other spillways of any type.

Quantitative Tests on the First Side Channel Spillway Model

The quantitative tests on the first model side channel spillway were largely to test the reliability of the theory ordinarily used for side channel spillway design. Some tests were made to determine the effectiveness of guide vanes in increasing the spillway capacity and to determine the air content.

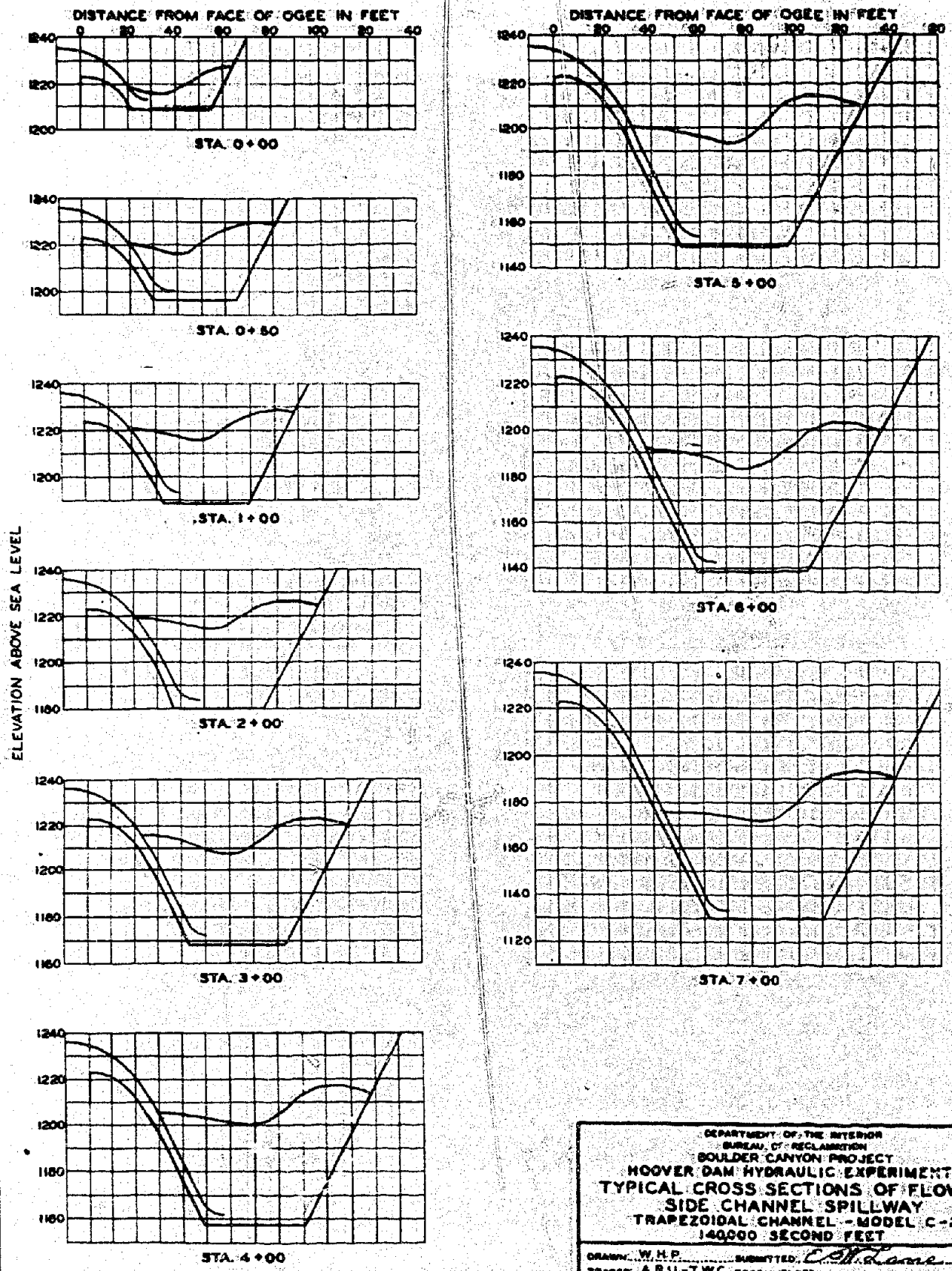
The observations to test the side channel theory consisted in measurements of the cross section of flow in the side channel for various discharges through the model. For this purpose two parallel bars were placed along the sides of the channel at about $1\frac{1}{2}$ ft. above the level of the crest. These bars were carefully leveled so that their tops were in the same horizontal plane. By means of a point gage extending down from a movable beam spanning from one bar to the other, the elevation of the water surface at any point in the channel could be determined. Ordinarily cross-sections were taken at 20-inch intervals, which for the 1:60 scale of the model, corresponded to 100 ft. intervals in the prototype.

In each section enough points were determined to permit the surface to be accurately plotted. The cross-sections were plotted to scale and the area of each was determined with a planimeter. A typical set of cross-sections is shown on Figure 29. This is for a discharge corresponding to 140,000 sec. ft. on the prototype. From the cross-section area, knowing the bottom width and side slopes, the mean depth of flow was computed. In the computations this value was used for the depth of flow in the channel. The surface on the spillway side was lower and on the opposite side higher than this mean.

By means of piezometer openings in the floor of the channel, connected to the glass tube manometers, observations were made of the downward pressure on the bottom of the channel at four points along the center line.

Comparison of Results of Model Tests with Spillway Theory

The quantitative results of observations on the model of the original design are shown on Figure 30. The first diagram on this page shows the mean water surfaces for the various discharges observed, between 50,000 and 200,000 sec. ft. The remaining diagrams of Figure 30 give analyses of the individual runs. For each discharge is shown the average water surface, determined as described above, the theoretical water surface, where it could be computed, the depth of water equivalent to the downward pressure on the channel bottom, as determined by the readings of the piezometers, and the critical depth of flow for



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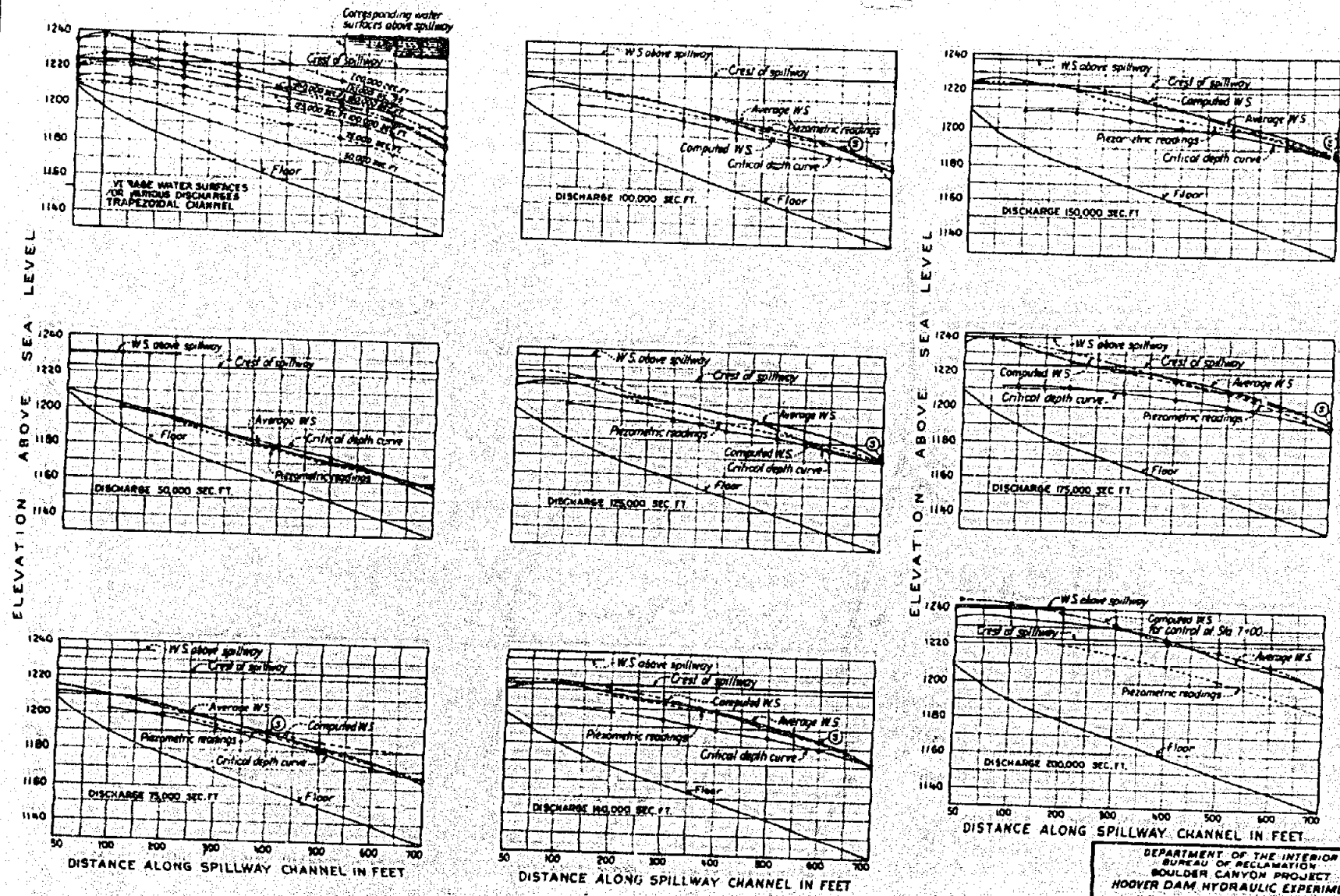
HOOVER DAM HYDRAULIC EXPERIMENTS
TYPICAL CROSS SECTIONS OF FLOW IN SIDE CHANNEL SPILLWAY
TRAPEZOIDAL CHANNEL - MODEL C-2
140,000 SECOND FEET

DRAWN: W.H.P. SUBMITTED: C.H. Lane
TRACED: A.R.U.-T.W.C. RECOMMENDED:
CHECKED: APPROVED:

DENVER, COLO., OCT. 31, 1931 | 45-D-2585



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HOOVER DAM HYDRAULIC EXPERIMENTS

RESULTS OF EXPERIMENTS ON SIDE CHANNEL SPILLWAY
TRAPEZOIDAL CHANNEL-MODEL C-2

DRAWN: W.S.R. TRACED: C.E.G. CHECKED: R.W.S. SUBMITTED: E. W. Lane RECOMMENDED: APPROVED:

DENVER, COLO., SEPT. 8, 1931 | 45-D-1412

FIG. 36

the given channel section and discharge. The level of the water surface in the reservoir above the spillway and the elevation of the spillway crest and floor are also indicated. The theoretical water surface was computed by the method given by Hinds*, which is based on the assumption that all of the energy of the water flowing over the

*Trans. Am. Soc. C. E., Vol. 89 (1926) pp. 898-899.

crest is lost. These curves were usually assumed to start at the height of the observed water surface at Station 7+00, the lower end of the overflow section of the side channel, or the lowest station at which the cross-section could be observed. The computation of the theoretical water surfaces was made by a cut and try method very similar to that in the ordinary cut and try method of computing a backwater curve. In fact, the operation is really the computation of a backwater curve using the laws of inelastic impact instead of the Bernoulli Theorem. There is a strong resemblence in some respects between the two types of curves; for example in the fact that the curves of both types cannot extend across the critical depth. In a backwater curve it is necessary to have a starting point, and since backwater curves can be worked upstream more effectively than downstream when the flow is at a supercritical depth, the logical place to start was at the known elevation at the lower end of the channel. The conformity of the observed results with the theory would then be shown by the agreement between the theoretical and actual water surface profiles.

It was not always found possible to determine the curve beginning at Station 7+00, since it was not possible to compute the surface curve where it passed through the critical depth. It was often found to be impossible also where the computed depth was near the critical depth. In these cases the curves were started at a station further upstream, the location of the point in each case being indicated by a letter "S" on the diagrams.

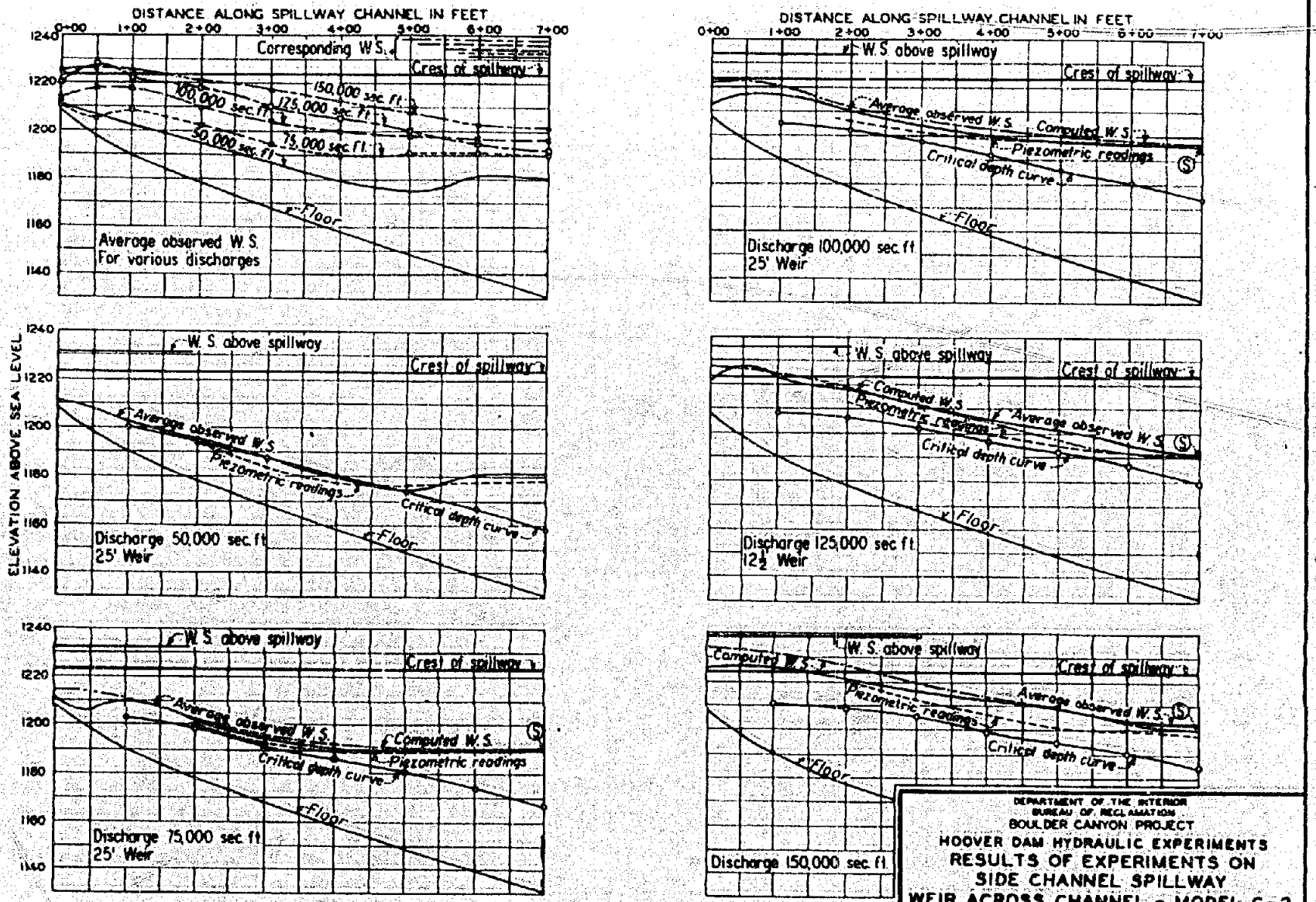
The flow line for 50,000 sec. ft. discharge in the channel, as originally designed, was so near the critical depth that no backwater curve could be computed. For the 75,000 sec. ft. discharge computation was made starting at Station 4+00 and working in both directions. It departed considerably from the observed water surface below Station 4+00, indicating that where the flow is near the critical depth the water surface cannot always be computed accurately in this manner.

For discharges of 100,000, 125,000, 140,000, 150,000 and 175,000 sec. ft. there is a satisfactory agreement between the observed and the computed water levels indicating the substantial soundness of the basic theory. For the 200,000 sec. ft. discharge the computed line falls considerably higher than the observed at the upper end. This may be accounted for by the fact that in the computation a uniform discharge per foot of crest was assumed, while in the actual case the crest was submerged to sufficient depth to reduce the flow at the upstream end.

The line representing the piezometric pressures on the bottom of the channel falls uniformly below the observed mean water surface. This has been assumed by some experimenters to be due to the air content of the water, but measurement of air by other apparatus has shown that this is not the case. The cause of the difference may be that the depth of water over the piezometer openings, which were on the center line of the channel was less than the mean depth of flow in the channel.

The experiments with a weir across the lower end of the channel, the results of which are shown on Figure 3, also indicate the substantial accuracy of the basic theory. In this setup also the flow for the 50,000 sec. ft. discharge was too near the critical to enable a water surface to be computed, but for the 75,000, 100,000 and 125,000 discharge there is a satisfactory agreement between the computed and observed water levels, except at the lower end of the channel, where a wave was formed which made measurement difficult. For the 150,000 sec. ft. flow the crest at the upper end of the channel was deeply submerged as in the case of the 200,000 sec. ft. discharge of the previous setup.

The results on the experiments on the channel made by placing a false bottom in the spillway channel, as shown on Figure 32, also confirm the theory developed by Mr. Hinds. Where the water surface could be computed there was a good agreement of the computed with the observed water surface. For the 150,000 sec. ft. discharge, the crest was also deeply submerged.



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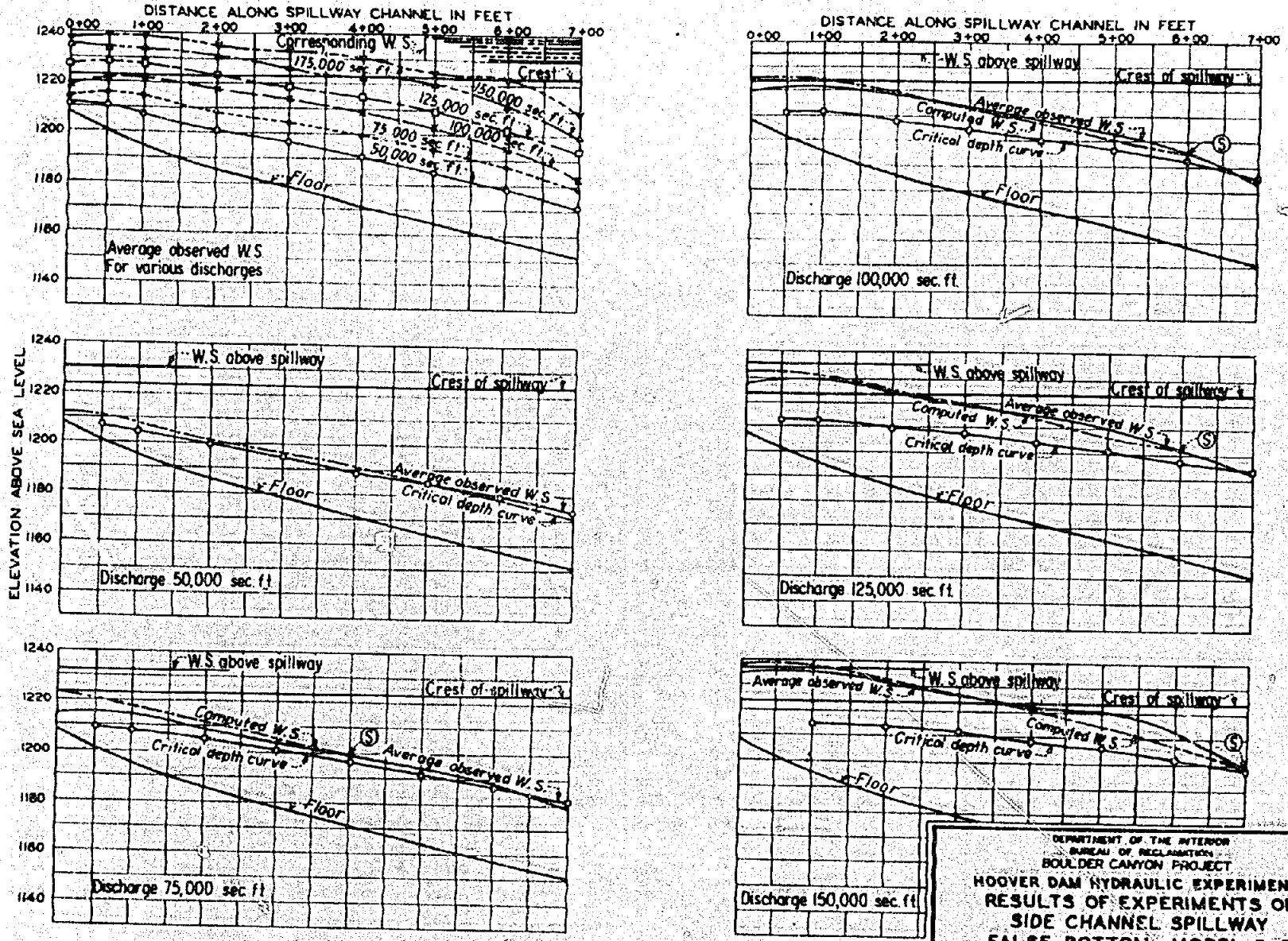
HOOVER DAM HYDRAULIC EXPERIMENTS
RESULTS OF EXPERIMENTS ON
SIDE CHANNEL SPILLWAY
WEIR ACROSS CHANNEL - MODEL C-2

DRAWN: W.H.P. SUBMITTED: C. M. Lane
TRACED: A.R.W.-T.W.C. RECOMMENDED:
CHECKED: W.M.G. APPROVED:

DENVER, COLO., OCT. 31, 1931

45-D-2589

FIG. 31



S Starting point for computed W.S.

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HOOVER DAM HYDRAULIC EXPERIMENTS
RESULTS OF EXPERIMENTS ON
SIDE CHANNEL SPILLWAY
FALSE BOTTOM - MODEL C-2

DRAWN: W.H.P. SUBMITTED: E.D. Lane
TRACED: A.R.H.-T.W.C. RECOMMENDED:
CHECKED: W.M.B. APPROVED:
DENVER, COLO., OCT. 31, 1934
45-D-2590

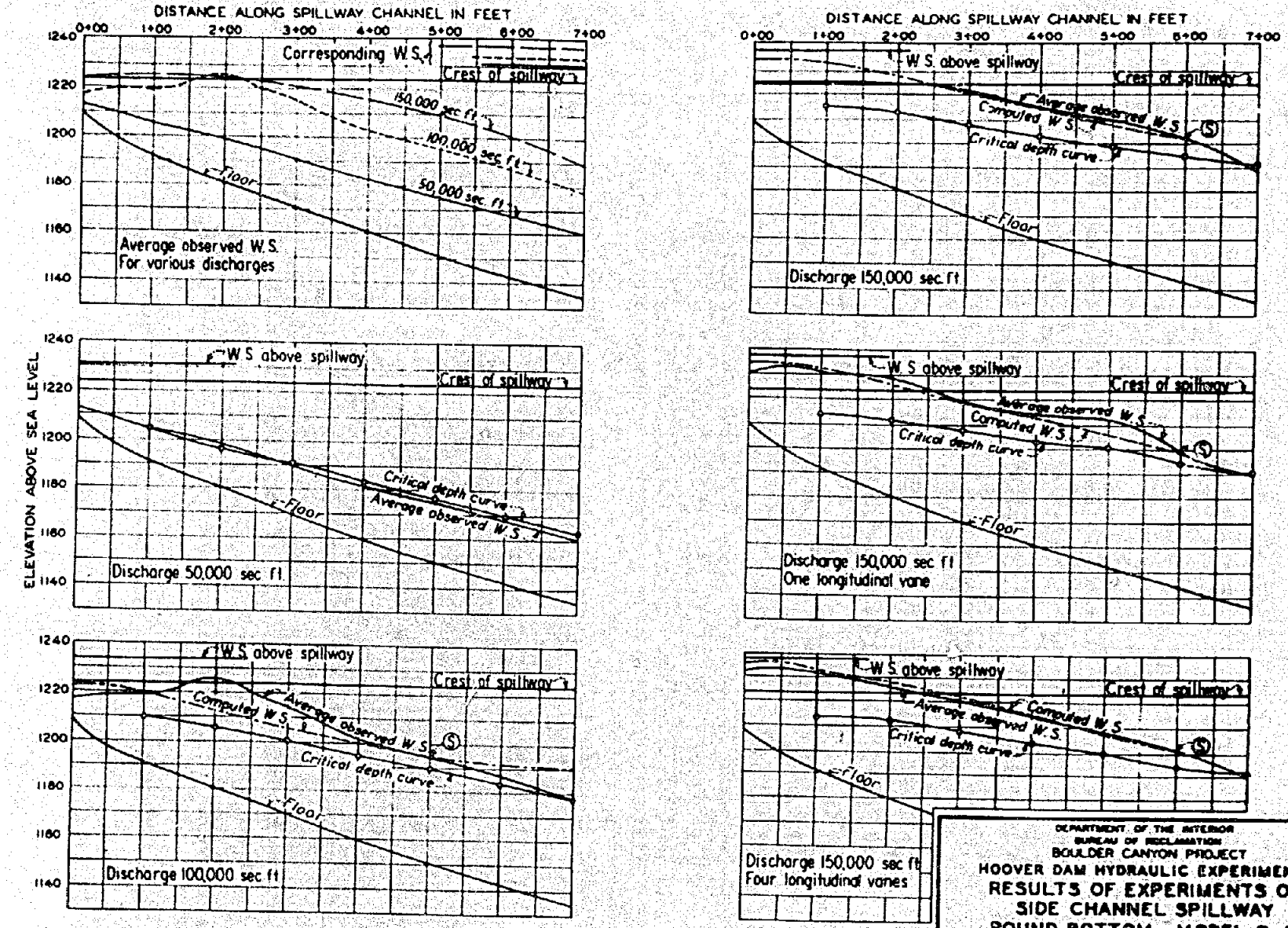
FIG. 32

The experiments on the channel formed by placing a round bottom in the original spillway as shown on Plate 13-A and B, Book 1, in general substantiate the fundamental theory. The results of these tests are shown on Figure 33. For a discharge of 100,000 sec. ft. however, a peculiar wave formation occurred, the explanation of which is not apparent.

Experiments on the Effect of Guide Vanes

A number of experiments were conducted to determine to what extent it was possible to decrease the cost of the spillway by introducing guide vanes into the current, to deflect the water in the direction of the spillway channel, and thus increase the efficiency of the spillway. Without vanes, practically all the energy of the water falling over the spillway crest is dissipated and flow along the spillway channel is caused only by the slope of the water surface in the channel itself. With vanes it is possible to deflect some of the water falling over the crest in the direction of the channel, thus increasing the velocity of the water in the channel and permitting the use of a smaller and cheaper trough. A series of experiments on the effect of vanes was therefore undertaken. The details of the setup of apparatus for all of these tests is filed with the office file copies of these reports.

The first series of guide vanes tried were placed on the bottom of the spillway channel. Observations with color injected into the



(S) Starting point for computed W.S.

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HOOVER DAM HYDRAULIC EXPERIMENTS
RESULTS OF EXPERIMENTS ON
SIDE CHANNEL SPILLWAY
ROUND BOTTOM - MODEL C-2

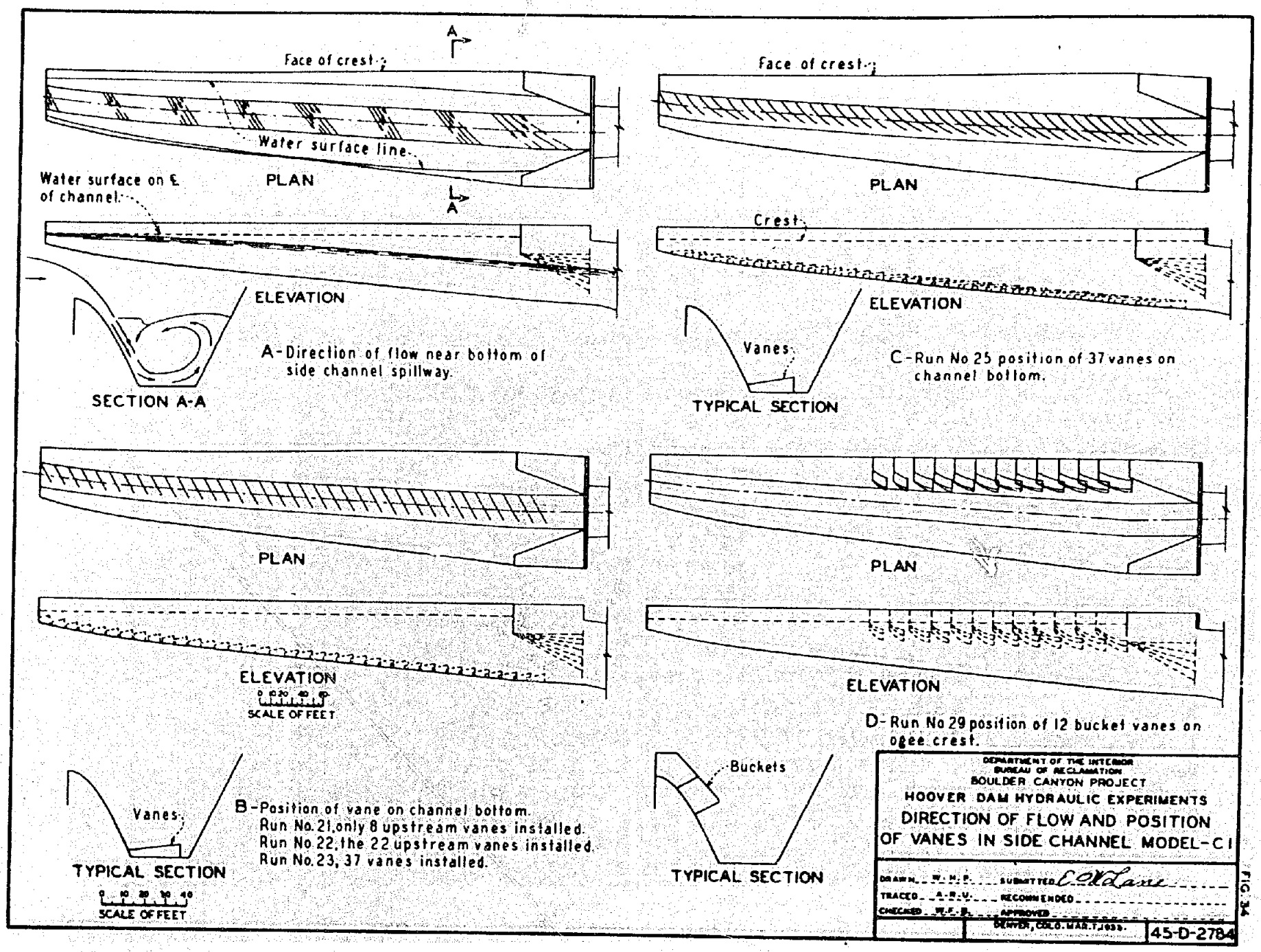
DRAWN W.K.P. SUBMITTED C.H. Lane
TRACED A.R.U.-T.W.C. RECOMMENDED
CHECKED W.M.B. APPROVED

DENVER, COLO., OCT. 8, 1931

45-D-2584

Fig. 19

water and with a current direction indicator showed that the stream of water falling over the weir plunges to the bottom of the channel, thence moves across the bottom and flows up the other side. The direction of the current in flowing across the channel bottom is shown on Figure 34-A. In the first test 8 vanes were placed on the bottom (Run No. 31) and their effect noted. (Fig. 34-B and 35.) More vanes, to totals of 22 and 37 were then added (Runs No. 22 and 23 respectively). The results of these tests are shown on Figures 34 and 35. The average water surface for those conditions was not appreciably changed, showing that the velocity of flow in the channel was not materially increased. It was concluded that the vanes were not sufficiently inclined downstream. Runs Nos. 24, 25, and 26 were then made with 37 vanes at various greater inclinations. The position of the vanes for Run 25 is shown on Figure 34-C. These all showed lower water surfaces than with no vanes, indicating that the vanes were effective in increasing the velocity of flow in the channel. In order to indicate the extent to which it would be possible to lower the water surface if all the energy of the water falling over the crest was used in producing velocity along the channel, the surface which the water would have if there was no loss of energy was computed. On Figure 35, the position of this surface is indicated by the line labeled "No Loss of Energy." The results of these experiments with vanes showed that it was possible to recover only a small part of the energy of the falling water. The recovery was not sufficient to greatly reduce the size of channel necessary, and it was


A
Face of crest
Water surface line
PLAN
Water surface on ℄ of channel
A
ELEVATION
SECTION A-A
A-Direction of flow near bottom of side channel spillway.
Face of crest
PLAN
Crest
ELEVATION
Vanes
TYPICAL SECTION
C-Run No 25 position of 37 vanes on channel bottom.
PLAN
ELEVATION
0 10 20 30 40
SCALE OF FEET
Vanes
TYPICAL SECTION
0 10 20 30 40
SCALE OF FEET
B-Position of vane on channel bottom.
Run No. 21, only 8 upstream vanes installed.
Run No. 22, the 22 upstream vanes installed.
Run No. 23, 37 vanes installed.
PLAN
ELEVATION
D-Run No 29 position of 12 bucket vanes on ogee crest.
Buckets
TYPICAL SECTION
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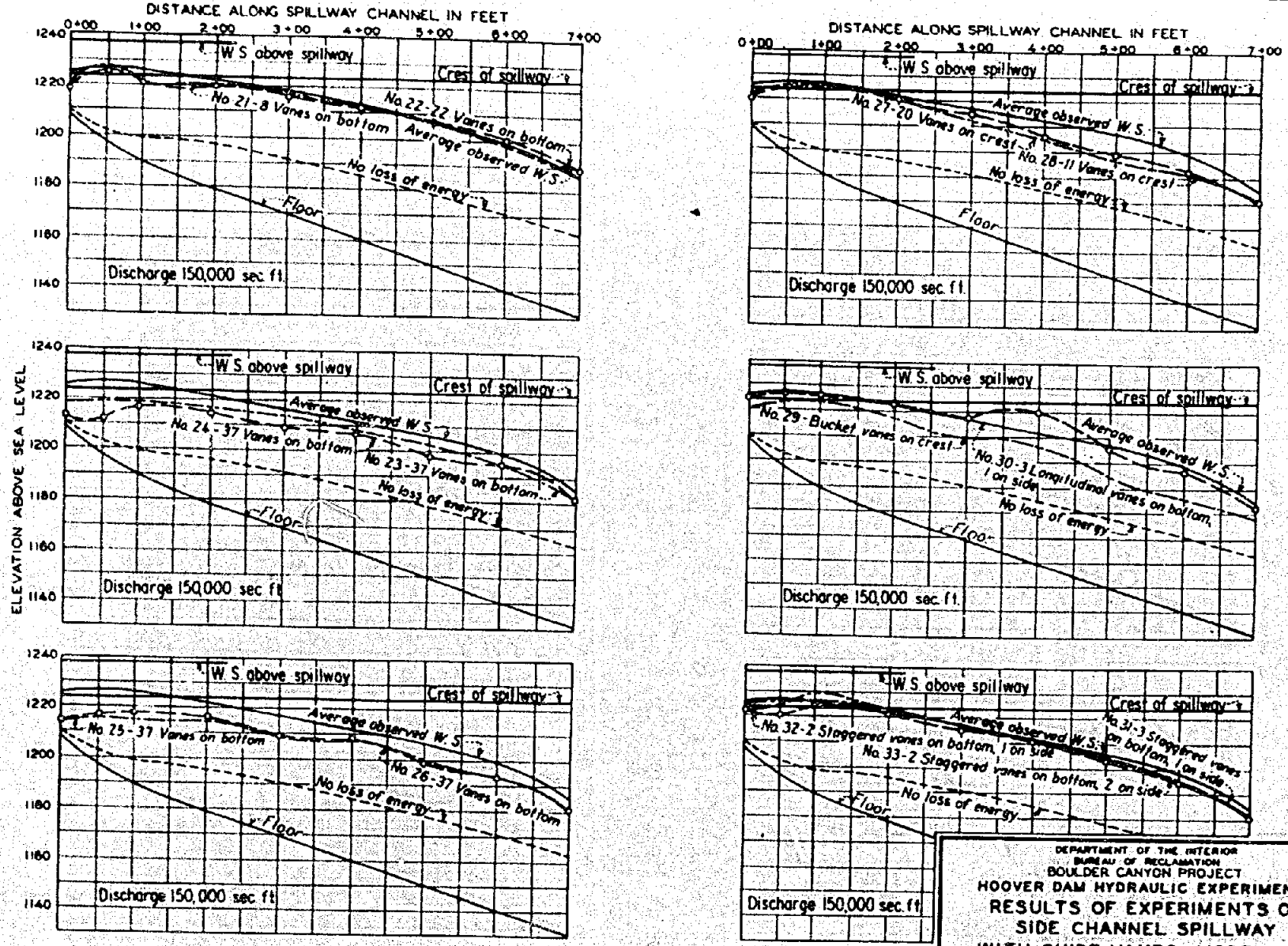
HOOVER DAM HYDRAULIC EXPERIMENTS
DIRECTION OF FLOW AND POSITION OF VANES IN SIDE CHANNEL MODEL-C I
DRAWN W. H. P.
SUBMITTED
TRACED A. R. V.
RECOMMENDED
CHECKED W. F. B.
APPROVED
DENVER, COLO. MAR. 7, 1933.
45-D-2784

FIG. 34



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BOULDER CANYON PROJECT

HOOVER DAM HYDRAULIC EXPERIMENTS

RESULTS OF EXPERIMENTS ON SIDE CHANNEL SPILLWAY WITH GUIDE VANES - MODEL C-2

DRAWN: W.H.P. SUBMITTED: C. W. Lane
TRACED: A.R.U.-T.W.C. RECOMMENDED:
CHECKED: W.M.B. APPROVED:

DENVER, COLO., OCT. 31, 1931

45-D-2591

FIG. 35

evident that the cost of installing guide vanes of sufficient strength to resist the forces involved would be greater than the saving that would be produced by them.

A series of vanes were also placed upon the ogee crest. The first of these were plane surfaces curved in the direction of the channel. (Runs 27 and 28) As the water passed off the side of these vanes, a set (Run No. 29) curved up at the edge was tested (Plate 13-C and D, Book 1 and Figure 34-D.) This gave a better recovery, but in neither case was the saving enough to justify the expenditure necessary to construct the vanes.

Experiments were also performed with various kinds of longitudinal vanes along the bottom and rear wall of the spillway, both with the trapezoidal bottom and with the round bottom. The water impinged on these vanes and did not rise so high against the rear wall of the channel. Their effect was to level out the surface of the flow in the channel, but they caused no recovery of energy. The mean water surface for these conditions was at practically the same elevation as it would have been without the vanes.

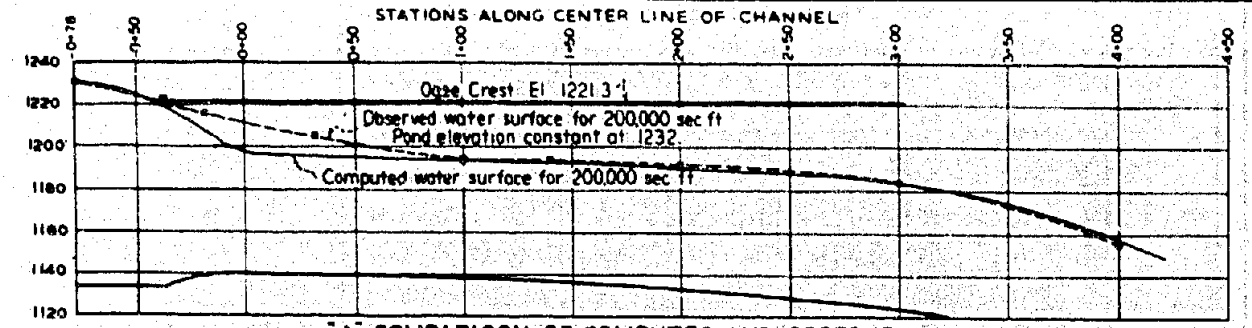
Flow in Channel of Stoney Gate Spillway Model

The design of the Stoney gate side channel spillway contemplated a discharge of 50,000 sec. ft. over the ogee weir and 150,000 through the Stoney gate at the end of the channel. The design (Figure 8) was based on the assumption that the water surface would drop as it

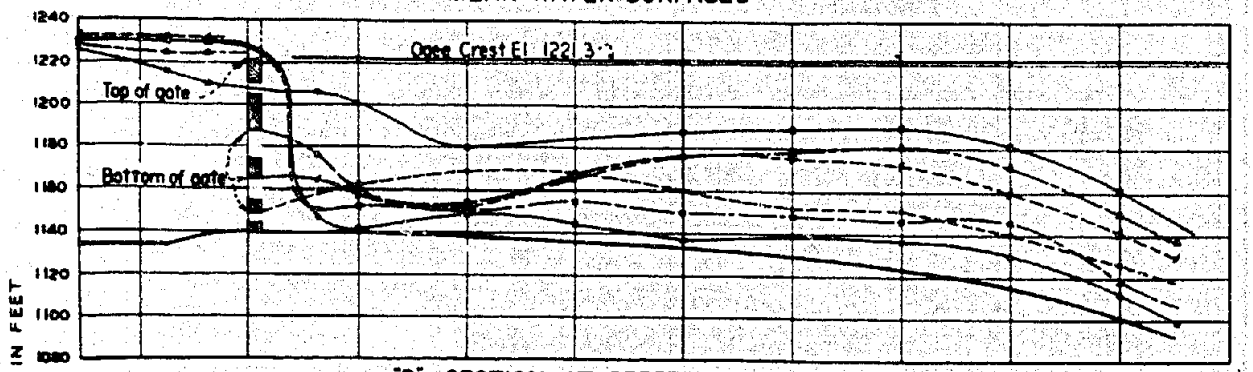
entered the Stoney gate opening to a depth of two-thirds the height of the reservoir level above the gate sill, as the commonly accepted theory of critical flow would indicate. To this was applied a slight friction loss correction. Downstream from the gate, the design was based on the ordinary Hinds' theory for side channel spillways. The flow line computed according to these assumptions is shown on Figure 36, Section A. In order to check those assumptions a comparison is made of this computed mean water surface profile with that actually observed on the model. The agreement is remarkably close, except in the vicinity of the Stoney gate, where the observed water surface was substantially higher than the computed height. This discrepancy had no effect on the flow over the weir portion of the spillway since the observed water surface was still below the crest of the weir. It does indicate however that there is some difference between the action of the water flowing through the gate and the theory ordinarily used for such cases. That the disagreement of theory and actual results is not a special case is shown by similar results for the case of flow through the gate only, as shown on Figure 37, and for experiments conducted elsewhere. This phenomenon will be discussed in succeeding paragraphs.

Although the mean water surface in the channel for most runs agreed well with the computed mean water surface, there were large differences in elevation of the water surface on the two sides of the channel. On Figure 36 are shown the water surface levels at the two

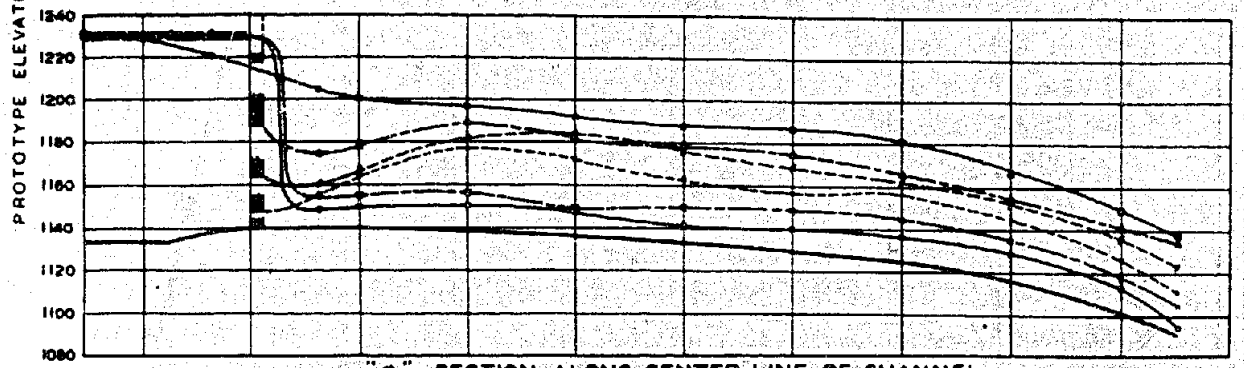
FIG. 36



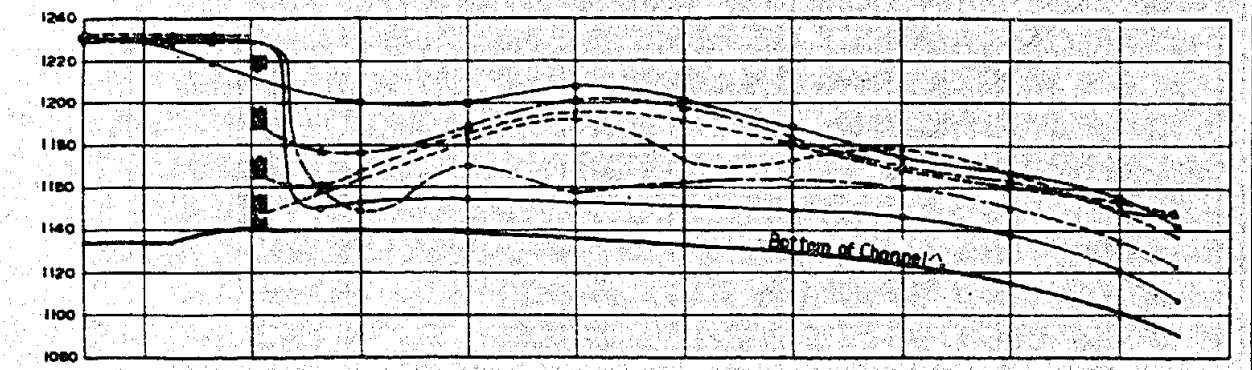
"A"-COMPARISON OF COMPUTED AND OBSERVED MEAN WATER SURFACES



"B"- SECTION AT CREST SIDE OF CHANNEL



"C"- SECTION ALONG CENTER LINE OF CHANNEL



"D"- SECTION ALONG REAR SIDE OF CHANNEL

EXPLANATION

Symbol	Quantity (Sec ft)
	20,000
	40,000
	80,000
	120,000
	160,000
	200,000

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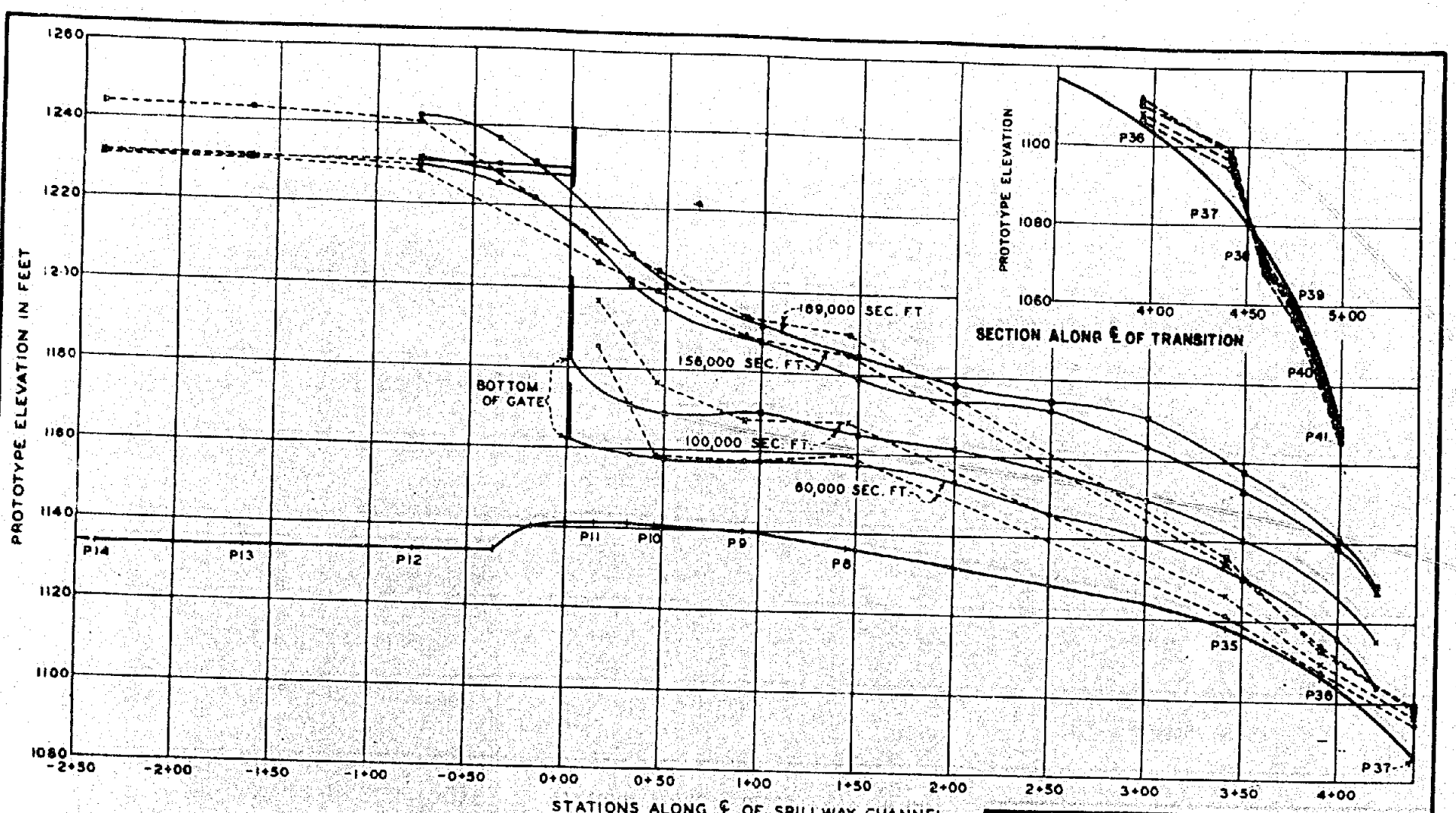
HOOVER DAM HYDRAULIC EXPERIMENTS
WATER SURFACE PROFILES OF
STONEY GATE SPILLWAY MODEL C-3
COMBINED FLOW OF GATE AND CREST

DRAWN W.P.B. SUBMITTED E.W. Lane
TRACED E.A.O. RECOMMENDED
CHECKED W.H.B. APPROVED

DENVER COLO., MAR. 23, 1938 45-D-3148



POINT GAGE READINGS - FULL LINES
PIEZOMETER PRESSURES - DASH LINES

Flow thru Stoney Gate only

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HOOVER DAM HYDRAULIC EXPERIMENTS

WATER SURFACE PROFILES AND PRESSURES ON CHANNEL BOTTOM STONEY GATE SPILLWAY MODEL C-3

DRAWN: W.F.S. SUBMITTED: E.M. Lane
TRACED: G.E.W. RECOMMENDED:
CHECKED: APPROVED:

DENVER, COLO. MAR. 28, 1933. 45-D-2880

FIG. 37

sides and center line of the channel for various flows over the entire range of capacity. These show in a quantitative way the action of the water as described in Book 1.

A comparison of the depth of flow in the spillway channel and the pressure on the bottom of the spillway is shown on Figure 37. The flow in this case was entirely through the gate. The solid lines show the water surface along the center line of the channel, which in this case is very close to the mean depth. The dotted lines indicate the pressure on the bottom of the channel, as shown by piezometers located on the center line, directly beneath the line along which the water surface was observed.

It will be noticed that at some points the pressure on the bottom is greater than would be indicated by the depth of flow, and at other points it is less. In general, the points where the pressure is greater than would be indicated by the depth are located where the water surface is concave, viewed from above, and pressures less than might be expected from the depth occurring where the water surface is convex. This difference is a result of non-parallel flow of the water particles. Where the surface is concave, the particles near the surface are moving in curved paths, beginning with a direction toward the channel bottom and being deflected from this direction as they move along. By Newton's laws of motion, they can only be diverted from the straight path by the application of a force. In this case the force is applied to them by the pressure of the particles moving along

beneath them. If these particles exert a force, there must be a corresponding reaction. This reaction, is exerted downward through the successive layers of particles until the bottom is reached and is there indicated by the pressure on the bottom, in excess of that due to the weight of the particles. This is registered in the difference in the height of the water surface in the piezometers connected to the channel bottom and the height of the surface of the flowing water in the channel. Similarly, when the surface is convex, pressure on the bottom is less than that indicated by the depth because part of the force of gravity is exerted in diverting the particles from their straight line directions, and it therefore does not cause the pressure which would be expected from the depth of flow.

Flow Through the Stoney Gate

Since a large part of the total capacity of the Stoney gate spillway was expected to pass through the gate, for a reliable design of this structure it was necessary to know with considerable accuracy what the discharge capacity of the gate would be. The general theories of discharge for these conditions were well known, but there were very few experimental data available from which accurate estimates could be made. In order to be assured that the full size spillway would have the capacity desired, and to furnish data which would enable future designs to be made with accuracy, a series of experiments on the flow through the Stoney gate were performed.

During the passage of a maximum flood through the reservoir, the Stoney gate would operate under four conditions: (1) gate closed and water discharging over its top; (2) gate partly open and water discharging both over and under it; (3) gate partly open and water discharging under it; (4) gate entirely open and not in contact with the water.

The flow in the first condition was similar to that over a flat crested weir. No detailed design for the gate had been prepared and since the discharge over the prototype would depend so largely on these details, it was believed that results obtained from the model would be no more reliable than could be obtained by using the coefficients derived from experiments on flat top weirs. Since this discharge was always small, accuracy in estimating it was not important and therefore no measurements for this condition were made on the model.

The second condition was practically a combination of the first and third condition, since the flows both under and over the gate were small and did not appreciably interfere with each other. As the discharges could be estimated with sufficient accuracy using this assumption, and since it would have been difficult to measure the two discharges when both were flowing at the same time, no quantitative results for this condition were secured.

Detailed experiments were made to determine the discharge coefficients for the third and fourth condition, as it is under these conditions that the greatest discharge would take place. Since different discharge formulas apply to the two cases, they will be discussed separately.

Stoney Gate Partly Open

When the Stoney gate was partly open and no water was passing over the top, the condition was that of flow through an orifice with suppressed contraction on the bottom and the two sides upstream from the orifice and confined on the sides and bottom on the downstream side, in which the downstream depth was not sufficient to submerge the orifice. Under these conditions the best general formula for discharge is

$$Q = CWD\sqrt{2gH'}$$

where C is the coefficient representing the ratio of the measured discharge to the theoretical discharge. (See Figure 38.)

W is the width of the gate opening.

H' is the head measured from the water surface in the reservoir, where the velocity of approach was negligible, to the bottom of the gate.

D is the opening of the gate or the vertical height of the orifice opening.

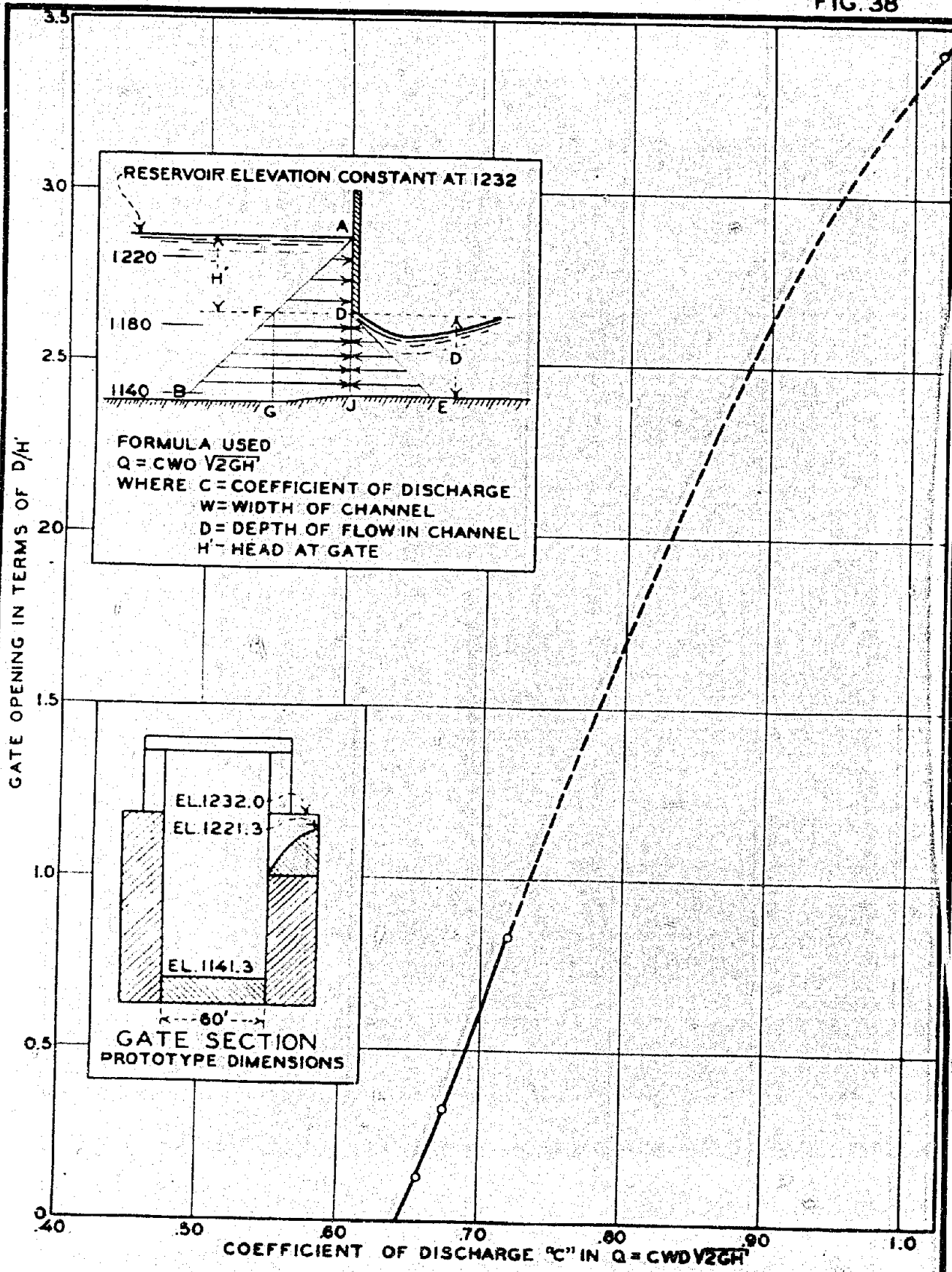
Since the outflowing stream was supported by the floor and confined by the sidewalls downstream from the opening, it did not fall freely as in the usual case of the orifice, and therefore it exerted a back pressure on the orifice as shown by the arrows in the triangle D J E of Figure 38. The pressure acting downstream and tending to cause flow is shown by the arrows of the trapezoid D F B J and the net effective pressure is therefore that shown by the heavy arrows of the rectangle F D J G, which is that due to a head H' measured to the bottom of the gate.

A better formula theoretically might be developed by measuring both D and H' from the most contracted section of the stream issuing from the orifice, but the elevation of this point was not observed. Moreover, such a form would have a much more limited practical application.

The results of the coefficient determinations are shown on Figure 38 and the numerical data is given in Appendix II, copies of which are filed with the office copy of this report. The curve gives the value of the coefficient C for various values of D/H' or the ratio of height of orifice opening to the head. This curve has been made up in this manner in order to make it as readily applicable as possible to other cases.

It is believed that because of the gradually contracting section of the channel leading to the orifice, the effect of the side contractions on the orifice is negligible. If this is the case, by

FIG. 38



HOOVER DAM HYDRAULIC EXPERIMENTS

DISCHARGE COEFFICIENTS

FLOW UNDER STONEY GATE

E.W.C.

the principle of similitude, the results on the model should be applicable to any case of suppressed side contractions and of a supported, confined stream below, having the same values of the ratio D/H'. In other words, to determine the coefficient for any such case, determine the value of D/H' and enter the curve of Figure 38 to obtain the coefficient.

The data on Figure 38 is fairly well established up to values of D/H' slightly above 0.8. No observations were made for higher values except for free flow conditions where the gate did not hold back the water but merely touched the surface of freely flowing stream. This occurred with D/H' = 3.42. The curve has therefore been extended as a dotted line above D/H' equal to 0.8, indicating that the values in this range are not well established.

Stoney Gate Entirely Open

With the probable plan of operation of the outlets of the Boulder Dam during flood, the Stoney gate would be opened entirely only if the maximum capacity of the spillways was required. In this case the reservoir would be at elevation 1232. In order to make the tests as complete as possible, however, discharges through the gate opening were made for reservoir water levels both above and below, as well as at El. 1232.

Since the channel bottom downstream from the Stoney gate structure sloped rather steeply and expanded slightly, it maintained the stream at a subcritical depth, the flow passing through the critical

depth in or near the Stoney gate structure. In other words, the gate structure acted as a control. Under these conditions, according to the well known theory of critical flow, the water surface as it passed through the gate should theoretically have dropped to give a depth of two-thirds the height of the reservoir water level above the gate sill and the velocity head should have been one-third of this height, or one-half the depth of the water passing over the sill. How close the water surfaces came to agreeing with this theory has been discussed on page 9. For the theoretical condition the velocity head would be $\sqrt{\frac{2gH}{3}}$ and the area of flow $W\frac{2H}{3}$, giving a discharge of $\frac{2}{3}WH\sqrt{2g\frac{H}{3}}$, in which W is the width of the gate opening and H is the height of the reservoir level above the gate sill, the velocity of approach in the reservoir being negligible. Substituting 32.2 as the value of g, there is obtained the formula

$$Q = 3.089\ WH^{3/2}$$

The results of the experiments have been worked up for the formula

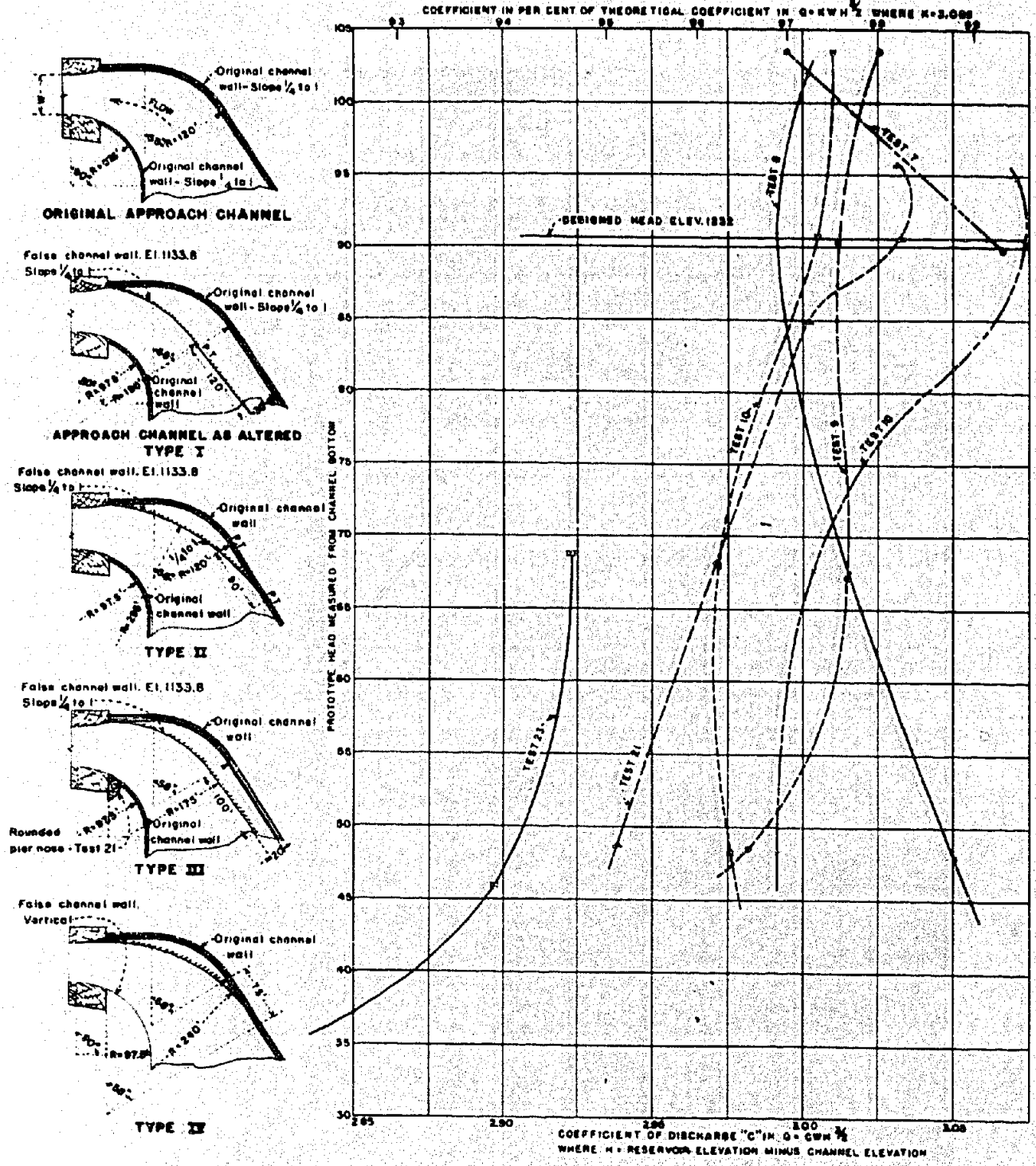
$$Q = 3.089\ KWH^{3/2}$$

in which K represents the ratio of the observed to the theoretical discharge, and also the more useful formula

$$Q = CWH^{3/2}$$

On Figure 39 is shown the forms of approach channel leading to the gate which were experimented upon and the values of the coefficients C and K observed.

FIG. 39



EXPLANATION

TEST NO.	SYMBOL	LAYOUT
7		Original channel design
8		Approach channel - Type I
9		Approach channel - Type II
10		Approach channel - Type III
18		Approach channel - Type IV 90° Curved pier wall vertical.
21		Original channel design Round false pier nose
23		Topography taken out

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BOULDER DAM HYDRAULIC EXPERIMENTS

DISCHARGE COEFFICIENTS OF STONEY GATE OPENING

MODEL C-3

GATE ENTIRELY OPEN

DRAWN F.H.B. SUBMITTED C.W.Lane
TRACED A.R.U. RECOMMENDED
CHECKED F.H.B. APPROVED
DENVER, COLO. AUG. 12, 1933 45-D-3340

The coefficient values for all types of entrance with the topography around the spillway vary but little, giving values of C ranging from 2.93 to 3.075 and values of K from 0.950 to 0.996. This indicates values near the theoretical can be obtained with any similar gradually contracting channel and that small differences in shape are relatively unimportant. The best results at the higher flows were obtained with the Type IV entrance (Run 18). This was to be expected, since the right wall was set to eliminate the sudden change of wall direction at the junction of the wall with the Stoney gate structure, and the left wall presented a much smoother surface than the model topography. The results without the model topography (Test 23) show considerably lower coefficients, due to the sharp corners (shown on Plate 14, Book 1) around which it was necessary for the water to flow in entering the spillway.

Although the discharge seems to agree closely with the theoretical formula, based on the ordinary assumptions of flow through a contraction in a channel, the water surface, as previously mentioned, departs considerably from the position which this theory would indicate. The theory indicated that the drop in water surface level from the reservoir to the gate should be one-third the difference in elevation between the reservoir surface and the gate sill, but the observed surface on Figure 36, Section A, shows a drop of only about one-sixth of the difference, or only half that indicated by the theory. Much the same conditions are shown in the observations of flow through the

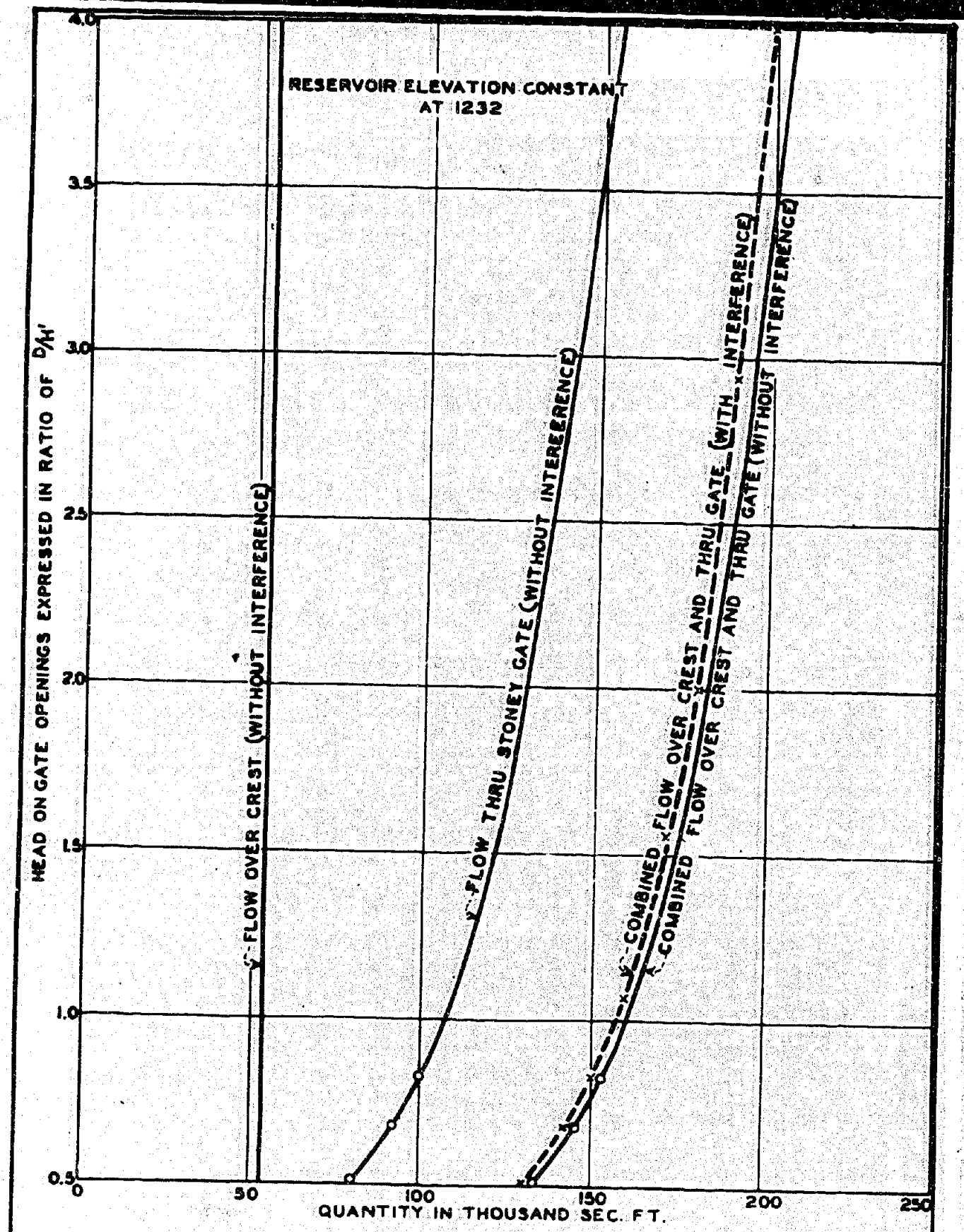
Stoney gate only, as shown on Figure 37. Experiments performed elsewhere show similar results.* The cause of this divergence of fact and theory is not definitely known, but the probable cause is the difference previously pointed out in the pressure in the water and that which would be expected from the depth. The theory which gives rise to the one-third drop assumes flow in parallel stream lines, which would produce pressures corresponding to the depth. The conditions of flow with the one-third drop are such that a slight difference of pressure could cause a considerable difference in the surface level without a material effect upon the discharge. Time was not available for a more through study of this phenomena, but it offers a worth while field for further experimentation.

*Trans. Am. Soc. C. E., Vol. 83, 1920, page 1183-1185.

Interference of Stoney Gate and Crest Discharge

One of the uncertainties in the design of the Stoney gate spillway was the extent to which there would be interference between the discharges of the Stoney gate and the spillway crest. Since the capacity of the gate is quite large, it was feared that the drawdown which would occur in the reservoir near the gate would greatly reduce the effective head on the weir crest and thus materially reduce the discharge over it. The drawdown of the weir would also reduce the flow through the Stoney gate. A study was made of the model results to determine the magnitude of this combined effect. The result of the

study is given on Figure 40 which shows the interference when the reservoir is held at El. 1232 and various flows are passed through the Stoney gate. The flow over the crest without interference was determined by computation using values of the crest coefficient determined from experiments with the Stoney gate closed and the flow through the Stoney gate without interference was computed using the value of the coefficient shown on Figure 37 which was determined with no flow over the crest. A comparison of the sum of these two computed flows with the flow observed with both crest and gate acting at the same time is given. This shows that at 200,000 sec. ft. discharge the interference reduced the capacity 5,000 sec. ft. or 2.5%. This represents 9.2% of the discharge of the weir. Since the observed profile of flow through the Stoney gate shows only a slight drop down upstream from the gate and since this acts near the adjacent end of the crest only, a reduction of 9.2% seems too large for the weir alone. This would indicate that a substantial part of the interference acted to reduce the flow through the gate. Since time was not available to run a special set of tests to determine this effect and since the data available from the other tests is meager, the interference effect may be somewhat in error. Visual observations of the model in action would lead one to believe that the interference was small and more likely to be less than 2.5% rather than more than that amount.



HOOVER DAM HYDRAULIC EXPERIMENTS

INTERFERENCE OF FLOWS THRU
STONEY GATE AND OVER SPILLWAY CREST

E.W.C.

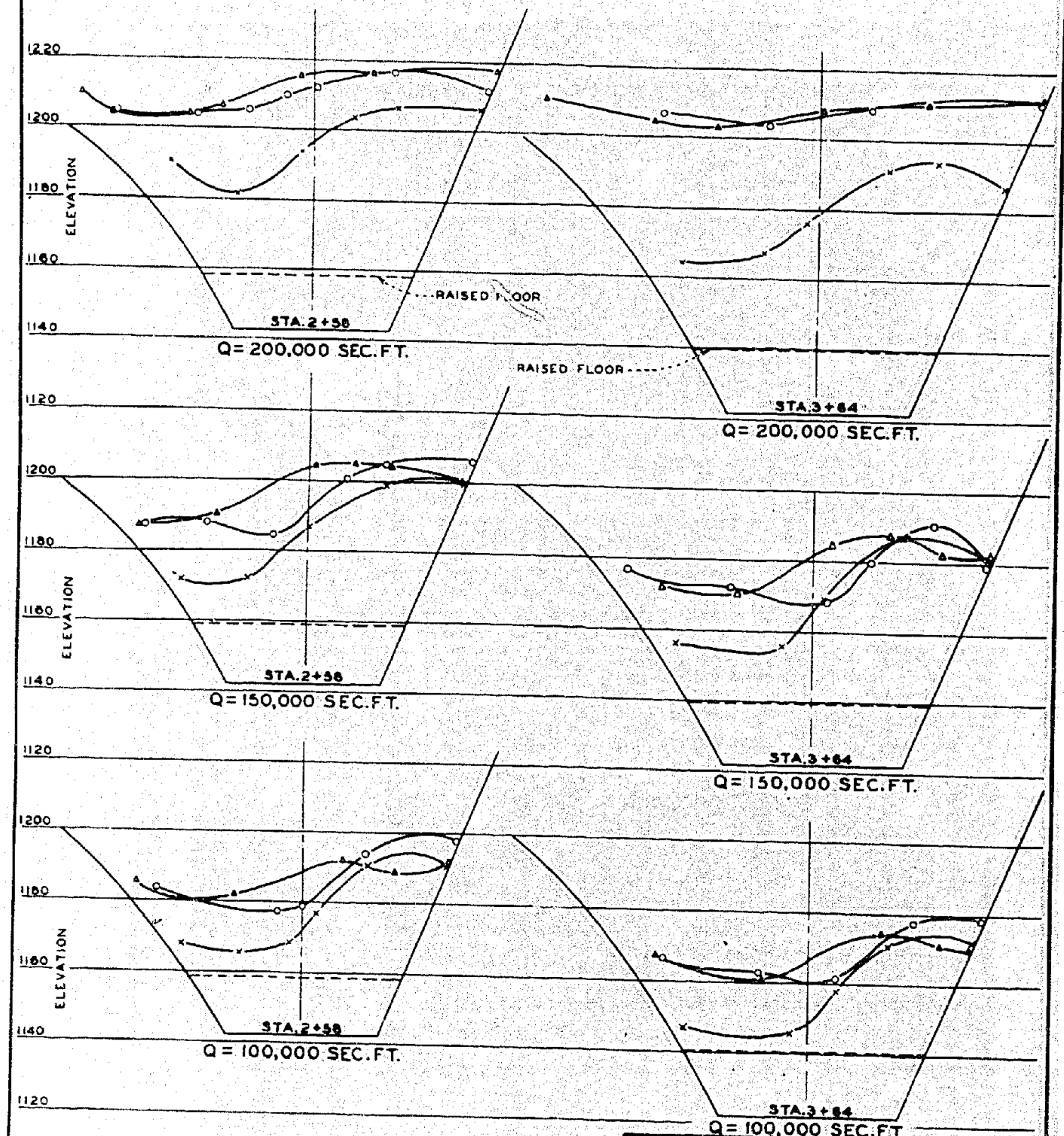
45-D-3448

Quantitative Results on Channel of Spillway Model M-1

On the 1:20 scale M-1 model at the Montrose laboratory, which was the first type of drum gate spillway investigated, cross sections of the water surface were taken at a number of points along the side channel, for several of the different conditions observed. Typical cross sections for three of these conditions are shown on Figure 41 and longitudinal profiles of the mean water surface for two conditions are shown on Figure 42.

The longitudinal profiles of mean water surface for four different discharges on the original design are shown on Figure 42. This profile shows that the mean water surface for the design discharge rose above the spillway crest at the upstream end of the channel, and to the top of the transition at the downstream end. The submergence of the crest probably had little or no effect on the discharge. The height at the lower end however indicated that the portal section of the transition was not large enough, and in later models it was made larger. The typical cross sections for the original design on Figure 41 show the high wave which formed against the back wall of the channel at all discharges. The effect of the raised floor in decreasing the height of the wave is also shown. In studying this diagram it should be remembered that the floor of the model for two of the runs was higher than for the original design. The effect of the coping on the back wall, in lowering the height of the wave and moving it toward the center of the channel is also shown.

FIG. 41



EXPLANATION

×——× Original design

○——○ Improved portal, flat top contraction in transition, and raised floor.

△——△ Same as the above with the addition of a coping on back wall.

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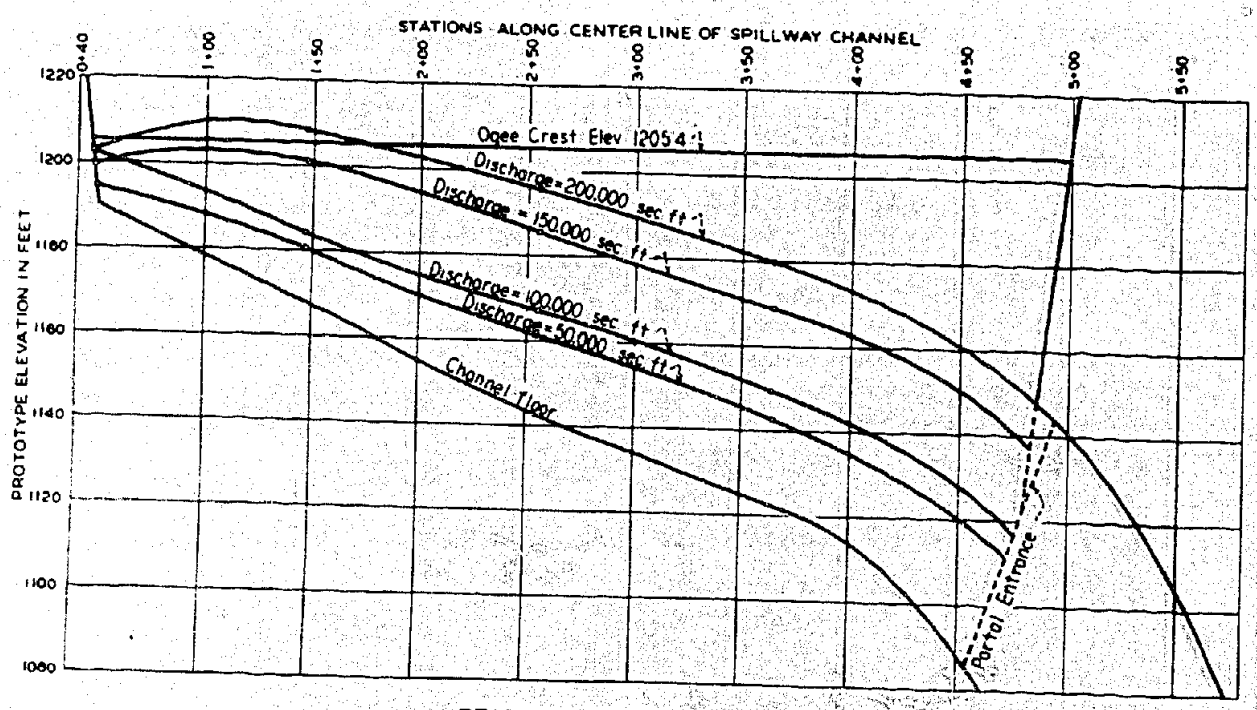
HOOVER DAM HYDRAULIC EXPERIMENTS

COMPARISON OF EFFECT OF RAISED FLOOR IMPROVED PORTAL, FLAT TOP CONTRACTION IN TRANSITION AND COPING ON BACK WALL
MODEL M-1

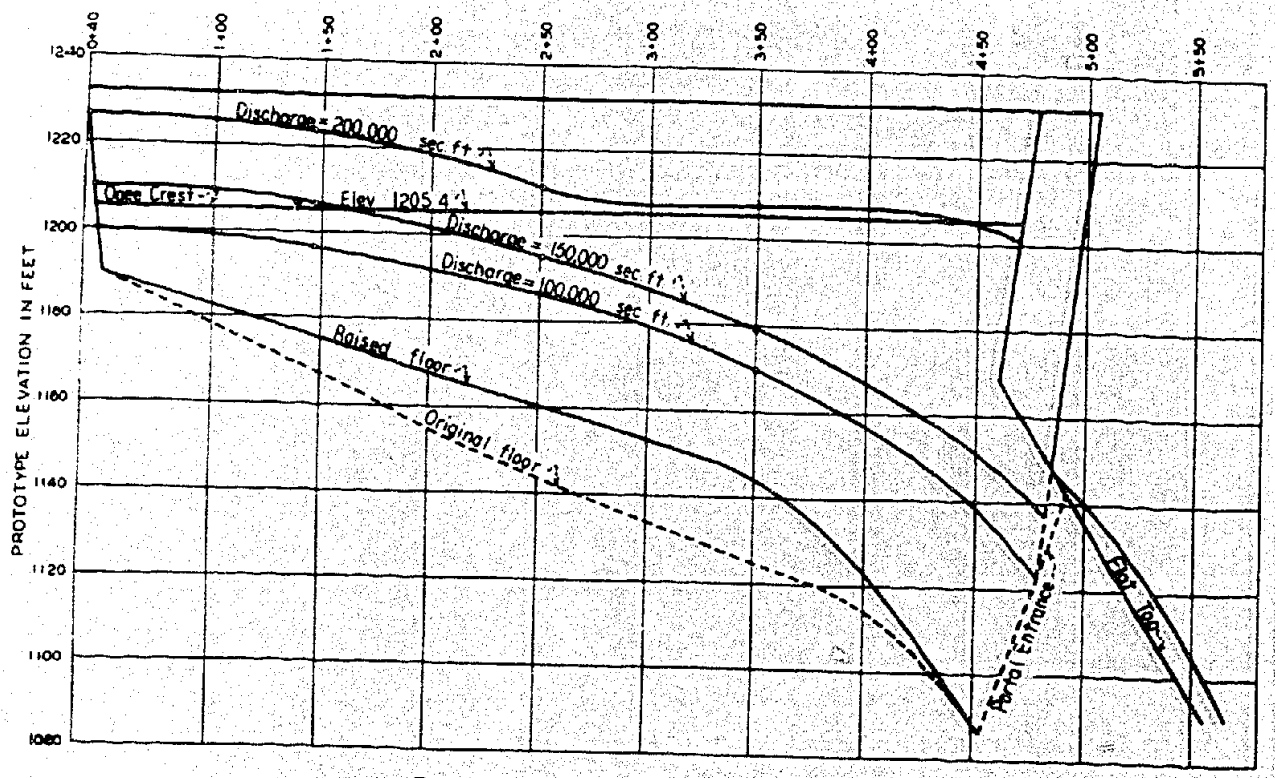
DRAWN W.M.B. SUBMITTED E.O. Lane
TRACED A.R.U. RECOMMENDED
CHECKED W.F.B. APPROVED

DENVER, COLO. MAR. 8, 1933. 45-D-2765

FIG. 43



ORIGINAL DESIGN



RAISED FLOOR-BELL MOUTH PORTAL WITH
20 FT. FLAT TOP IN TRANSITION

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HOOVER DAM HYDRAULIC EXPERIMENTS
MEAN WATER SURFACE PROFILES
SIDE CHANNEL SPILLWAY MODEL M-I

DRAWN W.M.D. SUBMITTED
TRACED E.A.G. RECOMMENDED
CHECKED APPROVED

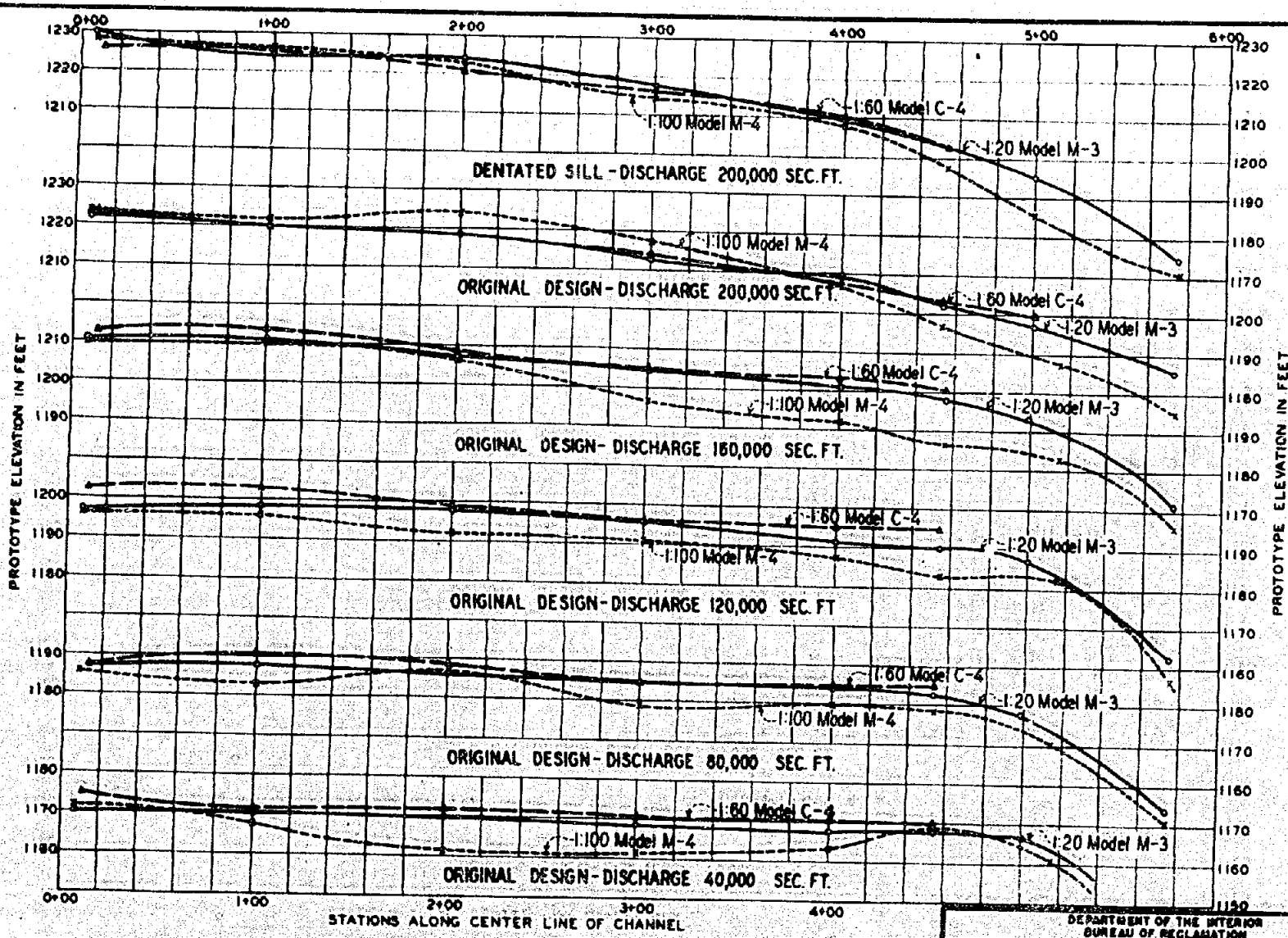
DENVER COLO. MAR. 22, 1933. 45-D-2877

Figure 42 also shows the longitudinal profile for the model with the raised floor and the flat top in the transition. This contraction in the transition did not come into action until a discharge of about 175,000 second feet was reached, since at the lower flows the water does not rise high enough to come into contact with this top. For the design discharge however it restricts the flow and causes the water surface at the lower end of the channel to rise considerably above the top of the portal. This rise is shown on both Figures 41 and 42.

Quantitative Results on Channel of Models C-4 and M-3

Cross sections of the flow in the channel of the C-4 model were taken for a great many different conditions and discharges but space does not permit them all to be included in this report. Longitudinal profiles and typical cross sections for a number of runs are shown in the comparisons of the results of this model and those obtained on the M-3 model on Figures 43 and 44.

Similar cross sectional measurements were also made on the M-3 model. Figure 45 shows the longitudinal profiles of the mean water surface for the semi-finally designed channel, as compared with those obtained with the dentated baffle on the back side of the channel. This comparison shows the effect of the baffle in obstructing the flow down the channel, as indicated by the fact that the water surfaces for the same flows are considerably higher with the baffle in place. As the crest of the weir in this model was at El. 1205.4 the



EXPLANATION

SYMBOL	MODEL
	1:100 Model M-4
	1:20 Model M-3
	1:60 Model C-4

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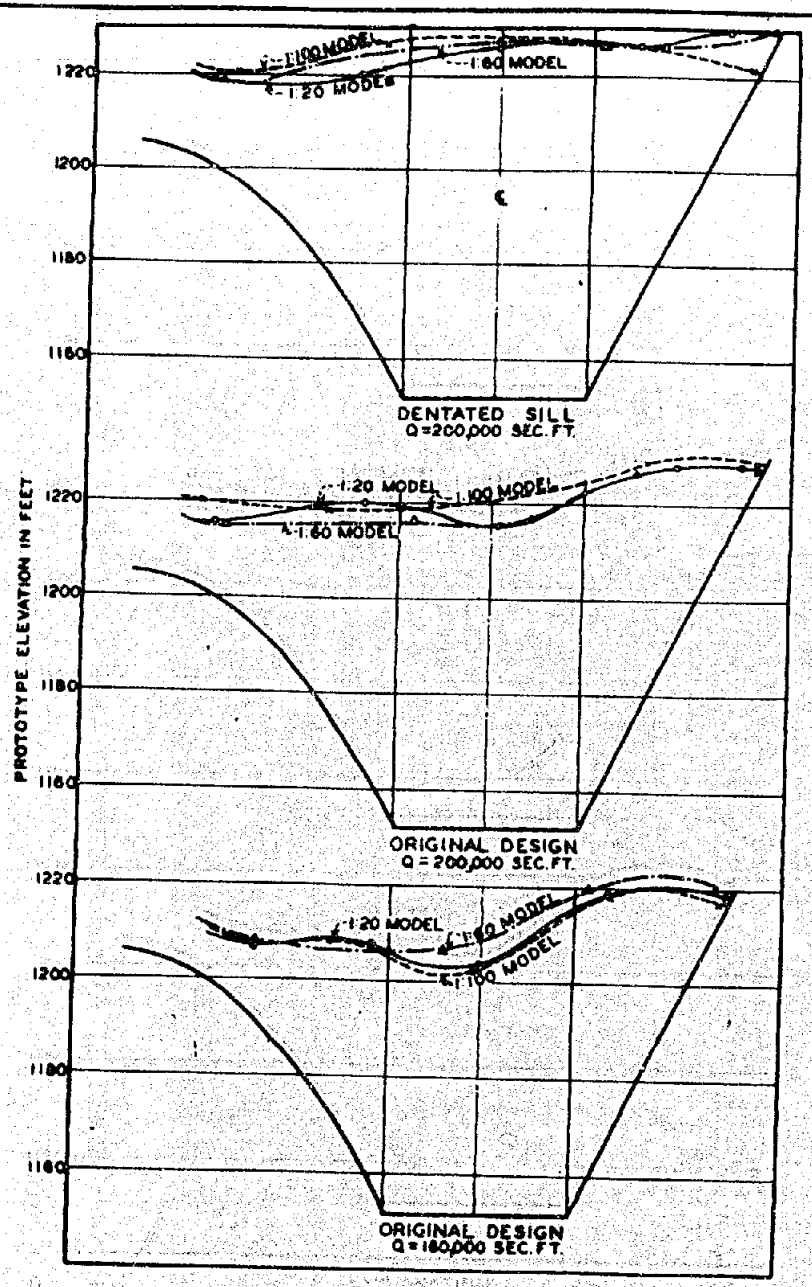
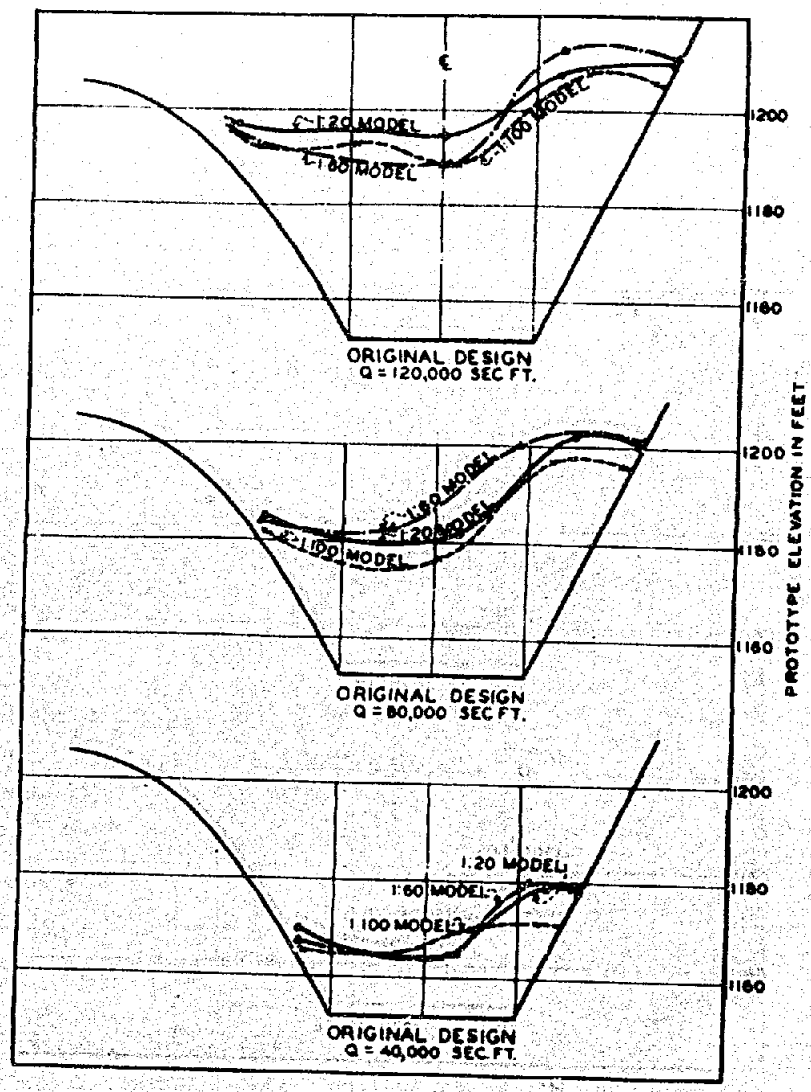
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILTUDE STUDIES
CHANNEL MEAN WATER SURFACE PROFILES
M-3, C-4 AND M-4 MODELS

DRAWN J.F.G. SUBMITTED E. M. Lane
TRACED A.R.U. RECOMMENDED
CHECKED J.W.H. APPROVED
DENVER, COLO. MAR. 16, 1933

45-D-3339

FIG. 43





EXPLANATION

- 1:20 MODEL M-3
- 1:60 MODEL C-4
- 1:100 MODEL M-4

HOOVER DAM HYDRAULIC EXPERIMENTS

SIMILITUDE STUDY

CHANNEL CROSS SECTION

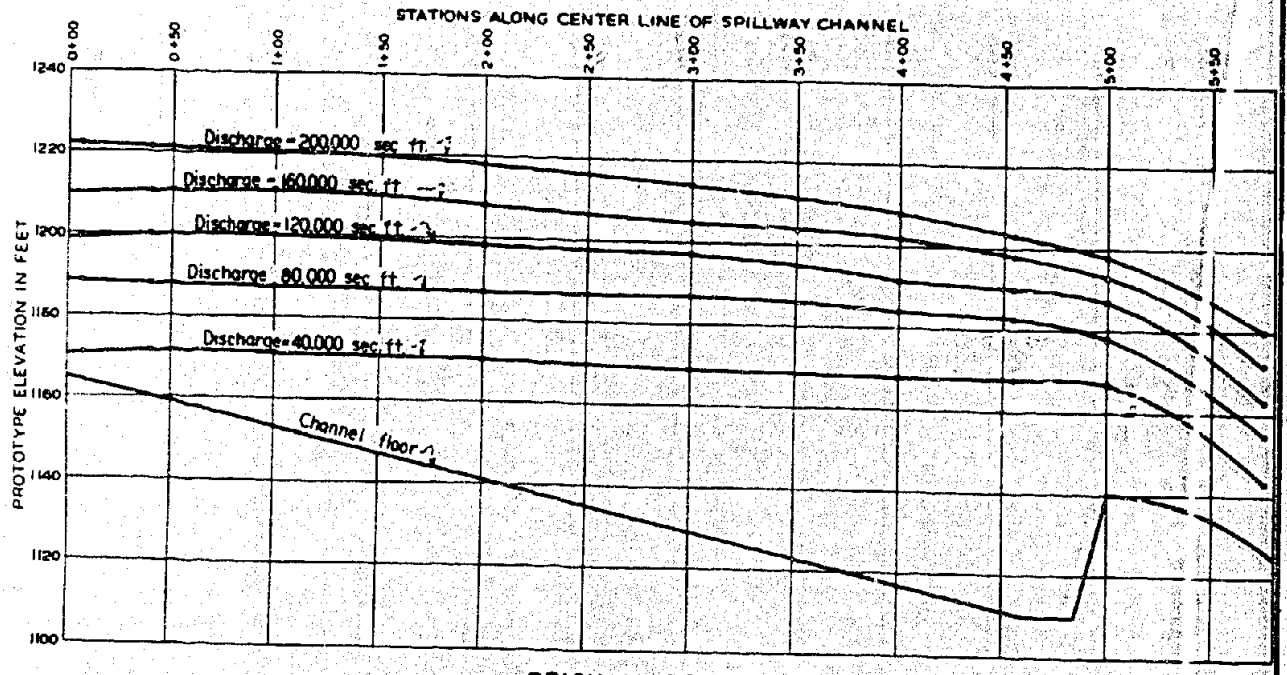
M-3, M-4 AND C4 MODELS

SECTION AT STATION 1+100

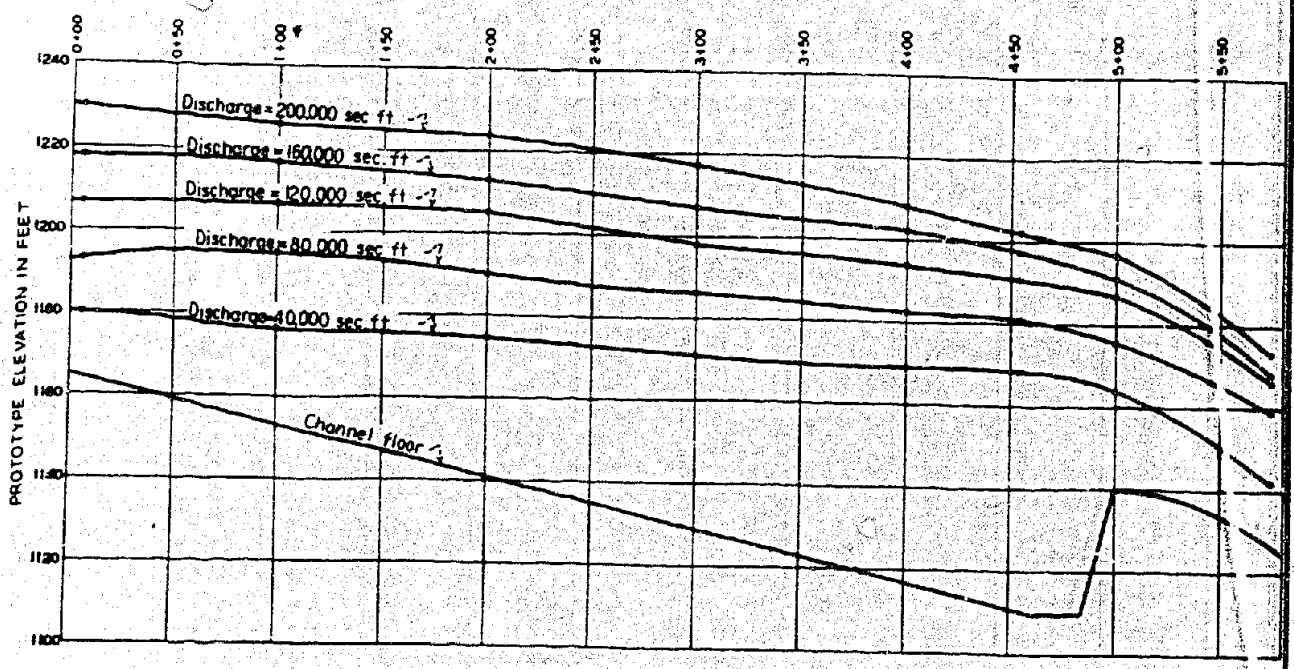
FIG. 4

E.W.C.

45-D-3449



ORIGINAL DESIGN



DENTATED BAFFLE IN BACK SIDE OF CHANNEL

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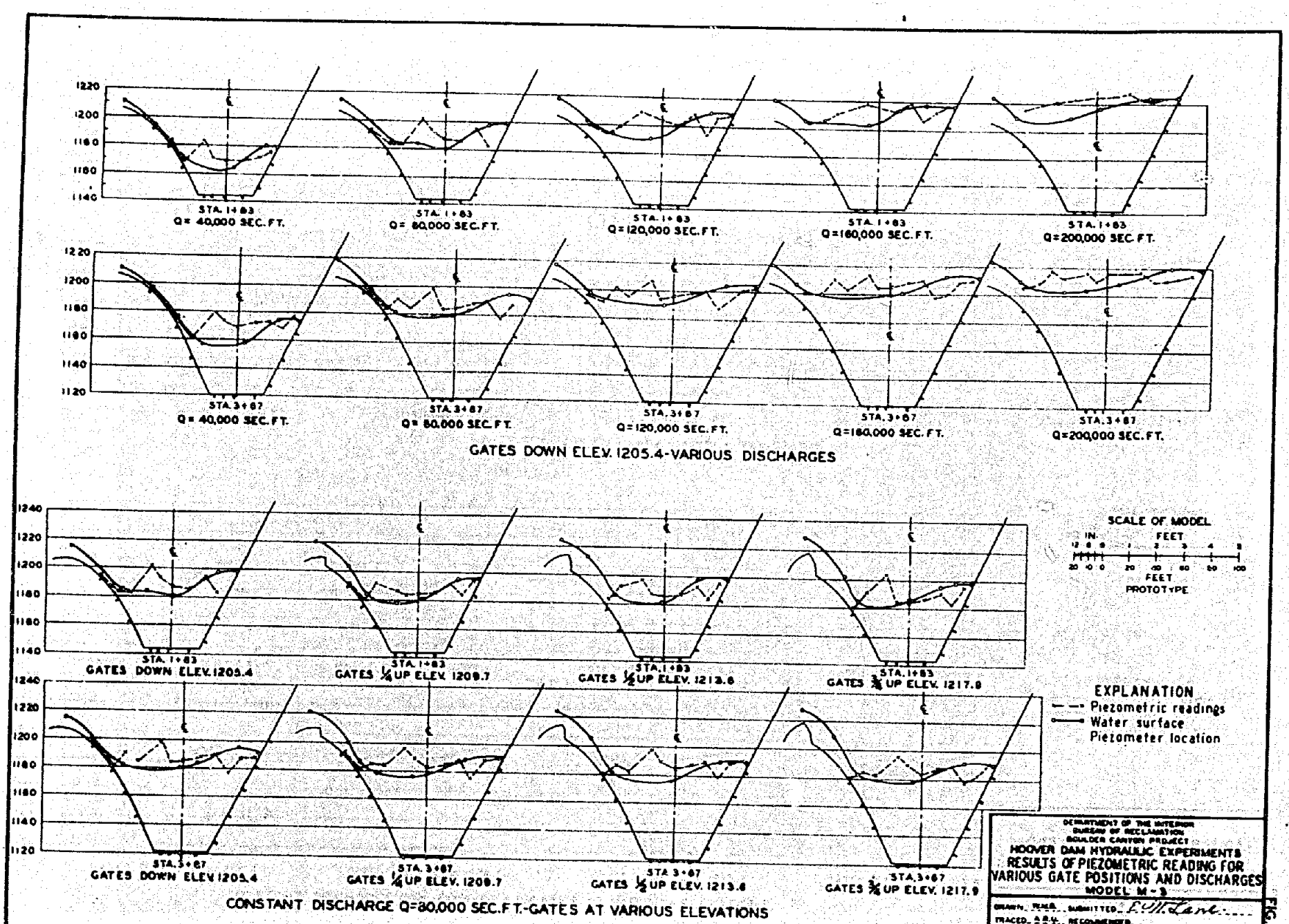
HOOVER DAM HYDRAULIC EXPERIMENTS
MEAN WATER SURFACE PROFILES
SIDE CHANNEL SPILLWAY MODEL M-3

COMPD. BSAB SUBMITTED E.O.T. Lane
TRACED BAO RECOMMENDED
CHECKED W.F.G. APPROVED

HOOVER DAM, ARIZ.-NEV. 45-D-2576

water surface in the channel at the upstream end was far above the crest of the weir and materially reduced the flow over the crest there even when there was no baffle. With the baffle still more reduction resulted.

Piezometers were installed in the sides of the channel of the M-3 model and observations of the pressures for various flows were made. For these runs there were no baffles in the channel. The results of most of these observations are shown on Figure 46. The solid line represents the observed water surface in the channel and the broken line connects the readings of the piezometer heights, the height of each piezometer being recorded vertically above the piezometer opening. The difference between the height of the piezometer surface and the height of the water surface represents mainly the excess or deficiency of pressure due to the impact of the flowing water. In this difference is also included the effect of air in the water of the channel, as explained on page 34. The locations of excess pressure, in general, occurred on the bottom near the foot of the ogee crest and on the rear side just above the bottom. These are the points at which the stream of water, first falling over the crest and then flowing across the channel bottom, is forced to change its direction of flow abruptly. This excess pressure is the reaction of the force exerted on the moving water to cause the change in direction of flow. In general the maximum intensity of excess pressure is a greater proportion of the depth in the channel for low flows, since the drop of the


STA. 1+83
Q= 40,000 SEC. FT.
STA. 1+83
Q= 80,000 SEC. FT.
STA. 1+83
Q=120,000 SEC. FT.
STA. 1+83
Q=160,000 SEC. FT.
STA. 1+83
Q=200,000 SEC. FT.
STA. 3+67
Q= 40,000 SEC. FT.
STA. 3+67
Q= 80,000 SEC. FT.
STA. 3+67
Q=120,000 SEC. FT.
STA. 3+67
Q=160,000 SEC. FT.
STA. 3+67
Q=200,000 SEC. FT.
GATES DOWN ELEV. 1205.4-VARIOUS DISCHARGES
STA. 1+83
GATES DOWN ELEV. 1205.4
STA. 1+83
GATES 1/4 UP ELEV. 1209.7
STA. 1+83
GATES 1/2 UP ELEV. 1213.6
STA. 1+83
GATES 3/4 UP ELEV. 1217.9
STA. 3+67
GATES DOWN ELEV. 1205.4
STA. 3+67
GATES 1/4 UP ELEV. 1209.7
STA. 3+67
GATES 1/2 UP ELEV. 1213.6
STA. 3+67
GATES 3/4 UP ELEV. 1217.9
CONSTANT DISCHARGE Q=80,000 SEC. FT.-GATES AT VARIOUS ELEVATIONS
SCALE OF MODEL
IN.
FEET
FEET
PROTOTYPE
EXPLANATION
Piezometric readings
Water surface
Piezometer location
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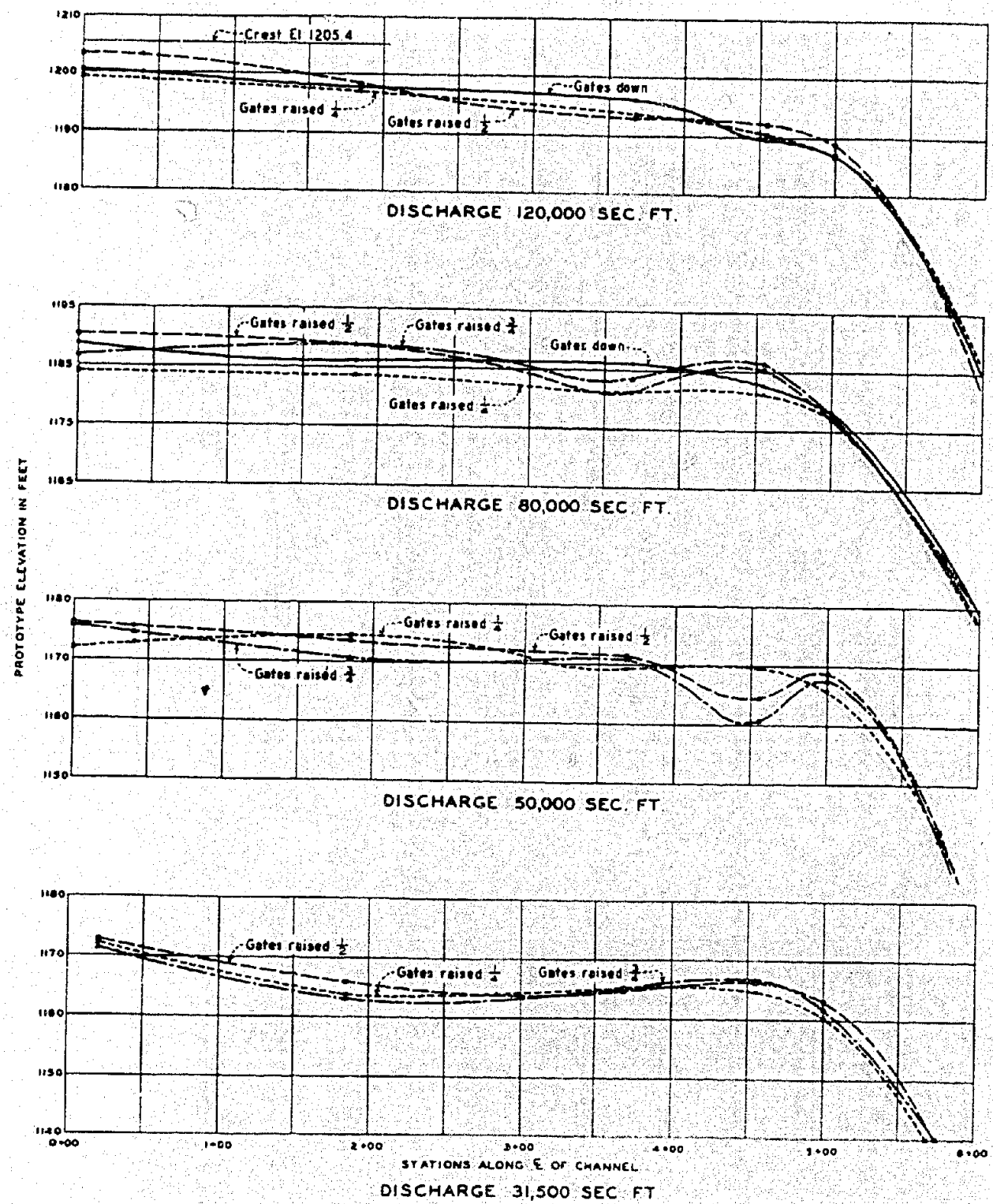
BOULDER CANYON PROJECT
HOOVER DAM HYDRAULIC EXPERIMENTS
RESULTS OF PIEZOMETRIC READING FOR
VARIOUS GATE POSITIONS AND DISCHARGES
MODEL M-3
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
DENVER, COLO. APRIL 1, 1933
45-D-3149
FIG. 48

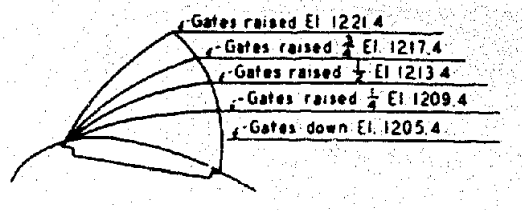
water passing over the weir was greater which caused a greater velocity and required a greater force to deflect it. The depth of water in the channel was also less for the low discharges, and therefore exerted less cushioning effect. The greater the depth of water, the more the stream tended to spread and the intensity of pressure was therefore smaller. This is no doubt also one reason why the maximum intensity of excess pressure is greater at the foot of the ogee than on the rear wall. Another contributing factor in this effect is the impact and friction losses of the water on the bottom of the channel which reduced the velocity of flow and therefore the pressure required to deflect the water.

One of the surprising results of the pressure observations is the existence of a reduction of pressure part way up the rear wall of the channel. This was not an observational error, since it appeared on all three models tested. The cause of this reduction in pressure is not apparent.

According to the theory of the side channel spillway, none of the energy of the water falling over the weir crest is available for causing flow down the channel. If this is the case, the mean water surface in the channel for a given discharge should be independent of the magnitude of the fall of the water coming in over the weir. That this was the case on the M-3 model is shown on Figure 47, on which are compared the mean surface profiles in the channel for various discharges with the drum gates in different positions. With the gates

FIG. 47





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HOOVER DAM HYDRAULIC EXPERIMENTS

COMPARISON OF CHANNEL WATER SURFACES PROFILES - MODEL M-3

GATES AT VARIOUS ELEV. WITH SAME DISCHARGES

DRAWN W.F.B. SUBMITTED E.F. Lane
TRACED R.S.W. RECOMMENDED
CHECKED R.F.B. APPROVED

DENVER COLO. MAR 27 1933 45-D-2879

raised the reservoir water levels were higher and therefore the falls to the channel level were greater. The profiles on Figure 47 show no consistent variation of level with position of the gate, which tends to substantiate the Hinds theory.

Similarity of Model and Prototype Results

The purpose of these studies of the Boulder Dam spillways was to develop the best form of spillway for the unusual conditions existing at that structure. The problem was attacked by means of hydraulic model tests, and the design developed was the one which these tests indicated would perform most satisfactorily within practical cost limitations.

The soundness of the conclusions reached however depends upon the accuracy with which the action of the prototype can be predicted from observations on models. This point has been the subject of considerable controversy, and although confidence in model testing results is rapidly growing, in view of the unprecedented nature of the structures involved, it was believed that some studies to further establish the reliability of model tests were also justified. This question can be investigated from two standpoints. One of these is from theoretical reasoning based on the application of the known laws of hydraulics and mechanics to the action of both model and prototype. The other is from the standpoint of experiment and observation. Because of the complexity of the problem it is not safe to depend entirely upon either one of these methods. Progress has been made in the past by using both.

The theory of model testing has been worked out in considerable detail, and excellent articles on it have appeared in many publications. These have very conclusively demonstrated the reliability of model tests, when certain practical limitations are kept in mind. In connection with these experiments considerable attention was given to this theoretical phase of the subject, but within the field covered by these tests the theoretical considerations have already been so well worked out that little was added by the experiments to existing knowledge. The principal emphasis was placed on the other angle of attack, the experimental confirmation of the theoretical deductions. In this field it is believed that a considerable addition to existing knowledge has been made. It was, of course, impractical to make comparisons of tests on the model and prototype. By making tests however on models extending over a considerable range of sizes, if nearly identical results were secured on all of them, a further extension of size to that of the prototype could be expected to produce a similar result. By making the largest size models as near as practical to the prototype size, any uncertainty in the comparative action of model and prototype was reduced to a minimum.

Due to the fact that the results of these experiments are so largely a confirmation of the conclusions reached by the model, theory, rather than additions to the theory itself, and to the fact that those who have delved deeply into the model theory need little proof of the reliability of model tests, the results of the experiments have been

presented without extensive discussions of model theory. In this form they are believed to be more useful to the engineer engaged in designing similar structures and more convincing to those who may not already be convinced of the reliability of model tests. It is believed however that those interested in the more theoretical phases of the subject will find the results of value, especially the quantitative comparisons of the results on the three sizes of final design models. For the benefit of those who are not familiar with, but would like to study the theoretical phases of model testing work, a bibliography of some of the best articles dealing with it is given in this report.

Similarity of Results on M-3, M-4 and C-4 Models

For hydraulic model tests to give the same results as the prototype, not only must the models be geometrically similar but there must also be similarity of the forces involved. In general, there are three types of these forces,* (1) inertia forces, (2) viscosity forces, and (3) gravity forces. It is not always feasible to obtain exact similarity of model and prototype, as it is often impractical to obtain an exact similarity of all of these forces. Usually however certain of these forces so predominate that the effect of not exactly duplicating the others is negligible from a practical standpoint. In the spillway models the viscosity forces were relatively insignificant. For such conditions geometrical similarity of the model and prototype

*The Use of Models in Aerodynamics and Hydro-dynamics. O. G. Tietjens. Trans. Am. Soc. M. E. Sept. 30, 1932, Vol. 54, No. 18, p.225

will insure comparable results. The only point in the spillway tests at which the viscosity forces were significant was in the model of the spillway tunnel at the Montrose laboratory. This case was taken care of by constructing the model tunnel to a grade which would compensate for the relatively greater viscosity effects of the model.

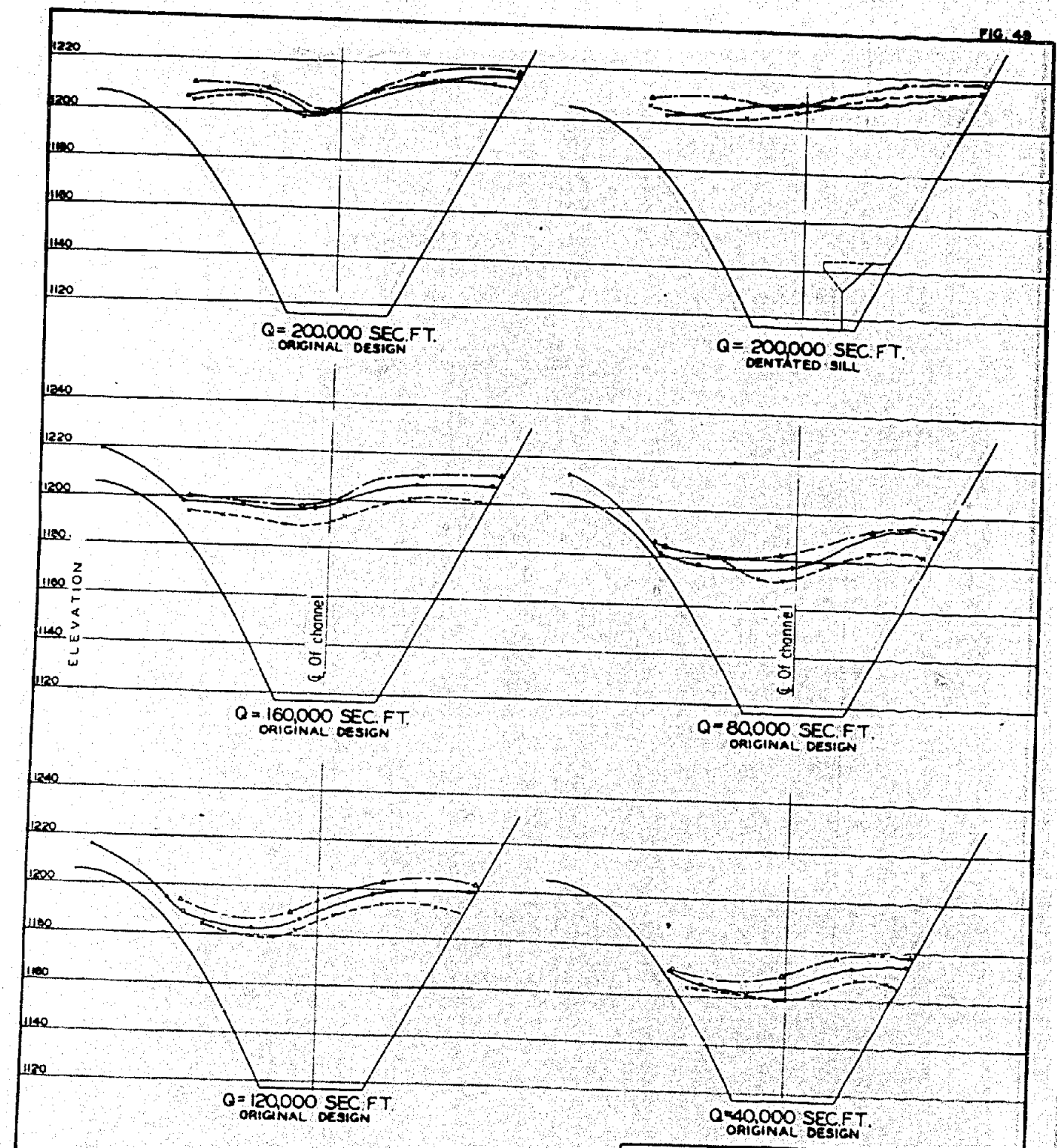
The similarity of the action of the M-3, M-4 and C-4 models is shown by the longitudinal profiles of mean water surface on Figure 43 and the typical cross sections on Figure 44. On the whole the agreement is very close. The 1:100 (M-4) model generally gave lower levels than the other two. In the tests on this model the discharges were not measured, but were estimated from the head on the model, using an assumed coefficient. For the lower discharges the value of the coefficient is now known to have been too high and consequently the discharges used were relatively less than on the other models, which probably accounts for the lower water surface levels obtained on the 1:100 models. This is also shown on Figure 48 which compares the water cross sections at Station 4+00 on all three models. This figure also shows the general similarity of the action of those models.

Similarity of Results on Final Design Models

M-5, C-5 and C-6

To further establish the reliability of predictions of the action of actual structures from the results of tests on models, extensive comparisons were made of the results obtained on the 1:20, 1:60 and 1:100 scale models of the final spillway design. In general,

FIG. 49



EXPLANATION

Symbol	Model Ratio	Model Number
———	1:20	M - 3
—△—△—	1:60	C - 4
- - - -	1:100	M - 4

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF CHANNEL CROSS SECTION
AT STATION 4+00
SEMIFINAL DESIGN AND DENTATED SILL

DRAWN W.M.B. SUBMITTED E.W. Lane
TRACED E.A.O. RECOMMENDED
CHECKED W.M.R. APPROVED
DENVER COLO. MAY. 23, 1933
45-D-3159

the results were very satisfactory and materially strengthen the case for the reliability of model tests. The results are shown in detail on Figs. 49 to 60.

Comparisons of the water levels for the various flows with the crest gates down are shown on Figs. 49 and 50. These show very close agreements of the water surface levels, except for a rise in the surface which seems to exist in the 1:20 model near the downstream end. The cause of this is not apparent and unfortunately time was not available for to study it in more detail.

Longitudinal profiles of comparable conditions on the 1:20 and 1:60 models with crest gates raised to various heights is shown on Figs. 51 and 53. Cross sections of the water prism in the channel for the same conditions are shown on Figs. 52, 54 and 55. The 1:100 model had no movable gates, and therefore it was not included in this study. The agreement in these cases was also good. The rise previously mentioned appears in these also.

On Fig. 56 is shown a comparison of the water surface for the same discharge with gates at various elevations, on each model separately. As previously explained, the agreement of the lines on a given model indicates the reliability of the assumption usually used in side channel spillway design that the entire energy of the overfalling water is dissipated without causing longitudinal flow in the spillway channel.

The pressures acting on the weir and in the side channel, as shown by the piezometric observations on the 1:20 and 1:60 models are

shown on Figs. 57 to 60. Considering the difficulties of obtaining consistent piezometric readings where high velocities and impacts are involved, the results are very consistent and comparable. In general they show the same pressure concentrations and reductions as the observations previously described on the M-3 model. The pressures on the weir and on the drum gates will be discussed in more detail in Book 4, of this report.

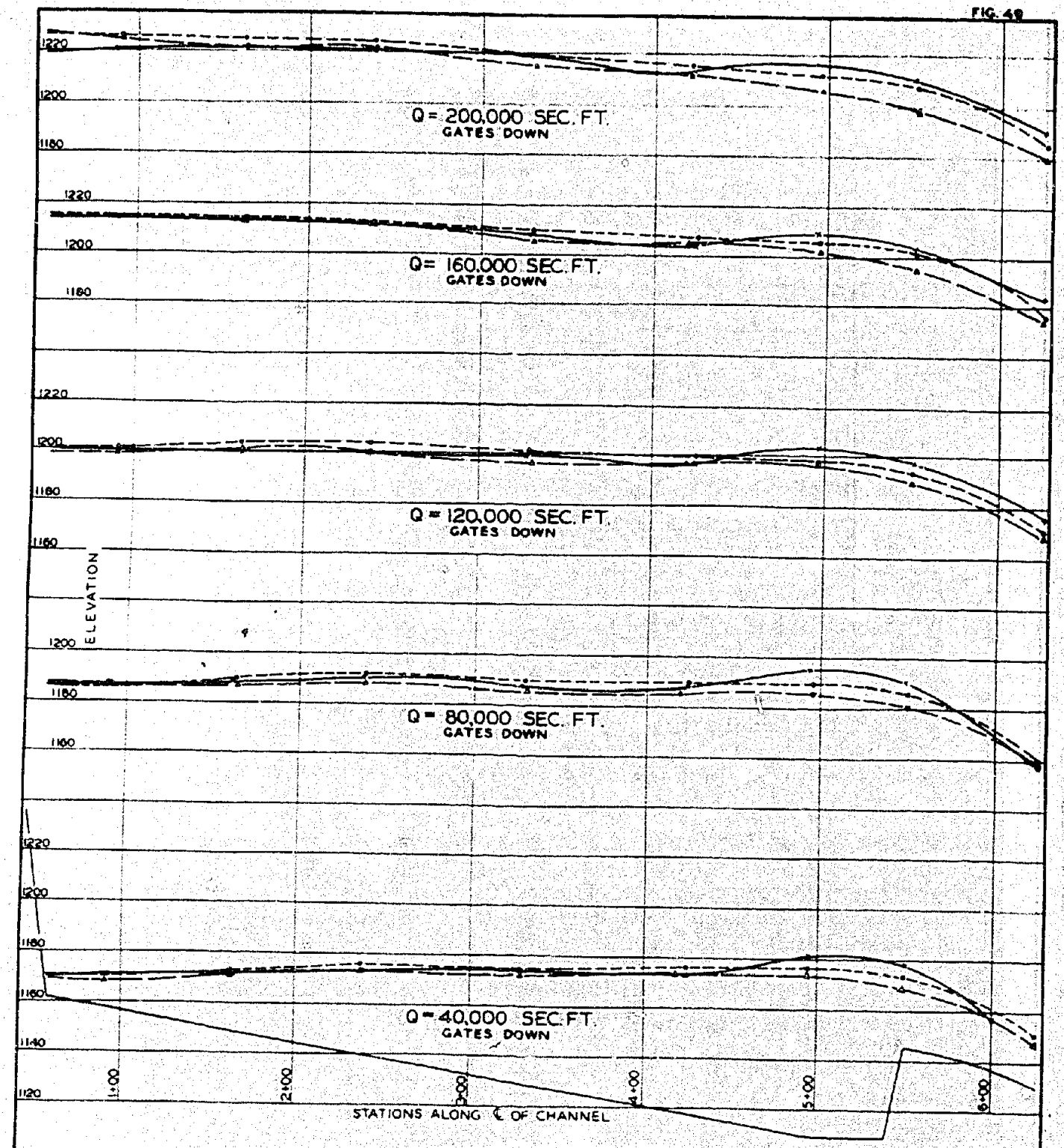
Final Gate Operating Program

A comparison of the action of the three gate operating programs studied in detail is shown on Plates 117 and 122. The similarity of action is shown by the close agreement of the water surfaces in all cases, both in the longitudinal profiles of mean water surface shown on Figure 61 and the cross sections shown on Figure 62. The only point at which the surface levels differ appreciably is at Station 4 + 25 for the 90,000 sec. ft. discharge. In this case the water surface for the program adopted rises somewhat higher than with the two other schemes. This is not believed to be important or especially undesirable.

Flow in the Spillway Transition Sections

The section of the side channel spillways in which the change was made from the trapezoidal section of the side channel to the circular section of the tunnel, referred to in this report as the "transition," was a transition section not only from the standpoint of the shape of the channel but also from the standpoint of direction of flow,

FIG. 49



EXPLANATION

Symbol	Model ratio	Model number
———	1:20	M-5
- - - -	1:60	C-5
—△—	1:100	C-1

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF WATER SURFACE PROFILES IN FINAL DESIGN SPILLWAY DRUM GATES DOWN

DRAWN W.M.B.	SUBMITTED	E. M. Lane
TRACED B.A.O.	RECOMMENDED	
CHECKED W.M.B.	APPROVED	

DENVER COLO., MAY 24 1933. 45-D-3161

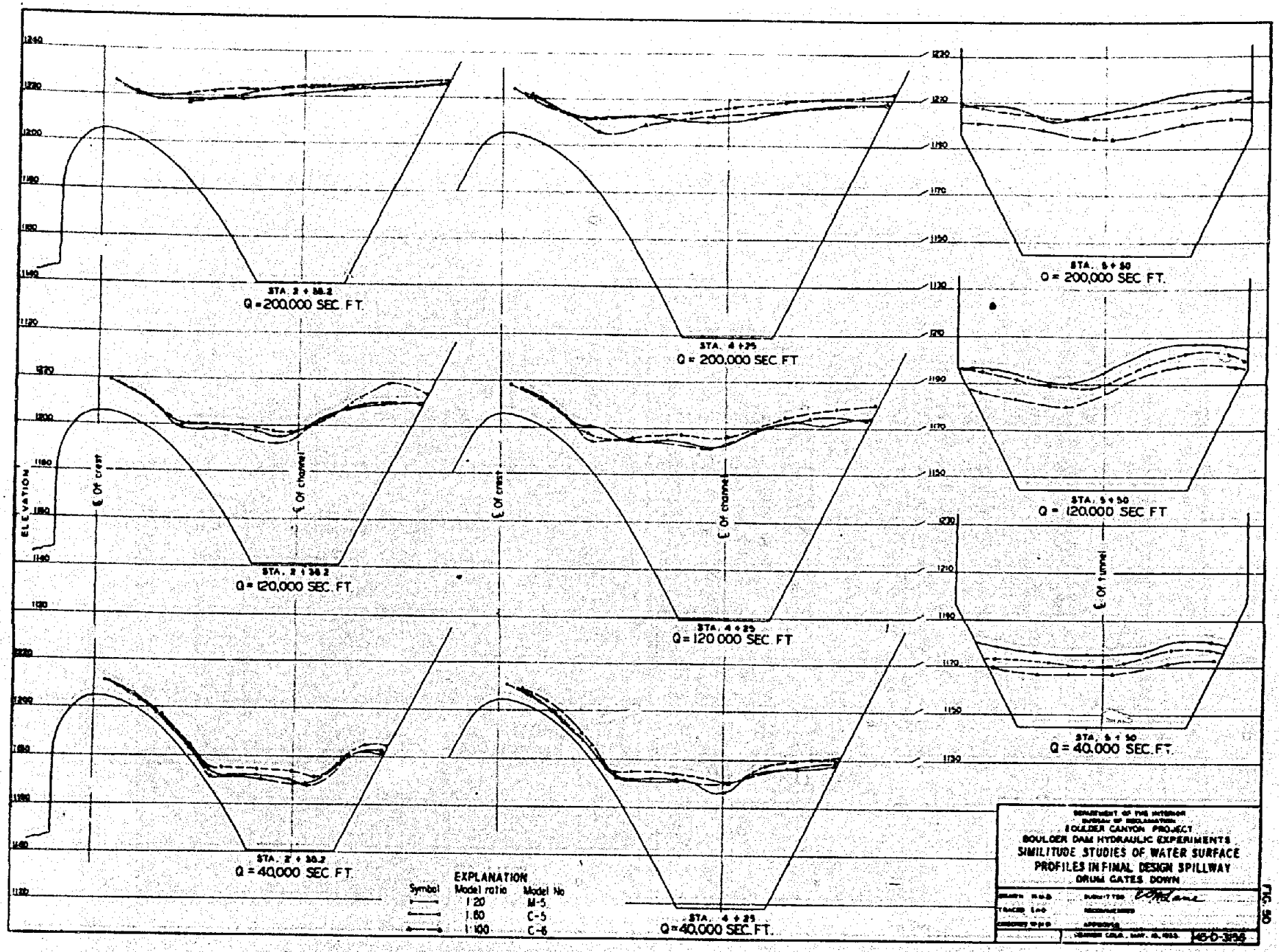
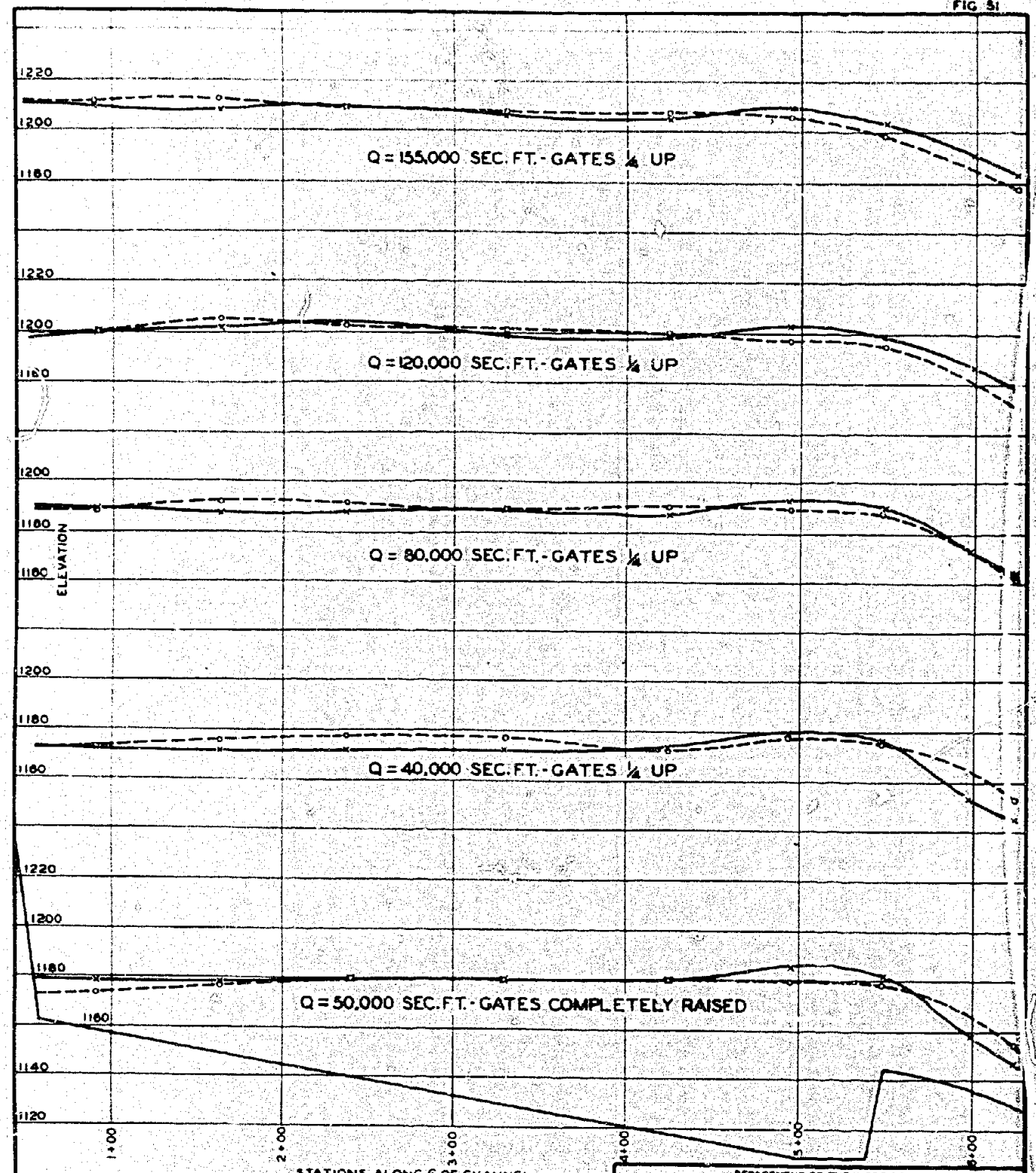

ELEVATION
℄ Of crest
℄ Of channel
℄ Of tunnel
STA. 2 + 38.2
Q = 200,000 SEC. FT.
STA. 4 + 25
Q = 200,000 SEC. FT.
STA. 5 + 50
Q = 200,000 SEC. FT.
STA. 2 + 38.2
Q = 120,000 SEC. FT.
STA. 4 + 25
Q = 120 000 SEC. FT
STA. 5 + 50
Q = 120,000 SEC. FT
STA. 2 + 38.2
Q = 40,000 SEC. FT.
STA. 4 + 25
Q = 40,000 SEC. FT.
STA. 5 + 50
Q = 40,000 SEC. FT.
EXPLANATION
Symbol
Model ratio
Model No
1:20
M-5
1:60
C-5
1:100
C-6
DEPARTMENT OF THE INTERIOR
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF WATER SURFACE
PROFILES IN FINAL DESIGN SPILLWAY
DRUM GATES DOWN
46-D-396

FIG. 85

FIG. 51



EXPLANATION

×———× Model ratio 1:20 M-5

o- - - - - o Model ratio 1:60 C-5

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

HOOVER DAM HYDRAULIC EXPERIMENTS

SIMILITUDE STUDIES OF WATER SURFACE PROFILES IN FINAL DESIGN SPILLWAY

DRAWN W.M.B. SUBMITTED E.M. Lane
TRACED A.J.H. RECOMMENDED
CHECKED W.M.B. APPROVED
DENVER, COLO. APRIL 3, 1933

45-D-3147

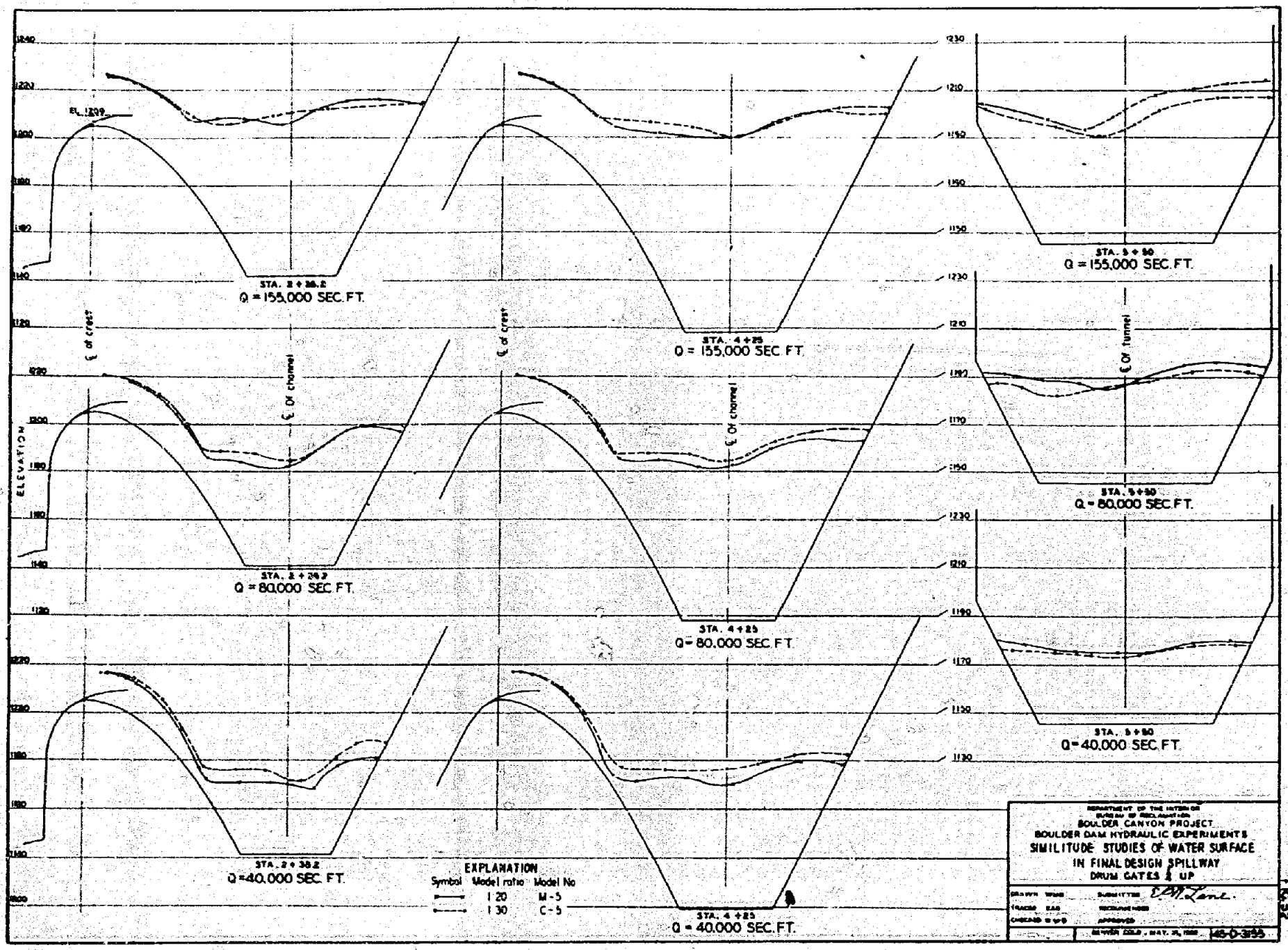
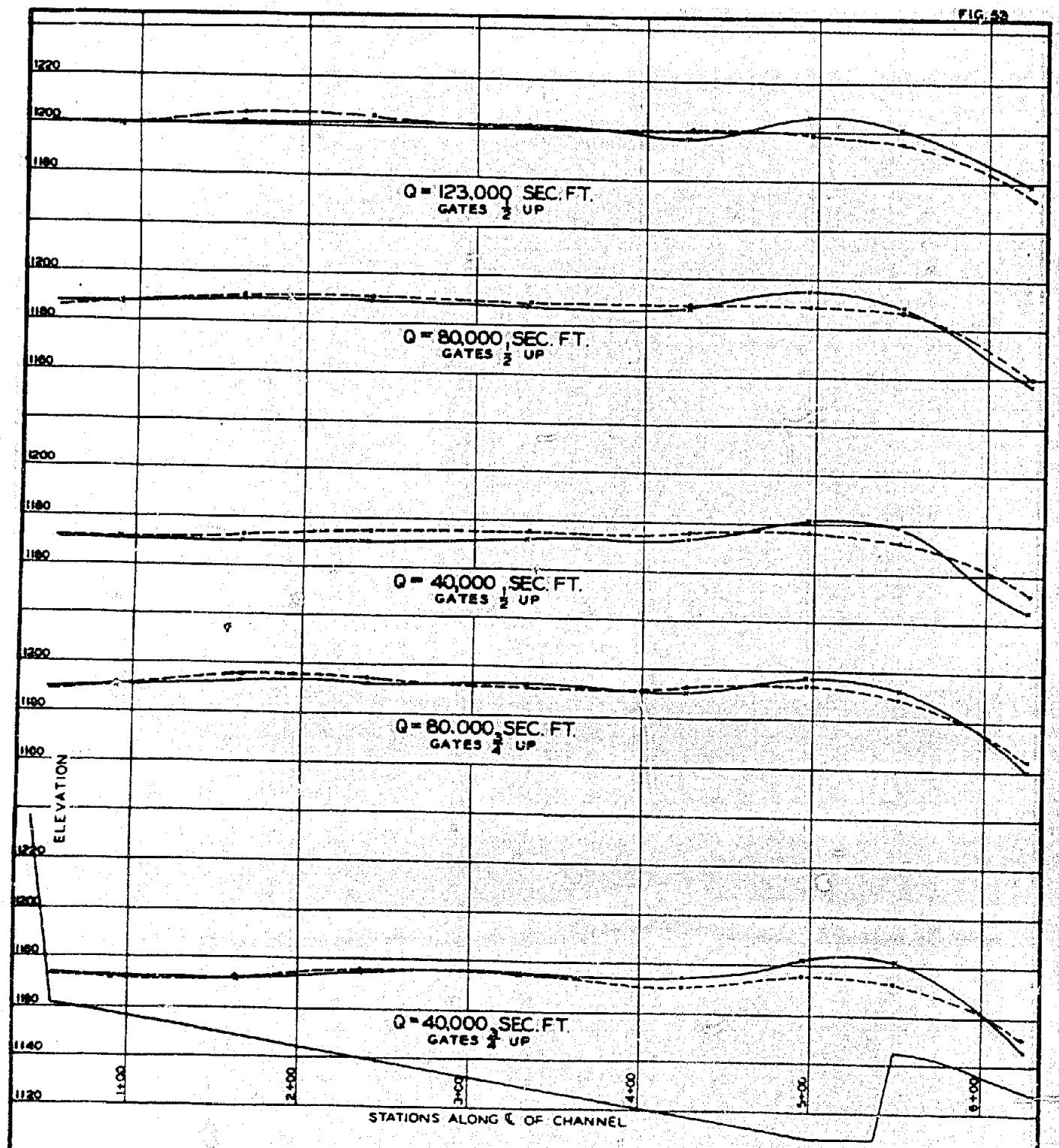

ELEVATION
℄ of crest
℄ of channel
℄ of tunnel
EL. 1239
STA. 2 + 38.2
Q = 155,000 SEC. FT.
STA. 4 + 25
Q = 155,000 SEC. FT.
STA. 5 + 50
Q = 155,000 SEC. FT.
STA. 2 + 24.2
Q = 80,000 SEC. FT.
STA. 4 + 25
Q = 80,000 SEC. FT.
STA. 5 + 50
Q = 80,000 SEC. FT.
STA. 2 + 38.2
Q = 40,000 SEC. FT.
STA. 4 + 25
Q = 40,000 SEC. FT.
STA. 5 + 50
Q = 40,000 SEC. FT.
EXPLANATION
Symbol Model ratio Model No
1:20 M-5
1:30 C-5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF WATER SURFACE
IN FINAL DESIGN SPILLWAY
DRUM GATES UP
DENVER, COLO., MAY 31, 1935
46-D-355

FIG. 52

FIG. 53



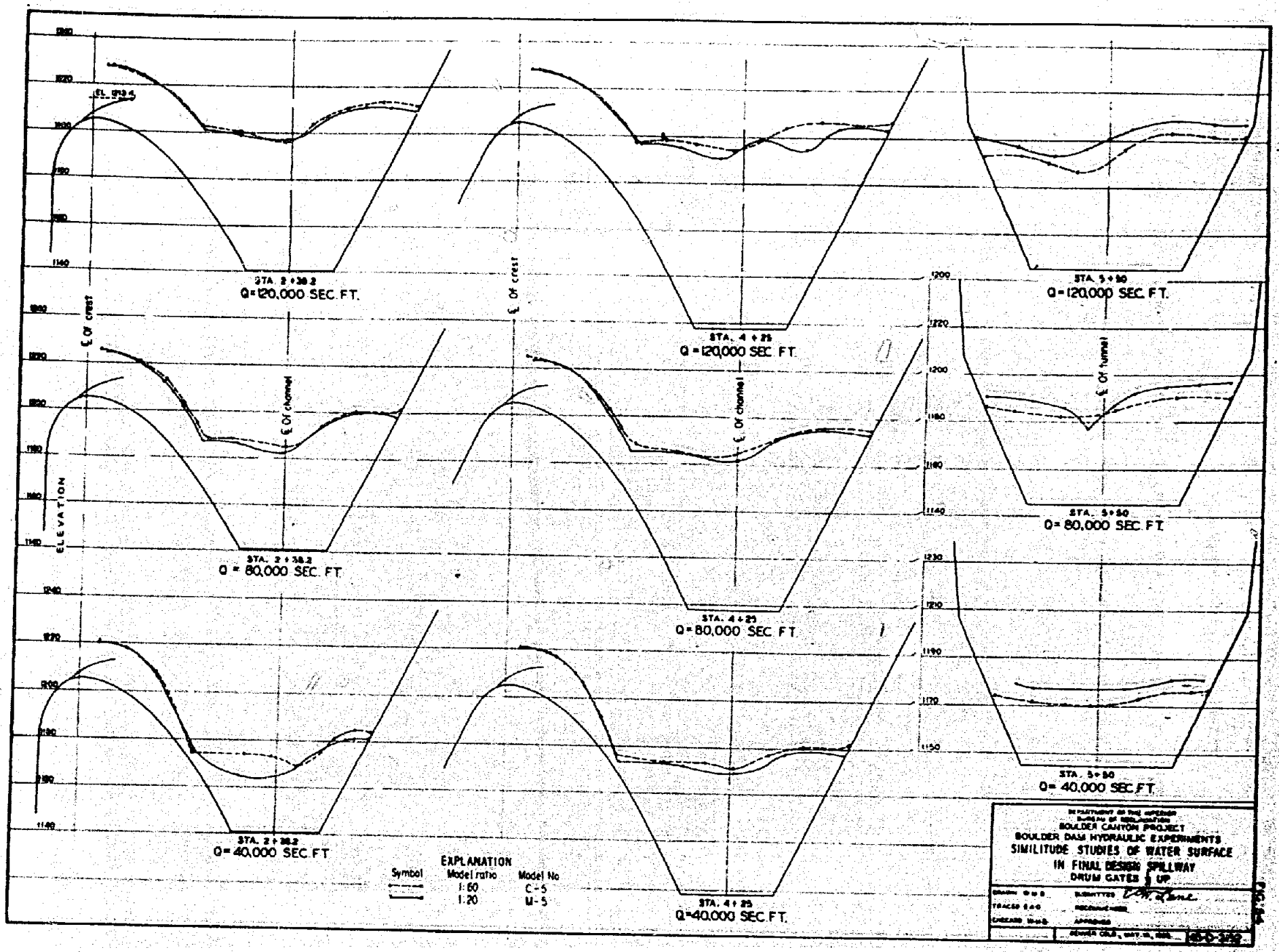
EXPLANATION

Symbol	Model ratio	Model number
———	1:20	M-5
- - - -	1:60	C-5

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF WATER SURFACE PROFILES IN FINAL DESIGN SPILLWAY
DRUM GATES 1/2 AND 3/4 UP

DRAWN W.M.B. SUBMITTED C.C. O'Leane
TRACED E.A.O. RECOMMENDED
CHECKED W.M.B. APPROVED

DENVER COLO., MAY 23, 1935 45-D-3162


ELEVATION
℄ Of crest
℄ Of channel
℄ Of tunnel
STA. 2+38.2
Q=120,000 SEC. FT.
STA. 4+25
Q=120,000 SEC. FT.
STA. 5+50
Q=120,000 SEC. FT.
STA. 2+38.2
Q=80,000 SEC. FT.
STA. 4+25
Q=80,000 SEC. FT.
STA. 5+50
Q=80,000 SEC. FT.
STA. 2+38.2
Q=40,000 SEC. FT.
STA. 4+25
Q=40,000 SEC. FT.
STA. 5+50
Q=40,000 SEC. FT.
EXPLANATION
Symbol
Model ratio
Model No
1:60
C-5
1:20
M-5
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF WATER SURFACE
IN FINAL DESIGN SPILLWAY
DRUM GATES UP

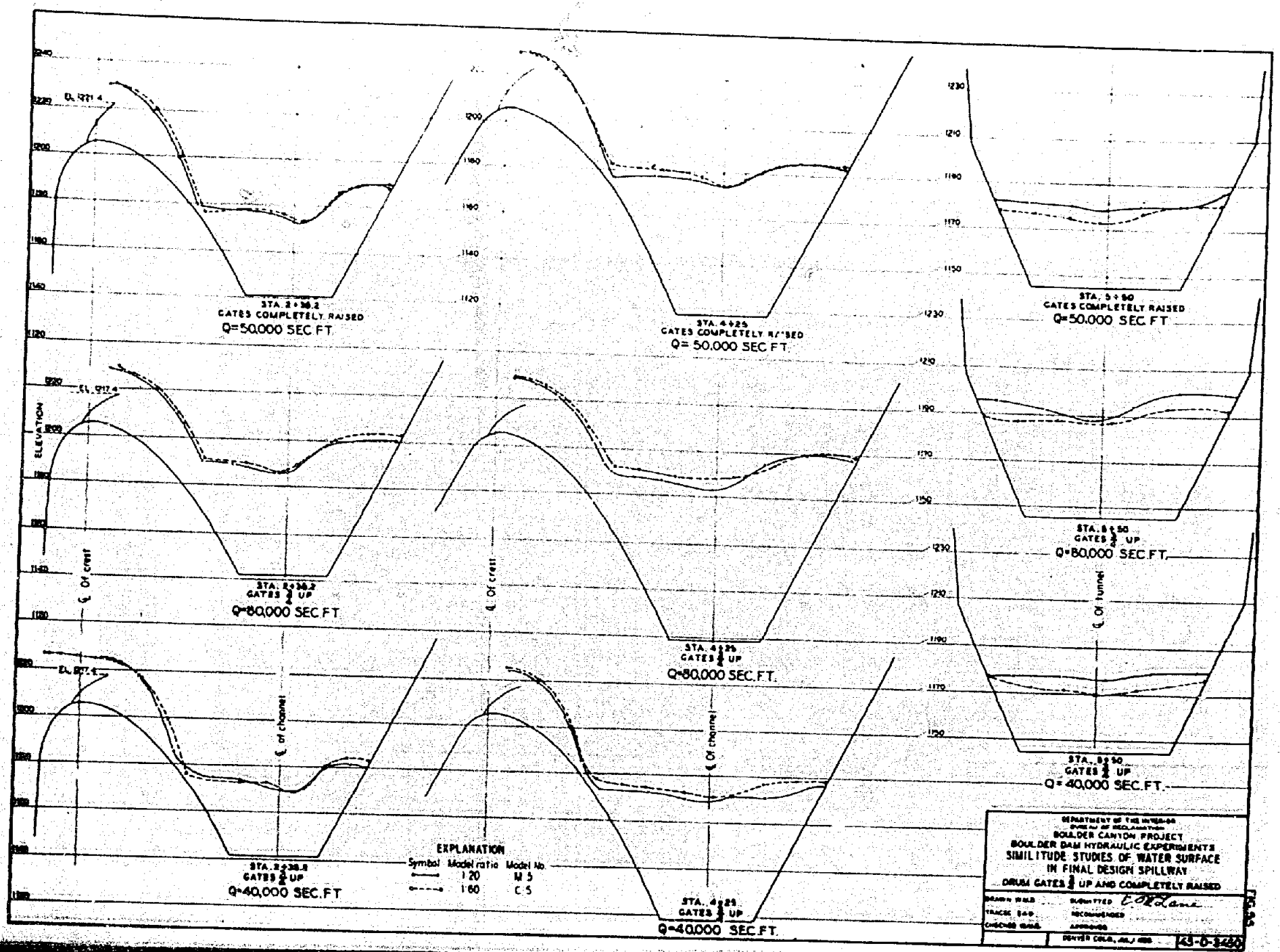
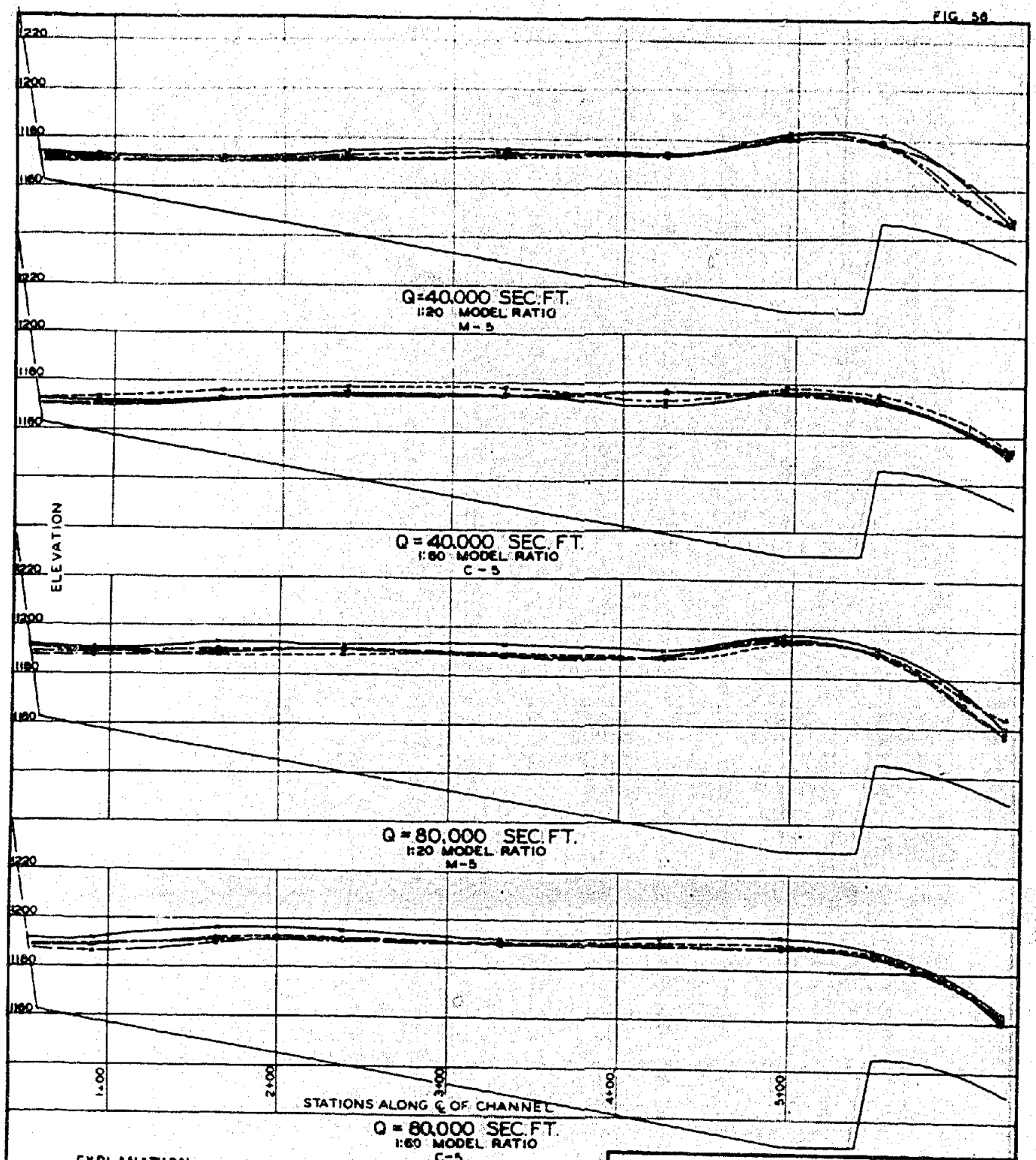

ELEVATION
EL. 1221.4
EL. 1217.4
EL. 1207.4
℄ Of crest
℄ Of channel
℄ Of tunnel
STA. 2+38.2
GATES COMPLETELY RAISED
Q=50,000 SEC.FT.
STA. 4+25
GATES COMPLETELY RAISED
Q=50,000 SEC.FT.
STA. 5+50
GATES COMPLETELY RAISED
Q=50,000 SEC.FT.
STA. 2+38.2
GATES ½ UP
Q=80,000 SEC.FT.
STA. 4+25
GATES ½ UP
Q=80,000 SEC.FT.
STA. 5+50
GATES ½ UP
Q=80,000 SEC.FT.
STA. 2+38.2
GATES ½ UP
Q=40,000 SEC.FT.
STA. 4+25
GATES ½ UP
Q=40,000 SEC.FT.
STA. 5+50
GATES ½ UP
Q=40,000 SEC.FT.
EXPLANATION
Symbol Model ratio Model No.
1:20 M 5
1:60 C 5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF WATER SURFACE
IN FINAL DESIGN SPILLWAY
DRUM GATES ½ UP AND COMPLETELY RAISED
DRAWN W.H.B.
TRACED E.H.P.
CHECKED
SUBMITTED E.O. Lane
RECOMMENDED
APPROVED
DENVER COLO. JUL. 1935
43-0-3420

FIG. 56



EXPLANATION

Symbol	Gate Position
	Down
	Raised 1/4
	Raised 1/2
	Raised 3/4

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
WATER SURFACE PROFILES CONSTANT DISCHARGE
WITH DIFFERENT GATE ELEVATIONS
FINAL DESIGN SPILLWAY

DRAWN W.F.B. SUBMITTED E. Otto Lange
TRACED G.H.O. RECOMMENDED
CHECKED W.H.B. APPROVED
DENVER COLO., MAY 23, 1933. 45-D-3160

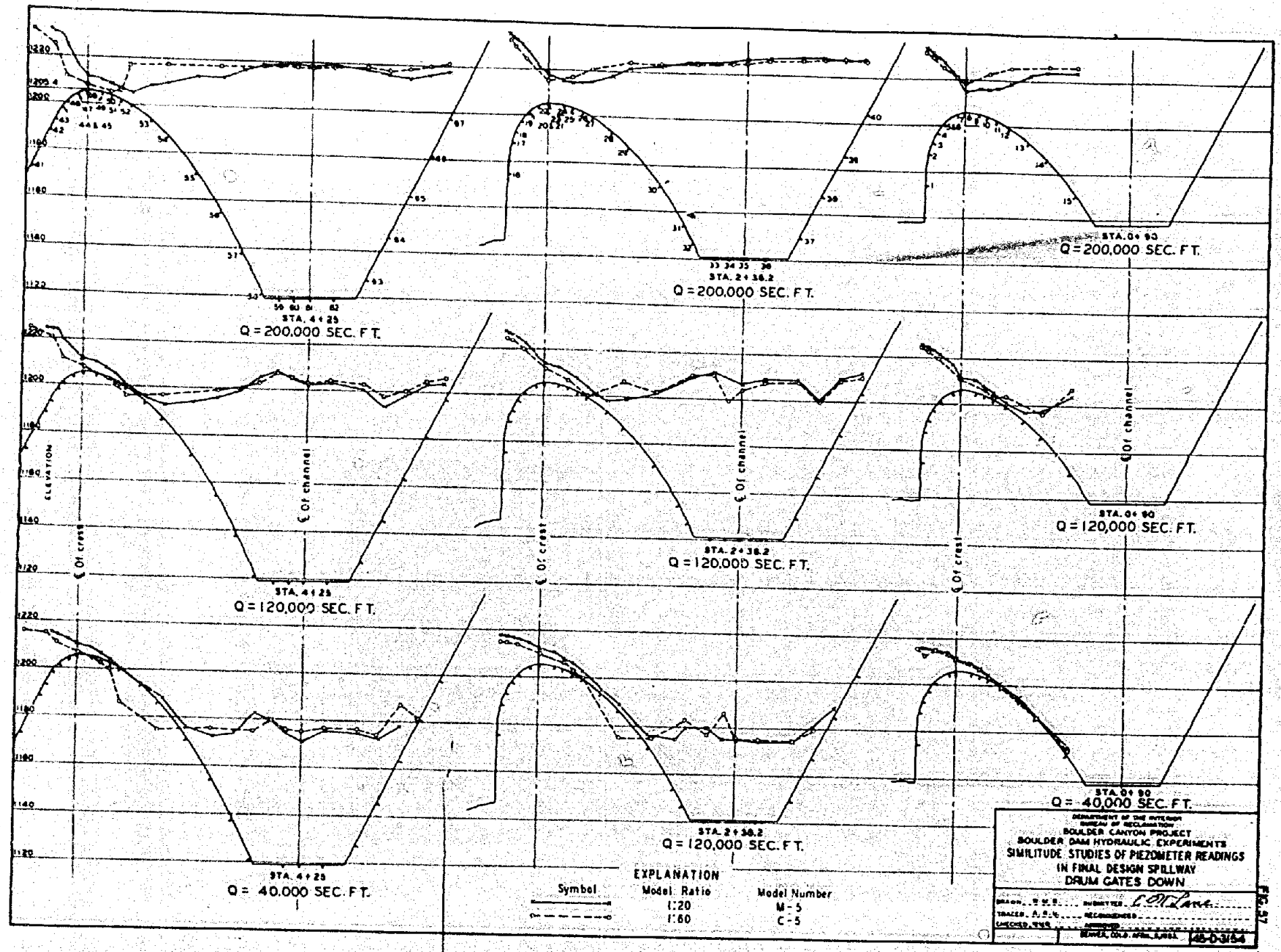
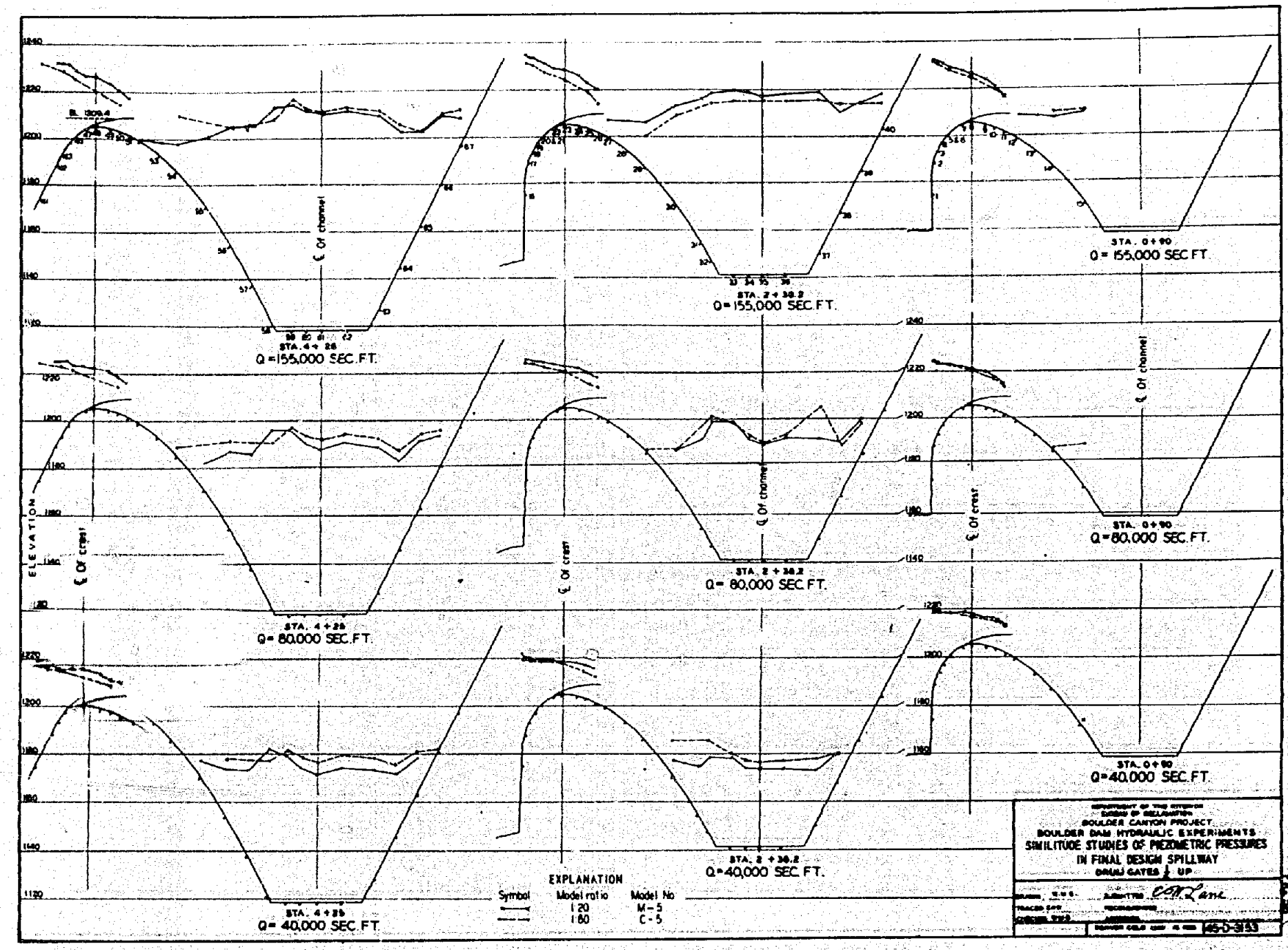
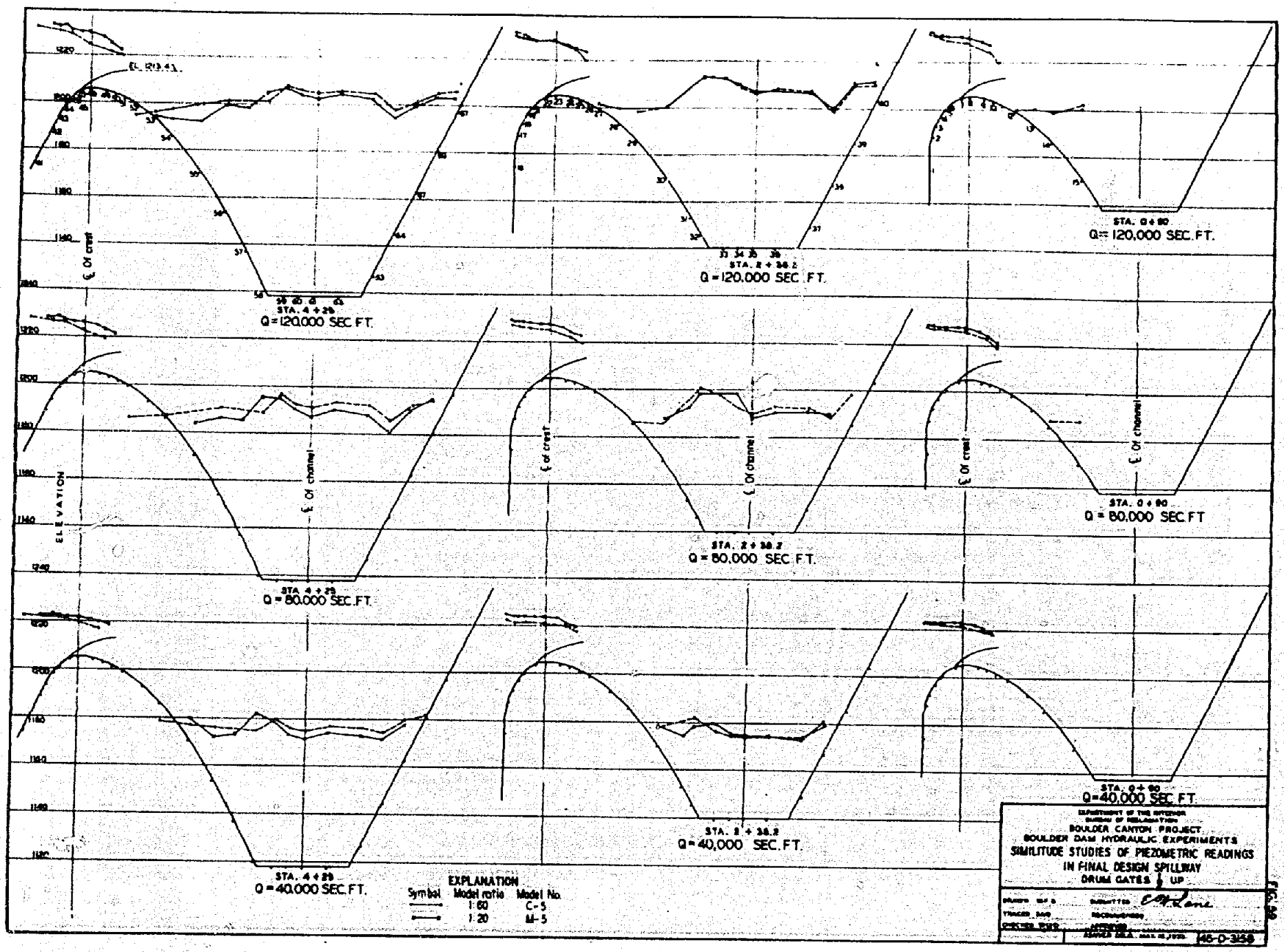

ELEVATION
℄ Of crest
℄ Of channel
STA. 4+25
Q = 200,000 SEC. F.T.
STA. 2+38.2
Q = 200,000 SEC. F.T.
STA. 0+90
Q = 200,000 SEC. F.T.
STA. 4+25
Q = 120,000 SEC. F.T.
STA. 2+38.2
Q = 120,000 SEC. F.T.
STA. 0+90
Q = 120,000 SEC. F.T.
STA. 4+25
Q = 40,000 SEC. F.T.
STA. 2+38.2
Q = 120,000 SEC. F.T.
STA. 0+90
Q = 40,000 SEC. F.T.
EXPLANATION
Symbol
Model Ratio
1:20
1:60
Model Number
M-5
C-5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF PIEZOMETER READINGS
IN FINAL DESIGN SPILLWAY
DRUM GATES DOWN
DRAWN R.W.B.
SUBMITTED C.O. Lane
TRACED A.B.C.
RECOMMENDED
CHECKED T.W.S.
APPROVED
48-D-3/54

FIG. 87


ELEVATION
℄ Of crest
℄ Of channel
STA. 4 + 25
Q = 155,000 SEC. FT.
STA. 2 + 30.2
Q = 155,000 SEC. FT.
STA. 0 + 90
Q = 155,000 SEC. FT.
STA. 4 + 25
Q = 80,000 SEC. FT.
STA. 2 + 30.2
Q = 80,000 SEC. FT.
STA. 0 + 90
Q = 80,000 SEC. FT.
STA. 4 + 25
Q = 40,000 SEC. FT.
STA. 2 + 30.2
Q = 40,000 SEC. FT.
STA. 0 + 90
Q = 40,000 SEC. FT.
EXPLANATION
Symbol
Model ratio
Model No
1:20
M-5
1:60
C-5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF PIEZOMETRIC PRESSURES
IN FINAL DESIGN SPILLWAY
DRUM GATES ↓ UP
45-D-353


EL. 1213.41
ELEVATION
℄ Of crest
℄ Of channel
STA. 4 + 29
Q = 120,000 SEC. FT.
STA. 2 + 38.2
Q = 120,000 SEC. FT.
STA. 0 + 90
Q = 120,000 SEC. FT.
STA. 4 + 29
Q = 80,000 SEC. FT.
STA. 2 + 38.2
Q = 80,000 SEC. FT.
STA. 0 + 90
Q = 80,000 SEC. FT.
STA. 4 + 29
Q = 40,000 SEC. FT.
STA. 2 + 38.2
Q = 40,000 SEC. FT.
STA. 0 + 90
Q = 40,000 SEC. FT.
EXPLANATION
Symbol Model ratio Model No.
1:60 C-5
1:20 M-5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF PIEZOMETRIC READINGS
IN FINAL DESIGN SPILLWAY
DRUM GATES UP
46-D-3458

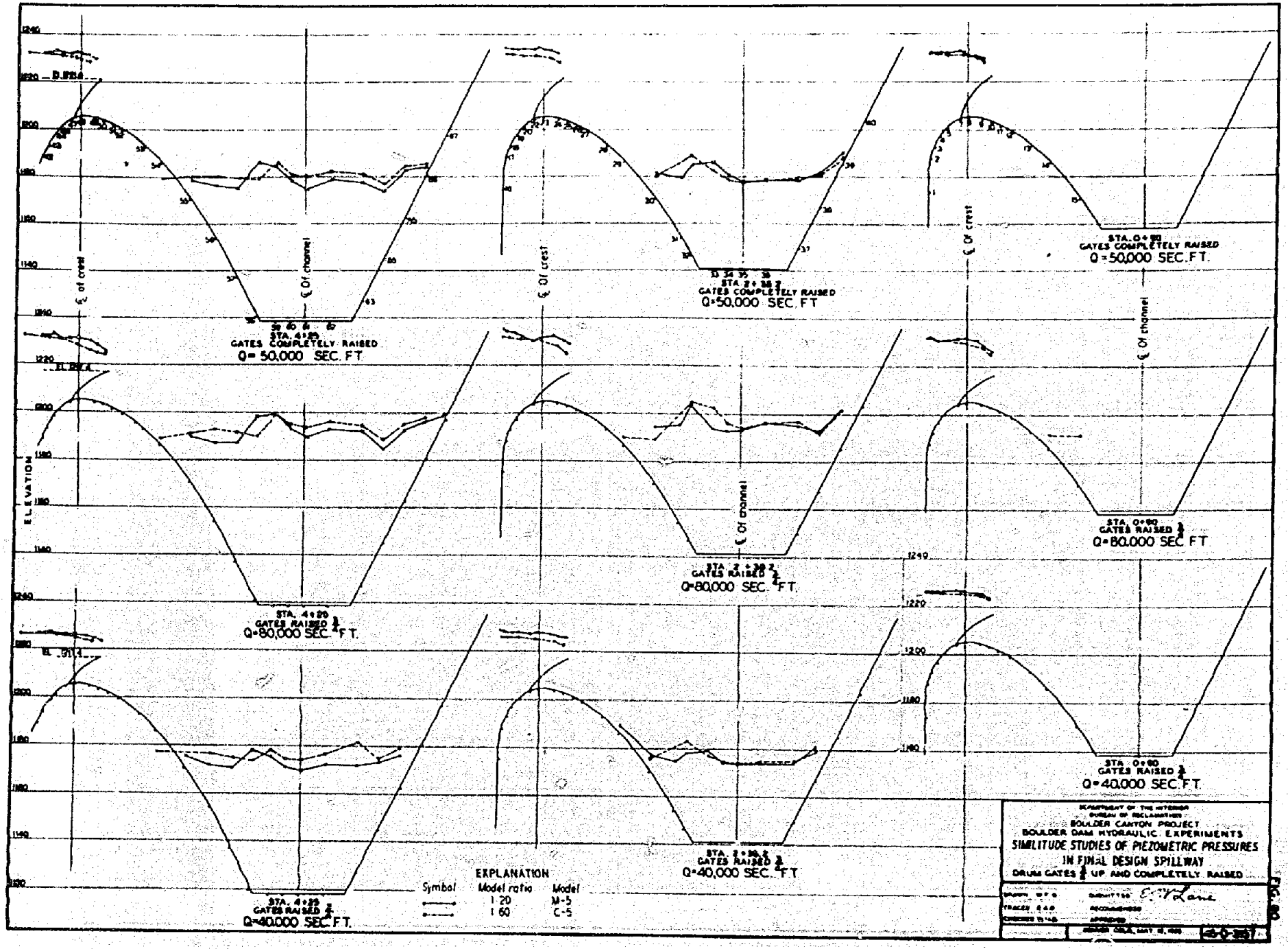
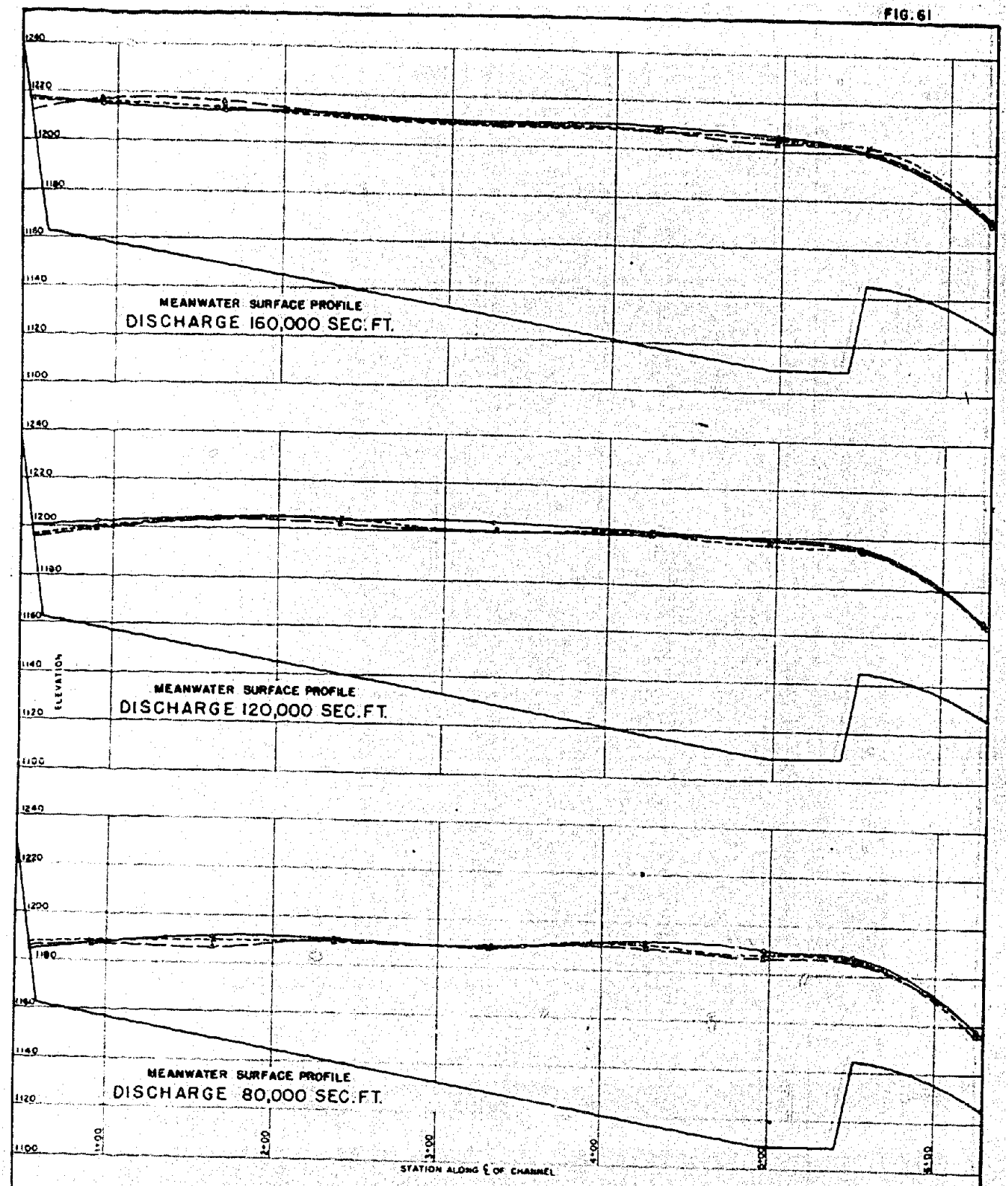

ELEVATION
℄ of crest
℄ of channel
STA. 4+29
GATES COMPLETELY RAISED
Q=50,000 SEC. FT
STA. 2+38.2
GATES COMPLETELY RAISED
Q=50,000 SEC. FT
STA. 0+80
GATES COMPLETELY RAISED
Q=50,000 SEC. FT.
STA. 4+29
GATES RAISED 3/4
Q=80,000 SEC. FT.
STA. 2+38.2
GATES RAISED 3/4
Q=80,000 SEC. FT.
STA. 0+80
GATES RAISED 3/4
Q=80,000 SEC. FT
STA. 4+29
GATES RAISED 3/4
Q=40,000 SEC. FT.
STA. 2+38.2
GATES RAISED 3/4
Q=40,000 SEC. FT
STA. 0+80
GATES RAISED 3/4
Q=40,000 SEC. FT.
EXPLANATION
Symbol
Model ratio
Model
1:20
M-5
1:60
C-5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
SIMILITUDE STUDIES OF PIEZOMETRIC PRESSURES
IN FINAL DESIGN SPILLWAY
DRUM GATES 3/4 UP AND COMPLETELY RAISED

FIG. 61



EXPLANATION

SYMBOL	PROGRAM	OPERATION
	A	Gate 1 & 4 are 5 ft. above gates 2 & 3
	B	Gate 1 is 5 ft. above gates 2, 3 & 4.
	C	All gates at same level. (Program adopted.)

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

BOULDER DAM HYDRAULIC EXPERIMENTS
COMPARISON OF GATE OPERATING PROGRAM IN FINAL DESIGN
MODEL 5-8

DRAWN W.R.S. SUBMITTED E.W. Lane
TRACED A.G.S. RECOMMENDED
CHECKED T.G.S. APPROVED

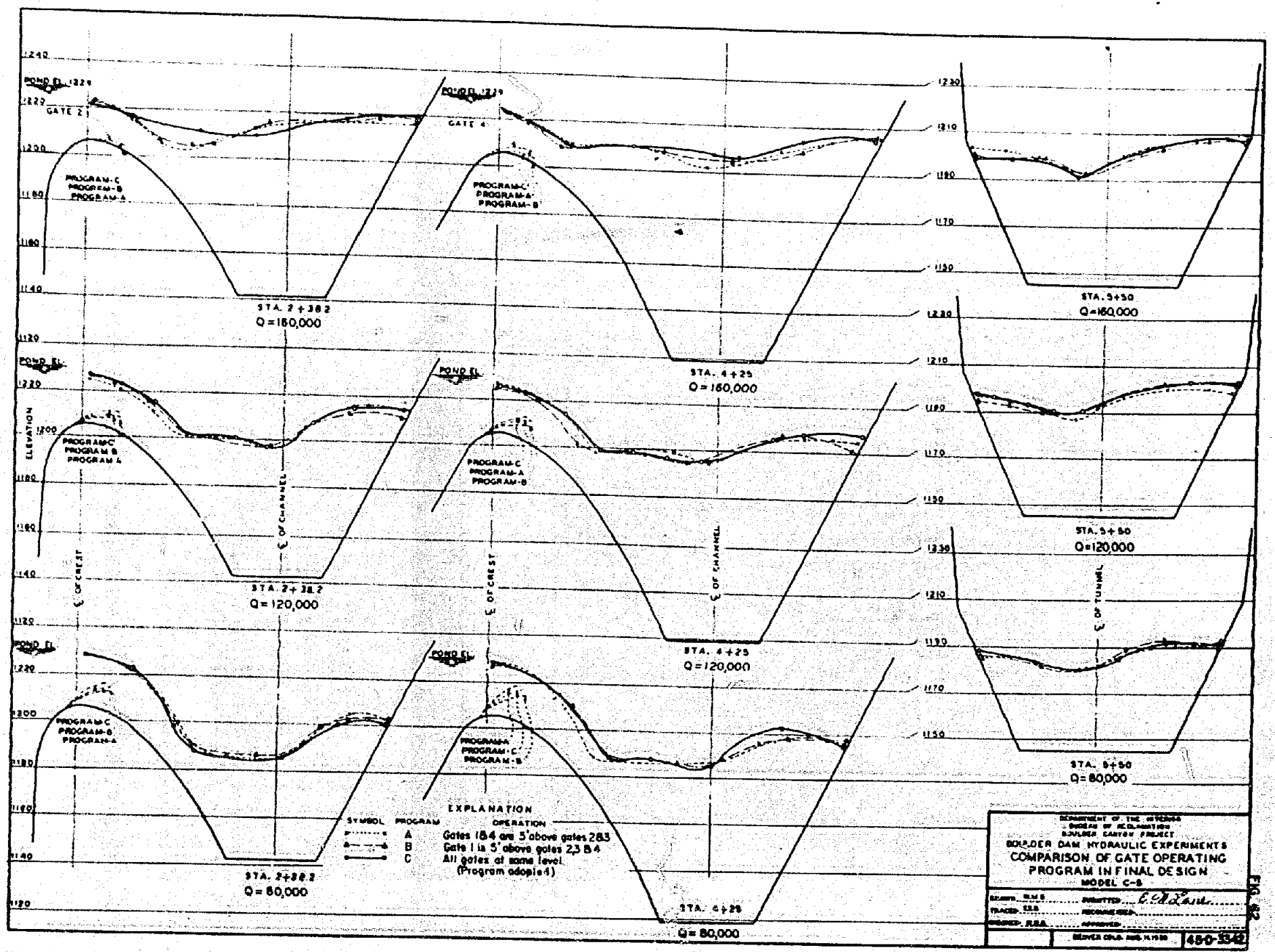

POND EL. 1229
GATE 2
GATE 4
PROGRAM-C
PROGRAM-B
PROGRAM-A
STA. 2+38.2
Q=160,000
STA. 4+25
Q=160,000
STA. 5+50
Q=160,000
STA. 2+38.2
Q=120,000
STA. 4+25
Q=120,000
STA. 5+50
Q=120,000
STA. 2+38.2
Q=80,000
STA. 4+25
Q=80,000
STA. 5+50
Q=80,000
ELEVATION
℄ OF CREST
℄ OF CHANNEL
℄ OF TUNNEL
1240
1230
1220
1210
1200
1190
1180
1170
1160
1150
1140
1130
1120
EXPLANATION
SYMBOL PROGRAM OPERATION
A Gates 1&4 are 5' above gates 2&3
B Gate 1 is 5' above gates 2,3 & 4
C All gates at same level
(Program adopted)
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
COMPARISON OF GATE OPERATING
PROGRAM IN FINAL DESIGN
MODEL C-5
DRAWN: R.M.S.
TRACED: E.L.B.
CHECKED: R.E.A.
SUBMITTED: E. W. Lane
RECOMMENDED:
APPROVED:
DENVER COLO. AUG. 14 1939
480-3542

FIG. 42

for in it was made the change from slightly inclined to steeply inclined flow. As the water entered this section with an initial velocity in a slightly inclined direction, the change to a steeply inclined direction could not be made suddenly without danger that the water would not remain in contact with the bottom of the channel, or might form partial vacuums between the water stream and the channel floor, which might make and break, causing hammering or pulsations. This situation is very similar to that of the flow over an ogee dam, but the undesirable results of such action might be more severe than for a dam, on account of being generated in a partially enclosed space. The slope of the transition bottom was therefore designed somewhat flatter than the trajectory of a freely falling stream having the initial velocity and direction of the water entering the section.

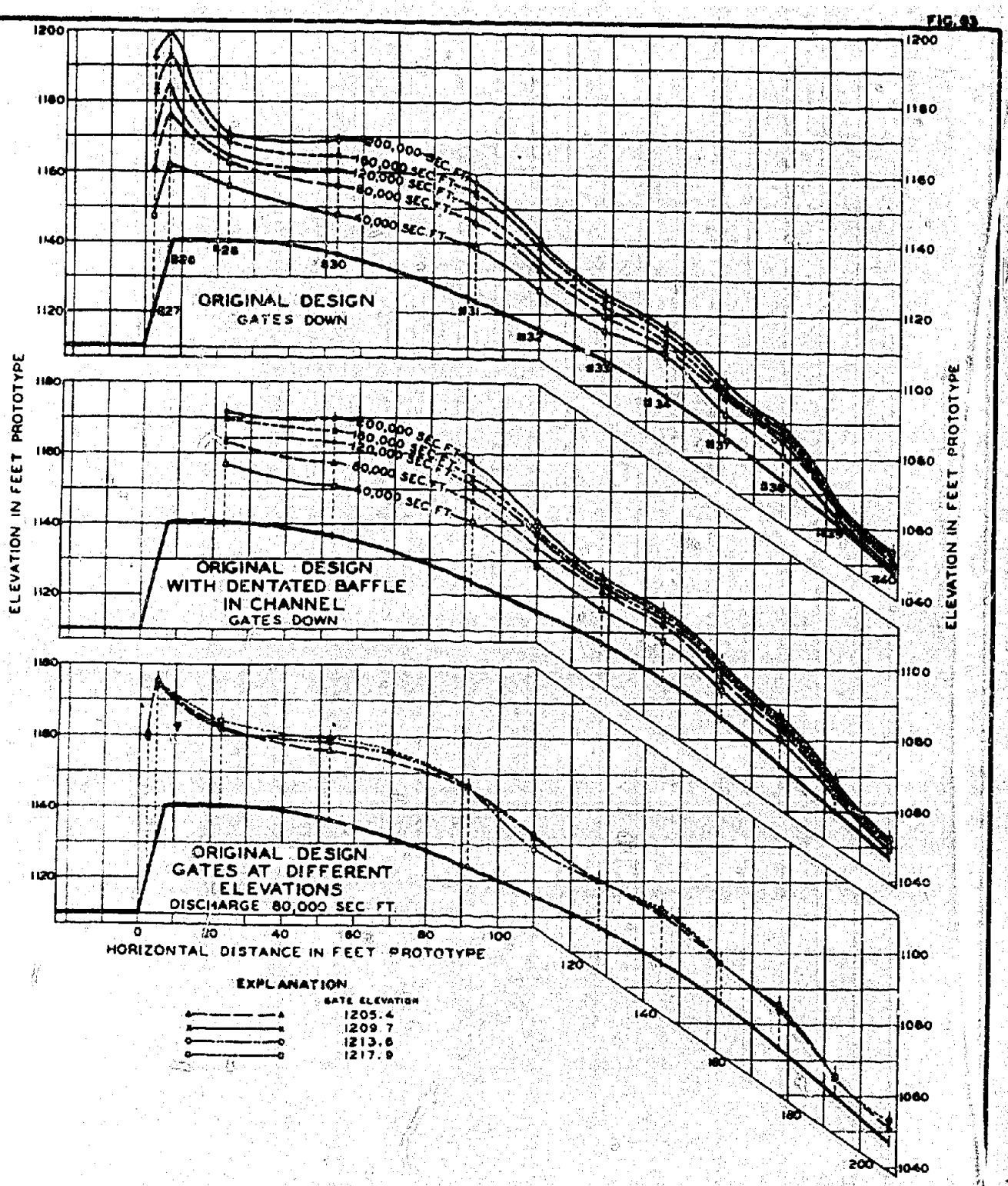
The action of the water in this case was somewhat complicated however by the contraction of the stream in a horizontal direction and by other uncertainties. In order to be sure that vacuum effects would be absent therefore, piezometers were installed in the bottom of the transition and the pressures were observed. Figure 37 shows the elevation of the water in open tubes connected to the piezometers at the positions indicated in the transition of the Stoney Gate Side Channel Model. The heavy line shows the elevation of the transition invert grade. The height of the piezometric levels above the invert grade gives the pressure of the water on the transition bottom and where piezometric heights are below the invert grade a partial vacuum is im-

dicated. The results indicate a slight vacuum at the lower end of the transition. The results at piezometer No. 37 are probably high, due perhaps to a defect in the piezometer opening. These observations were taken with the entire discharge passing through the Stoney gate. With part of the flow coming over the spillway crest, which is the normal operating condition, the initial velocity entering the transition would have been less and have resulted in a steeper trajectory, which would probably not have caused negative pressures.

The pressures on the transition bottom of the semi-final 1:20 model (M-3) as indicated by the piezometers along the center line are shown on Figure 63. The upper set show the results for various discharges on the original design, which included no baffles. The middle set shows the results with the dentated baffle and the lower set the effect on the original design with the same discharge and gates at various elevations.

The results show that under all conditions there are positive pressures on the transition bottom and therefore vacuum effects need not be expected. The high pressures in the sloping face at the entrance to the transition section was due to the velocity of approach of the water flowing in the channel section. The dentated baffles interferred with the piezometers at this point, and the pressures are therefore not shown by the middle set of results on Figure 63. That the pressures depended upon the flow, but were practically independent of the channel or weir crest conditions is indicated by the similarity

FIG. 83



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM HYDRAULIC EXPERIMENTS
PRESSURES ON BOTTOM OF TRANSITION
DRUM GATE SPILLWAY MODEL M-3

DRAWN W.F.B. SUBMITTED
TRACED S.W.C. RECOMMENDED
CHECKED W.F.B. APPROVED
DENVER, COLO. MAR. 15, 1938

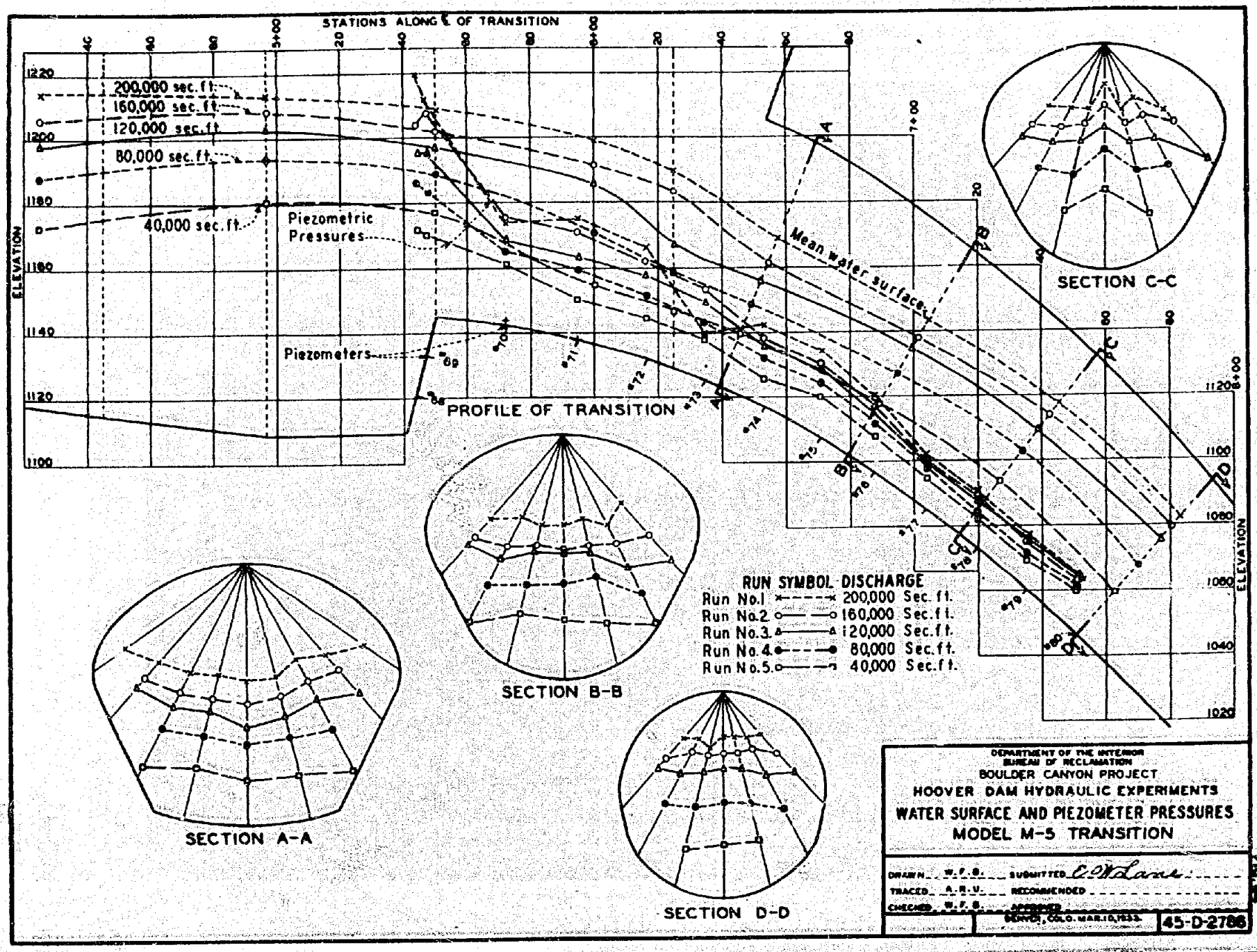
4-D-2795

of the results obtained with and without the dentated baffle, and the close agreement of the results for an 80,000 sec. ft. discharge for the drum gates held at various openings.

On Figure 64 is shown the water surface and piezometric line observed on the 1:20 final design model (M-5) for the gates down. The piezometric lines are considerably below their corresponding mean water surface lines since the water was accelerating under the action of gravity and did not have as much weight as normally, hence did not exert as much pressure. In all cases there was a positive pressure on the transition bottom. The water surface in directions normal to the flow are shown in the various cross sections given on Figure 64. Observations were also made on the surface levels and piezometric pressures for the complete range of flows with the drum gates at various positions. The results agree so closely with those obtained for the same discharge with the gates down that they are not given in this report.

Air Content in Side Channel Spillways

One of the uncertainties in the design of side channel spillways has been in regard to the air content of the water in the spillway channel. The flow over the spillway crest falls into the water in channel with considerable turbulence and carries into and mixes through the channel water quantities of air, which occupy space in the spillway channel and therefore necessitates that additional space be provided in


STATIONS ALONG ℄ OF TRANSITION
ELEVATION
200,000 sec. ft.
160,000 sec. ft.
120,000 sec. ft.
80,000 sec. ft.
40,000 sec. ft.
Piezometric Pressures
Piezometers
PROFILE OF TRANSITION
Mean water surface
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
RUN SYMBOL DISCHARGE
Run No.1 200,000 Sec. ft.
Run No.2 160,000 Sec. ft.
Run No.3 120,000 Sec. ft.
Run No.4 80,000 Sec. ft.
Run No.5 40,000 Sec. ft.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM HYDRAULIC EXPERIMENTS
WATER SURFACE AND PIEZOMETER PRESSURES
MODEL M-5 TRANSITION
DRAWN W.F.B.
TRACED A.R.V.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO. MAR. 10, 1933
45-D-2786

and the water was drawn off from the bottom. The water was measured volumetrically or by a water meter and the air volume was determined from the increase in the volume of the air in the tank, as observed on a gage glass on the side of the tank, care being taken to have the air in the tank at atmospheric pressure and at the same temperature whenever the volume was observed.

The third method was used in the final design spillway at Montrose, as a check on the second method. Through the windows in the rear wall of the spillway the size of the air bubbles and their average spacing was noted by several different observers. From this data the relative volume of water and air could be computed. In a similar manner as estimate was made of the air content in the highly disturbed flow at the contact between the overfalling nappe and the water in the channel, where the air content was probably a maximum.

Accuracy of Methods

The first method of measuring air content is based on the assumption that the flow of water in the channel in the vicinity of the piezometer is normal to the piezometer opening and that the flow is not accelerating. This is only approximately the case in the side channel spillway. Observations on piezometers located at various points in the channel described on page 29 indicate that high local pressures against the floor and walls exist at certain points. Whether or not such pressures would act at the location of the piezometers for measuring the air content is uncertain. If they did act, they would no doubt be of

lesser magnitude than the severe pressures shown on page 29 but the effect might be enough to cause appreciable error in the small difference in pressures from which the air content was determined. Near the lower end of the channel the water was accelerating and therefore did not exert as much pressure on the bottom as its depth and normal weight would indicate. This effect appeared in the difference between the two heights from which the air content was determined and produced serious errors, as the results at this point in some cases indicated negative air contents. In general it may be said that while this method might give good results under certain conditions, it was not very reliable for the conditions existing in the model spillways tested.

The second method should give reliable results if a representative sample is secured. Two of the possible sources of error were leaks in the air tank and the possibility that air escaped with the water discharged from the tank. The tank was tested by allowing it to remain long periods under pressure and since the pressure did not drop, it is believed that there was no leakage. No appreciable quantity of air escaped with the water, for when the outflow hose was placed in a vessel of water, no air bubbles could be seen coming from it.

The most representative sample would be secured if the orifice through which the sample was taken was so placed that it caused as little disturbance in the flow lines in its vicinity as possible. In these experiments the samples were drawn out through a 7/8-inch brass tube with its open end pointed upstream. If all the water flowing

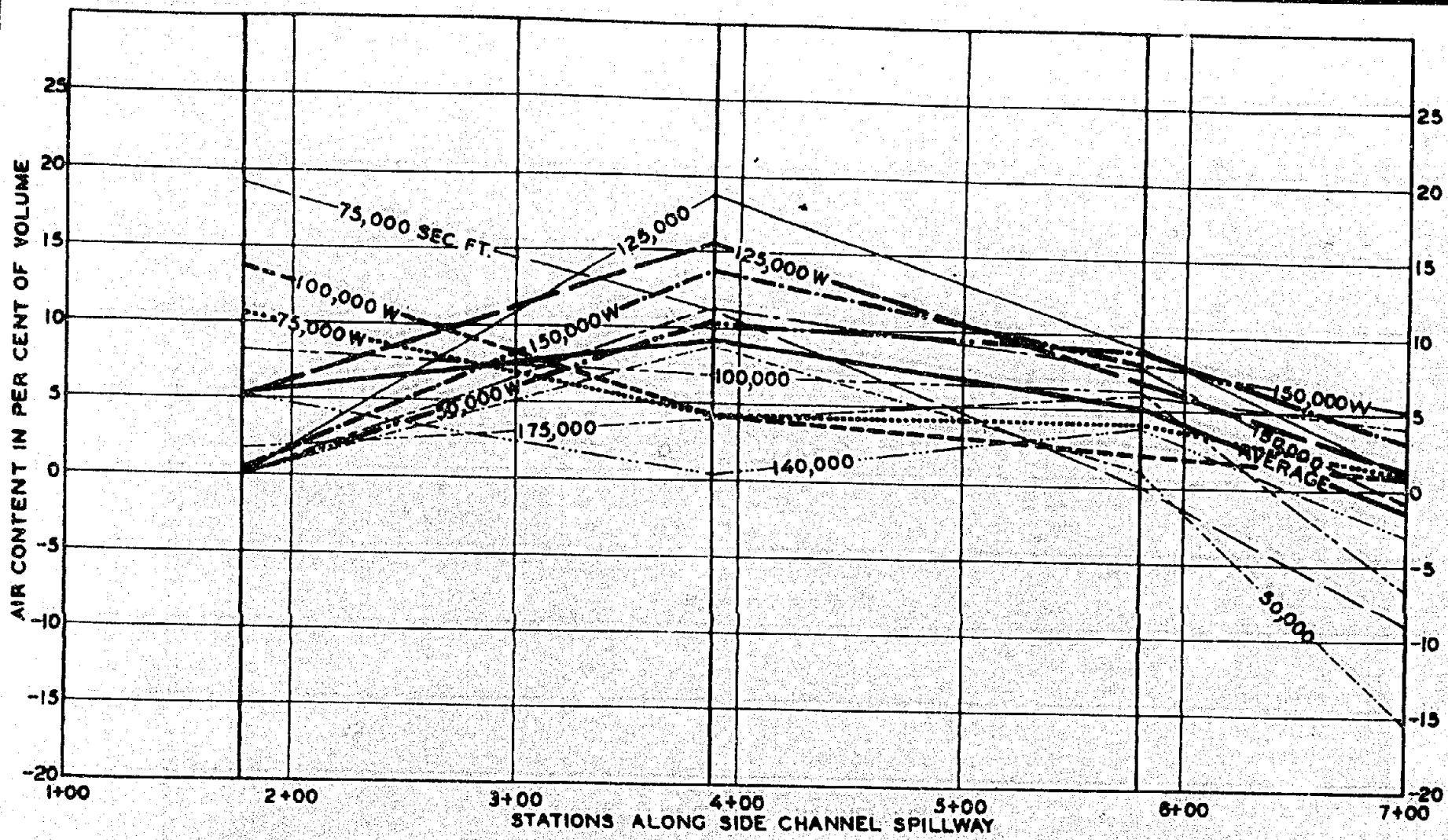
directly toward the end of the pipe was taken into the pipe it would cause no appreciable distribution. If more or less water was drawn in, some deflection of the stream lines would result. Since the air would have less mass per unit of volume than the water, it might be expected that the air would be more easily deflected than the water. If the sampling tube was taking in more of the mixture than was flowing directly towards its end, it would be drawing in the adjacent currents and might therefore draw in relatively more air than water. If it took in less water than was directly approaching its end, some fluid would be deflected away, and since the air would be more easily deflected than the water, relatively more air than water would be deflected and the sample would contain less air than it should. In the tests, no attempt was made to have the sampler take just the quantity of water which was directly approaching it, but the amount taken was believed to be sufficiently close to the desired amount to give results reasonably near the true conditions. In general it is believed that this method of measuring air content in the water was reasonably reliable and gave values accurate enough for practical purposes.

The third method, that of estimating the size and spacing of the air bubbles, can not be regarded as an accurate method, but was used as a check on the results obtained by the second method. By having a number of independent observers, a more reliable estimate was secured. It should be noted however that since the air contents were always low, a measurement recording only 75% of the true volume of air

would give a per cent of air content differing very little from the correct value. For this reason high accuracy of results was not necessary.

Results of Air Content Measurement

Observations were made by the first method on the air content in the first side channel spillway. (C-2 model) Piezometers were connected to the bottom of the channel on the center line of the channel at four points in its length. The per cent of air content was estimated from the difference between the elevation in the tubes connected with the piezometer openings and the average elevation of the water surface in the channel at that station. Observations were made with the trapezoidal channel (Figures 29 and 30) and with the weir across the channel Figure 31. The results of the estimates of air content obtained by this method is given on Figure 65. The results are quite variable, extending from +19% to -15%, with averages considerably higher than shown by the second method. It is possible that closer agreement would have been secured if the elevation of the surface in the channel had been taken as the water surface immediately above the piezometer opening, instead of the average surface level at the cross section. A distinct reduction of air content is indicated at the lower end of the channel, due no doubt to the reduction of pressure of the water on the bottom of the channel as it accelerated in starting down the inclined shaft. It is not believed that the results obtained by this method are as reliable as by the second method.



Note - The designation W indicate tests in which cross weir obstructions were placed in the channel

HOOVER DAM HYDRAULIC EXPERIMENTS

AIR CONTENT INSIDE CHANNEL SPILLWAY CHANNEL OF MODEL C-2

FIG. 65

E.W.C.

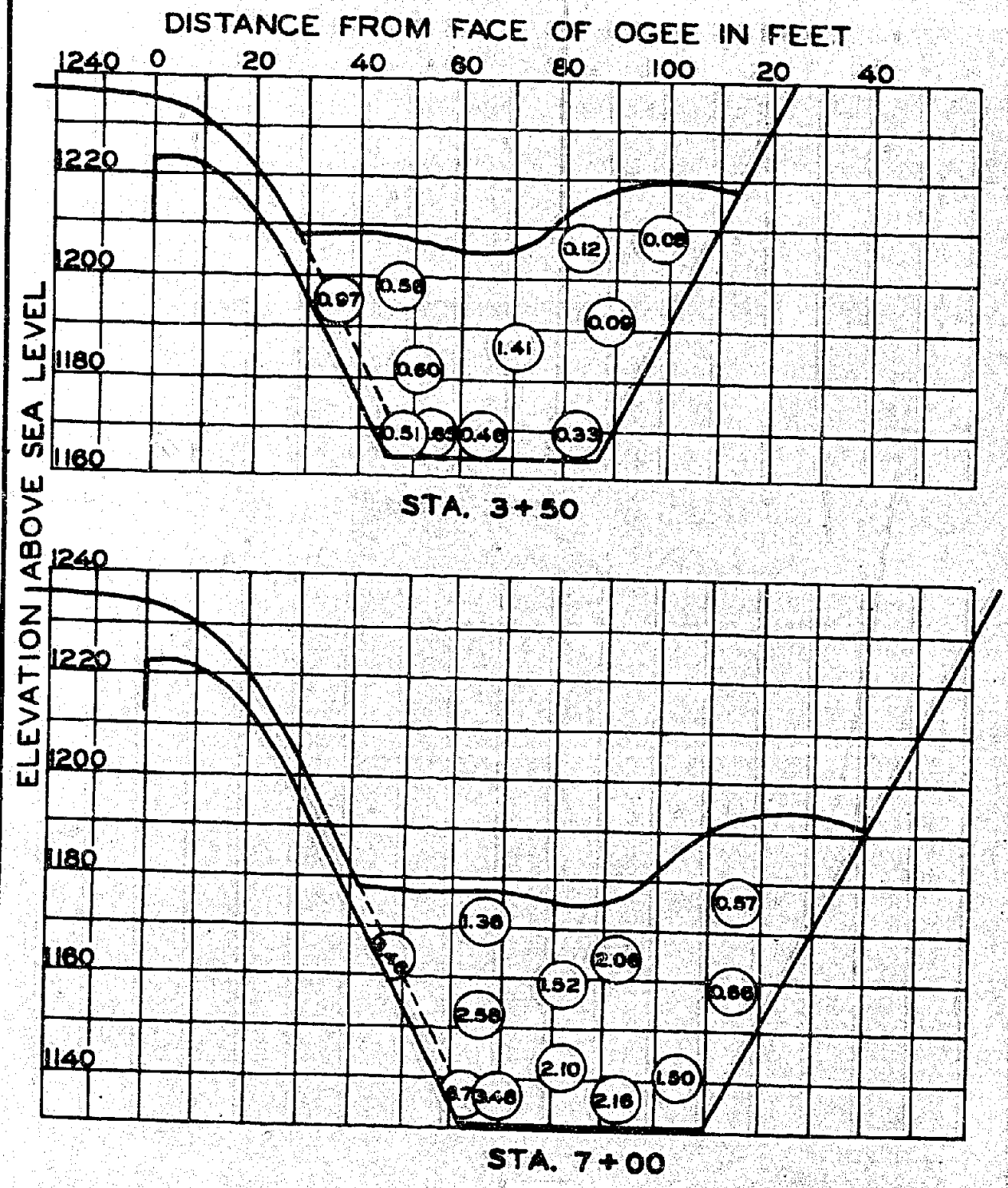
45-D-3451

Measurements of air content were made by the second method at two cross sections in the free crest side channel (Model C-2). The first one of these was at the middle of the length of the channel. The results are shown on Figure 66. The maximum air content at this section was 1.41%. This occurred in the center of the channel in the region of the core of air which ran through the center of the channel, as described on page 18 Book 1. The average content at this section was 0.5%. The second cross section was taken at Station 7+00, which is near the lower end of the channel. At this end of the channel the water passing over the weir crest fell a greater distance to the channel surface level, which caused more turbulence and greater air content. The maximum content in this case was 6.7% and the average was 2.2%. These contents are based on air at atmospheric pressure and are slightly larger than for the pressures which existed in the model.

On the M-3 model observations were made by the second method at three stations along the spillway, as shown on Figure 67. The results are not very definite, but tend to indicate more air toward the downstream end. The average discharge at the downstream section was the largest of the three and reached only 1.08%.

On the M-5 model observations were made by the second method at the downstream end of the channel for discharges of 200,000, 160,000 and 120,000 sec. ft. Here again the results were not definite, but the largest average content was only 1.09%.

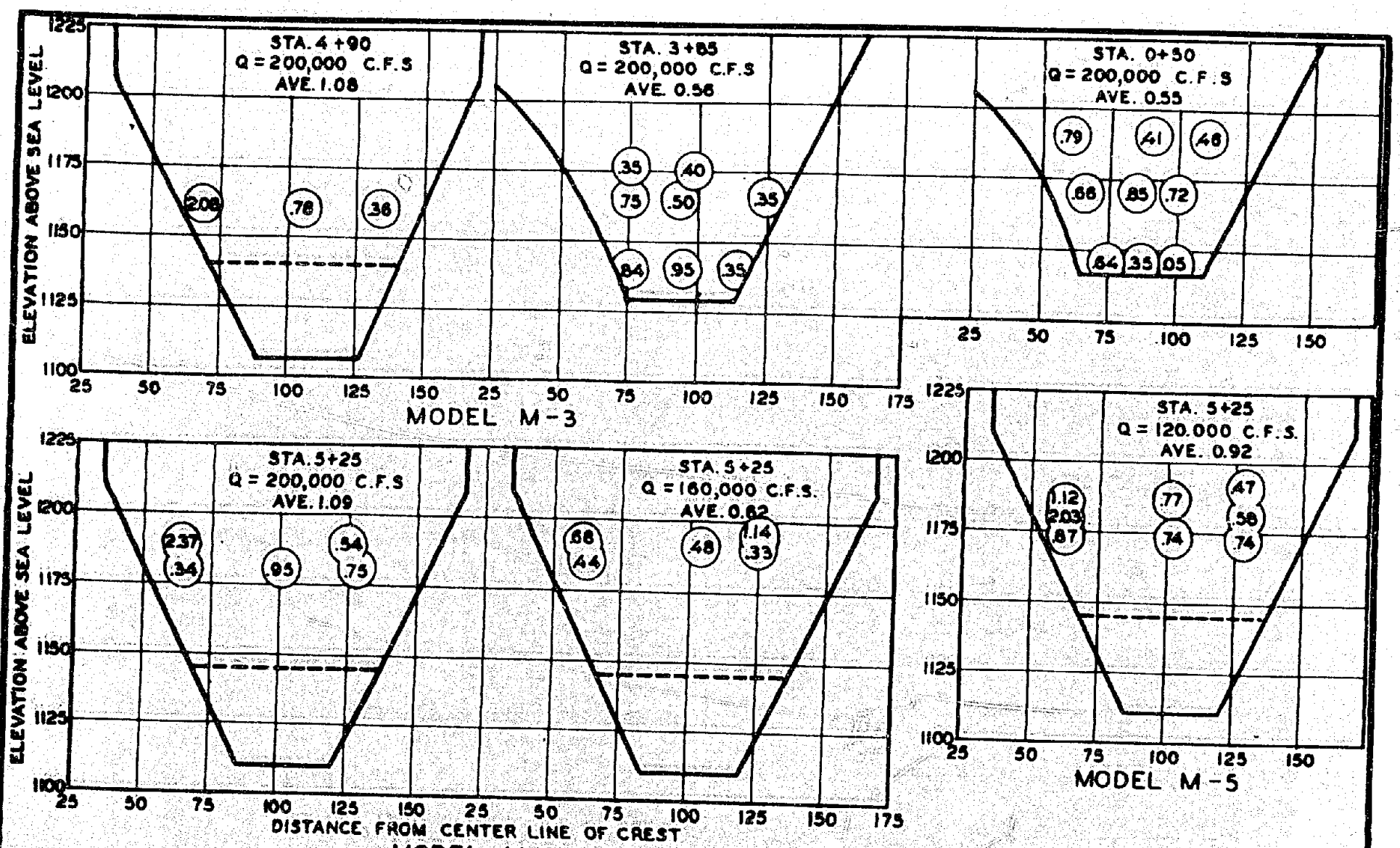
FIG. 66



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM HYDRAULIC EXPERIMENTS
PER CENT OF ENTRAINED AIR
SIDE CHANNEL SPILLWAY MODEL C-2

DRAWN: W.H.P. SUBMITTED:
TRACED: T.W.C. RECOMMENDED:
CHECKED: APPROVED:

DENVER, COLO., OCT. 31, 1934 AS-D-2513



HOOVER DAM HYDRAULIC EXPERIMENTS

PERCENT OF ENTRAINED AIR
SIDE CHANNEL SPILLWAY
MODEL M-3 AND M-5

E.W.C.

45-D-3452

FIG. 67

Observations by the third method of the air content for 200,000 sec. ft. discharge near the windows of the side channel averaged from the seven observers 0.16% at the upper, 0.85% at the middle and 1.30% at the lower window. The estimates by the various observers differed considerably relatively, but in no case more than 1½% of air content from the average. The point at which the 1.30% was observed at the lower window was very close to the locations at which the 0.54% and 0.75% contents were obtained by the second method for 200,000 sec. ft. discharge as shown on Fig. 67. Here again the agreement, although relatively not close, shows only about 0.65% difference in air content. The fact that the air content was small was further demonstrated by the comparative transparency of the water. If the water had contained a large proportion of air it would have been opaque or translusent, rather than nearly transparent.

The estimates, by the third method, of air content of the highly disturbed portion of the water near the overfalling nappe were taken near the center of the length of the channel. The content increased as the discharge became less, due no doubt to the greater fall at the lower discharges. The quantities were 2.8%, 4.3%, 6.1%, 9.0% and 9.0% for discharges of 200,000, 180,000, 120,000, 80,000 and 40,000 sec. ft. respectively. This shows that at the design discharge, even in the most highly disturbed portion of the channel, the air content is very small. The air contents at low flows are not important because under these conditions there is ample channel capacity.

Relation of Model and Prototype Results.

The relation between the air content on the model and what would occur on the prototype is not well established. Since the viscosity of the water is relatively greater for the model, the water in the model would be relatively much less disturbed and would not entrain as much air. Where opportunity has existed to observe both prototype and model of hydraulic structures, the flow in the prototype appeared to be relatively much rougher and carried more foam and mist.

In the model spillway channels the air, when carried down into the channel, was subjected to a pressure of at the most of only a few feet. In the prototype, however, it would be subjected to many times this pressure, and consequently the same quantity of air would occupy a relatively much smaller space in the prototype. Those two effects would tend to offset each other, and the model results are therefore probably not far from those which would result on the prototype.

Conclusions on Air Content in Side Channel Spillways

The model tests indicated that for the Boulder Dam spillways as finally designed, the air content at maximum discharge would be negligible and it was therefore neglected in the design of the side channel although an allowance of about 35% air content was made in the tunnel transition section to insure a free air space above the water. At low discharges the air content might be appreciable but this would not control the channel size. The conditions were favorable for low air

content at maximum flows however, as the spillway channel had a relatively large cross section and the drop of the water over the crest was small.

For designs at other dams it will also usually be economical to have a low drop, but ordinarily a smaller channel cross section will be used, because the desirability of smooth flow down the chute will usually not be so important as in the case of the inclined tunnels at Boulder Dam and smaller channels will be cheaper. With a smaller channel section or greater drop, the air content will probably be higher. In designing spillways where the air content will be great, it will be worth while to assume a varying air content along the channel, with a small air content at the upper end where the drop is small, and increasing content down the spillway.

Flow in the Spillway Tunnels

Because of the possibilities that serious damage might result from the flow of the water through the spillway tunnels, it was essential that careful consideration be given to this feature of the Boulder Dam project. For the design discharge of 200,000 second feet in each spillway the velocity of flow is expected to reach approximately 173 feet per second, or nearly 120 miles per hour, which represents an energy of 10,560,000 horse power in each tunnel, or several times that at the falls at Niagara.

To connect the inclined tunnels with the horizontal diversion tunnels involved a bend in a vertical plane with a central angle of about 50° and the most advantageous alignment for the spillway tunnel on the Nevada side involved a bend in a horizontal plane with a central angle of 32° 15'. The action of the water in passing around these bends could not be predicted without model studies.

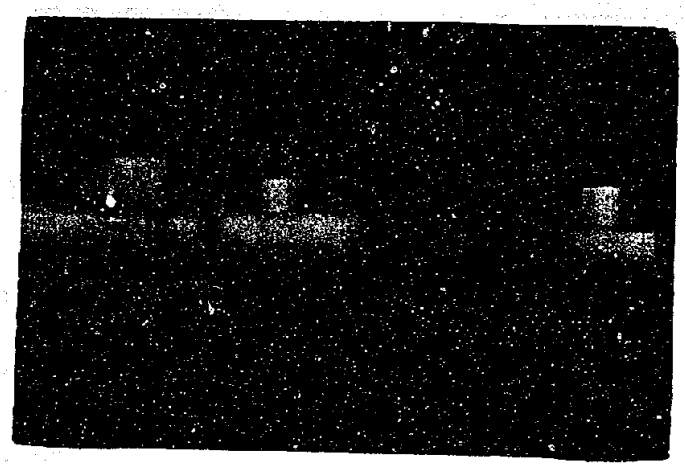
For the vertical bend it was desirable to use a fairly short radius, as long radii involved the excavation of additional tunnel length. A radius of approximately 800 feet for the horizontal bend seemed to fit in best with other conditions. In all cases the radii given are measured to the center line of the tube. Experiments were made on both vertical and horizontal bends to determine the best dimensions for each. At the Colorado Agricultural College laboratory space did not permit an exact duplication of the entire length of horizontal tunnel and the distance between the vertical bend and the horizontal one was therefore shortened in the 1:60 models. At the Montrose laboratory however the full length was constructed (Plate 33, Book 1). The close agreement between the action in the two models indicated that no material discrepancy was involved in the shortened length of the 1:60 model.

Flow in the Horizontal Bend - 1:60 Scale

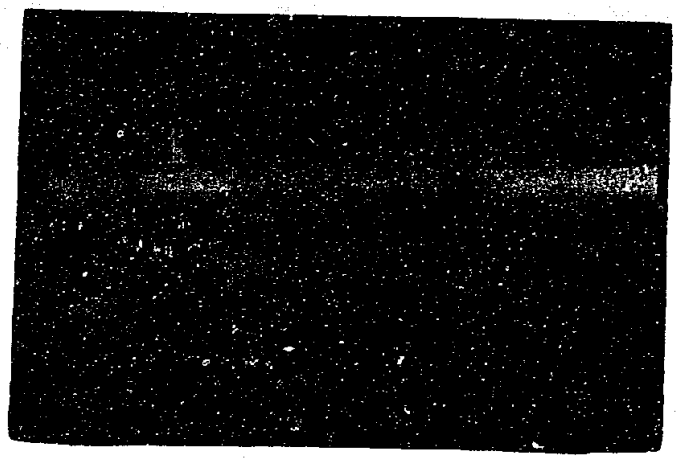
In order to determine what the action of the water would be in the horizontal bend in the spillway tunnel on the Nevada side of the river, several models of different degree of curvature were

experimented upon using models with a 1:60 scale. The first of these models were made with one of the curved sections of the pyralin tube. This had a central angle of 22½ degrees and a radius of 150 ft. These dimensions did not agree with any plan for the tunnel, but served to show what the action would be in a bend of short radius. As the water entered the bend it tended to preserve its original direction of motion, but, being restrained by the wall of the pipe, was forced up rapidly, creating a velocity in a circumferential direction which was sufficient to produce a centrifugal force large enough to keep most of the water at the outside of the pipe for a complete revolution. The combination of this circumferential velocity and the longitudinal velocity caused the water to take a spiral path around the outside of the pipe. Near the bend most of the water seemed to be against the outside of the pipe, but further downstream less followed the outside and more flowed along the bottom until about 8 diameters below the bend the spiral motion around the outside of the pipe ceased. Plate 123-A is a view of this bend taken from the concave side, and shows the abrupt lowering of the water surface at the inside of the bend as the surface rose on the outside of the bend. View B shows the spiral motion of the water moving around the outside of the tube downstream from the bend.

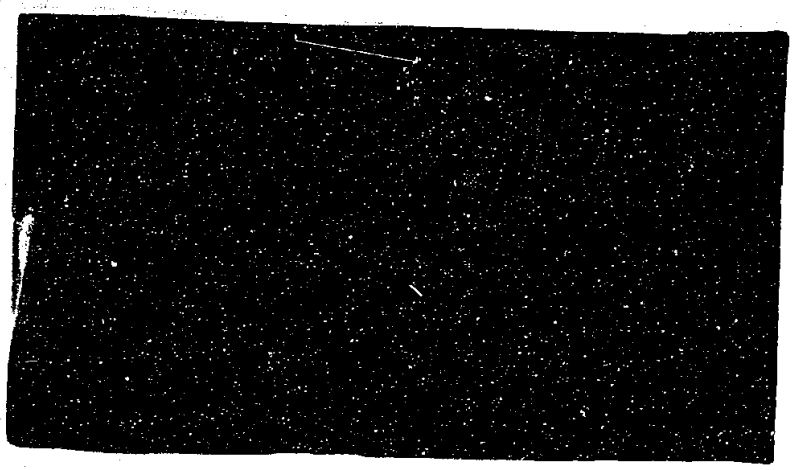
The bend shown in the contract drawings had a central angle of 32° 15' and a radius of 819 ft. on the center line. A model of this was constructed of 10" steel pipe. In order that the action of the water might be observed, the wall of the pipe was cut away with an



A-FLOW AT 150 FT. RADIUS BEND
VIEW OF CONCAVE SIDE



B-SPIRAL FLOW BELOW
150 FT. RADIUS BEND



C-FLOW LINES IN BEND FOR
200,000 SEC. FT. DISCHARGE

D-FLOW LINES IN BEND FOR
140,000 SEC. FT. DISCHARGE

FLOW IN HORIZONTAL TUNNEL BEND

PLATE 123

acetylene torch, to as great an extent as possible without permitting the water to escape. In this model the water also tended to pile up on the outside of the bend, but the radius of the bend was greater with respect to the velocity and the water was not forced so high nor so rapidly upward as in the one previously described. The circumferential velocity thus produced was much less than for the 150 ft. radius bend and was not sufficient to give a centrifugal force large enough to keep the water on the outside of the pipe. The water rose to near the top and then fell toward the bottom of the pipe at the same time continuing its motion in a longitudinal direction. The water which fell had however a component of flow across the pipe which tended to make the water pile up on the opposite side of the pipe to a certain extent, but much less than on the outside of the pipe. The action was therefore first to rise on the outside of the bend and further downstream to rise to a lesser extent on the other side of the pipe. This second rise occurred in the straight pipe downstream from the bend. This action was much the same at all discharges.

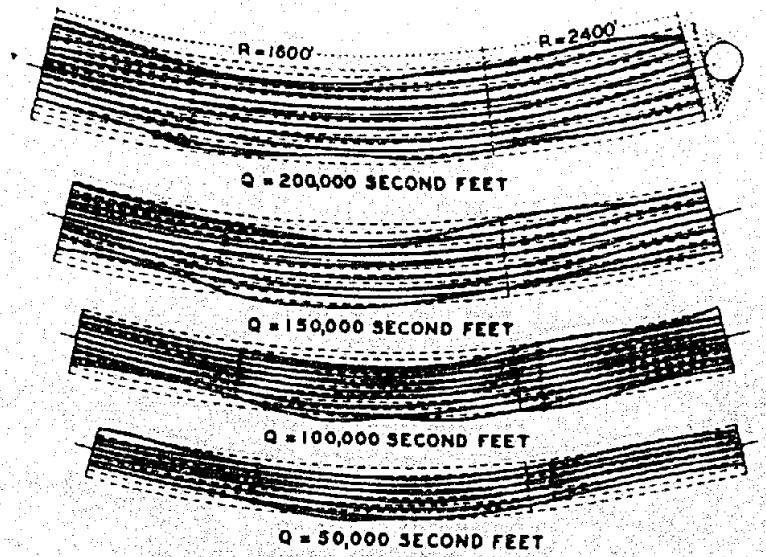
In order to determine the direction of the currents in the bends, the inside of the bend was coated with thick white lead paint, and before it had started to dry, the water was permitted to flow for a short time. This caused the soft paint to take a streaked form as shown on Plate 123-C and D. View C shows the rise of the surface on the outside of the bend for a discharge of 200,000 sec. ft. and View D shows the superelevation resulting from a flow of 140,000 sec. ft.

An attempt was made to record the direction of these flow lines, but the result was not entirely satisfactory. Figure 68 shows the main direction indicated by the paint streaks for the various types tested.

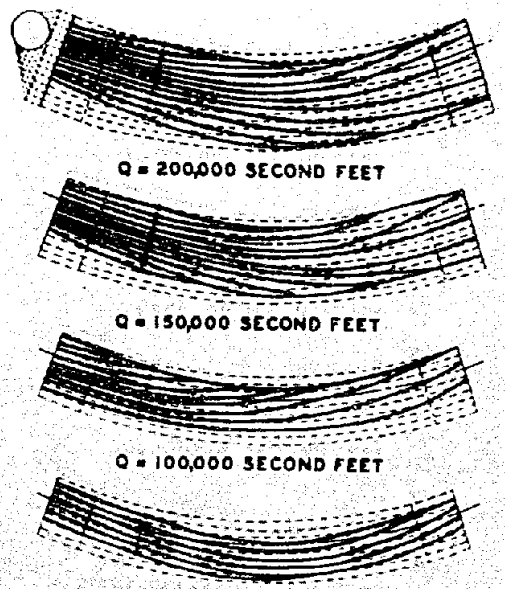
In order to try to improve the conditions of flow in the bend, one with a radius corresponding to 1600 ft. was built. This model was constructed of wood and cement mortar. The conditions of flow are shown on Plate 124 and the direction of the stream lines are shown in Figure 68. The action in the bend did not seem to be improved by this increase in radius. From observing the action on the 1600 ft. radius bend, it was thought that better results might be obtained by using a section of longer radius bend below a section of 1600 ft. radius. A model of this form was constructed, with the dimensions as shown on Figure 33. The action in this bend (Plate 125) did not indicate any improvement and it was decided to adopt the 819 ft. radius bend in the construction of the Montrose laboratory.

Flow in Vertical Bends - 1:60 Scale

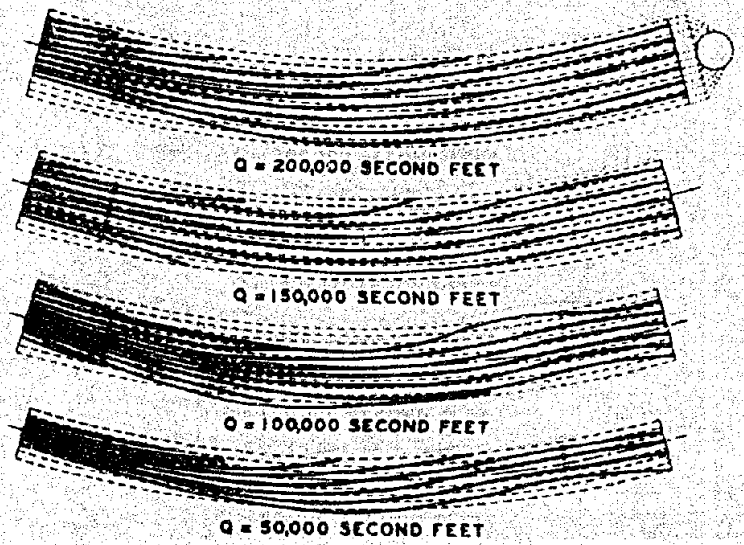
In general, the condition of flow in the vertical bend was quite satisfactory for all the radii of bend selected. The condition for the bend of 150 ft. radius is shown on Plate 126-A. The upper surface of the water was generally smooth. There was a tendency for a small part of the water to splash upward near the center of the pipe and form a ridge of detached particles of water moving down the horizontal pipe for some distance until they settled down to the level of the remainder of the water. Plate 126-B shows this condition for the



R = 1600', L = 600, I = 21° 24'
R_2 = 2400', L_2 = 300, I_2 = 7° 9'



R = 819', L = 416 + 120 = 581, I = 32° 15'



R = 1600', L = 900, I = 32° 15'

EXPLANATION
LINES OF FLOW
ELEMENTS OF TUNNEL

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM HYDRAULIC EXPERIMENTS

DIRECTION OF FLOW IN HORIZONTAL BEND OF SPILLWAY TUNNEL
DIAMETER - 50 FEET

DRAWN: W.H.P. SUBMITTED: E.W. Lane
TRACED: G.E.W. RECOMMENDED:
CHECKED: W.H.B. APPROVED:
DENVER, COLO. DEC. 18, 1932.

45-D-2592

FIG. 88

A-DISCHARGE 50,000 SEC. FT.

B-DISCHARGE 100,000 SEC. FT.

C-DISCHARGE 100,000 SEC. FT.

D-DISCHARGE 150,000 SEC. FT.

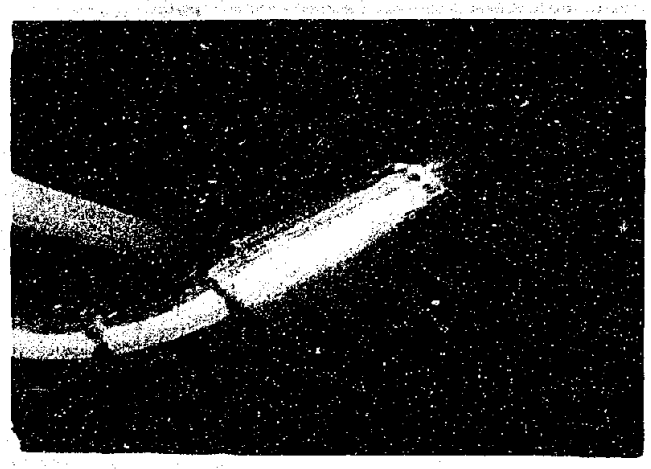
FLOW IN 1600 FT. RADIUS BEND

DISCHARGE 100,000 SEC. FT.

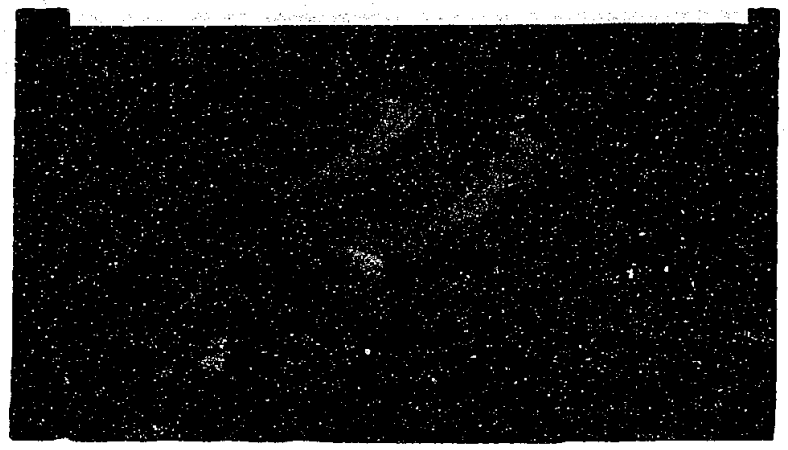
DISCHARGE 100,000 SEC. FT.

DISCHARGE 50,000 SEC. FT.

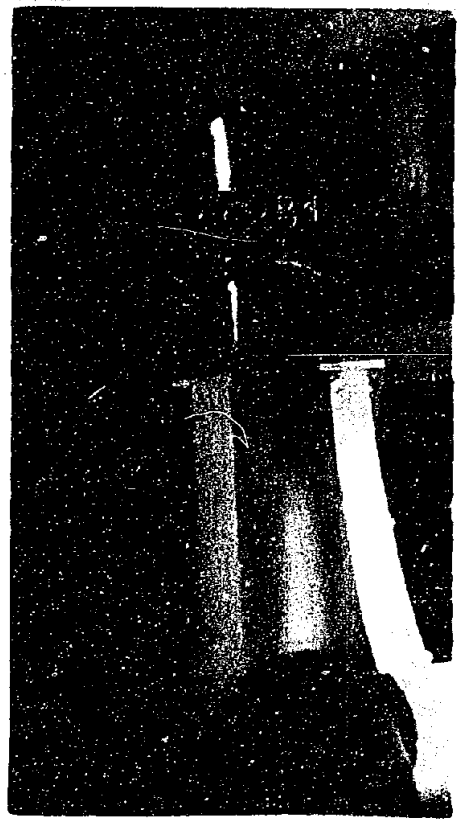
FLOW IN 1600-2400 FT. RADIUS COMPOUND CURVE



A-FLOW IN BEND OF 150 FT. RADIUS



C-ANOTHER VIEW OF 200 FT. RADIUS BEND
DISCHARGE 200,000 SEC. FT.



B-FLOW IN BEND OF 200 FT. RADIUS
200,000 SEC. FT. DISCHARGE

FLOW IN VERTICAL BENDS 1:60 MODEL RATIO

PLATE 126

bend of 200 ft. radius. This action was less pronounced for the 200 ft. radius bend than for the 150 ft. radius. Pitot tube observations showed a mean velocity of 172.5 ft. per sec. for the 150 ft. radius and 175.9 ft. per sec. for the 200 ft. radius, indicating slightly less loss for the longer radius bend.

It was desirable not to make the radius of the vertical bend larger than necessary as this increased the excavation in the tunnel and tests were made to determine whether the flow conditions were improved by using longer bends. For this purpose a 10-inch rubber pipe was used. The same length of pipe was experimented on in each case and the velocity of the water at the end of the pipe was observed with a pitot tube. With this pipe, radii of 300 ft., 468 ft. and 582 ft. were tested. The average velocities at the ends of the pipe for these radii were 138.3, 140.7 and 137.8 ft. per second respectively. These results indicated that there was practically no difference in the velocities at the end of the pipe for the various radii of bend and therefore that the loss of head and consequently the internal disturbance, was the same in each case. As a result of these experiments it was decided to use a vertical bend with 200 ft. radius in the prototype.

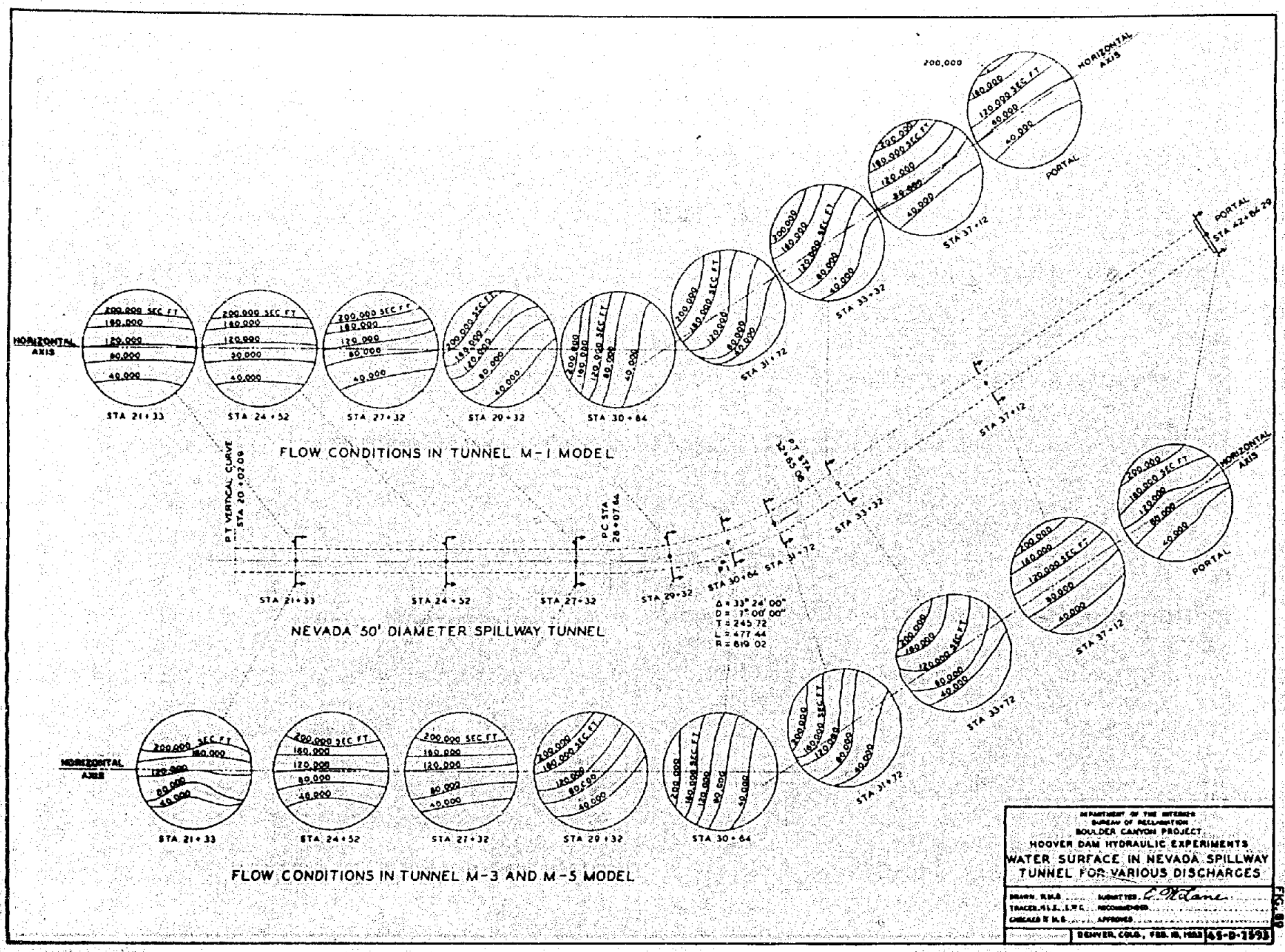
Flow in 1:20 Scale Model Tunnel

The flow in the 1:20 model vertical bend at Montrose was very similar to that in the 1:60 model, the same tendency to splash upward in the center of the pipe being noticeable. Unfortunately, it was not possible to get photographs of the flow in the tunnel model at Montrose.

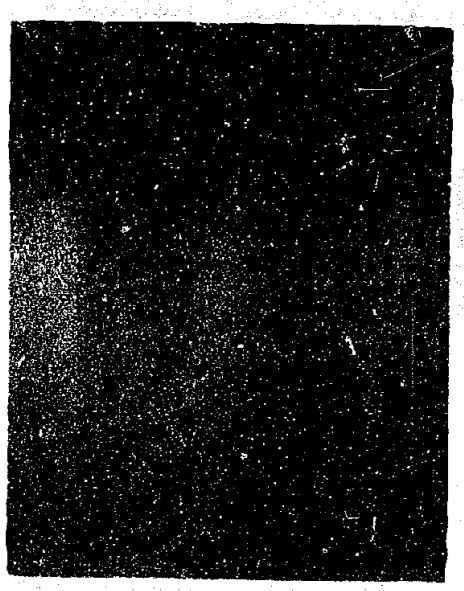
With the final design spillway model there was no splashing of the water out of the windows in the top of the vertical bend at any discharge, indicating a very satisfactory condition of flow. A description of a peculiar condition of flow in the vertical bend with one of the preliminary models, is given on page 45 of Book 1.

The nature of the flow in the 1:20 scale horizontal tunnel for the M-1, M-3 and M-5 models is shown on Figure 69. The radius of the horizontal bend was 819 ft. This shows the position of the water surface at various points along the horizontal tunnel, as observed through the windows in the top of the tunnel pipe. In nearly all cases these are visual determinations, but the water surface was comparatively rough and it was possible to estimate the mean position of the surface with substantially the same accuracy by either visual or measurement methods.

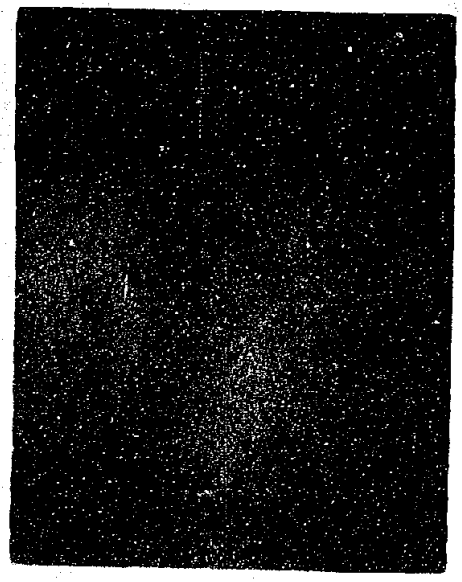
The conditions of flow in the tunnel for all three of these models was practically the same. In the horizontal section upstream from the horizontal bend the water surface was usually level for all flows. At the beginning of the horizontal bend the water surface rose at the outside of the bend and dropped at the inside of the bend, at the middle of the bend reaching a nearly vertical position. After passing the middle, the water surface dropped back toward a horizontal position, and did not swing on over in a spiral. In general the results in the 1:20 model were very similar to those obtained on the 1:60 model. In all cases on the final and semi-final models (M-3 and M-5)


HORIZONTAL AXIS
200,000 SEC FT
180,000
120,000
80,000
40,000
STA 21+33
STA 24+52
STA 27+32
STA 29+32
STA 30+64
STA 31+72
STA 33+32
STA 37+12
PORTAL
STA 42+86.29
FLOW CONDITIONS IN TUNNEL M-1 MODEL
P.T. VERTICAL CURVE STA 20+07.09
P.C. STA 28+07.64
P.T. STA 32+65.08
Δ = 33° 24' 00"
D = 7° 00' 00"
T = 245.72
L = 477.44
R = 819.02
NEVADA 50' DIAMETER SPILLWAY TUNNEL
FLOW CONDITIONS IN TUNNEL M-3 AND M-5 MODEL
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM HYDRAULIC EXPERIMENTS
WATER SURFACE IN NEVADA SPILLWAY TUNNEL FOR VARIOUS DISCHARGES
DENVER, COLO., FEB. 18, 1933
45-D-1593
FIG. 89

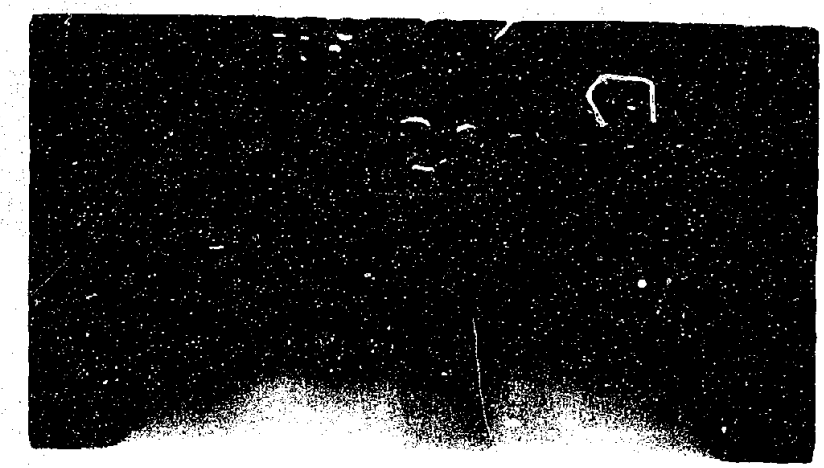
M-3 MODEL-DISCHARGE 40,000 SEC. FT.



M-3 MODEL-DISCHARGE 200,000 SEC. FT.



M-3 MODEL-DISCHARGE 160,000 SEC. FT.



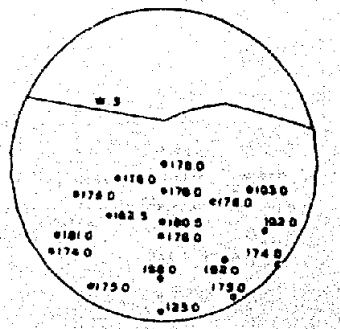
M-5 MODEL FINAL DESIGN
DISCHARGE 200,000 SEC. FT.

CONDITIONS OF FLOW AT END OF TUNNEL
M-3 AND M-5 MODELS

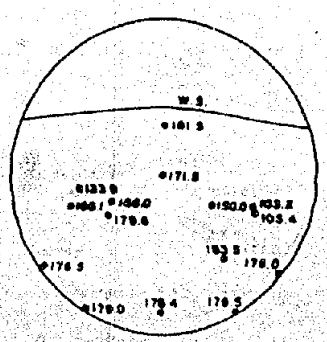
PLATE 127

there was ample air space at the top of the tunnel. There was a distinct pulsation in the M-3 model for certain conditions, during which at times the pipe was completely filled at the lower end. This action has been described in detail on page 39 of Book 1.

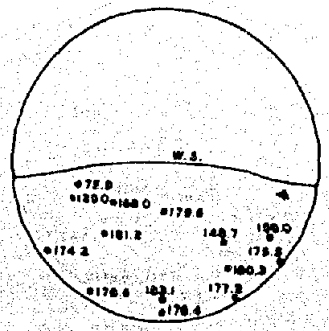
The velocities of flow in the tunnel for the M-3 model were measured for a wide range of flows as shown on Figure 70. Observations were made with a pitot tube with an impact opening, the pressures being observed by means of a mercury manometer. The velocities were observed 135 ft. (prototype dimension) downstream from the vertical bend and at the end of the pipe. The velocities were generally higher for the larger discharges, on account of the relatively smaller friction effects for the larger flows. The result obtained for the maximum discharge was very similar to that computed in the spillway design. The approximate average velocity observed (i.e. the mean of the observed velocities) for 200,000 sec. ft. was 174.4 ft. per sec. just below the vertical bend, and 134.7 ft. per sec. at the end of the pipe. The corresponding computed velocities were 173 and 124 ft. per sec. In this connection it should be noted that the friction losses in the model were relatively higher than they would be in the prototype and this difference was corrected for in the model by making the slope of the model tunnel steeper. This excess slope was computed for the 200,000 sec. ft. discharge assuming a value of n of 0.10 for the wood stave pipe and n of 0.13 for the concrete lined prototype. For lower discharges the friction effects in the model would be lower than in the prototype.



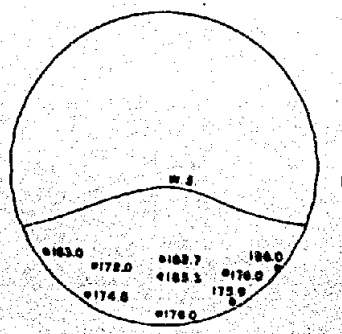
Q = 200,000 second feet
$V_{Ave.}$ = 174.4 Ft. per Sec.



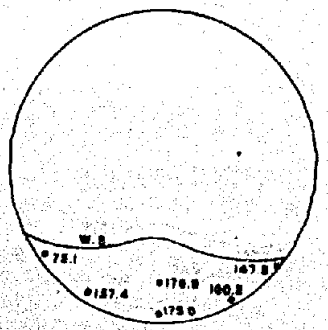
Q = 160,000 second feet
$V_{Ave.}$ = 162.1 Ft. per Sec.



Q = 120,000 second feet
$V_{Ave.}$ = 163.1 Ft. per Sec.

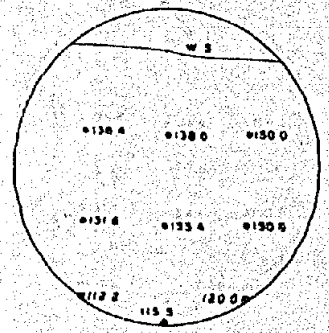


Q = 80,000 second feet
$V_{Ave.}$ = 174.6 Ft. per Sec.

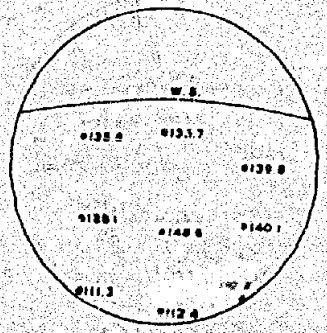


Q = 40,000 second feet
$V_{Ave.}$ = 148.6 Ft. per Sec.

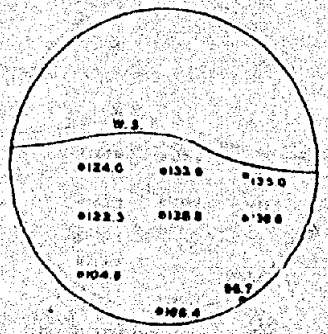
VELOCITIES IN TUNNEL, 135 FEET BELOW VERTICAL BEND
CROSS SECTION LOOKING DOWNSTREAM



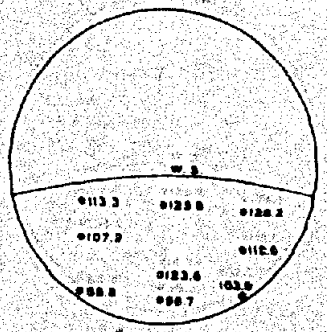
Q = 200,000 second feet
$V_{Ave.}$ = 134.7 Ft. per Sec.



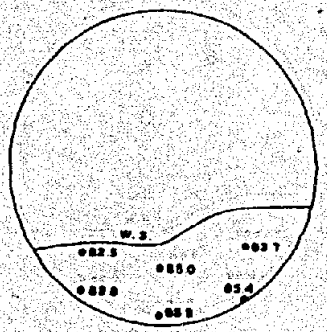
Q = 160,000 second feet
$V_{Ave.}$ = 129.1 Ft. per Sec.



Q = 120,000 second feet
$V_{Ave.}$ = 122.3 Ft. per Sec.



Q = 80,000 second feet
$V_{Ave.}$ = 111.9 Ft. per Sec.



Q = 40,000 second feet
$V_{Ave.}$ = 84.0 Ft. per Sec.

VELOCITIES AT OUTLET OF TUNNEL
CROSS SECTION LOOKING DOWNSTREAM

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

HOOVER DAM
DISTRIBUTION OF VELOCITIES
MODEL OF NEVADA SPILLWAY TUNNEL

DRAWN: W.F.B. SUBMITTED: E. N. Lane
TRACED: G.E.W. RECOMMENDED:
CHECKED: W.H.B. APPROVED:
DENVER, COLO. JAN. 8, 1935. 45-D-3148

FIG. 70

Conclusions from Side Channel Spillway and Tunnel Experiments

Although the experiments described in this report were primarily for the purpose of developing the best form of spillway for the unusual conditions existing at the Boulder Dam, many of the conclusions reached will be of value to designers of other spillways. The following are the most important of the conclusions reached regarding the side channel spillway and tunnel design. Those dealing with crest shapes will be discussed in Book 4 of this report.

It is believed that the very close similarity between the results obtained on all of the similar models, both in appearance and when analyzed quantitatively, justifies considerable confidence that the results on the models can be expected to be duplicated on the prototype and that the Boulder Dam spillways will therefore perform in a satisfactory manner. The results also furnish considerable evidence to the reliability of hydraulic model testing and the value of this method of solving complicated hydraulic problems.

The experiments, which were conducted on a wide range of sizes of models, showed such closely comparable results for all sizes that the value of observations made on comparatively small models was emphasized. It was found that the smaller models were so much easier and cheaper to construct and alter. It is concluded therefore that the greatest efficiency in model testing can be obtained by making the first tests on small scale models, without great precision of dimensions, and as the designs become more perfect, the tests should be carried out

on larger, more accurate models. The unprecedented magnitude of the work as compared with previous experience, which existed at the Boulder Dam will rarely be met, and such large models as were used for the investigation of it therefore will rarely be required.

The experiments on the C-2 model demonstrated the reliability of the Hinds* method of analysis which is based on the theory of the conservation of linear momentum, and it was used throughout in the design of these spillways. It was found however that it was difficult to compute the results by this method when the flow was near the critical depth, and that the results of computations under such conditions could not always be relied upon. Guide vanes on the weir crest or on the channel bottom reduce the size of channel necessary but the expense of constructing them would probably be greater than the saving affected.

*Trans. Am. Soc. C. E., page 881, Vol. 89, 1926.

If it is necessary or desirable to secure a relatively smooth flow in the spillway channel this may be done by increasing the cross section of flow or by the introduction of baffles. For the conditions existing at the Boulder Dam the former method proved most desirable and will probably be better in most cases. A steeply sloping bottom in the channel gives rise to a shallow depth of flow in the channel and a water surface much higher on the opposite side of the weir crest. This irregularity of surface is likely to produce undesirable roughness in the flow in the inclined tunnel or chute below the spillway proper.

Better conditions can be obtained by using a greater depth in the channel. Since the depth of flow is usually fixed by the presence of a control section, when greater depth is desired, it is advisable to place the control section at the downstream end of the overfall weir and design it to provide the desired channel depth. This may involve a cross weir in the channel, similar to that used in the Boulder Dam spillways. A smoothing out of the flow in the spillway channel may also be secured by means of baffles. Of the many forms tested, the overhanging dentated baffles shown on Plates 71 and 93, Book 2, and the coping baffle shown on Plates 36 and 37, Book 1, on the rear wall seemed to produce the best results, although other forms gave nearly as good results. These would no doubt prove expensive to construct and better results might possibly be obtained by an equal expenditure in channel enlargement. This could be determined for any case only by model tests.

The offset in the wall just downstream from the weir, which makes the side of the channel at this point continuous with the side of the effective cross section of the channel flow opposite the weir, as described on page 6/ Book 2 should be incorporated in side channel designs.

The factors in a side channel spillway leading to the low air content are small drop in the water passing over the weir and low turbulence in the spillway channel. At the upper end of the spillway channel, where the drop is smaller, the air content will be less than further downstream where the drop is larger. For the discharge for which the spillway was designed, the drop over the weir at the upper end is

usually small, and therefore the air content there is slight. Where the channel flow is comparatively quiet, it is likely to contain less air than if it is very turbulent. Since in the Boulder Dam spillway at design discharge, the fall over the weir is not great, especially at the upstream end, and the flow in the channel is comparatively quiet, it is not believed that the air content will be sufficient to necessitate consideration in designing the channel sections. At lesser discharges the air content is likely to be greater on account of the greater fall over the spillway, but since excess channel capacity is available under those conditions, the extra air content is not important. Another factor which will lead to low air content is great depth of the water, which will cause high pressure on part of the entrained air and therefore the air bubbles there will be compressed into smaller sizes. In designing side channel spillways where the conditions favor high air contents, it will be worth while to vary the air content assumed, beginning with a smaller amount at the upper end where the fall is small, and increasing it downstream as the fall increases.

In spillway forms using gates, as in the Stoney gate model, it is undesirable to have changes in the direction of flow downstream from the gate, where the water is moving at a velocity near or greater than the critical velocity, since it is difficult to change the direction of flow under those circumstances. The change can readily be made upstream from the gate, where the velocity is below the critical value.

A large wide approach channel to the gate is not necessary, as a comparatively narrow one gives as good results. The flow through the gate opening with the gate fully raised is very close to that which would be computed by the well known formula $Q = 3.089\ WH^{3/2}$ where W is the width of the opening and H is the head in the reservoir above the gate sill. The water surface drop at the most contracted section however is not one-third of the head, as the theory indicates, but nearer one-sixth of the head.

The experiments on the flow in the bends indicated that the radius of the vertical bend at the bottom of the inclined tunnel section did not have an important effect upon the character of the flow around this bend. In the horizontal bend however two small a radius caused a spiral flow around the periphery of the pipe. This did not occur with larger radii.

It was found that considerable care was necessary in designing the piers between the drum gates, in order to prevent vacuum effects which would prevent the gate from functioning as desired.

Partial Bibliography on

Theory of Model Testing and Similitude

Bean, Buckingham and Murphy.

Discharge Coefficients of Square-edged Orifices for Measuring the Flow of Air.

U. S. Bureau of Standards paper No. 49 - March 1929.

Chick, A. C.

Dimensional Analysis and the Principle of Similitude as Applied to Hydraulic Experiments with Models.

pp. 775-827 - Freeman's Hydraulic Laboratory Practice - A. S. M. E. - 1929.

Eisner, F.

Offene gerinne - Handbuch der Experimental physik - Vol. 4, Part 2 - by Wien and Hanns - Leipzig - 1931

Gives practical limits in field of hydraulic models.

Groat, B. F.

Ice Diversion, Hydraulic Models, and Hydraulic Similarity.

A. S. C. E., pp. 1128-1191 - Vol. 82, Dec. 1918.

Groat, B. F.

Theory of Similarity and Models.

A. S. C. E., pp. 273-387, Vol 96 - 1932.

Includes mechanical models.

Hodgson, J. L.

The laws of Similarity for Orifice and Nozzle Flows.

A. S. M. E. - Vol. 56 - Paper A. P. M. 50-3 - 1928.

Marzolo, F.

Some Consideration Regarding Hydraulic Models.

pp. 743-658 - Hydraulic Laboratory Practice - A.S.M.E. 1929.

Reynolds, K. C.

Notes on the Laws of Hydraulic Similitude as Applied to Experiments with Models.

pp. 759-773 - Freeman's Hydraulic Laboratory Practice - A. S. M. E. 1929.

Smith, Jr., E. S.

Quantity - Rate Fluid Meters.

A. S. M. E. - Paper HYD 52-7b - 1930.

Spitzglass, J. M.

Similarity - Limitations in its Application to Fluid Flow.

pp. 111-122, A. S. M. E., Vol. 52, part 1, 1930.

Thompson, P. W.

The Technique of Hydraulic Model Experimentation.

Proc. Louisiana Eng. Soc., pp. 125-134 - Nov. 14, 1932.

Tietjens, O. G.

The Use of Models in Aerodynamics and Hydrodynamics.

A. S. C. E., p. 225, Vol. 54, Sept. 30, 1932.

Partial Bibliography on Side Channel Spillways

Blee, C. E.

Lining an Earth Spillway Channel with Concrete - Eng. News-Rec., p. 777, May 13, 1926.

Q = 30,000; see Canadian Engineer of about same date for plan and sections, Alouette Project, B. C. E. Ry.

Christopher, W. C.

Tieton Dam - Dams and Control Works, U. S. Bureau of Reclamation, p. 45, 1929.

Plan and Section of Spillway with Dimensions. Q = 40,000.

Daro, H. H.

The Spillways of the Burrinjuck Dam - Proc. Inst. C. E., Vol, 214, Part II, 1921-1922.

Results of Model Tests.

Davis and Wilson

Irrigation Engineering - Wiley, New York, Seventh Edition, pp. 272-3, 278, 279, 1919.

Several U. S. Bureau of Reclamation Canal Spillways.

Engineering Record

Construction of the Waste Weir at the Ashokan Reservoir - Eng. Rec., pp. 109-110, Vol. 64, 1911.

Details of Section but no flow lines or quantities.

Spillway for about 50,000 sec.-ft.

Hinds, Julian

Side Channel Spillways: Hydraulic Theory.

Design, Economic Factors and Experimental Determination of Losses - with discussion.

Trans., Am. Soc., C. E., pp. 881-939, Vol 89, 1926.

Miller, C. N.

Design of Wash Water Troughs for Rapid Sand Filters.

Water Purification - by J. W. Ellms, Appendix B, Edit. 1917.

Contains theory for rectangular channels and some experimental verification.

Nalder, W. H.

Guernsey Dam, North Platte Project - New Reclamation Era, p. 184, Dec., 1928.

General details of spillway.

Nimmo, W. H. R.

Side Spillways for Regulating Diversion Canals - with discussion.

Trans. Am. Soc., C. E., pp. 1561-1588, Vol. 92, 1928.

Gives application to other than rectangular flumes.

Paul, C. H.

Excavation for the Arrowrock Dam - Eng. News, pp. 93-100, July 17, 1913.

General plan of spillway only.

Smith, F. F.

Building the Guernsey Irrigation and Power Dam - Eng. News-Record, pp. 264-268, Vol. 100, 1928.

Shows plan of dam and spillway. Q = 52,000 sec.-ft.

Steele, B. W.

McKay Dam.

Dams and Contol Works, U. S. Bureau of Reclamation, p. 53, 1929.

Q = 10,000 - Plan and section.

Steele, B. W.

Echo Dam.

Dams and Control Works, U. S. Bureau of Reclamation, p. 81, 1929.

Q = 15,000. Plan and sections.

Steele, I. C.

High Rockfill Dam Designed with Jointed Concrete Face - (Salt Springs Dam) - Eng. News- Rec., p. 92, Jan. 16, 1930.

Details of side channel spillway for Q = 48,000.

Taylor, P. I.

Arrowrock Dam.

Dams and control works, U. S. Bureau of Reclamation, p. 35, 1929.

Spillway designed for 40,000 sec.-ft. Partial dimensions of channel.