PAP 98

FC11
Bureau of Reclamation

HYDRAULICS BRANCH
OFFICIAL FILE COPY

FILE COPY
WHEN BORROWED RETURN PROMPTLY

AUTHOR

Memorandum

Denver, Colorado

H. M. Martin

November 26, 1954

J. N. Bradley and C. W. Thomas

Modification of water measuring station downstream from outlet works--Flatiron Afterbay Dam--Colorado-Big Thompson Project

HISTORY OF STRUCTURE

Previous to the 1954 irrigation season, it was learned that the gaging station immediately downstream from the outlet works at Flatiron Afterbay Dam did not perform satisfactorily. A control section was not provided at the station as it was intended that gaging be accomplished by merely measuring the depth of water in the rectangular channel. Two distinct difficulties were encountered in water measurement: First, waves as much as 2 feet in height were present in the flume; and secondly, the level in the long reach of canal downstream acted as the control and adjustment was very slow. A lag of from 1 to 2 hours was required to reach a steady gage reading at the station after a change in discharge. To deliver the correct amount of water on time, the measuring station at Flatiron Afterbay Dam must not only be accurate, but it is imperative that an immediate indication of the discharge be available at all times. On February 16, 1954, Messrs. H. E. Stradley, Jr., A. D. Soderberg, and Ted Sundquist of Region 7, together with R. W. Whinnerah and J. N. Bradley from the Chief Engineer's office, inspected the gaging station for the purpose of proposing corrective measures for the 1954 operating season. An account of the findings and corrective measures recommended is contained in Field Trip Report No. 1472 to the Chief Engineer, dated March 10, 1954, subject "Gaging station downstream from outlet works--Flatiron Afterbay Dam--Colorado-Big Thompson Project."

It was recommended in the field trip report that a rectangular weir be installed as a control, together with a pair of rafts upstream to reduce wave action. The rectangular weir was installed, but this was later replaced with a notched check weir 6 feet high (Figure 1). The check weir provided better flow conditions to the siphon downstream and allowed a greater discharge to pass for low reservoir heads than the rectangular weir, but the notched check weir has a disadvantage in that it operates partially submerged for all flows. In addition to the check weir, two floating rafts were installed some 75 feet upstream and timber grills were placed in the stoplog slots at the downstream ends of the outlet conduits in order to reduce wave action. These installations reduced wave heights to a maximum of about 4 inches at the measuring section. This system was used for water measurement throughout the 1954 season with fair success. The principal objections

PAP 98

PAP 98

were, the excessive head loss, and the fact that a delay of about 40 minutes was necessary to obtain a steady gage reading after a reduction in discharge. A delay in steady gage reading was likewise apparent after an increase in flow, but since this delay was shorter the water master did not consider this phase as objectionable.

Near the end of the season, C. W. Thomas and J. N. Bradley, accompanied by H. E. Stradley and A. D. Soderberg, made a second trip to the project for the purpose of planning permanent modifications to the structure. A field trip report by Thomas and Bradley, dated September 10, subject "Observation of hydraulic conditions at Flatiron Afterbay Dam outlet works--Flatiron pump turbine bypass and Carter Lake outlet works, Colorado-Big Thompson Project," gives an account of this trip.

MODEL STUDIES OF 6.0-FOOT CHECK WEIR

Subsequent to the second field trip, a rough model was set up in an existing small laboratory flume to study the structure with respect to permanent modifications. The flume was approximately 2 feet wide, which dimension set the model scale at 1 to 10.3 (Figure 8). The first step was to install the 6-foot high check weir in the model and calibrate it for various degrees of submergence. The dimensions of the weir, the points at which measurements were made, and the resulting calibration are shown on Figure 1. The abscissa is depth upstream from the check weir, H_1; the ordinate is depth downstream from the check weir, H_2; and the curved lines represent constant values of discharge. H_1 was measured far enough upstream from the weir to be above the drawdown effect. H_2 was measured downstream from the depression in the water surface caused by the drop over the weir. It is evident from the chart that the check weir operates partially submerged for practically all conditions of operation. The lower straight line shows the depth of flow, H_n, for various discharges previous to installation of the check weir. This was based on a very limited amount of field data. Completion of the field tests outlined in Appendix 1 of the September 10, 1954 field trip report will greatly enhance the value of the curves given in this figure and should be accomplished before definite steps are taken to modify the presently existing structure. The line labeled "stabilized operation" shows the depths H_1 and H_2 for the 6.0-foot check weir for different discharges after the flow has stabilized. Approximately 40 minutes is now required for the stage recorder in the field to stabilize after a change in discharge. Without further explanation with regard to the present check weir, the results of a second check weir will be considered.

MODEL STUDIES OF 7.5-FOOT CHECK WEIR

The height of the second weir studied in the model was 7.5 feet and the proportions are shown on Figure 2. The principal purpose of the higher check weir was to raise the level upstream and hence reduce the submergence effect. It was felt that increasing the depth upstream would also help the wave problem. The model calibration curves for the check weir 7.5 feet high are shown on Figure 2. The method of obtaining the curves on this figure was the same as explained for Figure 1. A comparison of the calibration curves for the two weirs (Figures 1 and 2) definitely indicates that the submergence effect has been reduced with the higher weir; stabilization Points A, B, C, D, and E more nearly approach the flat portion of the curves for Figure 2 than for Figure 1.

Once the calibration curves are definitely established it is only necessary to consider two variables, the depth upstream, H_1, and the discharge, so long as the depth measurement, H_1, is allowed to stabilize before a second discharge is set. For example, suppose a discharge of 200 cfs has been flowing through the gaging station sufficiently long for the gage to stabilize, the recorder should then show a depth of 5.8 feet (Point A, Figure 2). Should the operator wish to increase the flow to 400 cfs, he projects a vertical line from Point A on the 200-cfs curve up to Point X on the 400-cfs curve. By this procedure it is assumed that H_2 remains constant during the initial adjustment. The gates are raised and the discharge increased as rapidly as possible to give a depth of flow, H_1, of 7.5 feet. Upon arriving at this depth, the discharge flowing will be 400 cfs. An adjustment of the depths, both upstream and downstream from the check weir, will occur over the next 20 to 30 minutes. At the end of this period, the recorder should stabilize at Point B, or a depth of 7.85 feet.

As a second example, suppose the operator wishes to reduce the flow from 800 to 600 cfs. To do so, a vertical line is projected downward from Point D on the 800-cfs curve to Point Y on the 600-cfs curve. The depth upstream from the weir is reduced as quickly as possible from 9.8 to 9.4 feet. Upon reaching the second setting, the flow will be 600 cfs. In a matter of 20 to 30 minutes, the depths upstream and downstream will adjust along the 600-cfs line from Point Y to Point C. It should be remembered that a prolonged delay in properly adjusting the depth H_1 can lead to an error in discharge as it is assumed that the depth H_2 does not change during the initial adjustment. It is imperative that the present gate position indicators be revised or replaced and that the gates be weighted so that small movements can be detected while making adjustments.

It was necessary to utilize both model results and a very limited amount of prototype information to construct the calibration curves shown on Figure 2. It is quite likely that adjustments in the curves will be necessary, but these should be based on actual field measurements as was stated under the discussion of the 6-foot weir. A chart is included as Figure 3 to aid the operator in taking check points. The upper curve gives the experimental depth upstream from the 7.5-foot check weir, H_1, with respect to discharge after the stage recorder has stabilized. The intermediate curve gives the same experimental information upstream from the 6.0-foot check weir. The lower curve shows the downstream depth, H_2, based on a very limited number of field points, with respect to discharge after stabilization of the stage recorder. It is suggested that the operator check the lower curve and one other; the upper one depending on whether the 6.0-foot weir is allowed to remain or whether it is replaced by the 7.5-foot weir, as follows: Record the depth of flow upstream and also downstream from the check weir for a sufficient number of discharges to establish the validity of two curves. The discharge in each case should be measured by means of a current meter traverse. Plot the check points on Figure 3 and return a copy to the Hydraulic Laboratory. Upon receipt of the field results, the laboratory will prepare a complete rating table.

The proposed method of operation requires a special interpretation of the water stage recorder chart. The configuration of the chart will be the same as in the past (Figure 4). As the operator can set the correct discharge in a matter of 3 to 5 minutes, vertical lines should now replace the transition portion of the record, as indicated by the dotted lines on Figure 4. In other words, it is now assumed that a change in flow from 200 to 400 cfs or from 800 to 600 cfs occurs immediately. Although this procedure will require drawing additional lines on the chart, the recorded flow should be easier to read and simpler to compile.

WAVE SUPPRESSOR

Various types of wave suppressors were tried in the model, principally in the form of floating rafts. As the water level upstream from the check weir will vary as much as 10 feet, it was felt that floating rafts were the only possible solution. The majority of these performed quite well if held stationary, but they proved of little value when floating freely on the surface. This experience led to experimentation with a short-tube underpass type of suppressor which is currently being developed in the laboratory for application where the stage variation is small. The short-tube suppressor, as developed for this installation, showed promise even

at these greater depths. The short tube, proper, is 10 feet long, Figure 5, while an additional slab downstream on a slope of 4:1 gives a diffuser effect which reduces the loss through the short tube structure and helps iron out surface roughness downstream. The dimensions of the suppressor are the same for both the 6.0- and 7.5-foot check weirs with the exception that the bottom is set 5.0 feet above the floor of the channel for the 6.0-foot weir (Figure 5) and 5.75 feet above the channel floor for the 7.5-foot check weir (Figure 6). The short tube suppressor is being used beyond its usual application at both the higher and lower flows, thus the final setting in each case represents a compromise. In either case the model showed that the suppressor was effective for flows from 930 to 200 cfs.

The length of the suppressor must exceed the wave length to be most effective. Observation of the prototype with the 6.0-foot weir in place indicated that the waves upstream from the rafts were about 1 foot high with a wave length of approximately 14 feet. An attempt was made to generate waves of this magnitude by hand. The procedure was rather crude but this was in keeping with the model which was also crude. To illustrate the effectiveness of the wave suppressor, wave heights both upstream and downstream are shown on Figure 7 for several discharges with the 7.5-foot check weir installed. The information is strictly relative as the model channel was quite short downstream compared to the prototype. It can be observed from Figure 7 that for a discharge of 200 cfs the suppressor attenuated the waves to 0.2 of their original amplitude. The greatest effectiveness was apparent at a flow of 400 cfs at which the waves were practically eliminated. Wave amplitude was reduced to 0.13 of the original for discharges of 600 and 800 cfs while the initial waves were reduced to 0.30 of their original height for the maximum flow of 930 cfs. The suppressor will be above the water surface so will be ineffective for flows of less than 200 cfs. The chart in the lower right hand corner of Figure 7 was made without generation of artificial waves upstream. It shows that there is more disturbance downstream from the suppressor than upstream for a discharge of 930 cfs. This is also true to a lesser extent for the 800-cfs discharge. This is because the short tube suppressor is being used beyond its usual application. The downstream disturbances would be more objectionable were it not for the diffuser.

The loss of head through the wave suppressor for the 5.0-foot and for the 5.75-foot clearances is given by the curves on Figures 5 and 6, respectively. The head loss varies from 0.75 feet at 930 cfs to no measureable loss at 300 cfs, the lower limit of effectiveness, for the suppressor used in conjunction with the 6.0-foot check weir (Figure 5). When used in connection with the 7.5-foot check weir the

head loss varies from 0.75 foot at 930 cfs to no noticeable loss at 200 cfs, the lower limit of effectiveness (Figure 6). In the case of the latter, the water surface upstream from the suppressor will be about 1 foot below the top of the wall. To accommodate the wave action the walls should be raised about 2 feet upstream from the suppressor. With this in mind, the head wall of the suppressor is shown as close to the upstream end of the channel as feasible or Station 5+40. In the case of the suppressor used in conjunction with the 6.0-foot weir, the head wall is shown farther downstream, Station 5+60.

CONCLUSIONS

Two schemes have been discussed for improving the measurement of water downstream from the outlet works. Each scheme has certain advantages and disadvantages which will be considered here.

Advantages of 6.0-foot check weir:

1. Present weir can remain in place.

2. Head loss is 1.5 feet less than for 7.5-foot weir.

3. No need to raise walls of channel.

Advantages of 7.5-foot check weir:

1. Submergence effect is smaller, thus discharge can be set more readily with slightly increased accuracy than for the 6.0-foot weir.

2. The effect of any change occurring in the carrying capacity of the canal downstream would be less noticeable for the 7.5-foot weir than for the 6.0-foot weir. This is a very important consideration in that growth during the 1954 season was known to have altered the normal regimen of the canal.

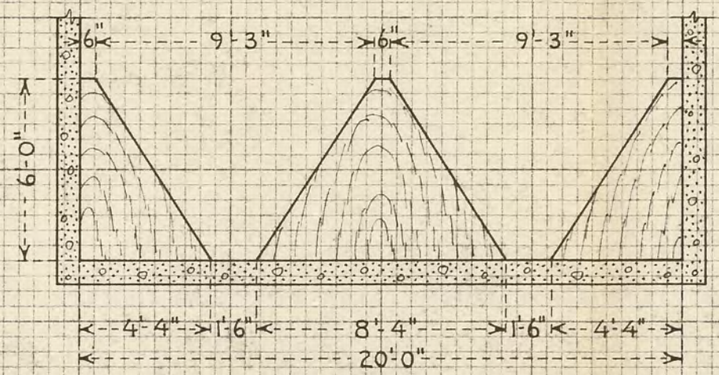
Both check weir arrangements have the disadvantage that they operate partially submerged at all flows. The higher the weir, of course, the less will be the submergence effect. Nevertheless, if conditions downstream from the weir can be held reasonably constant the method of measurement outlined herein will be found accurate and convenient for either check weir.

On the other hand, should wide fluctuations be expected in the carrying capacity of the canal downstream, we suggest that

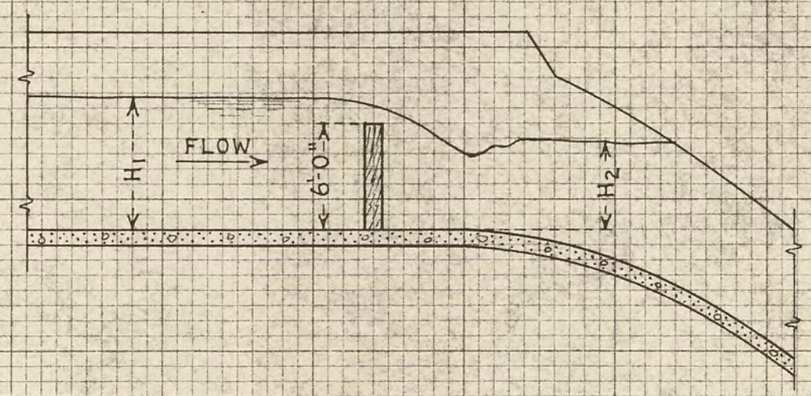
reconsideration be given to a proposal made during the first field trip. At that time it was suggested that a propeller meter, such as Sparling manufacturers, be installed in a convenient location in the inverted siphon. In this location the meter would give an instantaneous reading of the discharge at all times. The meter will also totalize the flow. The indication and totalized flow can be telemetered to a remote location.

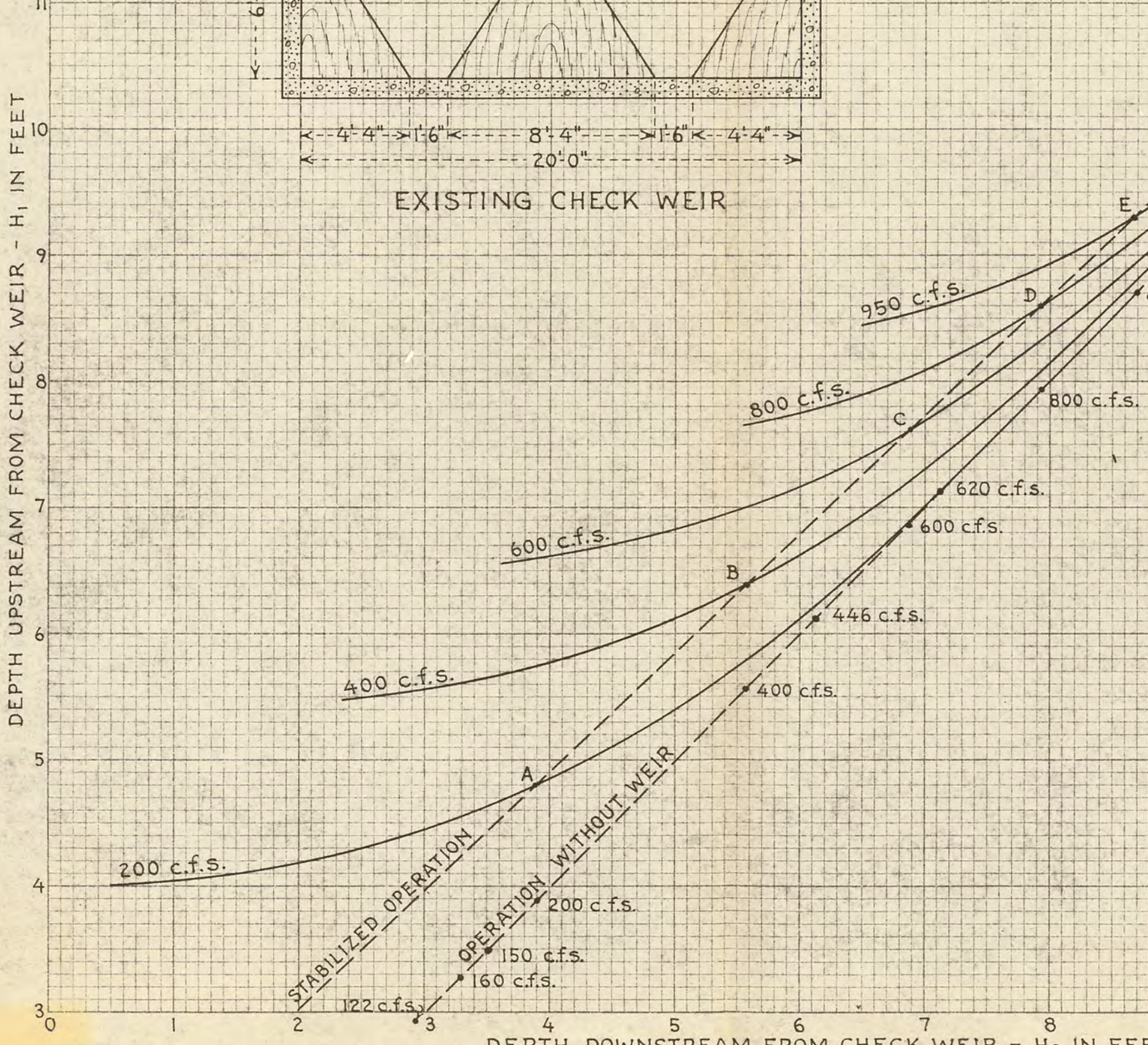
J. N. Bradley
C. W. Thomas

FIGURE 1



EXISTING CHECK WEIR



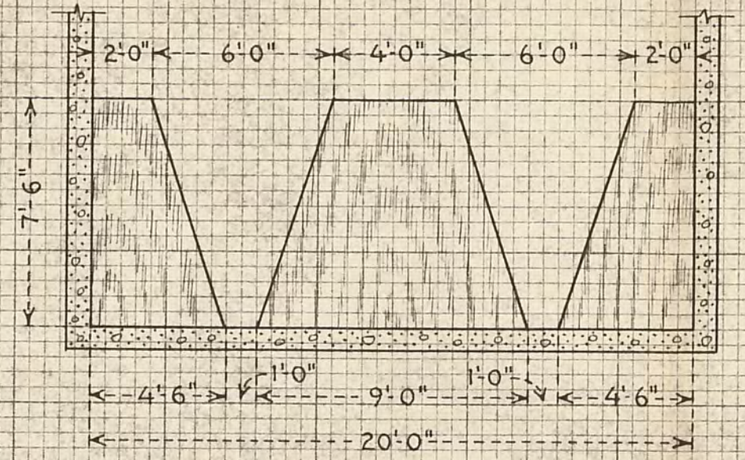


COLORADO - BIG THOMPSON PROJECT
FLATIRON AFTERBAY DAM OUTLET WORKS

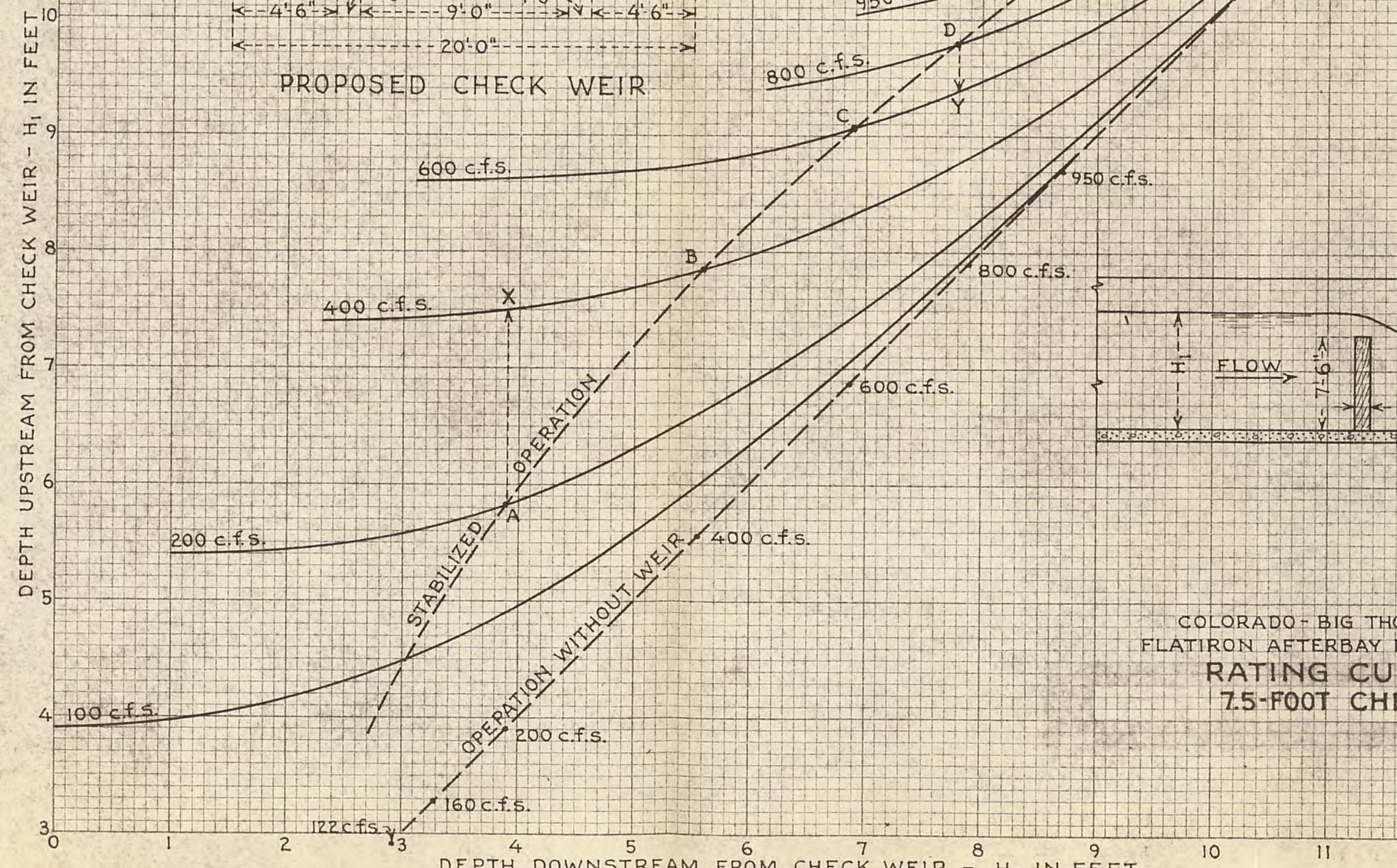
RATING CURVES FOR EXISTING CHECK WEIR

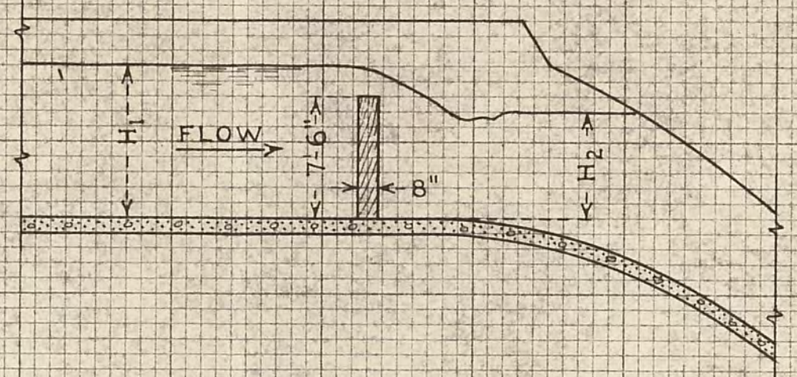
359-SL KEUFFEL & ESSER CO.
10 × 10 to the inch.
MADE IN U.S.A.

FIGURE 2



PROPOSED CHECK WEIR





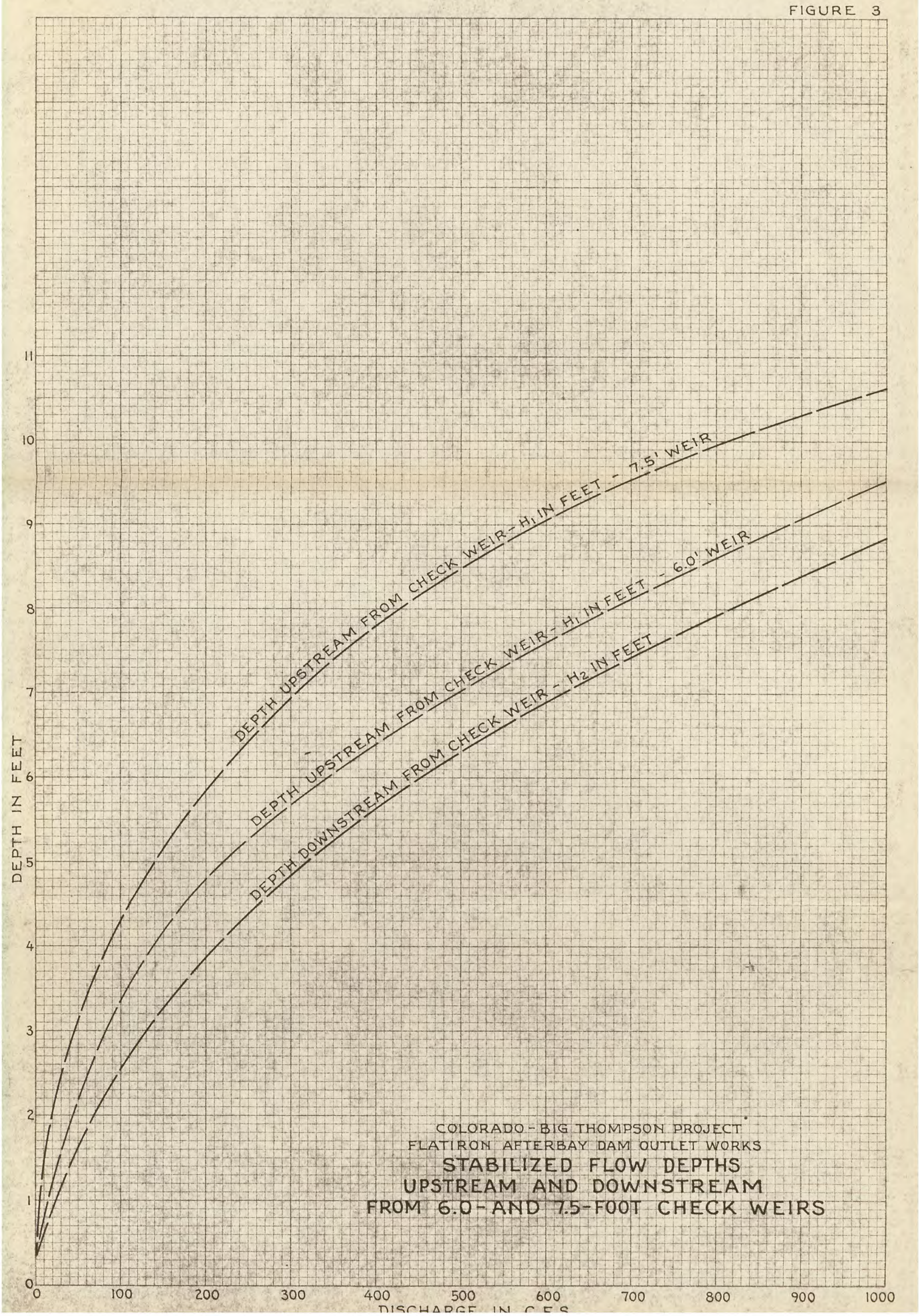
COLORADO - BIG THOMPSON PROJECT
FLATIRON AFTERBAY DAM OUTLET WORKS

RATING CURVES FOR 7.5-FOOT CHECK WEIR

349-SL KEUFFEL & ESSER CO.
10 X 10 to the Inch.
MADE IN U.S.A.

359-5L KEUFFEL & ESSER CO.
10 × 10 to the inch.
MADE IN U. S. A.

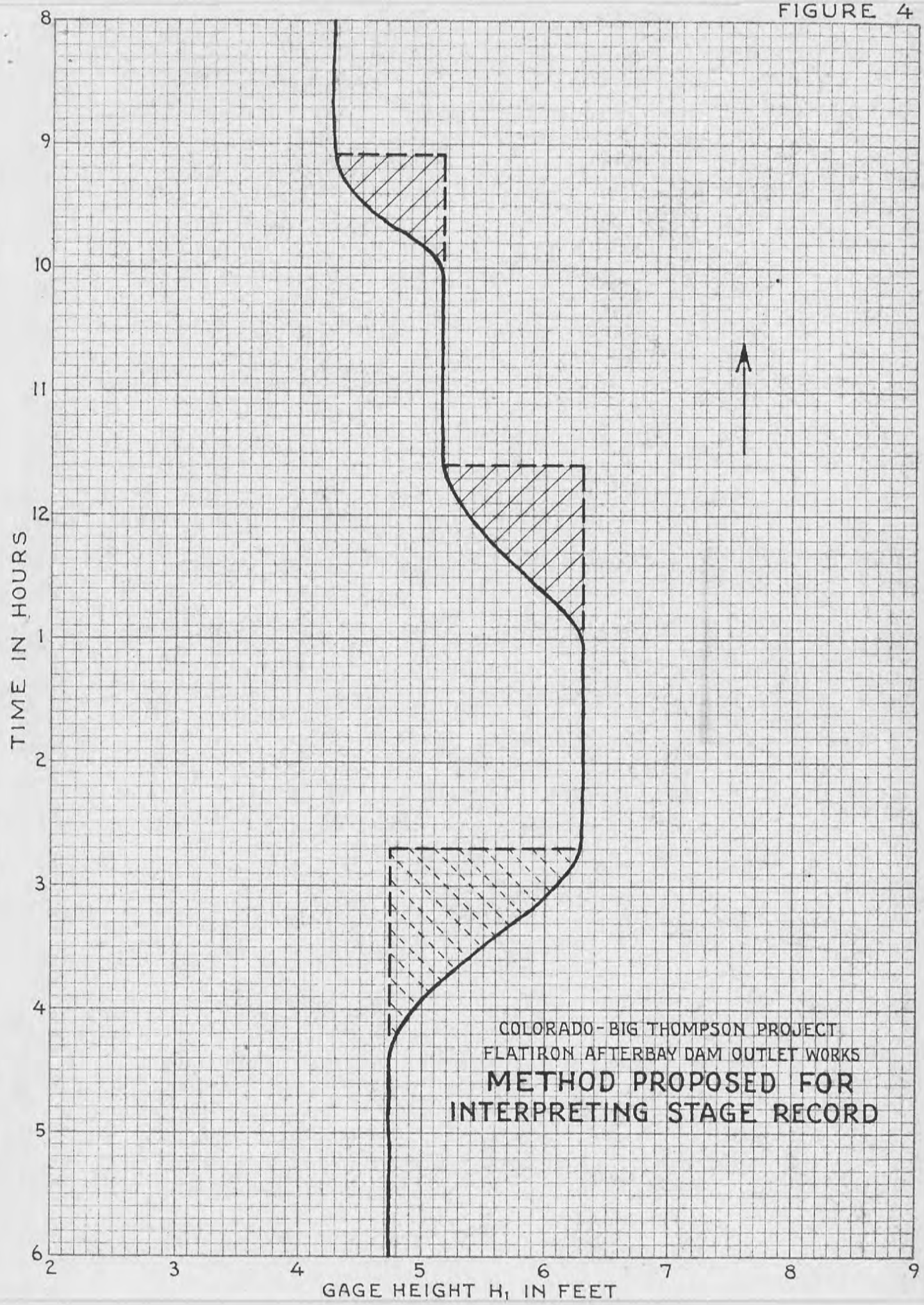
FIGURE 3



COLORADO - BIG THOMPSON PROJECT
FLATIRON AFTERBAY DAM OUTLET WORKS

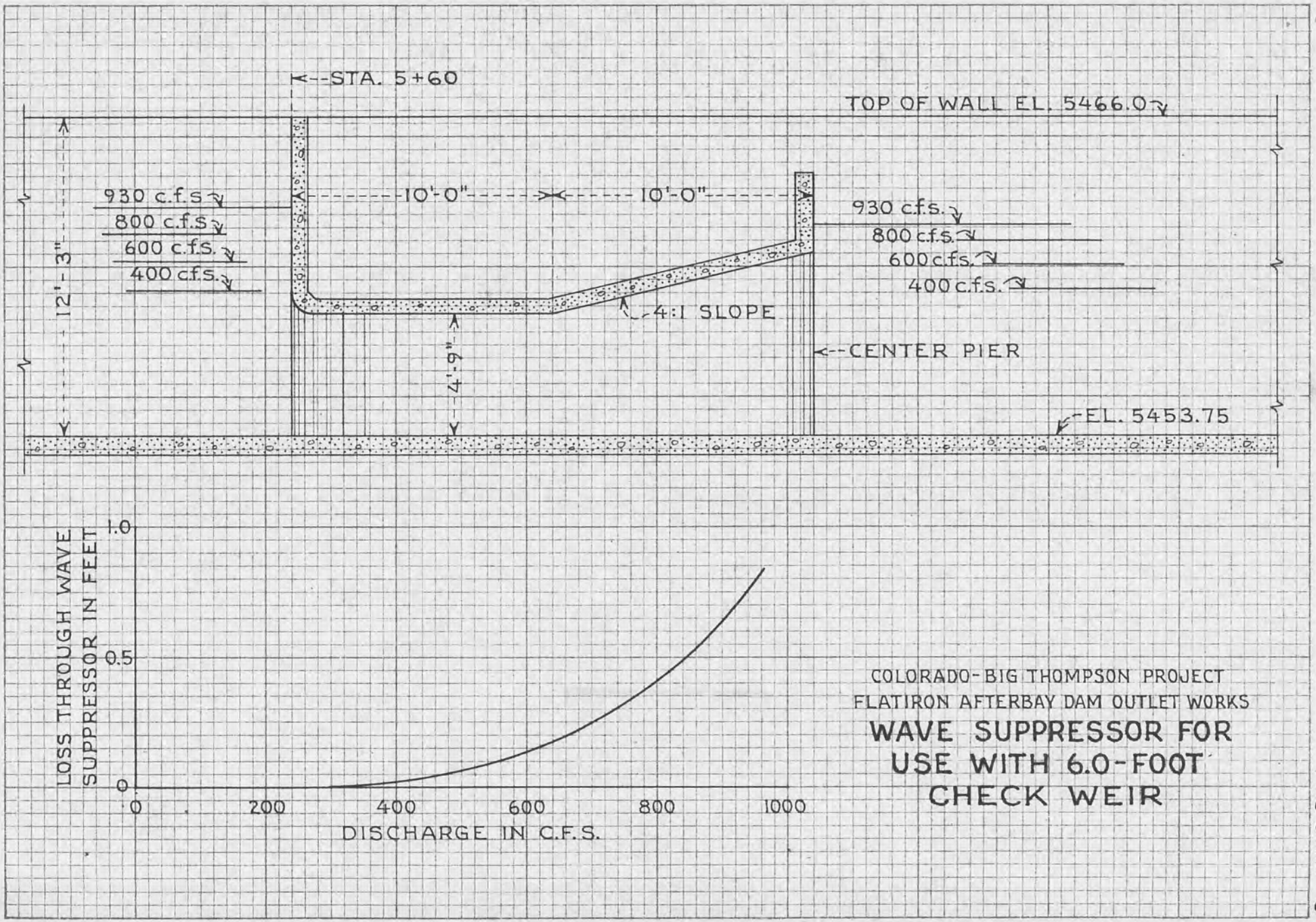
STABILIZED FLOW DEPTHS UPSTREAM AND DOWNSTREAM FROM 6.0- AND 7.5-FOOT CHECK WEIRS

FIGURE 4



359-3 KEUFFEL & ESSER CO.
10 × 10 to the inch.
MADE IN U.S.A.

359-5 KEUFFEL & ESSER CO.
10 × 10 to the inch.
MADE IN U. S. A.

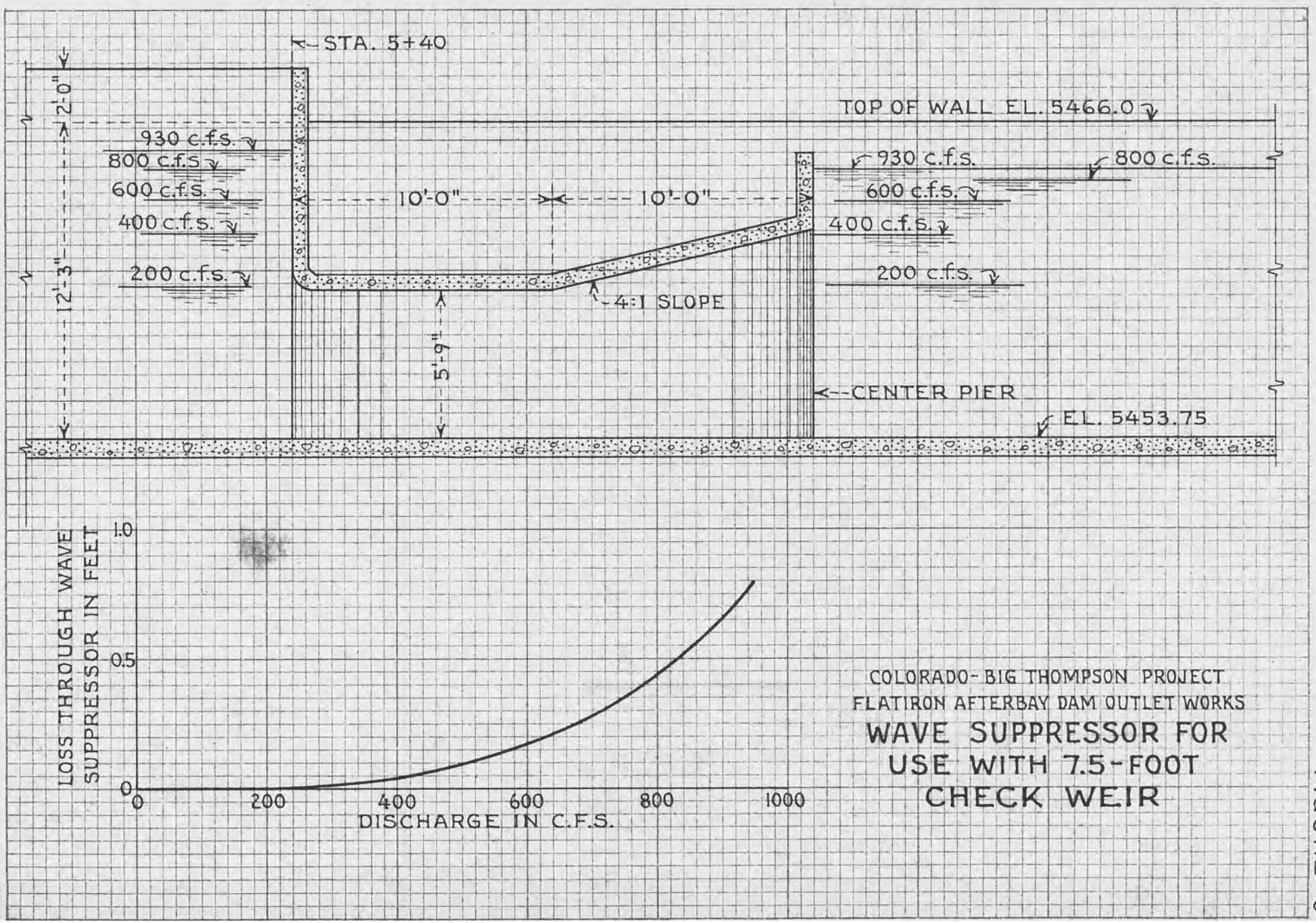


COLORADO-BIG THOMPSON PROJECT
FLATIRON AFTERBAY DAM OUTLET WORKS

WAVE SUPPRESSOR FOR USE WITH 6.0-FOOT CHECK WEIR

FIGURE 5

359-S KEUFFEL & ESSER CO.
10 × 10 to the Inch.
MADE IN U. S. A.

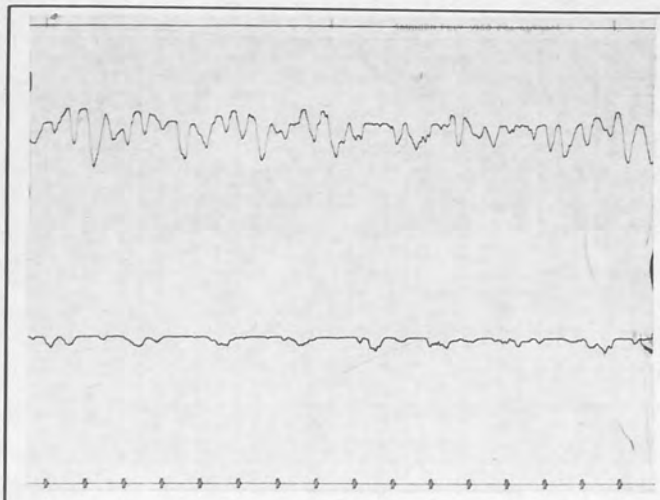


COLORADO-BIG THOMPSON PROJECT
FLATIRON AFTERBAY DAM OUTLET WORKS

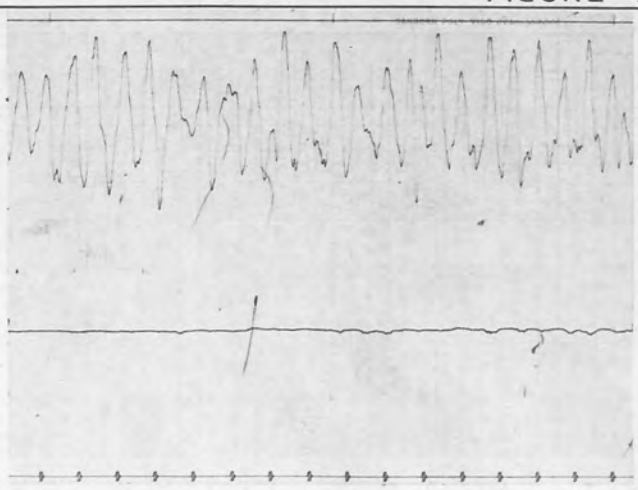
WAVE SUPPRESSOR FOR USE WITH 7.5-FOOT CHECK WEIR

FIGURE 6

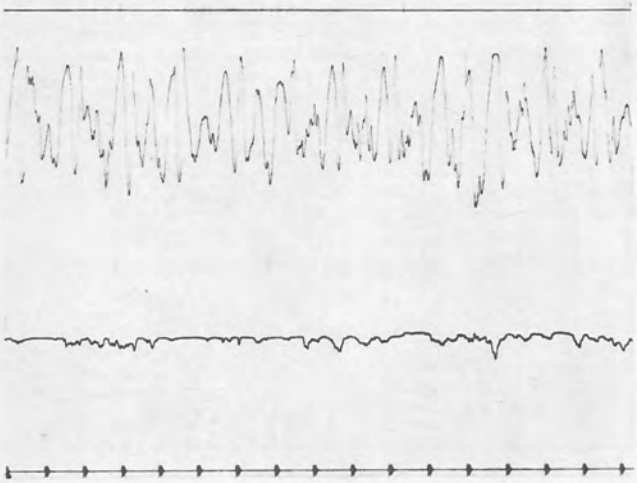
FIGURE 7



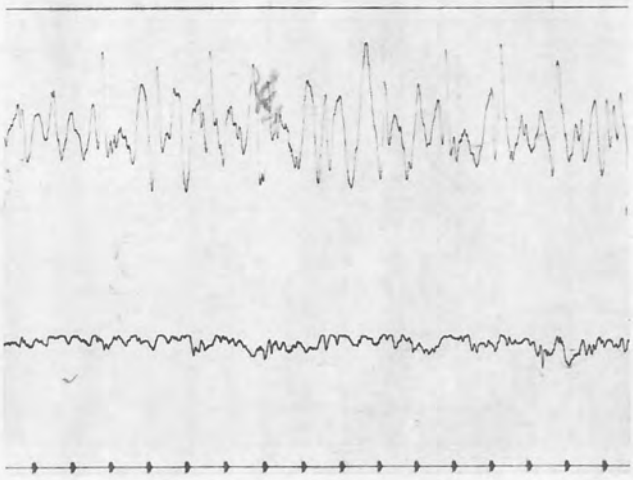
200 c.f.s., $\frac{M}{O}$ = 0.20



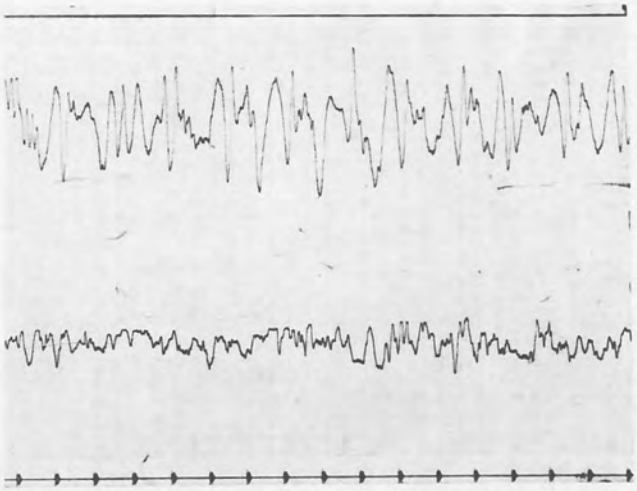
400 c.f.s., $\frac{M}{O}$ =0.05



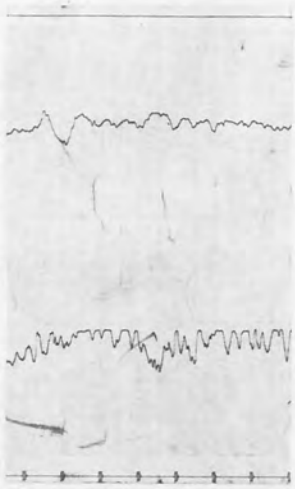
600 c.f.s., $\frac{M}{O}$ = 0.13



800 c.f.s., $\frac{M}{O}$ =0.13



930 c.f.s., $\frac{M}{O}$ = 0.30



930 c.f.s., $\frac{M}{O}$ = 1.33
WITHOUT GENERATION OF
ARTIFICIAL WAVE UPSTREAM

NOTE
O = ORIGINAL AMPLITUDE
M = MODIFIED AMPLITUDE

COLORADO-BIG THOMPSON PROJECT
FLATIRON AFTERBAY DAM OUTLET WORKS

COMPARISON OF WAVE HEIGHTS UPSTREAM AND DOWNSTREAM FROM WAVE SUPPRESSOR

FIGURE 8

Model on 1:10.3 scale showing check weir
7.5 feet high and wave suppressor