UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

PROGRESS REPORT V
RESEARCH STUDY ON STILLING BASINS ENERGY DISSIPATORS - AND ASSOCIATED APPURTENANCES

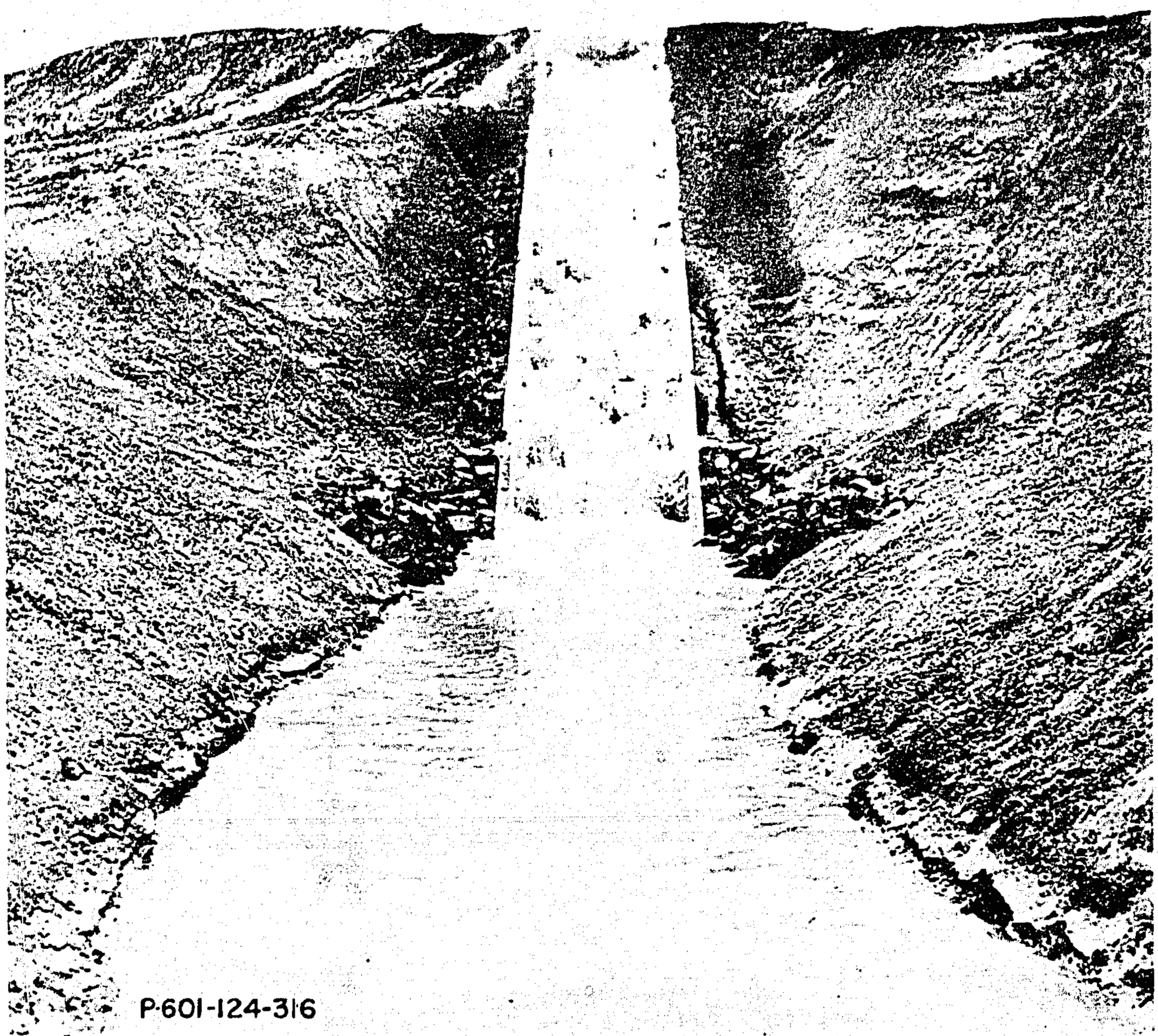
Section 9
Baffled Apron on 2:1 Slope for Canal or Spillway Drops (Basin IX)

Hydraulic Laboratory Report No. Hyd-445

DIVISION OF ENGINEERING LABORATORIES

OFFICE OF ASSISTANT COMMISSIONER AND CHIEF ENGINEER
DENVER, COLORADO

April 28, 1961



Main East Canal - Michaud Flats Project - Idaho
Kopp Wasteway Discharging 25 cfs (1/3 capacity)
Baffle piers - 18" wide & high - 18" spaces
Rows - 6' spacing
Chute - 9' wide & 90' long - 2:1 slope
- Training Walls 5' high

CONTENTS

CONTENTS--Continued

Prototype Photographs

FOREWORD

This report, Progress Report V, is the fifth in a series on research subjects included under the title of this report. Progress Report I, was superseded by Progress Report II, Hyd-399, which contains the first six sections listed below. Progress Report III, Hyd-415, contains Section 7. Progress Report IV, Hyd-446, contains Section 8.

Section 9 is contained in this report and covers Item 10 given in the "Scope" of the research program as originally planned and given in Progress Report II, page 3 Other numbered items will be completed and reported in future progress reports as time and funds permit.

Section 1--General Investigation of the Hydraulic Jump on a Horizontal Apron (Basin I)

Section 2--Stilling Basin for High Dam and Earth Dam Spillways and Large Canal Structures (Basin II)

Section 3--Short Stilling Basin for Canal Structures, Small Outlet Works and Small Spillways (Basin III)

Section 4--Stilling Basin and Wave Suppressors for Canal Structures, Outlet Works, and Diversion Dams (Basin IV)

Section 5--Stilling Basin with Sloping Apron (Basin V)

Section 6--Stilling Basin for Pipe or Open Channel Outlets--No Tail Water Required (Basin VI)

Section 7--Slotted and Solid Buckets for High, Medium, and Low Dam Spillways (Basin VII)

Section 8--Stilling Basin for High Head Outlet Works Utilizing Hollow-jet Valve Control (Basin VIII)

Section 9--Baffled Apron on 2:1 Slope for Canal or Spillway Drops (Basin IX)

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Office of Assistant Commissioner
and Chief Engineer--Denver
Division of Engineering Laboratories
Hydraulic Laboratory Branch
Denver, Colorado
April 28, 1961

Laboratory Report No. Hyd-445
Compiled by: A. J. Peterka
Reviewed by: J. W. Ball
Submitted by: H. M. Martin

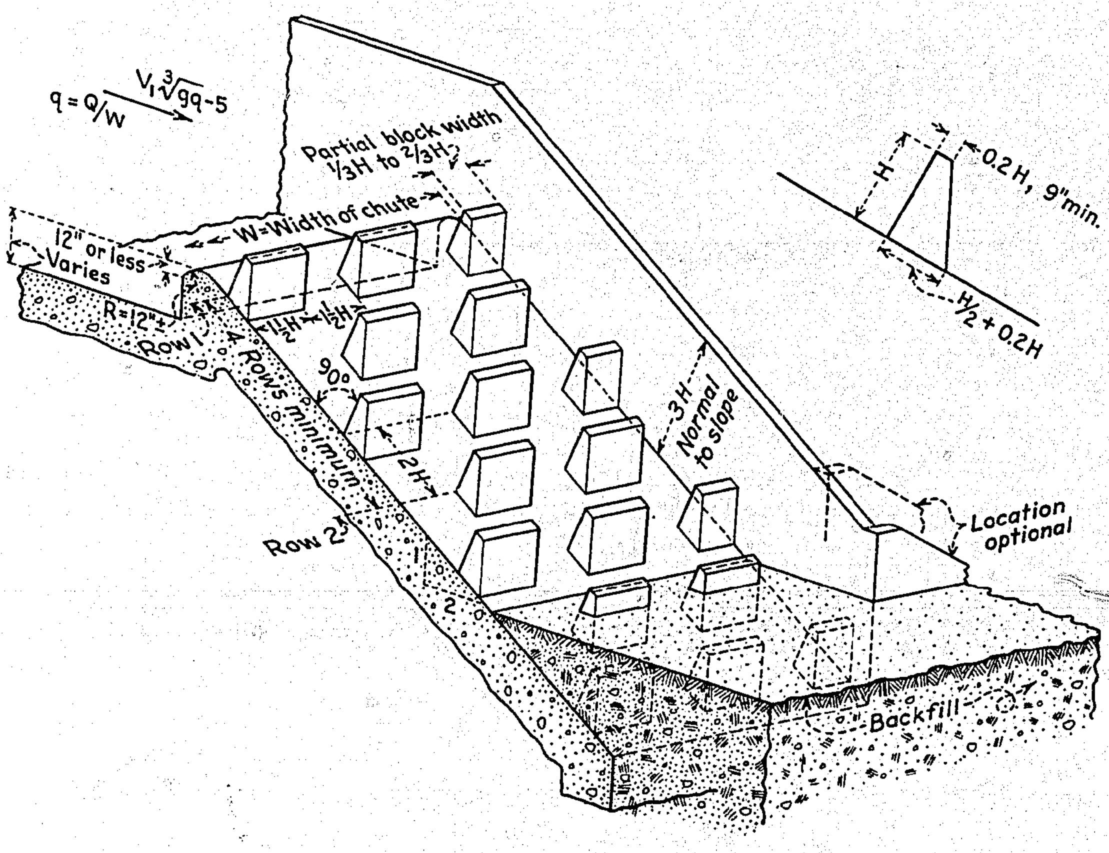
Subject: Progress Report V--Research study on stilling basins, energy dissipators, and associated appurtenances--Section 9, Baffled apron on 2:1 slope for canal or spillway drops--(Basin IX)

ACKNOWLEDGMENTS

This study was conducted in the Hydraulic Laboratory of the Bureau of Reclamation in Denver, Colorado, by Hydraulic Laboratory personnel. During the course of the hydraulic model tests, close cooperation with Canals Branch personnel resulted in a recommended structure which was acceptable from a structural as well as a hydraulic viewpoint. Cooperation by field personnel, particularly those in Region 7, helped to evaluate the prototype performance and the model-prototype conformance by supplying data, comments, and photographs of prototype installations. Foreign trainee engineers from Thailand, Tweechai Makaman and Chareuk Nonthathum helped obtain the model data.

SUMMARY

Baffled aprons or chutes are used in flow ways where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. Since the flow velocities entering the downstream channel are relatively low, no stilling basin is required. The chute, on a 2:1 slope, may be designed to discharge up to 60 cubic feet per second per foot of width, and the drop may be as high as structurally feasible. The lower end of the chute is constructed to below streambed level and backfilled as necessary. Degradation or scour of the streambed, therefore, does not adversely affect the performance of the structure. The hydraulic design procedure (simplified) is given in the numbered steps. More detailed explanations are given in the text of this report. Also discussed are the hydraulic laboratory tests used to develop the hydraulic design and the performance of prototype structures in operation.



DESIGN PROCEDURE

(Refer to Sketch in Summary)

1. The baffled apron should be designed for the maximum expected discharge, Q.

2. The unit design discharge $q = \frac{Q}{W}$ may be as high as 60 cubic feet per second per foot of chute width, W. Less severe flow conditions at the base of the chute exist for 35 cubic feet per second and a relatively mild condition occurs for unit discharges of 20 cubic feet per second and less.

3. Entrance velocity, V_1, should be as low as practical. Ideal conditions exist when $V_1 = \sqrt[3]{gq} - 5$, Curve D, Figure 23. Flow conditions are not acceptable when $V_1 = \sqrt[3]{gq}$, Curve C, Figure 23.

4. The vertical offset between the approach channel floor and the chute is used to create a stilling pool or desirable V_1 and will vary in individual installations; Figures 1, 3, 5, and 7 show various types of approach pools. Use a short radius curve to provide a crest on the 2:1 sloping chute. Place the first row of baffle piers close to the top of the chute no more than 12 inches in elevation below the crest.

5. The baffle pier height, H, should be about 0.8 D_c, Curve B, Figure 23. The critical depth on the rectangular chute is $D_c = \sqrt[3]{\frac{q^2}{g}}$, Curve A. Baffle pier height is not a critical dimension but should not be less than recommended. The height may be increased to 0.9 D_c, Figure 23.

6. Baffle pier widths and spaces should be equal, preferably about 3/2 H, but not less than H. Other baffle pier dimensions are not critical; suggested cross section is shown. Partial blocks, width 1/3 H to 2/3 H, should be placed against the training walls in Rows 1, 3, 5, 7, etc., alternating with spaces of the same width in Rows 2, 4, 6, etc.

7. The slope distance between rows of baffle piers should be 2 H, twice the baffle height H. When the baffle height is less than 3 feet, the row spacing may be greater than 2 H but should not exceed 6 feet.

8. The baffle piers are usually constructed with their upstream faces normal to the chute surface; however, piers with vertical faces may be used. Vertical face piers tend to produce more splash and less bed scour, but differences are not significant.

9. Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. Additional rows beyond the fourth maintain the control established above, and as many rows may be constructed as is necessary. The chute should be extended to below the normal downstream channel elevation as explained in the text of this report, and at least one row of baffles should be buried in the backfill.

10. The chute training walls should be three times as high as the baffle piers (measured normal to the chute floor) to contain the main flow of water and splash. It is impractical to increase the wall heights to contain all the splash.

11. Riprap consisting of 6- to 12-inch stones should be placed at the downstream ends of the training walls to prevent eddies from undermining the walls. Figures 24 to 36 show effective and ineffective methods of placement on field structures.

INTRODUCTION

Baffled aprons or chutes have been in use in the field for many years. The fact that many of these structures have been built and have performed satisfactorily indicates that they are practical and that in many cases they are an economical answer to the problem of dissipating energy. Baffled chutes are used to dissipate the energy in the flow at a drop and are most often used on canal wasteways or drops (see frontispiece). They require no initial tail water to be effective although channel bed scour is not as deep and is less extensive when the tail water forms a pool into which the flow discharges. The multiple rows of baffle piers on the chute prevent excessive acceleration of the flow and provide a reasonable terminal velocity, regardless of the height of drop. Since flow passes over, between, and around the baffle piers, it is not possible to define the flow conditions in the chute in usual terms. The flow appears to slow down at each baffle pier and accelerate after passing the pier, the degree depending on the discharge and the height of the baffle piers. Lower unit discharges result in lower terminal velocities on the chute.

The chute is constructed on an excavated 2:1 slope extending to below the channel bottom. Backfill is placed over one or more rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occur, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur, the scour creates a stilling pool at the downstream end of the chute, stabilizing the scour pattern. If excessive degradation occurs, it may become necessary to extend the chute.

A number of baffled chutes have been constructed and tested in the field. Some of the existing designs were obtained from hydraulic model tests made for the particular structure. Other existing designs have been obtained by modifying model tested designs to the extent believed necessary to account for local changes in topography and flow conditions. The generalized design procedures discussed in this report were obtained from test results on several models of baffled chutes and from one model which was modified as necessary to obtain information of value in designing a chute for any installation.

From a study of the existing baffled chutes, it was apparent that certain features of the design, such as the 2:1 chute slope, had been utilized in each installation. Thus, when the present series of tests to generalize the overchute design were begun, these features were considered to be standard and did not need to be evaluated as variables.

All existing data are for 2:1 sloping baffled aprons. It is believed that the general rules of this report may be used to design baffled aprons having flatter slopes. Preliminary laboratory tests indicate that a more-than-adequate design results when the rules are applied to 3:1 slopes. No information of any kind is available for slopes steeper than 2:1. Slopes flatter and steeper than 2:1 will be investigated and reported in the future.

DEVELOPMENT OF BAFFLED APRON FEATURES

In tests made prior to the generalization tests, individual models were constructed and tested to provide a stilling basin upstream from the baffled chute and to develop the baffled chute and stilling basin as a complete unit. Three of these model tests are reported in detail in Hydraulic Laboratory Report No. Hyd-359 "Hydraulic Model Studies of the Outlet Control Structure; Culvert Under Dike; and Wash Overchute at Station 938+00--Wellton-Mohawk Division, Gila Project, Arizona." The fourth study--Hydraulic Model Studies of the Check Intake Structure--Potholes East Canal, Columbia River Basin Project, Washington, is reported in Hydraulic Laboratory Report No. Hyd-411.

A brief summary of the parts of the individual studies which influenced the generalization test procedure is given below.

Wash Overchute at Station 938+00

The structure shown in Figure 1 was developed from hydraulic model tests on a 1:12 scale model. The design discharge was 1,250 cubic feet per second and the chute was 36 feet wide, making the unit discharge about 35 cubic feet per second. After tests had been made to develop the stilling basin upstream from the chute, six different arrangements of baffle piers on the chute were tested, Figure 2.

For Design 1, the missing row of baffle piers at the top of the chute permitted the flow to continue to accelerate, strike the second row, and jump over the third row of piers. In Design 1A, the top row of baffle piers was in place; the resulting scour depth in the sand bed at the base of the chute was 7 feet, 5 feet less than for Design 1. In Design 2A, the spacing of the rows was reduced from 6 feet to 4 feet 3 inches. This resulted in no apparent difference in the operation of the structure. Scour depth was 7 feet. In Design 3, a greater number of narrow baffle piers were used. These produced a rougher water surface and a scour depth of 8 feet. Stepped face baffle piers were substituted in Design 4. Flow appearance was good and scour depth was 7 feet. For Design 5, the upstream row of baffle piers was reduced to 2 feet in height. Flow appearance was good and scour depth was 5.5 feet. In Design 6, baffle piers 6 feet high and 2 feet square in cross section were used. Flow appearance was poor and scour depth was 9 feet.

Considering all factors including stilling basin performance, flow appearance, scour depth and extent, and structural problems, it was concluded that the arrangement shown in Figure 1 was most desirable. The piers were 3 feet high and 4 feet 6 inches wide placed in staggered rows 6 feet apart. Water surface profiles and baffle pier pressures for this arrangement are shown in Figure 1.

Culvert Under Dike

The structure developed from 1:12 scale hydraulic model tests is shown in Figure 3. The design discharge was 1,250 cubic feet per second and the chute width was 31 feet 6 inches, making the unit design discharge approximately 40 cubic feet per second. After tests had been made to develop the culvert and the stilling basin upstream from the chute, scour tests were made with baffle piers 3, 4, and 5 feet high on the chute. Results of these tests disclosed the depth of scour for the 4- and 5-foot piers to be approximately the same as that obtained for the 3-foot high piers. Piers 3 feet high provided the best overall performance. The appearance of the design flow and the resulting scour pattern are shown in Figure 4. Water surface profiles and baffle pier pressures for the recommended structure are shown in Figure 3.

Outlet Control Structure

The outlet control structure stilling basin and baffled chute were developed from 1:24 scale hydraulic model tests on a half model and are shown in Figure 5. The chute width is 140 feet and the design discharge is 7,000 cubic feet per second, making the unit discharge 50 cubic feet per second. Tests showed the stilling basin to be adequate for the design flow released through the control notches, Figure 6A. Baffle piers 3 feet high in rows spaced 6 feet apart provided satisfactory flow in the chute. Scour depth was about 5 feet, as shown in Figure 6B.

Potholes East Canal

A 1:16 scale model was used in this study. Figure 7 shows the developed design which includes the gated control structure, stilling basin, and baffled chute. The chute is 64 feet wide and the discharge is 3,900 cubic feet per second, making the unit discharge about 61 cubic feet per second. Baffle piers 4 feet 6 inches high were tested in horizontal rows spaced at intervals of 9 and 6 feet. No differences in the appearance of the flow were apparent for the two spacings, but the scour depth over most of the area was 2 feet less with the larger row spacing. Figure 8 shows the structure in operation and the scour test results.

Figure 9 shows the flow appearance and the resulting scour for a unit discharge of 50 cubic feet per second and the 9-foot row spacing. The scour depth is about 1 foot less than for 60 cubic feet per second. Also shown in Figure 9 are flow conditions for unit discharges of 31 and 16 second-feet.

Normal Versus Vertical Pier Faces

Tests were made to determine the effect of constructing the pier faces vertical rather than normal to the chute, Figure 10. For a unit discharge of 35 cubic feet per second there was very little difference in performance between vertical and normal placement. Figure 10 shows that the splash was about 5 feet lower with vertical face piers as indicated by the darker wetted area in the photographs. Figure 10 shows the scour patterns obtained after 1/2 hour of model operation. There was slightly less scour in the vicinity of the wing wall when normal pier faces were used. The scour pocket (Elevation 906) along the wall of symmetry in the model probably would not have occurred if the full width of the model had been built.

The same scour tendencies were prevalent for a unit discharge of 61 cubic feet per second, Figure 11. There was less overall erosion with the pier faces normal to the slope although the scour depths were the same.

GENERALIZATION TESTS

The Model

A 1:16 scale model of a 171-foot length of the Potholes East Canal between Stations 1367+69 and 1369+40 was used for the generalization tests. Included were a reach of approach canal, the gate control structure upstream from the baffled apron, the 2:1 sloping apron, and approximately 80 feet of outlet channel. To make the model features as large as possible, only one-half of the structure was built and tested, Figure 12. The wall on the right in the photograph is the wall of symmetry and is on the centerline of the full-sized structure. The gate structure, shown in Figure 7, was made removable so that studies could be made for low as well as high velocities at the top of the baffled chute. The channel downstream from the baffled chute was molded in sand having a mean diameter of about 0.5 millimeters. Discharges were measured through calibrated venturi meters. Velocities were measured with a pitot tube.

In the final tests, a painted splash board was installed along the wall of symmetry to record the height of splash. The paint on the board absorbed the splash and showed the splash area as a darker color.

Testing Procedure

The tests were concerned primarily with the effectiveness of the baffled chute in preventing acceleration of the flow down the chute. This was judged by the appearance or profile of the flow in the chute, the depth and extent of scour in the downstream channel, and by the height of splash shown on the splashboard. For each test, the channel was molded level at the base of the chute at Elevation 914 and the model was operated for 30 minutes, after which the erosion in the channel bed was measured. Relative depths were made visible with contour lines of white string. The tailgate in the model was set to provide a tail-water depth of 2 feet (Elevation 916) in the downstream channel for a discharge of 20 cubic feet per second per foot of width of chute. The tailgate setting was not changed for larger discharges so that the tail-water depth did not build up as much as it normally would in a field structure. The resulting depths for discharges of 35, 50, and 60 cubic feet per second were 2.5, 3.0, and 3.5 feet, respectively. For tests with gate controlled flow 15.3 feet of depth was maintained upstream from the gates. For the free flow tests, the gate structure was removed and the normal depth for the particular flow being tested was maintained in the canal. The elevations shown in the drawings and photographs are compatible and apply for a model scale of 1:16.

Four baffle pier heights were included in the original testing program: 3 feet, 4 feet, 5 feet, and 6 feet, measured normal to the 2:1 sloping chute, Figures 13, 14, 15, and 16. Each height was tested with the spacing between rows fixed at twice the baffle height. The baffle pier widths and spacing within each row were equal to one and one-half times the baffle height. For each baffle pier arrangement, individual tests were made for 20, 35, 50, and 60 cubic feet per second per foot of width.

Water surface measurements were made with a point gage and a scale taking the maximum water surface at each measured point. Since the water surface at any point on the chute varies with respect to time, the profiles obtained are higher than the profiles shown in a photograph of the same test. The measured profiles of Figures 13 through 16 are believed to be more dependable for estimating necessary wall heights than the photographs in the report which portray the appearance of the flow at a particular moment.

Velocity measurements were attempted in the locations shown in Figures 13 through 16. At Stations 0 and 1, the flow was smooth and uniform; the data are accurate. On the slope, where turbulence and unsteadiness are characteristic of the flow, only the measurements at Point 3 were considered to be dependable. Even these showed some inconsistencies, but velocity curves for the range of discharges were determined using general knowledge and judgment to adjust the obviously incorrect measured values. The curves shown in Figure 17 are believed to be reasonably accurate and were found useful in evaluating the height of the baffle piers in terms of general performance. The velocity measurements in other parts of the chute are summarized in the notes of Figure 17.

Test Results

For all baffle pier heights and a test discharge of 60 cubic feet per second per foot of width, the flow entering the chute had a bottom velocity of about 1.8 feet per second and reached a maximum of 5.5 feet per second at Point 2. At Point 3, the velocity was dependent on the baffle pier height as shown in Figure 17. The average velocity, $V = Q/A$, at the top of the chute was 7 feet per second. For a unit discharge of 20 cubic feet per second, the initial bottom velocity was about 1.1 feet per second, reached a maximum of about 4.5 feet per second at Point 2 and was reduced at Point 3 as shown in Figure 17. The average velocity at the top of the chute was 3 feet per second. The velocities in themselves are not important in generalizing the design of the baffled chute, but do help the reader to visualize the velocity distribution on the chute. With low baffles and high discharges, the bottom velocity at Point 3, Figure 17, is considerably higher than when higher baffles are used with the same discharge. This is because a larger volume of water passes over the tops of the low baffles and the decelerating effect of the baffles on the entire volume of flow is less, Figure 18.

Although the velocity at Point 3, for 60 cubic feet per second per foot and the 6-foot baffles, was considerably less than for the 3-foot baffles, the erosion was more severe. When the 6-foot baffles were used, erosion was to Elevation 900, exposing the end of the chute. When the 3-foot baffles were used, erosion was only to Elevation 905; the extent of the erosion was also less. Appearance of the flow on the chute and in the downstream channel for the 5-foot baffles, Figure 19B, was better than for the 6-foot baffles; appearance for the 4-foot baffles was still better, Figure 19A. The erosion patterns for the 4- and 5-foot baffles were better than for the 3- or 6-foot baffles. The least splash occurred with the 3- and 4-foot baffles.

The same relative performance was evident for the 50 cubic feet per second per foot flow. The 4- and 5-foot baffles produced the best flow appearance and the 5-foot baffles produced the most favorable scour and splash patterns. Figure 19 shows the flow for 50 cubic feet per second per foot with the 4- and 5-foot baffles.

At 35 cubic feet per second per foot, the flow patterns were all satisfactory in appearance. The most favorable erosion patterns occurred with the 3- and 4-foot baffles, the deepest erosion being to Elevation 906. The deepest erosion hole with the 5-foot baffles was to Elevation 905. Splash was least with the 4-foot baffles but was not much greater with the 3-foot baffles. Figure 20A shows the flow pattern and erosion for the 3-foot baffles and 35 cubic feet per second per foot of width.

For 20 cubic feet per second per foot, flow appearances were all good but the 3-foot baffles showed a slightly better flow pattern. The scour pattern was also most favorable with the 3-foot baffles. The deepest erosion hole was to Elevation 908. Erosion with the 4-foot baffles was to Elevation 907, 5-foot baffles to Elevation 905, and 6-foot baffles to Elevation 906. The 4-foot baffles produced the least erosion near the wing wall at the end of the chute. The splash patterns for the 3-, 4-, and 5-foot baffles were almost identical, but the splash for the 6-foot baffles was somewhat greater. Figure 20B shows the flow pattern and erosion for the 3-foot baffles and 20 cubic feet per second per foot of width.

After partial analysis of the test data, it was apparent that baffles 2 feet high might provide ample scour protection for a design discharge of 20 cubic feet per second per foot of width. Scour tests showed this to be true, although scour depths were about the same as found for the 3-foot high baffles. For a discharge of 35 cubic feet per second per foot, the scour depth exceeded that for the 3-foot baffles and the flow appearance was not good; too much high velocity flow passed over the tops of the piers.

A summary of scour test data is given in Table 1. Listed are the lowest scour-hole elevations (1) at the wing wall visible in the photographs, (2) downstream from the chute, and (3) the average of the elevations in (1) and (2). Scour along the wall of symmetry was not considered because the adjacent wall affected the scour depth adversely.

Figures 21 and 22 show three groups of curves, A, B, and C, plotted from the data in Table 1, one group, D, plotted from the velocity curves of Figure 17, and one group, E, plotted from the splash tests. In Group A, the scour depth at the wing wall is a minimum for the 2- and 3-foot high piers for a discharge of 20 cubic feet per second. At 35 cubic feet per second, the 3- and 4-foot piers provided the minimum scour depth and at 60 cubic feet per second, the 4- and 5-foot piers provided minimum scour depth. In Groups B and C, the depth of scour at the end of the chute, and the average of the maximum depths show the same general trend except that the 3- and 4-foot piers show minimum scour for the maximum design discharge of 60 cubic feet per second.

If envelope curves were drawn in A, B, and C to determine the height of baffle pier which produces the least scour, the pier heights would vary from 2 feet for 20 cubic feet per second in all cases to 3, 4, or 5 feet in the other cases for 60 cubic feet per second. An envelope curve drawn on the velocity curves to determine the height of pier to produce the lowest velocity on the chute would indicate baffle piers 6 feet high for all discharges. Since 6-foot piers produce maximum scour depth, a compromise must be made. Scour depth is more important than the velocity on the chute, and since the water surface profiles of Figures 13 to 16 favor the lower baffle piers, the most practical height for the baffle piers is indicated by the circles, shown in Figures 21 and 22. The circles have been plotted on each set of curves and represent baffle piers 2 feet high for design discharge 20 cubic feet per second; 3 feet high for design discharge 35 cubic feet per second; 3.8 feet high for design discharge 50 cubic feet per second and 4.3 feet high for design discharge 60 cubic feet per second.

Piers of this height produce near minimum depths of scour for all design discharges and near minimum velocity on the chute. In addition, they produce near minimum splash for all discharges as shown by Curves E of Figure 22. Finally, an inspection of the photographs made of each test (only a few representative photographs are reproduced in this report) show that the flow appearance is satisfactory for each of the recommended piers.

The height of baffle piers shown by the circles in Figures 21 and 22 may be expressed as 0.8 D_c where $D_c = \sqrt[3]{\frac{q^2}{g}}$ = critical depth on the chute. Curve B, Figure 23, shows the recommended height of baffle piers.

GENERALIZATION OF THE HYDRAULIC DESIGN

The general rules for the design of baffled overchutes have been derived from tests on individual models, prototype experiences, and on the verification tests described in detail in this report. Since many of the rules are flexible to a certain degree, an attempt has been made in the following discussion to indicate how rigidly each rule applies.

Design Discharge

The chute should be designed for the full capacity expected to be passed through the structure. The maximum unit discharge may be as high as 60 cubic feet per second. Generally speaking, however, unit discharges in the range of 35 cubic feet per second provide better flow conditions on the chute and in the downstream channel, and a unit discharge of 20 cubic feet per second provides a relatively mild condition.

In installations where downstream degradation is not a problem and an energy dissipating pool can be expected to form at the base of the chute, more acceptable operation for a unit discharge of 60 cubic feet per second will occur than in steeper channels where no energy dissipation occurs. The design maximum unit discharge may be limited by the economics of baffle pier sizes or chute training wall heights. A wider chute with a correspondingly reduced unit discharge may provide a more economical structure.

Reports have been received from the field that baffled aprons designed for a unit discharge of 60 cubic feet per second have operated at estimated values up to 120 cubic feet per second for short periods without excessive erosion and spillage over the walls. This is mentioned only to indicate that a baffled apron can discharge more than the design flow without immediate disaster; it is not intended to suggest that baffled aprons should be underdesigned as a matter of general practice.

Chute Entrance

Flow entering the chute should be well distributed laterally across the width of the chute. The velocity should be well below the critical velocity, preferably the values shown in Curve D of Figure 23. The critical velocity in a rectangular channel is $V_c = \sqrt[3]{gq}$. Velocities near critical or above cause the flow to be thrown vertically into the air after striking the first baffle pier. When the initial velocity is high, the flow has been observed in a model to pass completely over the next row or two of baffle piers. The baffled apron is not a device to reduce the velocity of the incoming flow; rather, it is intended only to prevent excessive acceleration of the flow passing down the chute.

To insure low velocities at the upstream end of the chute, it may be necessary to provide a short energy dissipating pool similar to the ones shown in Figures 1, 3, 5, and 7. A hydraulic jump stilling basin may be suitable if the flow is discharged under a gate as shown in Figure 7. The sequent or conjugate depth in the basin should be maintained to prevent jump sweep out, but the basin length may be considerably less than a conventional hydraulic jump basin since the primary purpose of this pool is to reduce the average velocity. This is accomplished in the upstream portion of the stilling basin. The downstream third of the basin may therefore be eliminated since the purpose of this portion of basin is to complete the jump action to provide a smoother water surface. A basin length of twice the sequent depth will usually provide ample basin length. The end sill of the pool may be used as the crest of the chute as shown in Figures 1, 3, 5, and 7.

Again, it is very important that proper flow conditions be provided at the entrance to the baffled apron. In fact, satisfactory performance of the entire structure may hinge on whether entrance flow conditions are favorable. If unusual entrance problems are encountered or if any doubt exists, a hydraulic model study is recommended.

Design of Chute

The drop section, or chute, is constructed on a 2:1 slope. Minor variations from this slope would probably cause no difficulties but the effect of major variations in slope is not known. The upstream end of the chute floor should be joined to the horizontal floor by a curve to prevent excessive vertical contraction of the flow. However, the radius should be sufficiently small that the curved surface does not interfere with the placement of the first row of baffles. The upstream face of the first row should be no more than 1 foot (vertically) below the high point of the chute. It is important that the first row of baffles be placed as high on the chute as practical, since half of the water will not be intercepted until the flow strikes the second row of baffles. To prevent overtopping of the training walls at the second row of baffles, a partial baffle (one-third to two-thirds of the width of a full baffle) should be placed against the training walls in the top or first row. This will place a space of the same width adjacent to the walls in the second row. Alternate rows are then made identical. (Rows 1, 3, 5, 7, etc., are identical; Rows 2, 4, 6, 8, etc., are identical.) Four rows of baffles are necessary to establish the expected flow pattern at the base of the chute.

The height of the training walls on the chute should be three or more times the baffle height, measured normal to the chute floor. Walls of this height will contain the main flow and most of the splash. The greatest tendency to overtop the walls occurs in the vicinity of the second and third rows of baffles as indicated in the profiles and photographs in the report. If it is important to keep the adjacent area entirely dry, it may be desirable to increase the wall height near the top of the chute.

Several rows of baffle piers are usually constructed below the channel grade and backfill placed over the piers to provide original bottom topography. To determine the depth below channel grade to which the chute should be constructed, the following methods have been used. When the downstream channel has a control, the slope of a stable channel from the control upstream to the structure should be used to determine the elevation of the end of the chute. Usually, data are not available or sufficient to compute a stable channel grade; a slope of 0.0018 is then used. Experience has shown that a slope of 0.015 is much too steep. If a stable downstream control does not exist, the probable stable channel must be determined by estimating the amount of material which will be moved during the maximum design flood.

Baffle Pier Heights and Spacing

Curve A of Figure 23 shows the critical depth in a rectangular channel. The curve was plotted from the equation

$$D_c = \sqrt[3]{\frac{q^2}{g}}$$

Curve B gives values for 0.8 D_c; a curve for 0.9 D_c is also shown. Baffle pier heights for unit design discharges up to 60 cubic feet per second may be obtained from Curve B. As indicated by the tests, the baffle pier heights are not critical and the height may be varied by several inches to provide a convenient dimension.

The width of the baffle piers should equal the width of the spaces between baffles in the same horizontal row and may vary between one and one and one-half times the block height; preferred width one and one-half times the block height. Greater baffle widths may result in too few baffles to thoroughly break up the flow while narrower widths do not intercept enough of the flow at one place and also may result in slots too narrow for easy passage of trash.

As a general rule, the slope distance between rows of baffles (measured face to face on the 2:1 slope) should be twice the baffle height. When baffles less than 3 feet in height are used, the row spacing may be increased but should not exceed 6 feet. Greater spacing with small baffles allows the shallower flows to accelerate excessively before being intercepted by a baffle pier. Alternate rows should be staggered to provide a space below a block and vice versa.

The baffles may be constructed with their upstream faces normal to the chute or truly vertical; the difference in performance is hardly measurable in a model. There is a tendency, however, for the vertical faces to produce more splash and less scour than the normal faces, Figure 10. Other dimensions of the blocks are not important except from the structural viewpoint. The proportions shown in Figure 13 have been found acceptable for both structural and hydraulic requirements and are recommended for general use. The forces on a baffle pier may be estimated from the baffle pier pressure measurements shown in Figures 1 and 3.

PROTOTYPE PERFORMANCE

Field performance of baffled aprons, designed and constructed according to the suggestions given in this report, has been excellent in most installations. This has been verified by inspection teams working out of design offices and by field personnel responsible for operating the structures. Where deficiencies in performance have been noted, the cause was as obvious as the deficiency and simple remedial measures have resulted in satisfactory performance. The only difficulties reported, have been associated with unstable channel banks, lack of riprap, or both. Proper bank protection in all cases has resulted in a satisfactory structure.

Figures 24 through 36 show various installations in the field and indicate construction techniques, completed baffled aprons which have operated for several years, and structures performing for various fractions of the design flow. Each structure shown has been reported as satisfactory, either at the outset of operation or after bank stabilization had been accomplished. Each structure was built according to the general rules given in this report.

Baffle pier dimensions, spacing and arrangement, wall heights, and other rules for baffled aprons on a 2:1 sloping chute were followed precisely. Table 2 contains other structures which have been built following the general rules. Although no favorable reports on the performance of the tabulated aprons have been received, it is believed that they are operating as expected. No adverse comments on their performance have been forthcoming.

Figure 24 shows construction techniques used on two baffled aprons and operation of another at partial capacity. In the latter photograph, a small quantity of riprap on the earth bank would have prevented undermining and sloughing of the soft earth at the downstream end of the right training wall.

In Figure 25, the baffled apron on the Boulder Creek Supply Canal has operated many times over a range of discharges approaching the design discharge. As a result, a shallow pool has been scoured at the base of the structure. This is desirable since the pool tends to reduce surface waves and make bank protection downstream from the structure unnecessary. Note that a relatively small quantity of riprap has been placed to do the maximum good. Also note the wetted area (darker color) adjacent to the training walls, starting at about the second row of baffles. This is caused by a small amount of splash which rises above the walls and is carried by air currents. No reports have ever been received that this splash or water loss is of any consequence.

Figure 26 shows a low-drop baffled apron on the Bostwick Courtland Canal. It appears that grass has stabilized the banks sufficiently for the height of fall indicated. Little, if any, riprap is evident and the structure has performed satisfactorily for a number of years with little maintenance. Note the shallow scoured pool at the base of the apron.

Figure 27 shows another structure on the Bostwick Courtland Canal. Note the trash accumulated near the base of the structure. Field reports indicate that trash tends to collect during a falling stage and is removed by the water during the rising stage. Generally speaking, trash is not a problem on baffled aprons and does not contribute materially to maintenance costs. Note the well-placed riprap at the base of the structure.

Figure 28 shows two baffled aprons on the Bostwick Franklin Canal which have been in operation for over 4 years. In each case, grass has stabilized the downstream channel banks sufficiently to prevent bank erosion.

The series of photographs in Figure 29 show the progress of downstream scour from October 1956 to the spring of 1959. It may be noted that between October 1956, and September 1957, scouring occurred which exposed one row of the buried blocks. The bed material which was carried away consisted of fines; the coarse material which resembles riprap was left in place as shown in the photographs.

Figure 30 shows the Bostwick Superior Canal Drain after only a few months of operation. The soft earth banks were badly eroded, both upstream and downstream from the structure. The small amount of riprap placed downstream did much to protect the structure from complete failure. Stabilization of the banks with a grass cover eliminated sloughing of the banks. Figure 31 shows the same structure 6 years later, operating satisfactorily for a fraction of the design discharge. Now that the banks are stable, there is no maintenance problem.

Figure 32, upper photograph, shows Frenchman-Cambridge Drain 8C after 4-1/2 years of operation. Performance has been excellent. Riprap originally placed at the base of the walls is covered by weed and grass cover. The shallow energy dissipating pool has helped to reduce bank maintenance downstream. In the lower photograph, the Culbertson Canal Wasteway 3.3 is shown in operation shortly after construction was completed. The need for riprap at the waterline near the base of the baffled apron is beginning to become apparent. Figure 33 shows closer views of this same structure and indicates how energy dissipation is accomplished on the chute. Action in hydraulic models of baffled aprons is identical to that shown here. Figure 34, top photograph, shows the wasteway after the discharge was stopped. It appears that additional riprap protection would be desirable, particularly if the discharge is greater than 75 cubic feet per second.

Figure 34, lower photograph, shows the Robles-Casitas Canal discharging 500 cubic feet per second into a baffled apron. Note that the riprap affords adequate protection to the structure. Operation is excellent.

Figure 35 shows a drop on the Frenchman-Cambridge Wasteway. The upper photograph shows how wingwalls can be used to protect the structure and how a small amount of riprap can be used to protect the wingwalls from undercutting. The lower photograph shows the action of the water on the baffled apron for a very small discharge. There is practically no turbulence at the base of the apron (see upper photograph also).

Figure 36, upper photograph, shows the North Branch Wasteway--Picacho Arroyo System discharging at about half capacity after a violent rainstorm. The water is carrying a high concentration of sediment. After the flood, lower photograph, it was found that the downstream channel had aggraded rather than scoured, partially covering one row of blocks which had been more exposed before the runoff. In this case, the reduction in velocity at the base of the apron caused sediment to settle out of the wasteway water.

Table 1

SCOUR TEST RESULTS

Baffle pier height ft	Discharge per foot of width q in cfs	(1) At wingwall	(2) End of chute	Average 1 and 2
		Elevation of deepest erosion		
2	20	910-	908-	909-
	35	907-	906	906.5
3	20	910	908	909
	35	908	907	907.5
	50	906	906+	906.1
	60	905	906-	905.5
4	20	909	907	908
	35	908	906	907
	50	907	906-	906.5
	60	906	905	905.5
5	20	908	905	906.5
	35	907	905	906
	50	907	904	905.5
	60	906	904	905
6	20	906	906	906
	35	903	904	903.5
	50	902	904	903
	60	900	904	902

Table 2

BAFFLED APRON STRUCTURES IN USE

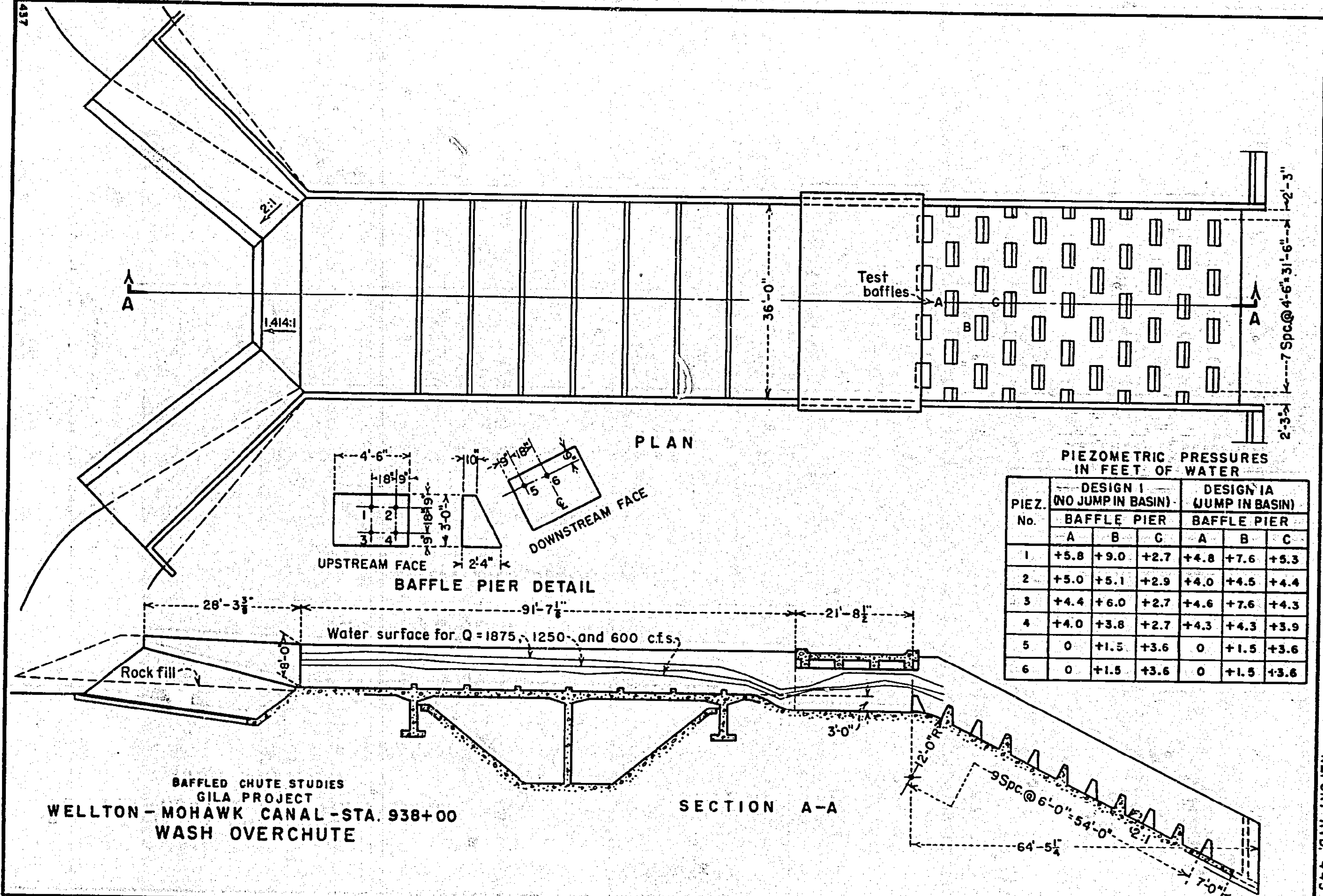
Spec. No.	Drawing No.	Location	Station	Chute width, feet	Design discharge, cfs
		FRANKLIN CANAL			
DC-3720	271-D-549	Drain F-1.5-D	0+50	8 Trap.	85
DC-3720	271-D-549	Drain F-10.1-U	1+10	8 Trap.	80
DC-3720	271-D-550	Drain F-1.9-D	1+25	6 Trap.	64
DC-3720	271-D-550	Drain F-10.1-D	2+00	6 Trap.	51
DC-3720	271-D-551	Drain F-10.1	84+68+	18 Rect	625
DC-3891	271-D-648	Drain F-14.1-D	1+44	10 Trap.	100
DC-3891	271-D-649	Drain F-14.9	5+20	32 Rect	1,100
DC-3891	271-D-650	Drain F-14.9-D	23+20	14 Rect	280
DC-3891	271-D-651	Drain F-15.8	5+00	23 Rect	800
DC-3891	271-D-653	Drain F-23.5-U	2+80	10 Trap.	100
		COURTLAND CANAL			
DC-4501	271-D-1031	Drain C-42.3-U	2+80	10 Trap.	120
		COURTLAND WEST CANAL			
DC-4874	271-D-1344	Drain CW-0.7-D	3+100	10 Rect	123
DC-4874	271-D-1344	Drain CW-1.4-U	2+00	6 Rect	123
DC-4874	271-D-1344	Drain CW-10.5	8+00	6 Rect	46
		SARGENT CANAL			
DC-4681	499-D-263	Airport Wasteway	16+00	11.5 Rect	130
DC-4681	499-D-263	Airport Wasteway	36+50	11.5 Rect	130
DC-4681	499-D-264	Airport Wasteway	51+20	17 Rect	300
DC-4681	499-D-264	Airport Wasteway	98+00	17 Rect	300
DC-4681	499-D-229	Big Oak Drain	11+25	11 Rect	220
DC-4681	499-D-230	Big Oak Drain	13+00	12.5 Rect	150
DC-4681	499-D-248	Drain S-21.9	4+60	25.5 Rect	800
DC-4681	499-D-249	Drain S-22.6	4+00	19.5 Rect	650
DC-4681	499-D-250	Drain S-22.6-U	0+60	14.5 Rect	165
DC-4681	499-D-260	Drain S-36.0	29+35	15 Rect	180
		GILA PROJECT			
DC-2688	50-D-2417	Wellton-Mohawk Canal	7+14.48	84 Rect	35 cfs
DC-2688	50-D-2432	Wellton-Mohawk Canal	151+39.25	52 Rect	per
DC-2688	50-D-2438	Wellton-Mohawk Canal	234+60	36 Rect	foot
DC-2972	50-D-2668	Mohawk Dike No. 1	0+00	140 Rect	of
DC-2972	50-D-2679	Mohawk Dike No. 1	12+30	25 Rect	width
DC-2972	50-D-2646	Mohawk Canal	1125+95.74	180 Rect	
DC-2972	50-D-2654	Mohawk Canal	1406+22.25	124 Rect	
DC-2972	50-D-2659	Mohawk Canal	1479+78.47	46 Rect	

Table 2--Continued

Spec. No.	Drawing No.	Location	Station	Chute width, feet	Design discharge, cfs
		GILA PROJECT--Continued			
DC-2972	50-D-2661	Mohawk Canal	1546+90	8 Rect	35 cfs
DC-3683	50-D-2982	Radium Hot Springs	179+84.91	18 Rect	per foot
DC-4983	50-D-3359	Wellton-Mohawk Canal	661+16	90 Rect	of width
DC-2822	50-D-2446	Wellton-Mohawk Canal	489+21.71	65 Rect	
DC-2822	50-D-2453	Wellton-Mohawk Canal	563+50	39 Rect	
DC-2822	50-D-2456	Wellton-Mohawk Canal	614+21.71	65 Rect	
DC-2822	50-D-2459	Wellton-Mohawk Canal	660+00	62 Rect	
DC-2822	50-D-2470	Wellton-Mohawk Canal	822+17.17	200 Rect	
DC-2822	50-D-2473	Wellton-Mohawk Canal	938+00	36 Rect	
DC-5019	50-D-3366	Texas Hill Floodway	113+00	11 Rect	200
DC-5019	50-D-3368	Texas Hill Floodway	133+00	28.5 Rect	1,000
		EDEN PROJECT			
DC-3558	153-D-152	Means Canal	7+30.77	18 Rect	630
		COLUMBIA BASIN PROJECT			
DC-4888	222-D-19589	WB5WW1	36+90	18 Rect	226
DC-4888	222-D-19596	WB5WW1	564+95	7 Rect	85
DC-4888	222-D-19596	WB5WW1	280+10	7 Rect	85
DC-4888	222-D-19596	WB5WW1	286+60	11 Rect	127
DC-4888	222-D-19596	WB5WW1	303+10	11 Rect	127
DC-4888	222-D-19596	WB5WW1	329+10	11 Rect	127
DC-4888	222-D-19596	WB5WW1	346+25	11 Rect	127
DC-4888	222-D-19596	WB5WW1	363+10	11 Rect	127
DC-4888	222-D-19596	WB5WW1	396+60	11 Rect	127
DC-4888	222-D-19597	WB5WW1	410+10	14 Rect	172
DC-4888	222-D-19597	WB5WW1	420+60	14 Rect	172
DC-4888	222-D-19597	WB5WW1	432+10	14 Rect	172
DC-4888	222-D-19597	WB5WW1	441+45	14 Rect	172
DC-4888	222-D-19597	WB5WW1	456+75	14 Rect	172
DC-4888	222-D-19597	WB5WW1	465+70	14 Rect	172
DC-4888	222-D-19597	WB5WW1	472+90	14 Rect	172
DC-4888	222-D-19598	WB5WW1	481+85	14 Rect	172
DC-4888	222-D-19598	WB5WW1	489+60	14 Rect	172
DC-4888	222-D-19598	WB5WW1	497+10	14 Rect	172
DC-4888	222-D-19598	WB5WW1	505+10	14 Rect	172
DC-4888	222-D-19598	WB5WW1	513+40	14 Rect	172
DC-4888	222-D-19598	WB5WW1	520+40	14 Rect	172
DC-4888	222-D-19598	WB5WW1	527+60	14 Rect	172
DC-4696	222-D-18763	EL68DWW	321+55.70	14 Rect	146
DC-4696	222-D-18817	EL68DWW	551+07AH	22 Rect	450
DC-4696	222-D-18775	EL81WW	202+02	18 Rect	365

Table 2--Continued

Spec. No.	Drawing No.	Location	Station	Chute width, feet	Design discharge, cfs
		COLUMBIA BASIN PROJECT--Continued			
DC-4696	222-D-18776	EL68DWW	Dike No. 1	9 Rect	96
DC-4696	222-D-18776	EL68DWW	Dike No. 4	14 Rect	198
DC-4696	222-D-18776	EL68DWW	Dike No. 5	14 Rect	198
DC-4696	222-D-18776	EL68DWW	Dike No. 6	14 Rect	198
DC-4696	222-D-18776	EL68DWW	Dike No. 7	14 Rect	198
DC-4696	222-D-18776	EL68DWW	Dike No. 9	18 Rect	313
DC-4696	222-D-18776	EL68DWW	Dike No. 10	20 Rect	363
DC-4696	222-D-18776	EL68DWW	Dike No. 11	22 Rect	414
DC-4696	222-D-18776	EL83WW	Dike No. 12	11 Rect	220
DC-4696	222-D-18776	EL83WW	Dike No. 13	11 Rect	220
DC-4696	222-D-18776	EL83WW	Dike No. 14	11 Rect	220
DC-4696	222-D-19601	WB5WW1	531+17.53	14 Rect	172
DC-4696	222-D-19601	WB5WW1	535+80	14 Rect	172
DC-4571	222-D-18422	PE16.4WW	1594+63	22 Rect	770
DC-4749	222-D-19090	Potholes East Canal	1369+11	46.5 Rect	3,900
		COLORADO BIG THOMPSON PROJECT			
DC-3657	245-D-6645	St. Vrain Supply	513+86	18 Rect	575
DC-4150	245-D-7137	Boulder Creek Supply	667+78	10 Rect	200
		SOLANO PROJECT			
DC-4881	413-D-513	Putah South Canal	1099+79	13 Rect	156
DC-4555	413-D-317	Putah South Canal	263+50	6 Rect	48

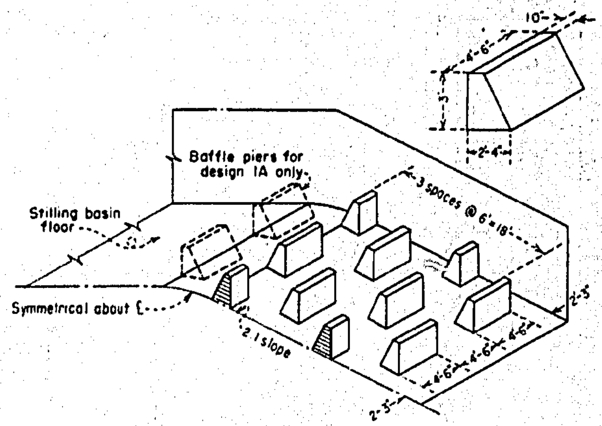


PIEZOMETRIC PRESSURES IN FEET OF WATER

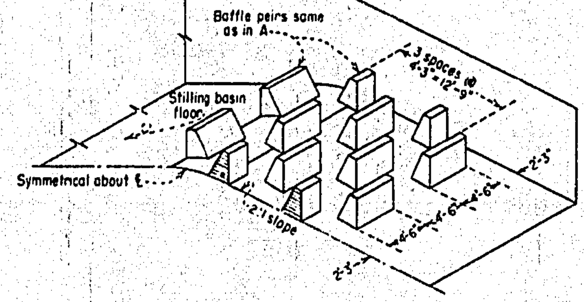
PIEZ. No.	DESIGN I (NO JUMP IN BASIN)			DESIGN IA (JUMP IN BASIN)		
	BAFFLE PIER			BAFFLE PIER		
	A	B	C	A	B	C
1	+5.8	+9.0	+2.7	+4.8	+7.6	+5.3
2	+5.0	+5.1	+2.9	+4.0	+4.5	+4.4
3	+4.4	+6.0	+2.7	+4.6	+7.6	+4.3
4	+4.0	+3.8	+2.7	+4.3	+4.3	+3.9
5	0	+1.5	+3.6	0	+1.5	+3.6
6	0	+1.5	+3.6	0	+1.5	+3.6

BAFFLED CHUTE STUDIES
GILA PROJECT
WELLTON-MOHAWK CANAL-STA. 938+00
WASH OVERCHUTE

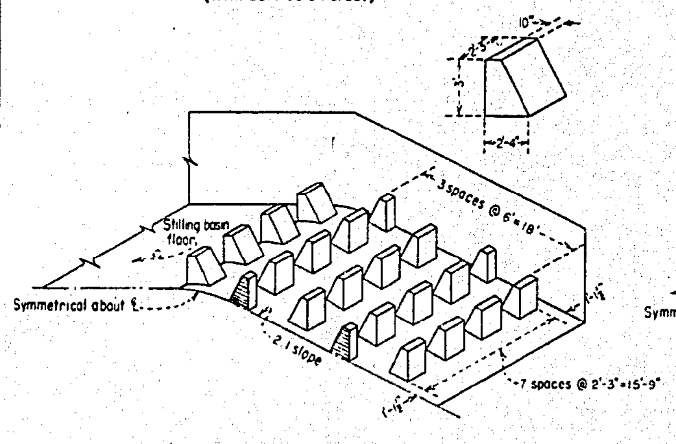
FIGURE 1
REPORT HYD. 445



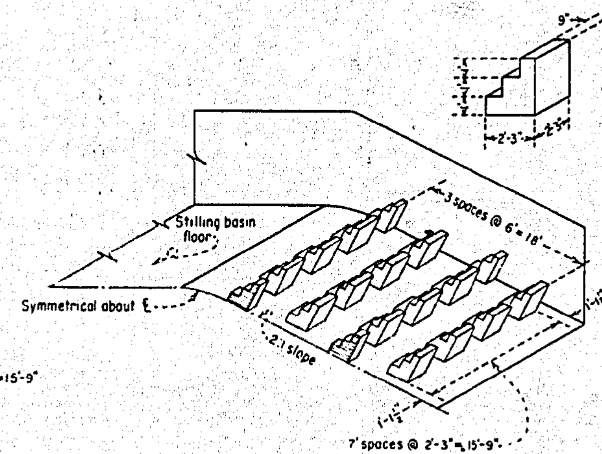
A. DESIGN 1
(Without baffles on crest)
DESIGN 1A
(With baffles on crest)



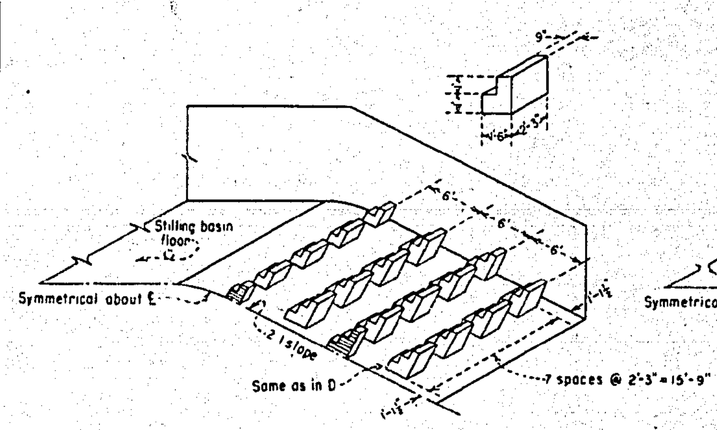
B. DESIGN 2A



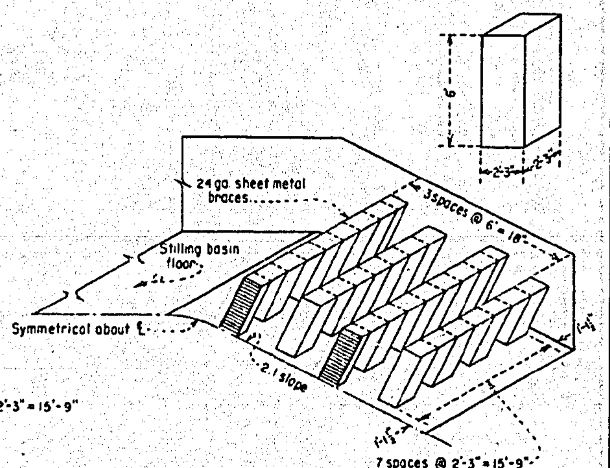
C. DESIGN 3



D. DESIGN 4



E. DESIGN 5



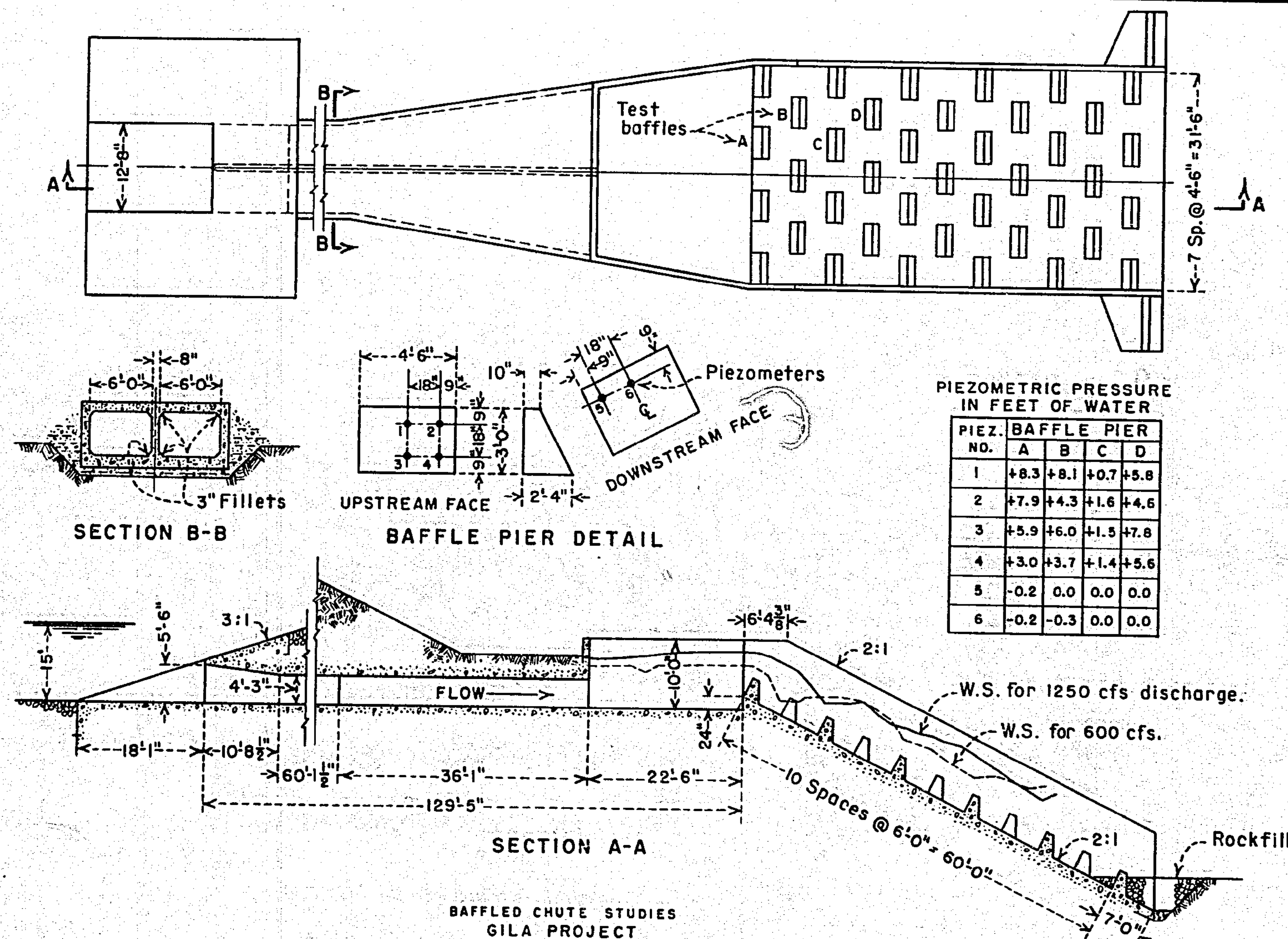
F. DESIGN 6

BAFFLED CHUTE STUDIES
GILA PROJECT
WELLTON-MOHAWK DIVISION

WASH OVERCHUTE AT STA. 938+00

DIFFERENT BAFFLE PIER ARRANGEMENTS ON 2:1 SLOPING APRON

1:12 SCALE MODEL

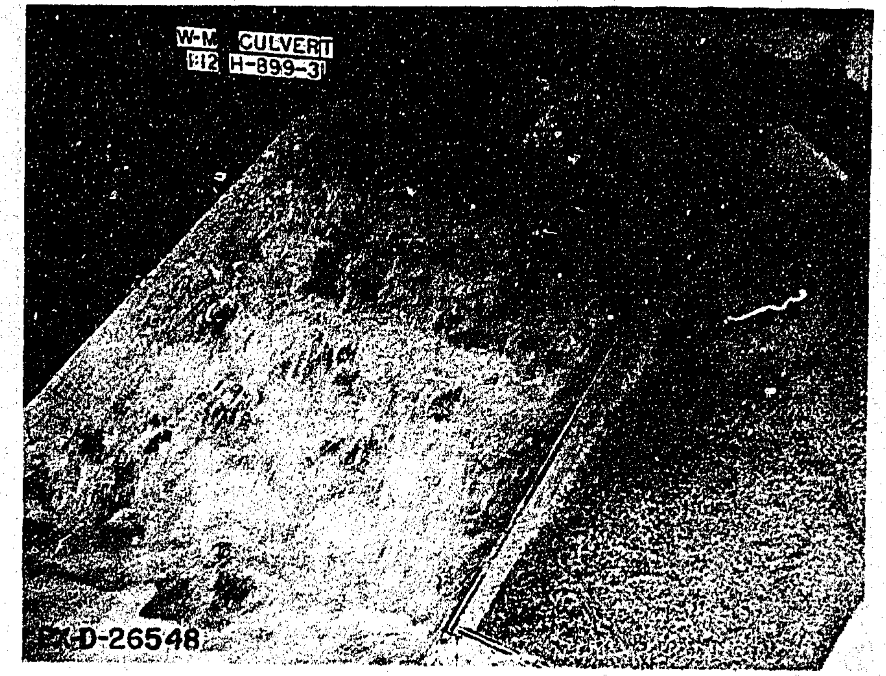


PIEZOMETRIC PRESSURE IN FEET OF WATER

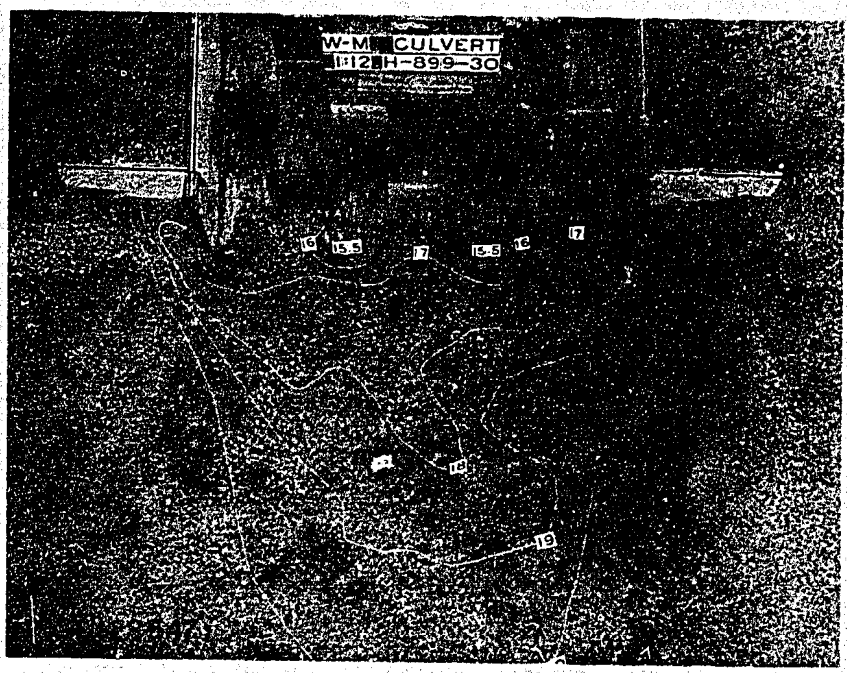
PIEZ. NO.	BAFFLE PIER A	B	C	D
1	+8.3	+8.1	+0.7	+5.8
2	+7.9	+4.3	+1.6	+4.6
3	+5.9	+6.0	+1.5	+7.8
4	+3.0	+3.7	+1.4	+5.6
5	-0.2	0.0	0.0	0.0
6	-0.2	-0.3	0.0	0.0

BAFFLED CHUTE STUDIES
GILA PROJECT
MOHAWK PROTECTIVE DIKE No. 1, STA. 12+30
CULVERT UNDER DIKE

Figure 4

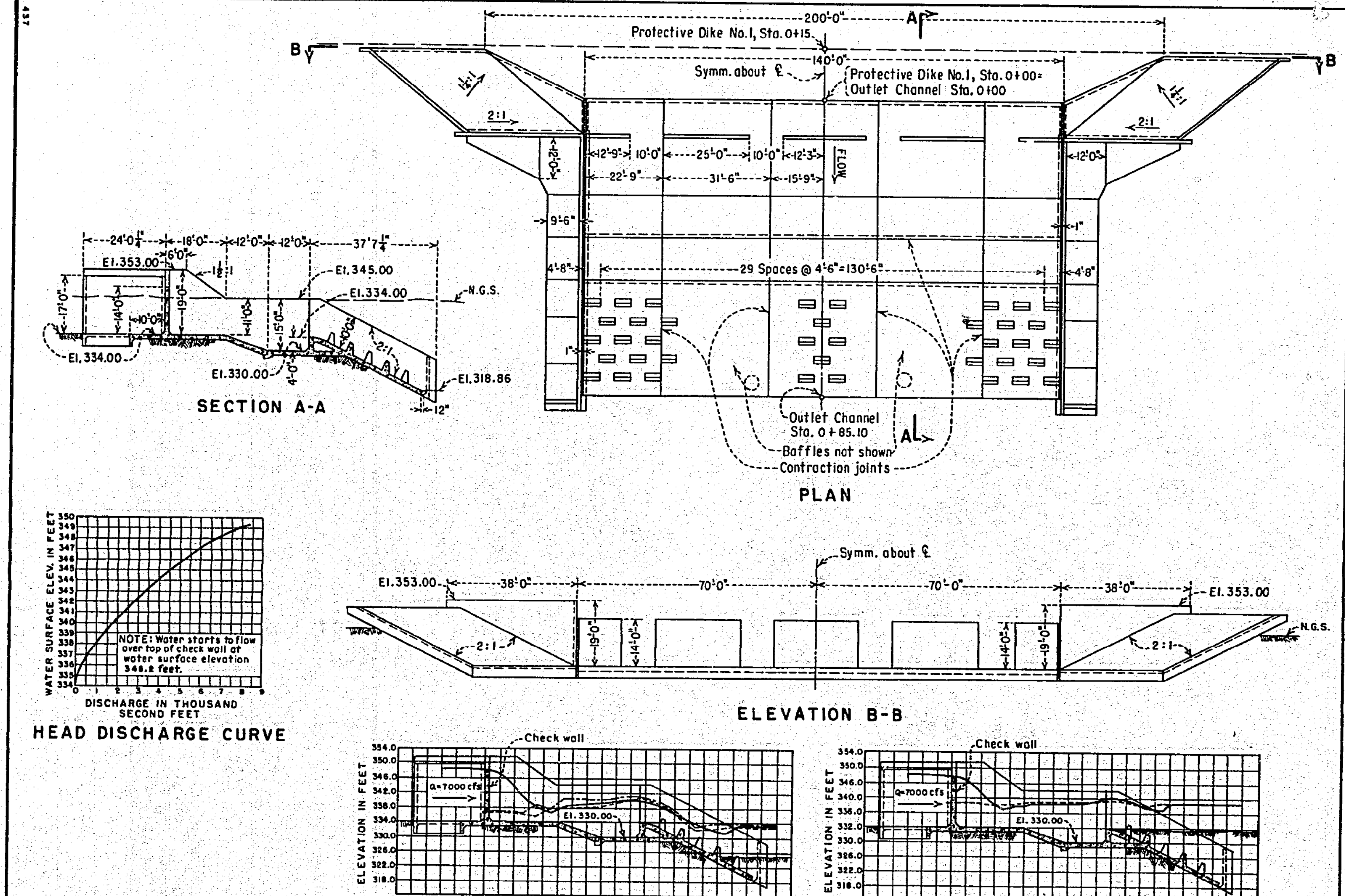


Discharge 1, 250 cfs
Unit discharge is 40 cfs per foot of width



Scour Pattern
(for above flow)

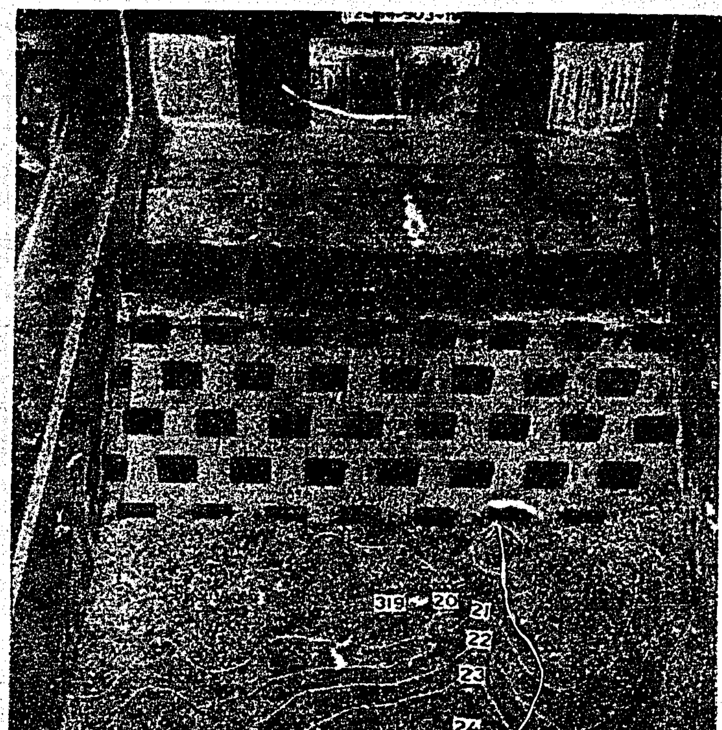
Culvert Under Dike Model
(See Figure 3 for details)



BAFFLED CHUTE STUDIES
GILA PROJECT
MOHAWK PROTECTIVE DIKE No. 1-STA. 0+00
OUTLET CONTROL STRUCTURE

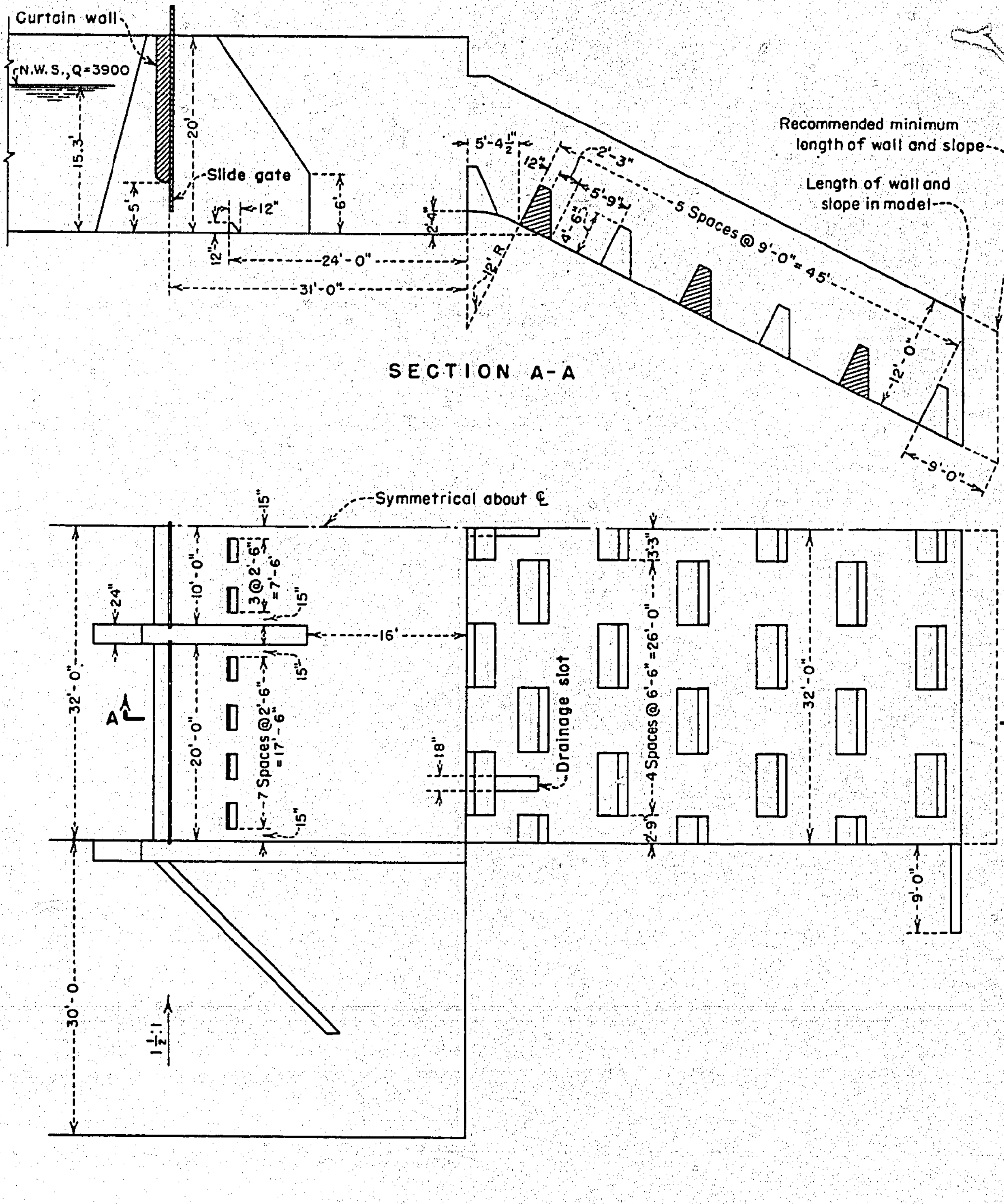
Figure 6

A. Discharge 7,000 cfs
Unit discharge
50 cfs per foot width



B. Scour Pattern

Outlet Control Structure Model
(See Figure 5 for details)



BAFFLED CHUTE STUDIES

POTHOLES EAST CANAL

CHECK INTAKE STRUCTURE AT STATION 1369+40±

COLUMBIA BASIN PROJECT

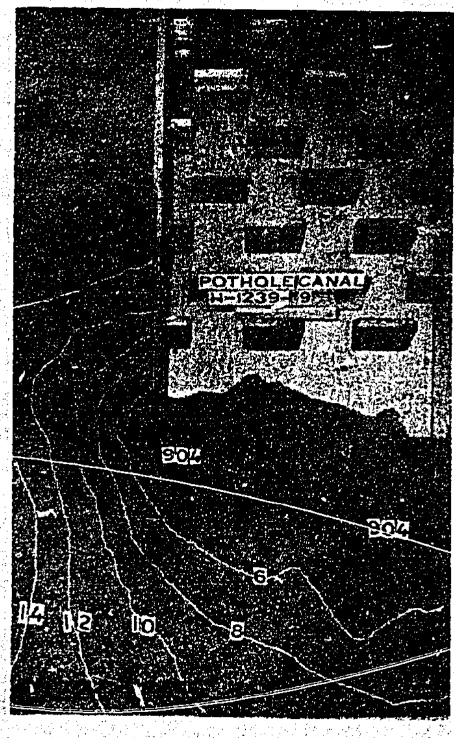
1:16 SCALE MODEL

437

Figure 8

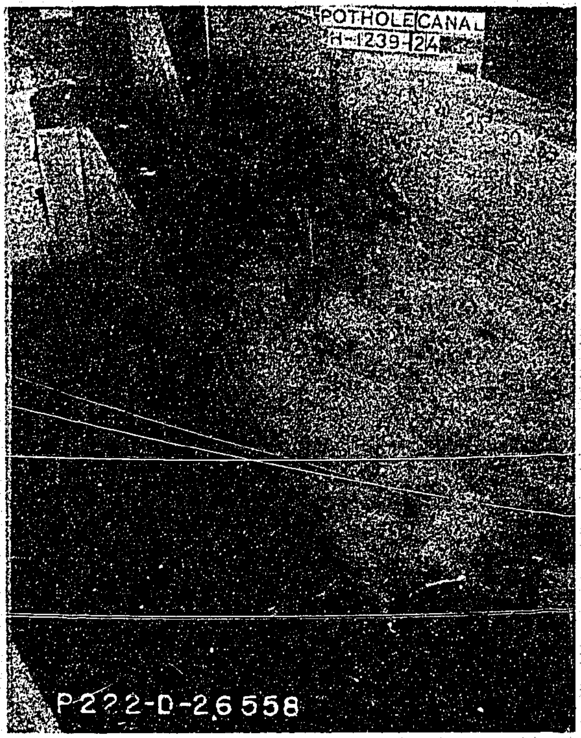
Baffle piers 4'-6" high, row spacing 9'-0"

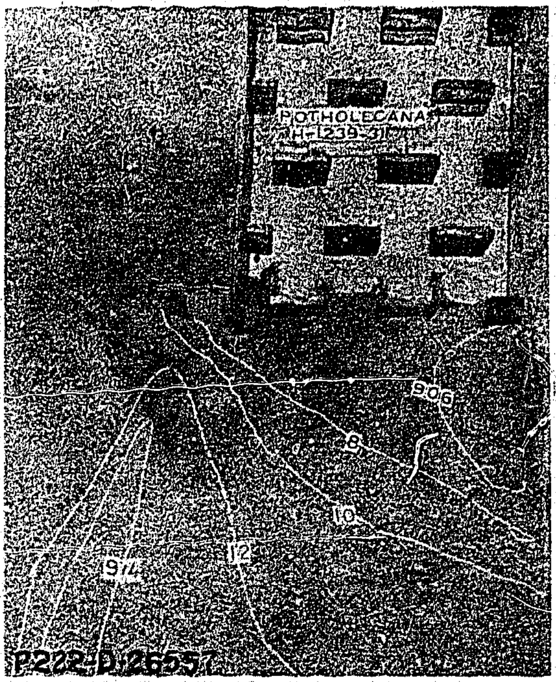
Note: Bed was at Elev. 914
at start of 30-minute test



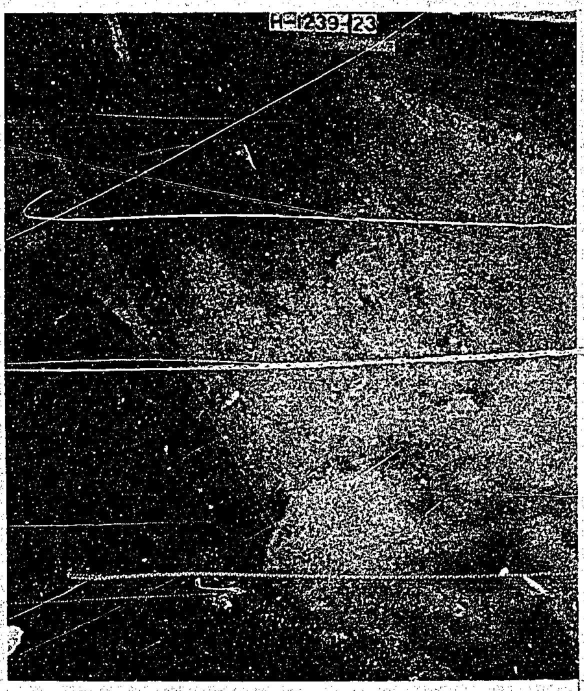
Baffle piers 4'-6" high, row spacing 6'-0"

Potholes East Canal Model
Discharge 3,900 cfs
Unit discharge 61 cfs per foot width





Flow Scour
Discharge 3,200 cfs, unit discharge 50 cfs per foot width

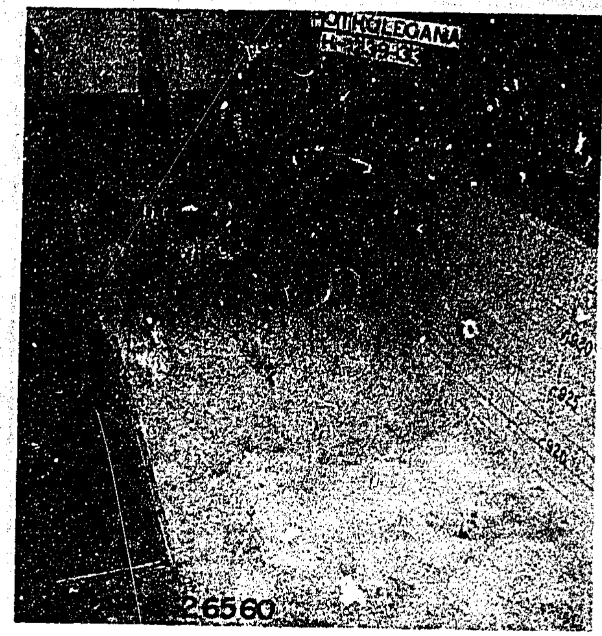


31 cfs per foot width

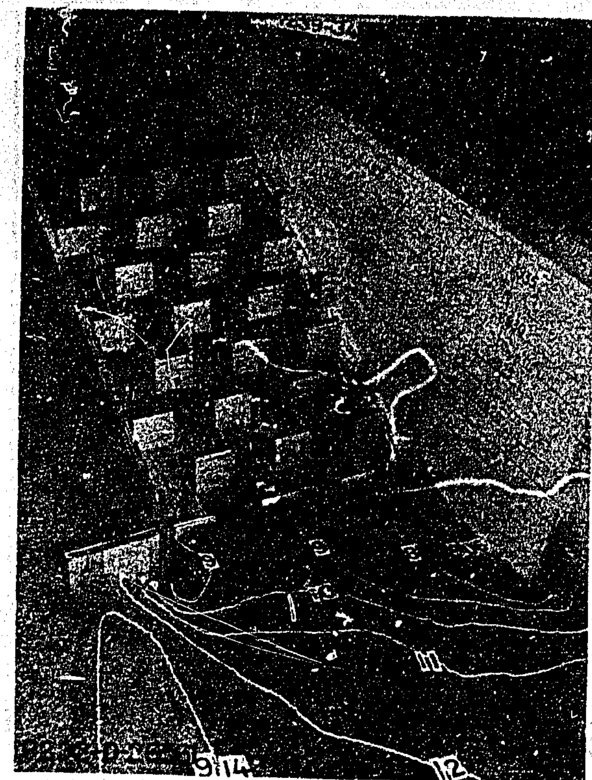
16 cfs per foot width

Potholes East Canal Model
Baffle piers 4'-6" high, row spacing 9'-0"

Figure 10

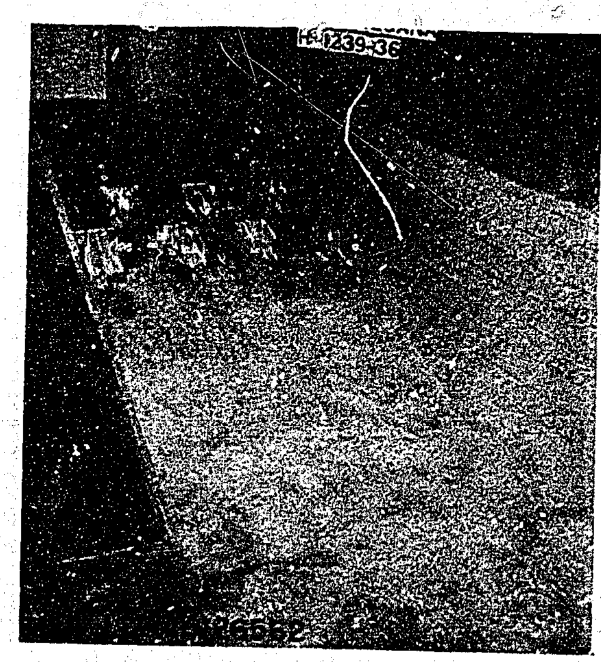


Flow
Note splash area on wall on right

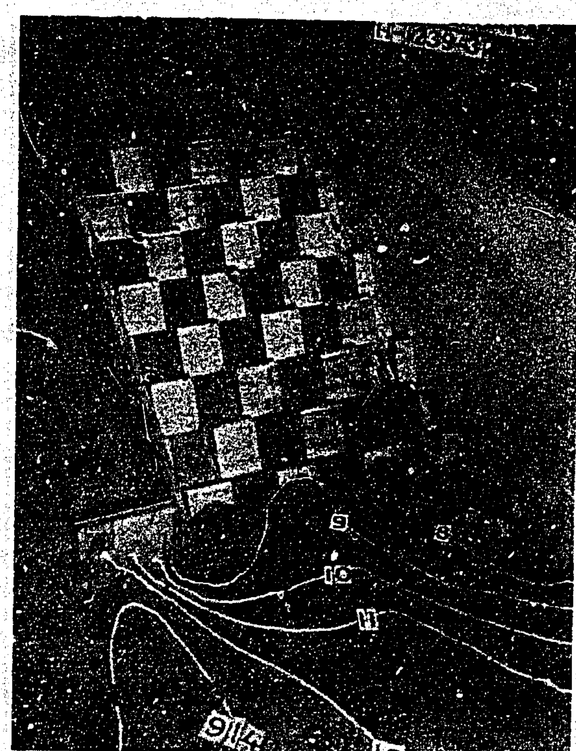


Scour

Normal-face Piers



Flow

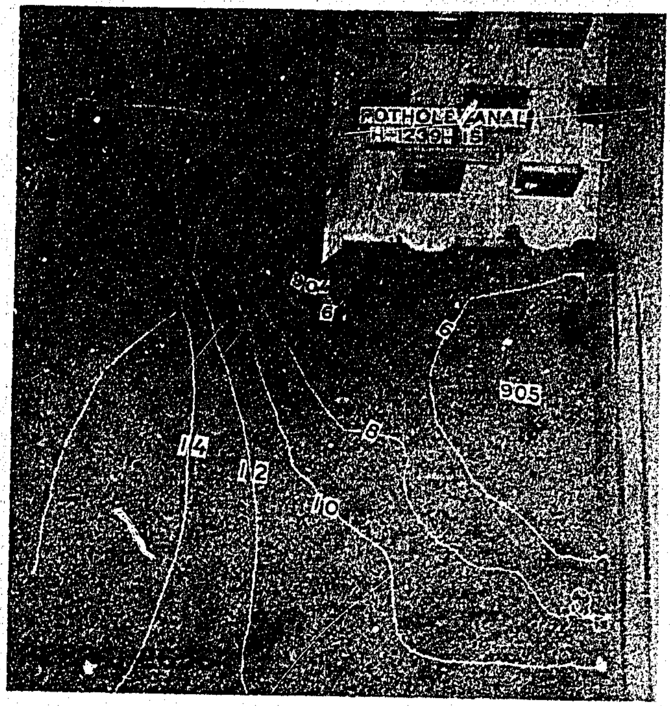


Scour

Vertical-face Piers

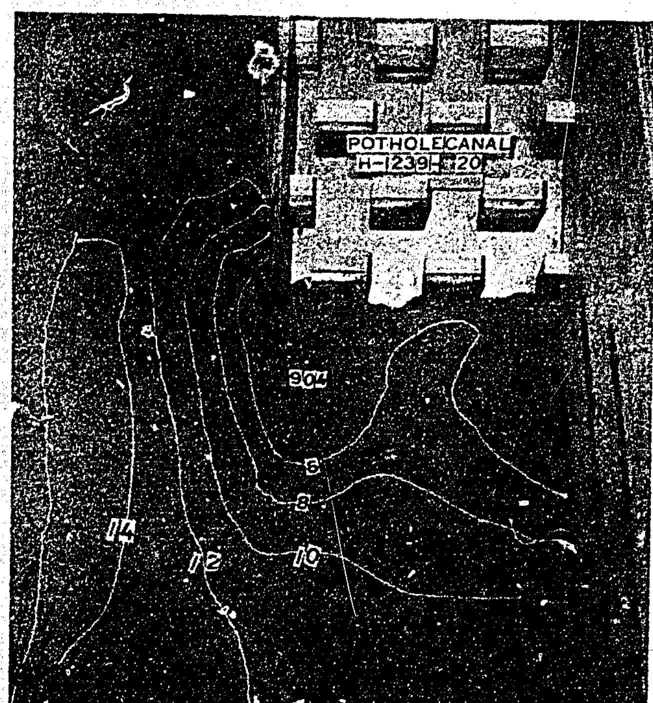
Potholes East Canal Model
Discharge 35 cfs per foot width

Figure 11



Normal-face Piers

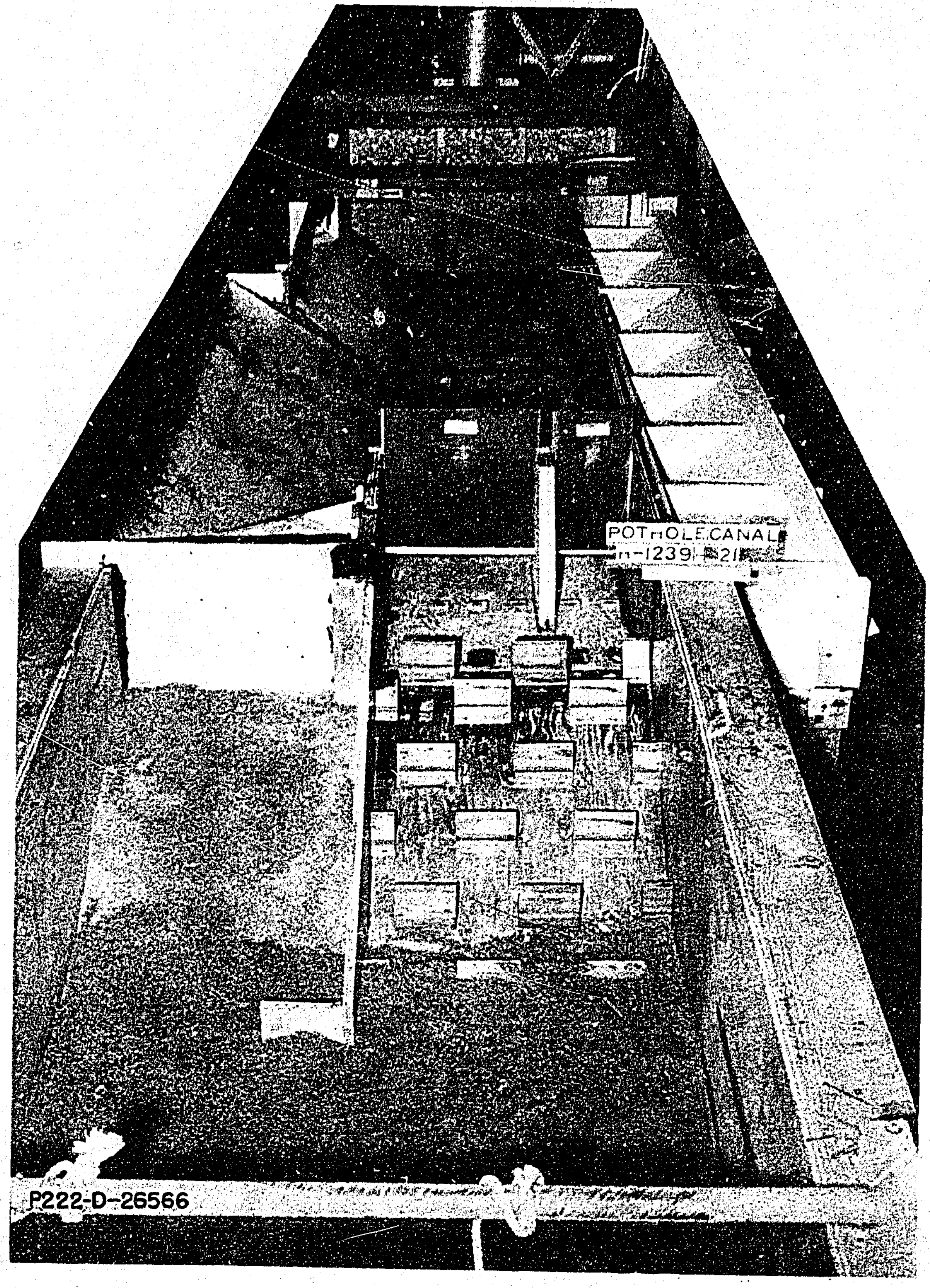
Note: Bed was at Elev. 914
at start of 30-minute test



Vertical-face Piers

Potholes East Canal Model
Discharge 61 cfs per foot width

Figure 12



Generalization Tests
Potholes East Canal Model used for general tests

FIGURE 13
REPORT HYD. 445

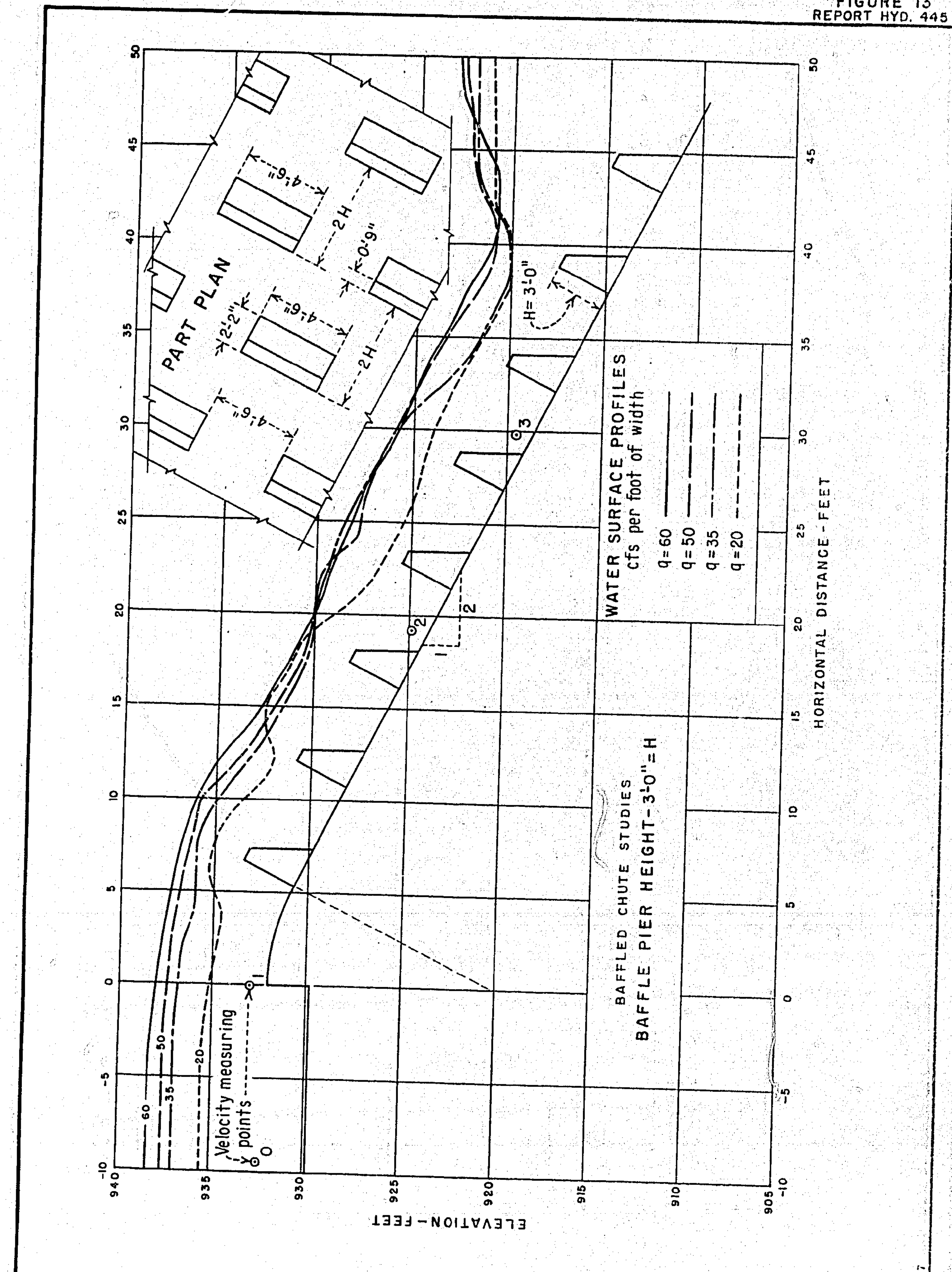
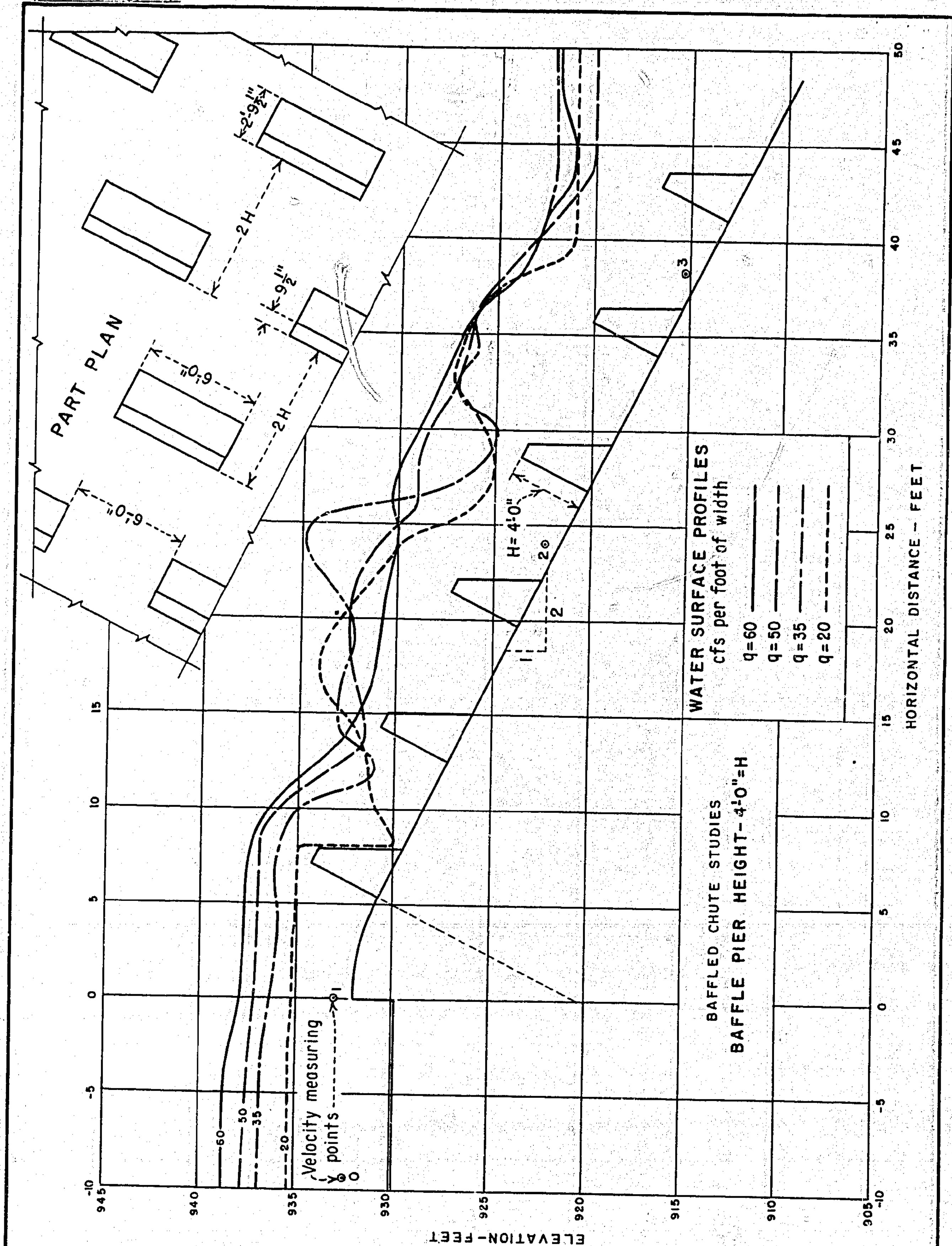


FIGURE 14
REPORT HYD. 445



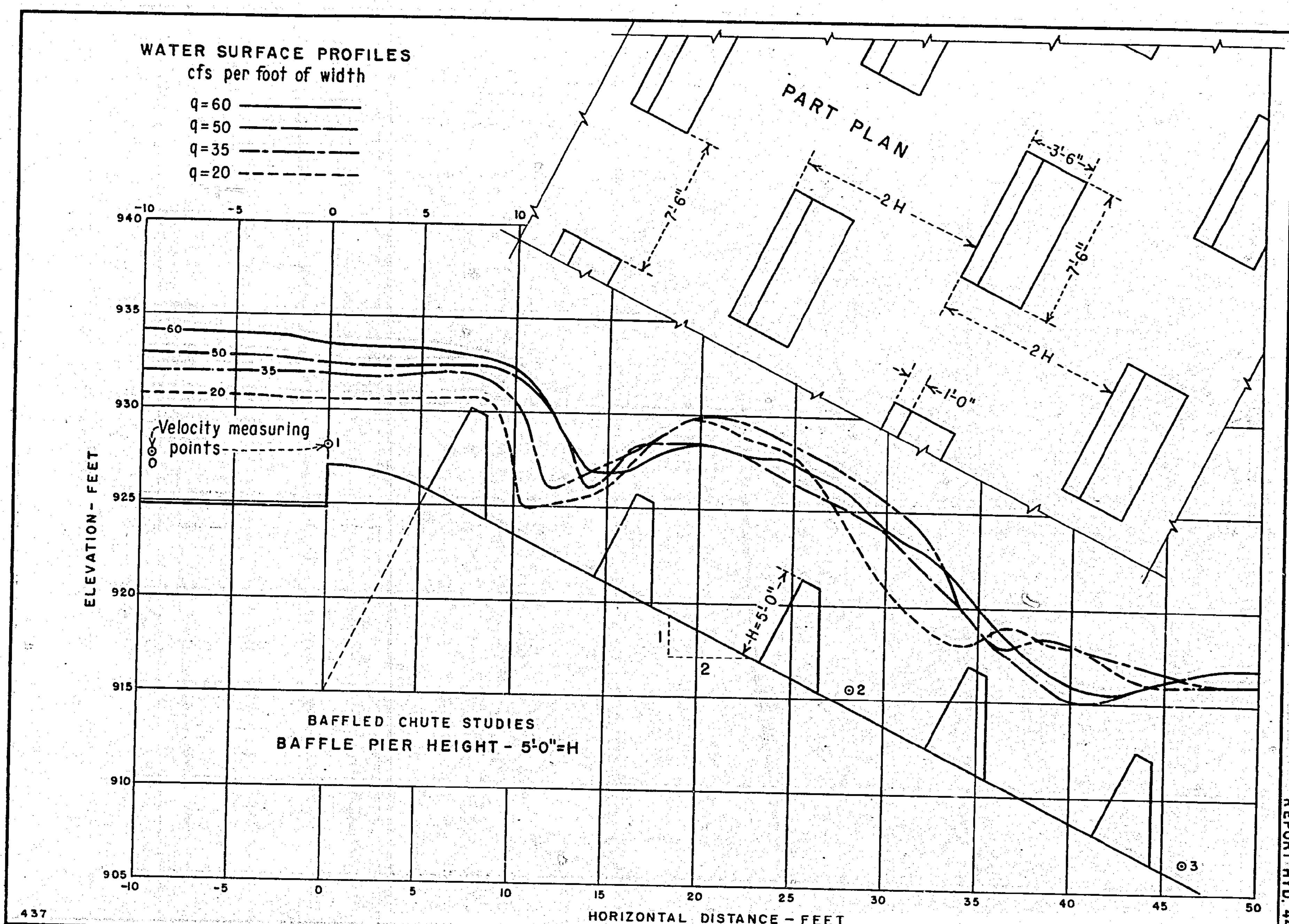
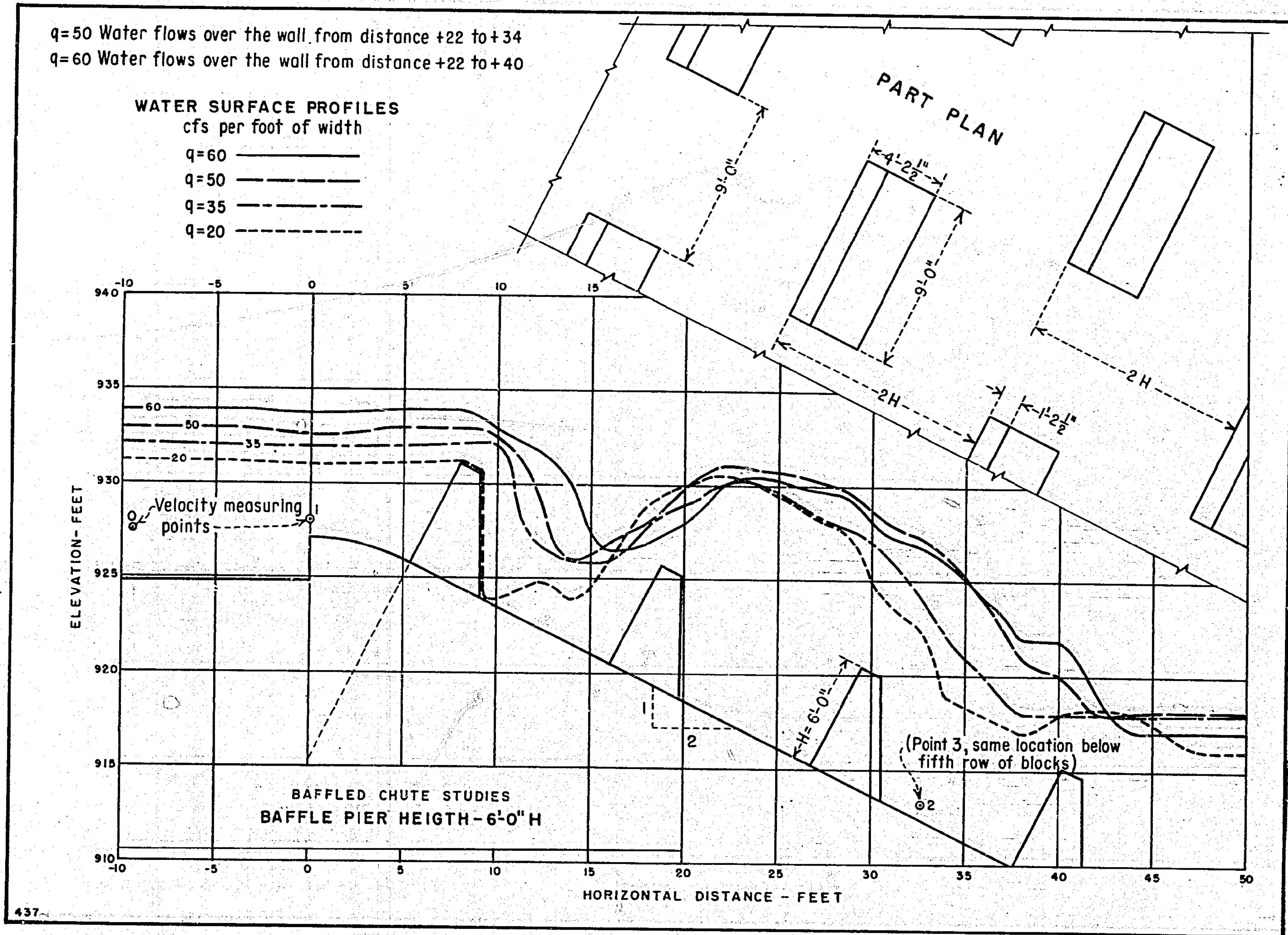
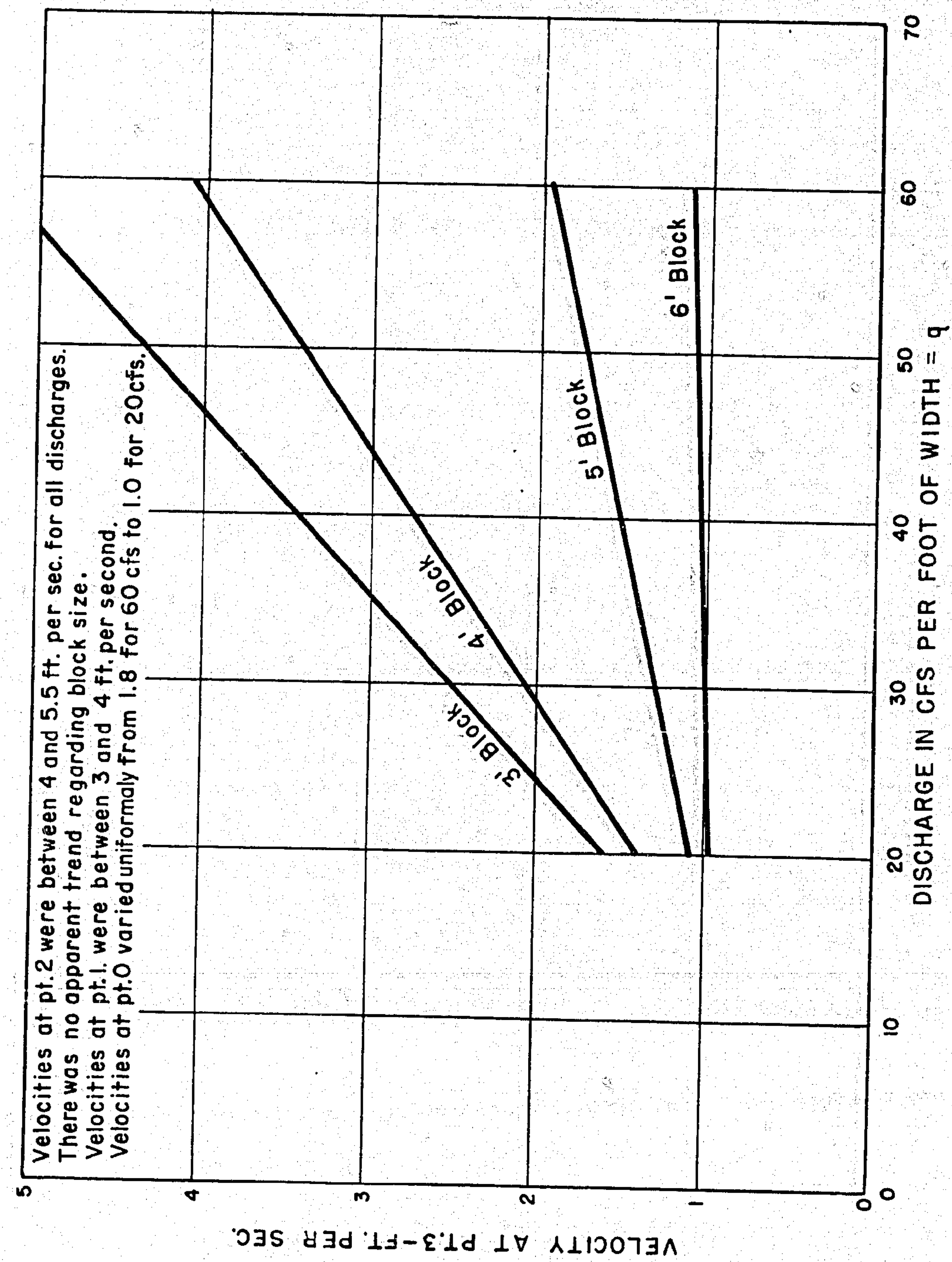


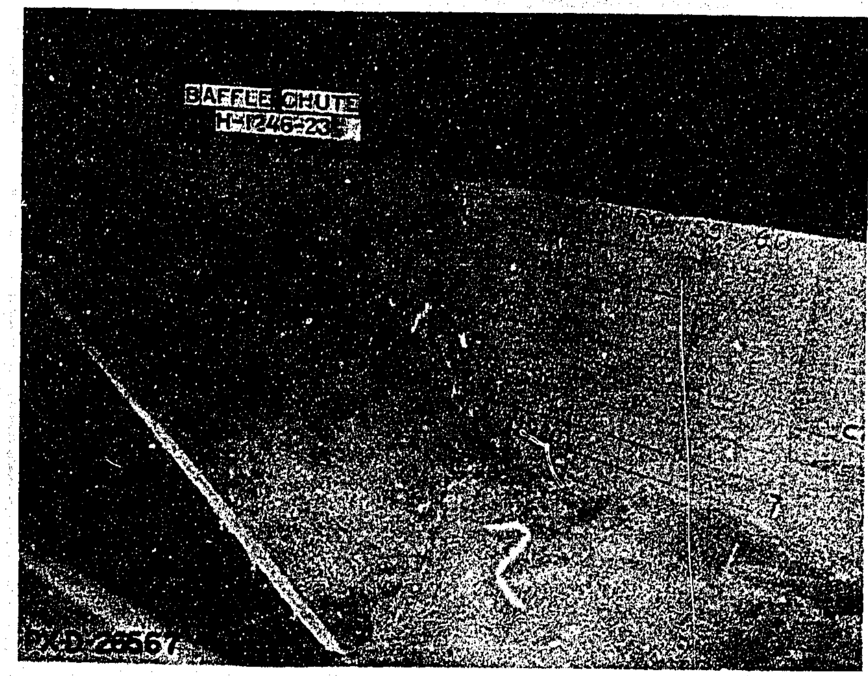
FIGURE 15
REPORT HYD. 445


q=50 Water flows over the wall from distance +22 to +34
q=60 Water flows over the wall from distance +22 to +40
WATER SURFACE PROFILES
cfs per foot of width
q=60
q=50
q=35
q=20
PART PLAN
9'-0"
4'-2 1/2"
9'-0"
2H
2H
1'-2 1/2"
Velocity measuring points
H=6'-0"
(Point 3, same location below fifth row of blocks)
ELEVATION - FEET
HORIZONTAL DISTANCE - FEET
BAFFLED CHUTE STUDIES
BAFFLE PIER HEIGTH-6'-0" H
437



BAFFLED CHUTE STUDIES
VELOCITIES AT POINT 3
(SEE FIGURES 13, 14, 15 AND 16)

Figure 18



Baffle piers 6'-0" high

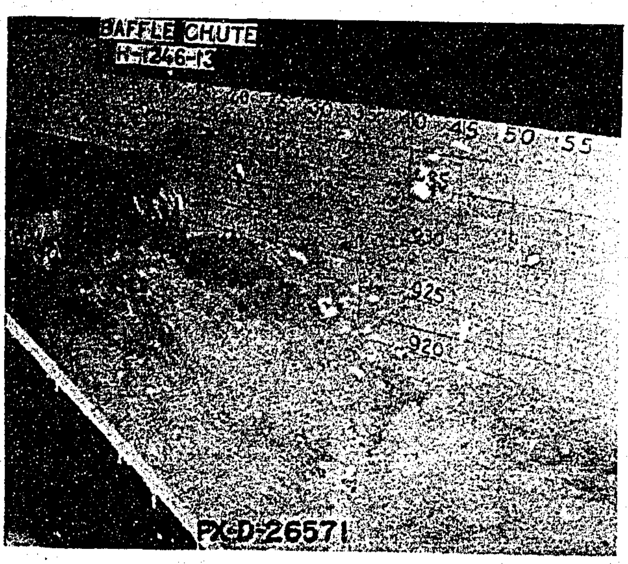
Baffle piers 3'-0" high

Baffled Chute Studies
Discharge 60 cfs per foot of width

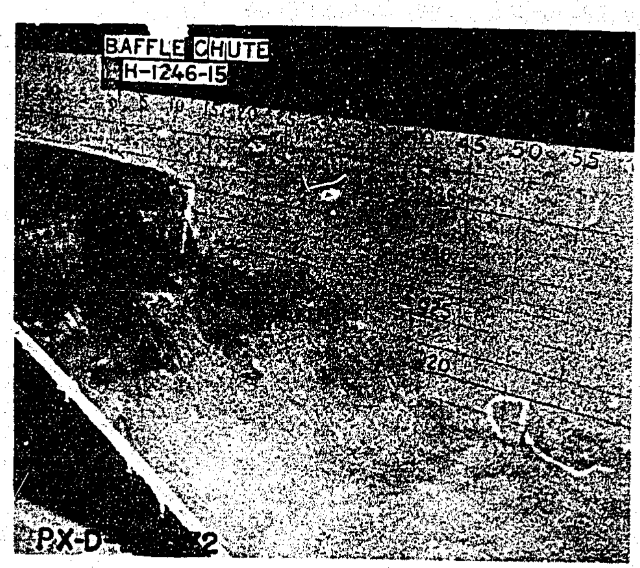
50 cfs per foot of width

60 cfs per foot of width

A. Baffle piers 4'-0" high



50 cfs per foot of width

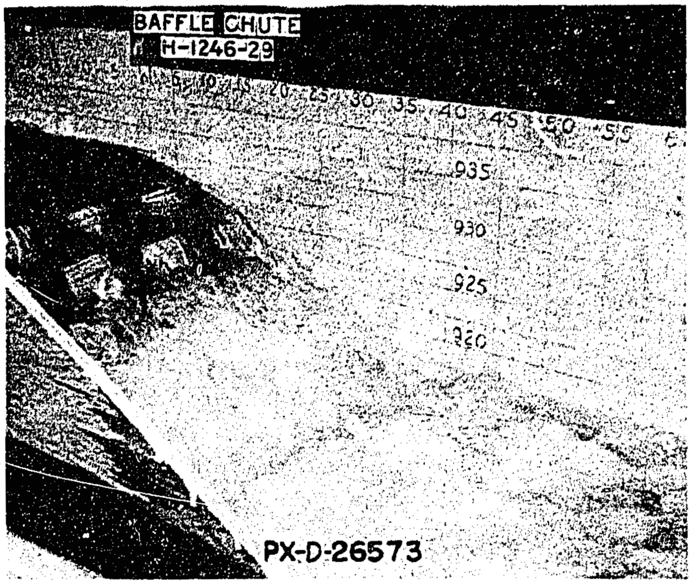


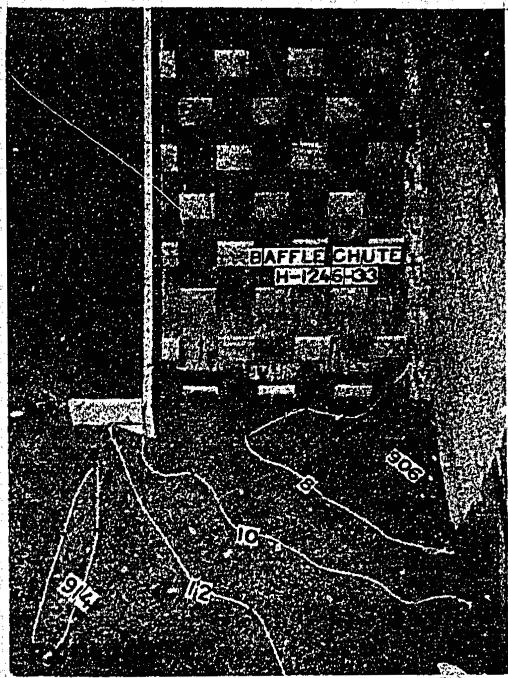
60 cfs per foot of width

B. Baffle piers 5'-0" high

Baffled Chute Studies

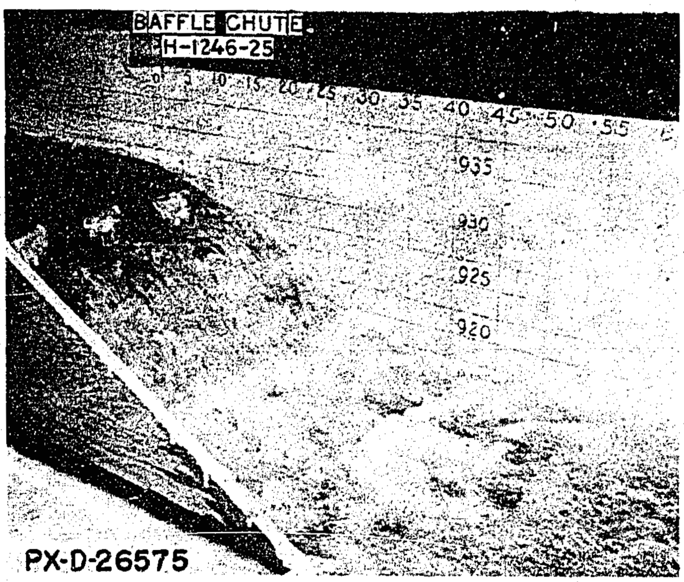
Figure 20

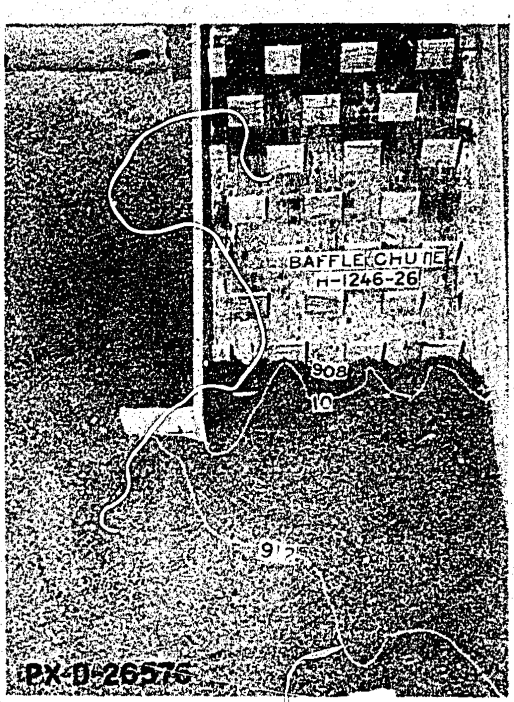




A. Discharge 35 cfs per foot of width

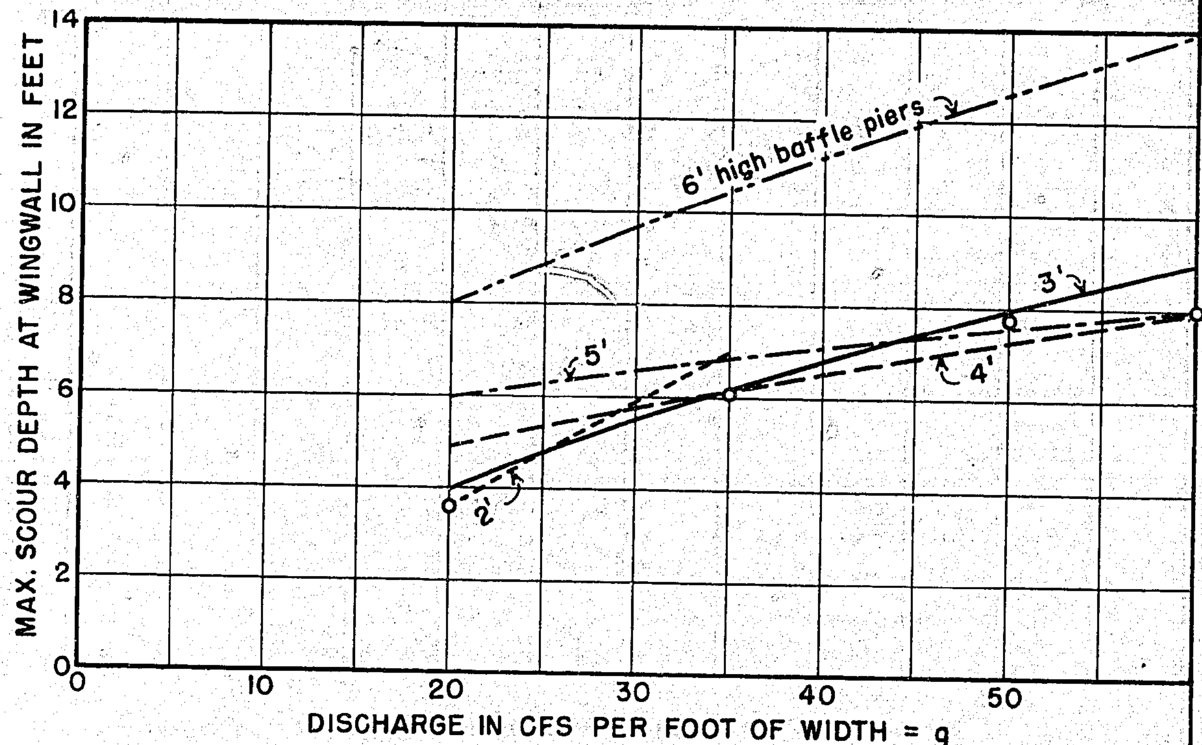
Note: Bed at Elev. 914 at
start of 30-minute test



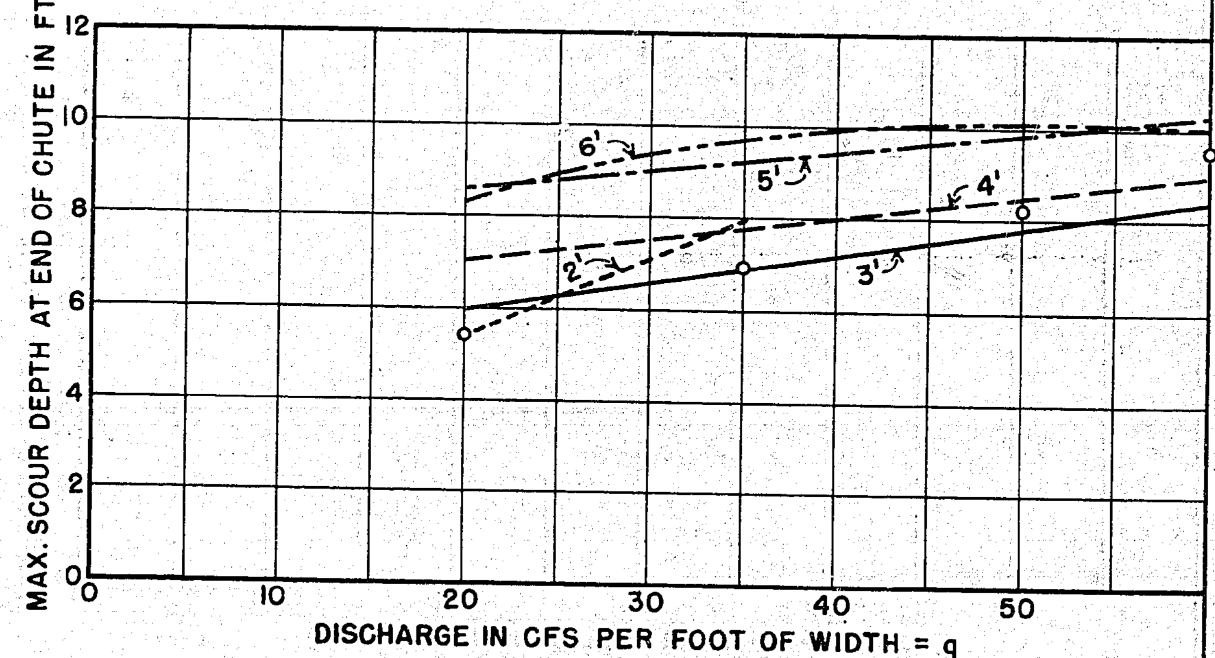


B. Discharge 20 cfs
per foot of width

Baffled Chute Studies
Baffle piers 3'-0" high



A



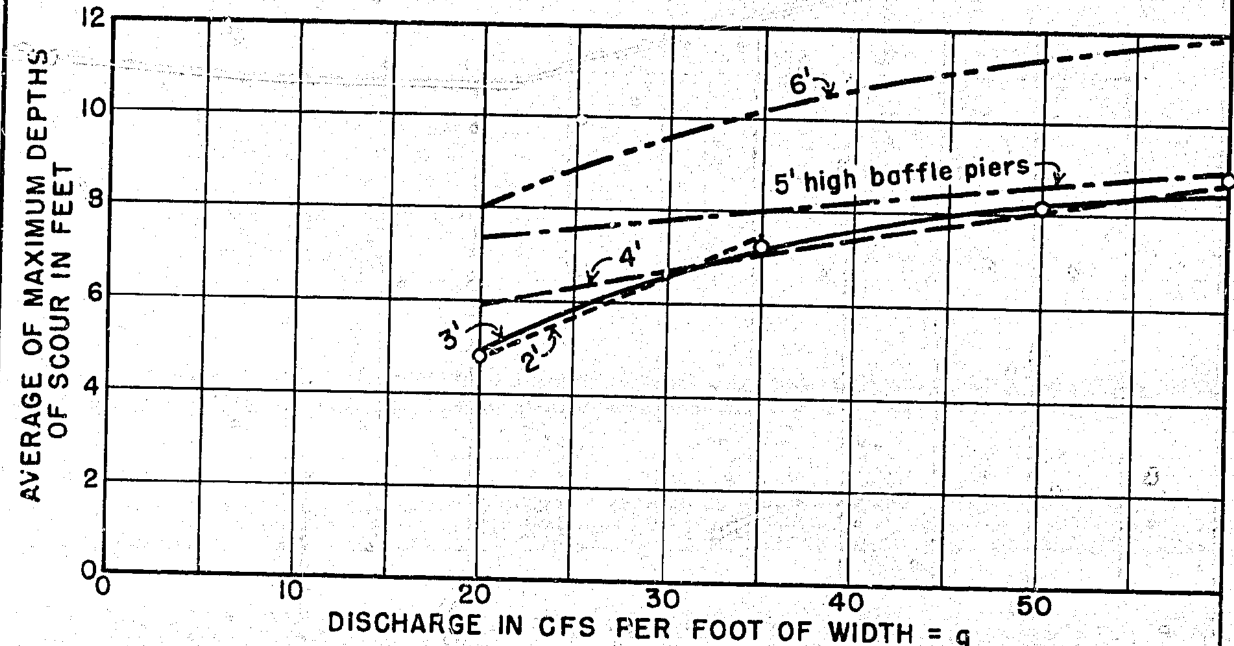
B

o's Show performance of recommended baffle piers

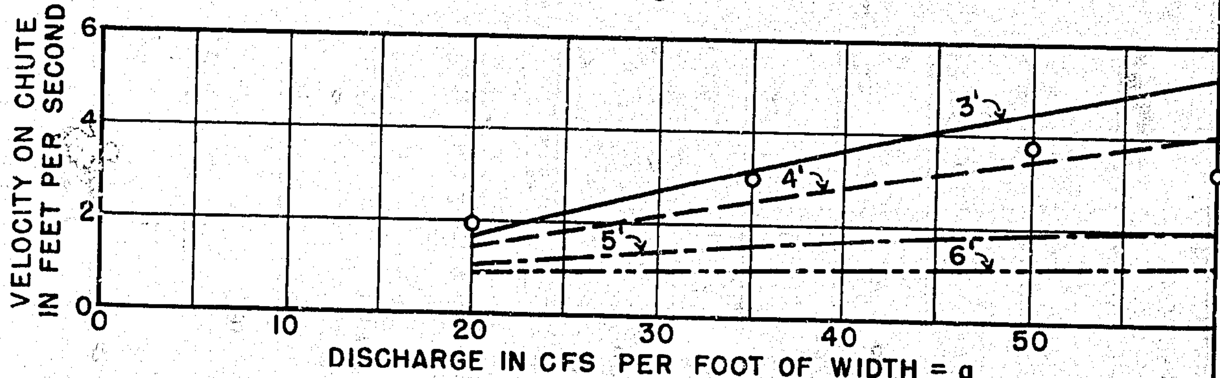
BAFFLED CHUTE STUDIES

SCOUR TEST RESULTS

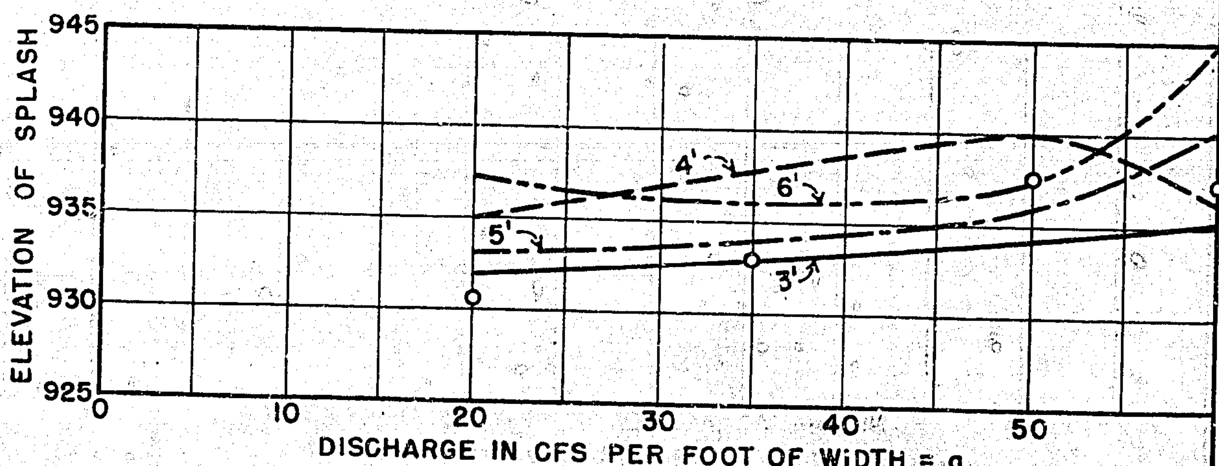
FIGURE 22
REPORT HYD 445



C



D

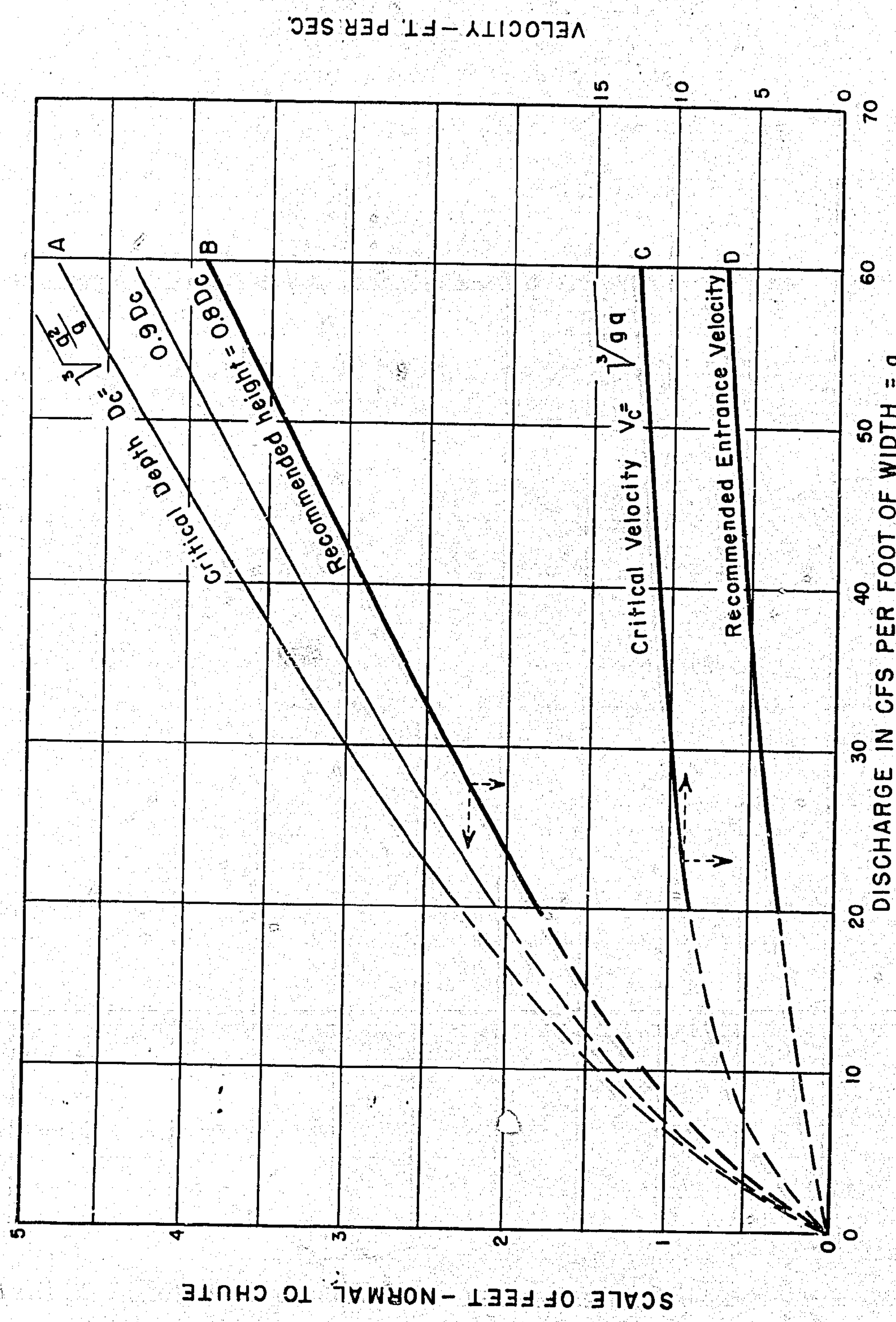


E

O's Show performance of recommended baffle piers

BAFFLED CHUTE STUDIES

SCOUR, VELOCITY AND SPLASH TEST RESULTS

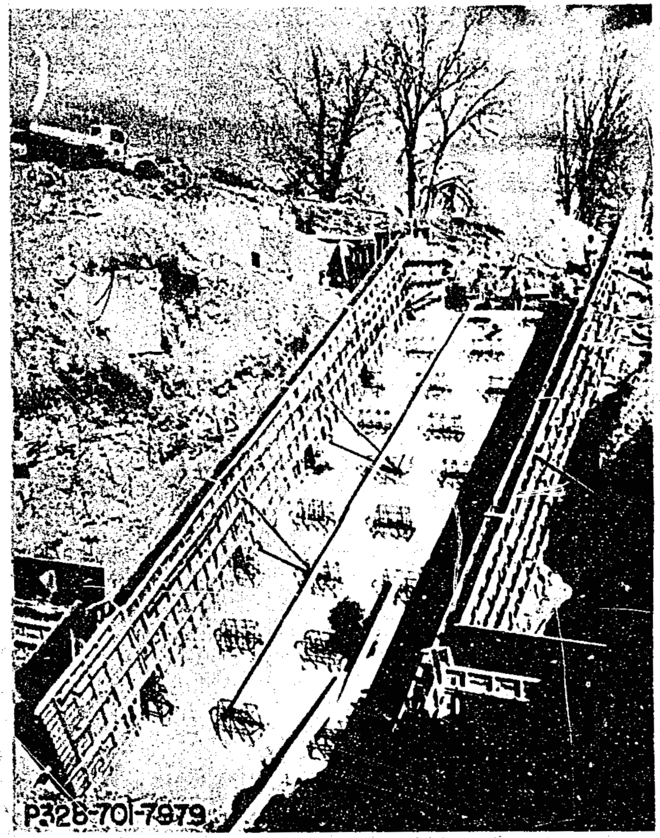


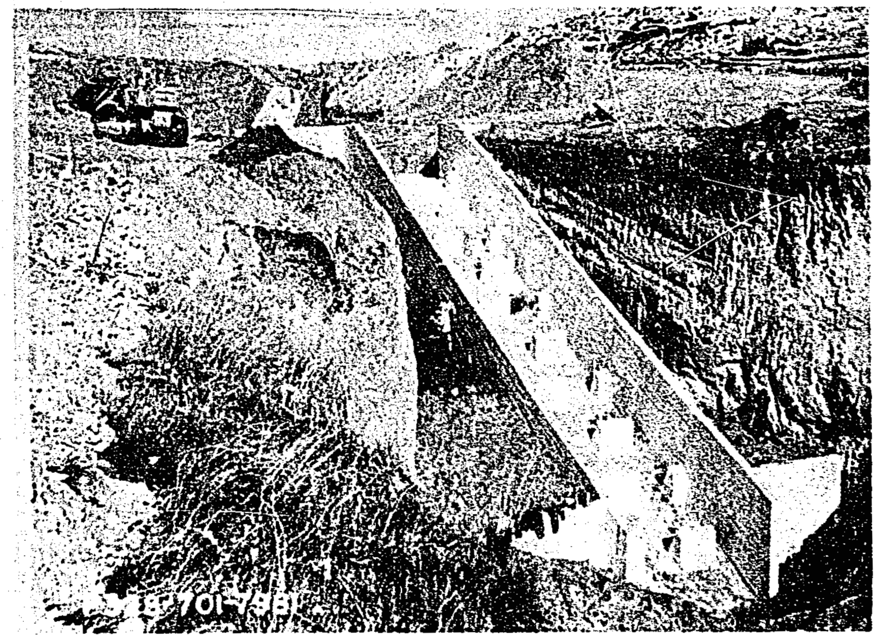
BAFFLED CHUTE STUDIES

RECOMMENDED BAFFLE PIER HEIGHTS AND ALLOWABLE VELOCITIES

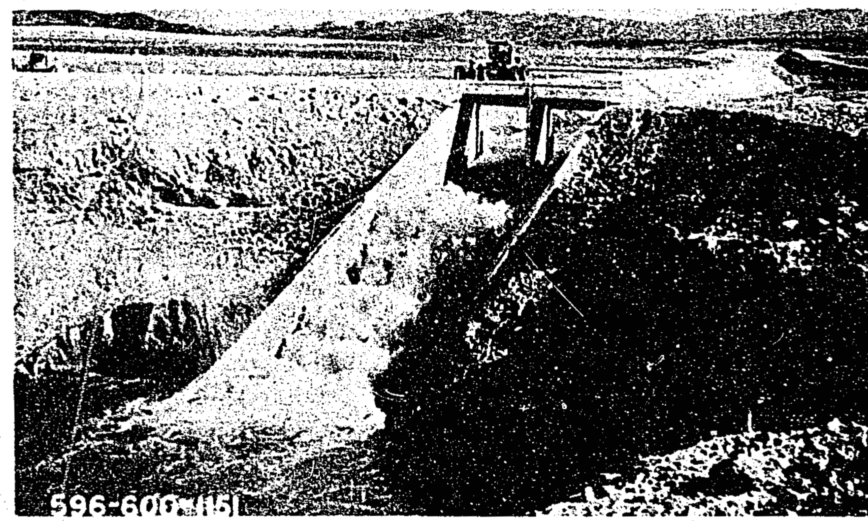
437

Figure 24

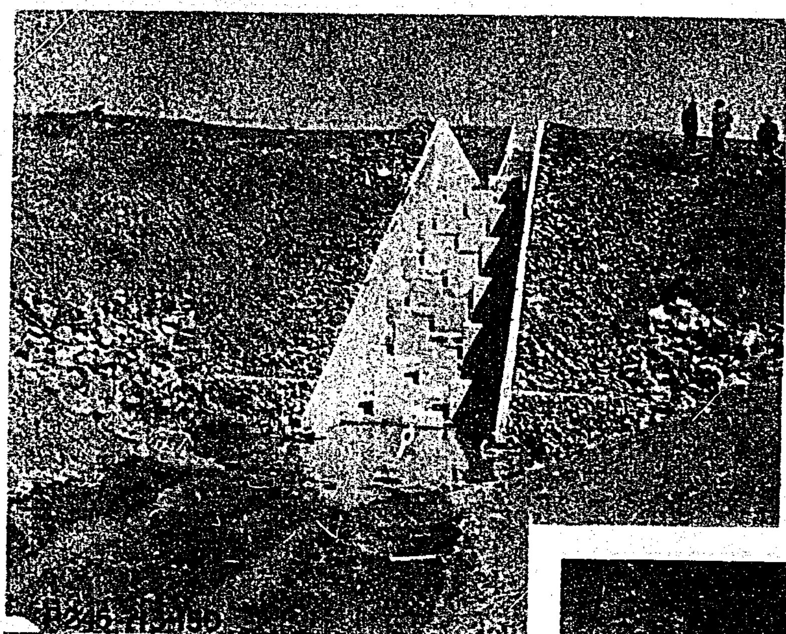


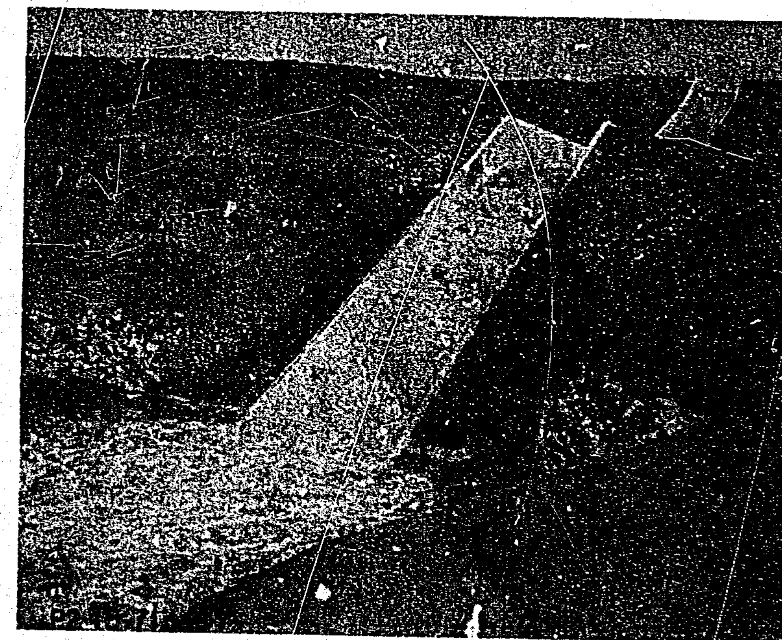


Setting forms for baffled apron
at Sta. 3+35 of Wasteway 10.7
and compacting backfill at
Sta. 2+85 of Wasteway 11.1,
Culbertson Canal, MRBPN,
Spec. No. DC-5101. October 1959



63 second feet flowing into Helena
Valley Regulating Reservoir. May
1959, from Helena Canal. Note erosion of soft earth bank; performance
otherwise excellent



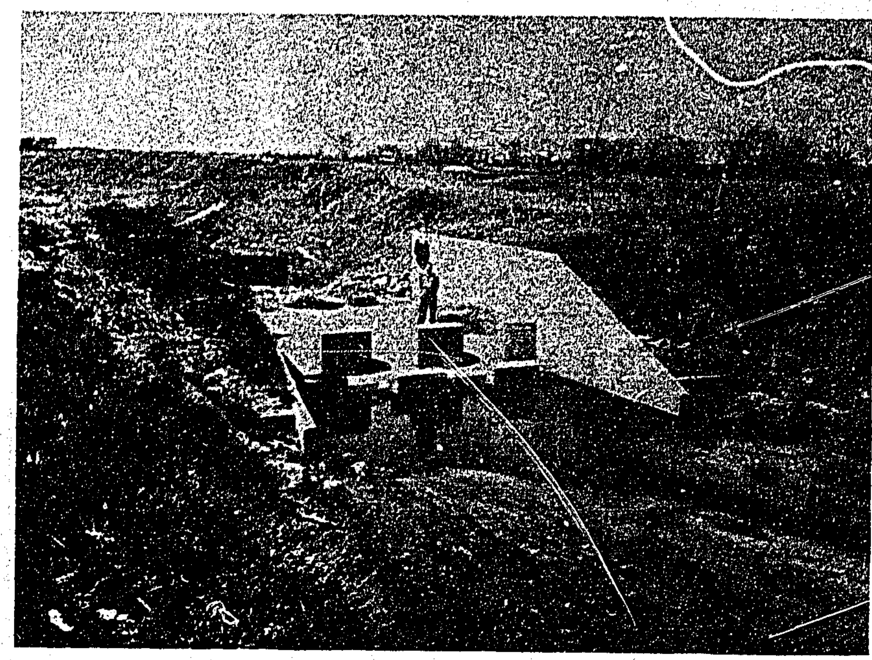


Baffled apron at Sta. 667+78, Extension Boulder Creek Supply Canal, designed for 200 second-feet; discharging 150 second-feet in upper photo and 100 second-feet in lower photo, April 1955. Performance excellent

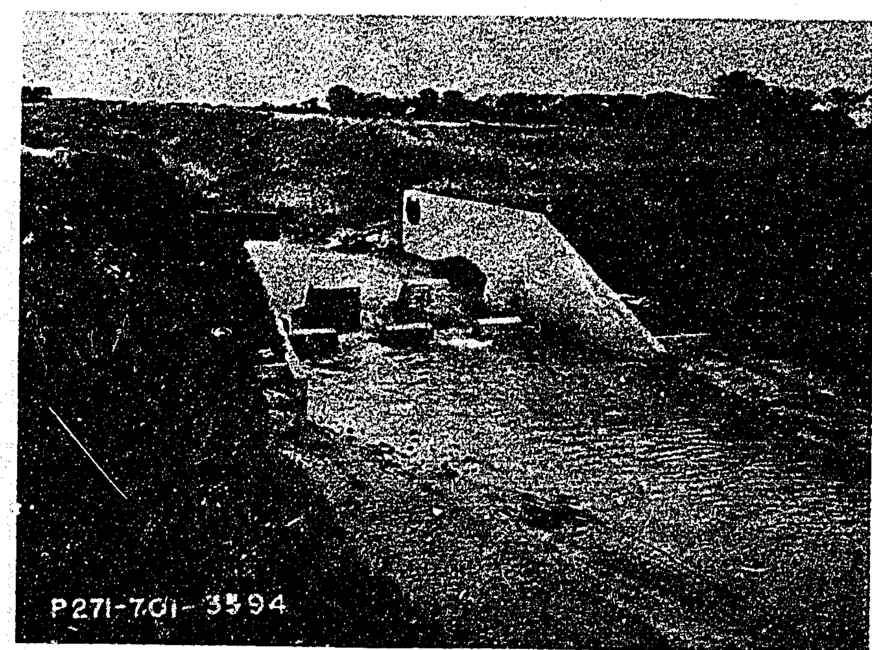
Note wet rock area adjacent to training walls.

Figure 25

Figure 26



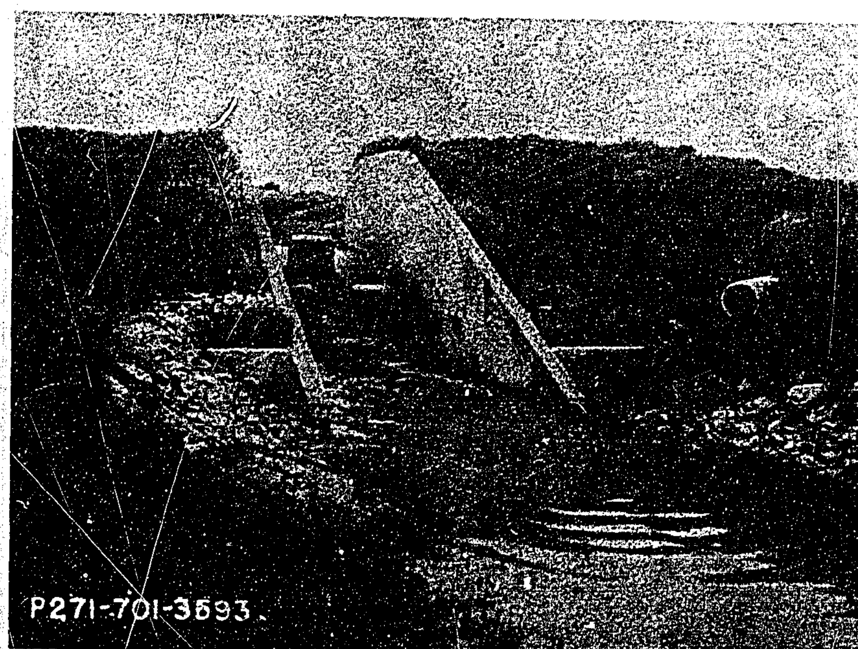
No flow. Structure designed for 924 second-feet
April 1959



Discharge about 5 second-feet in May 1959.
Field reports that structure performs well
for larger flows

Bostwick Courtland Canal
Drain A - Sta. 6+08 Spec. No. DC-4021

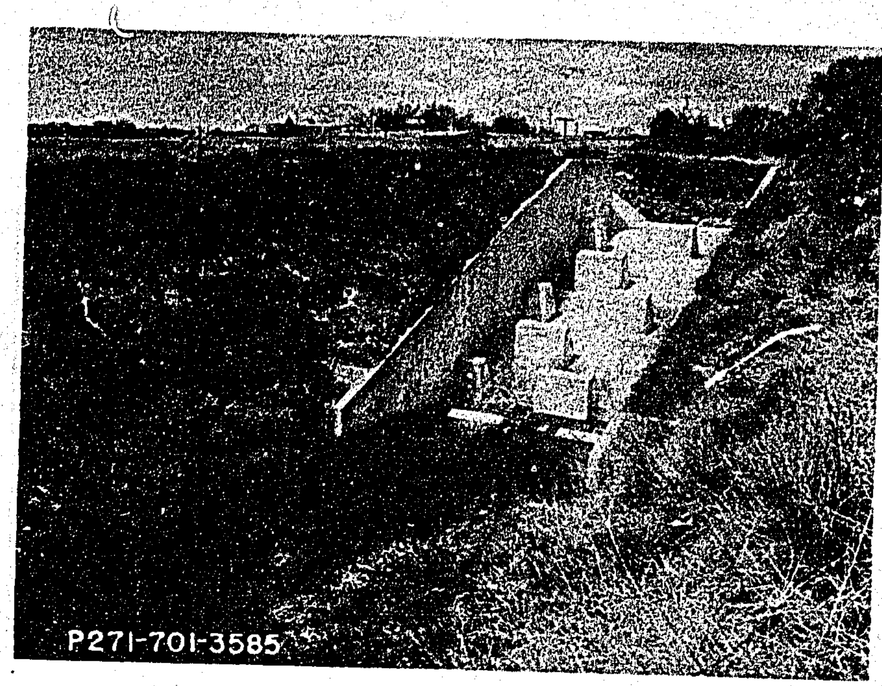
No flow. Note trash. April 1959
Spec. No. DC-4021 Design Discharge
277 second feet



Discharge about 3 second-feet in May 1959
Field reports that structure performs well
for large discharges

Bostwick Courtland Canal
Drain A Sta. 67+93

Figure 28



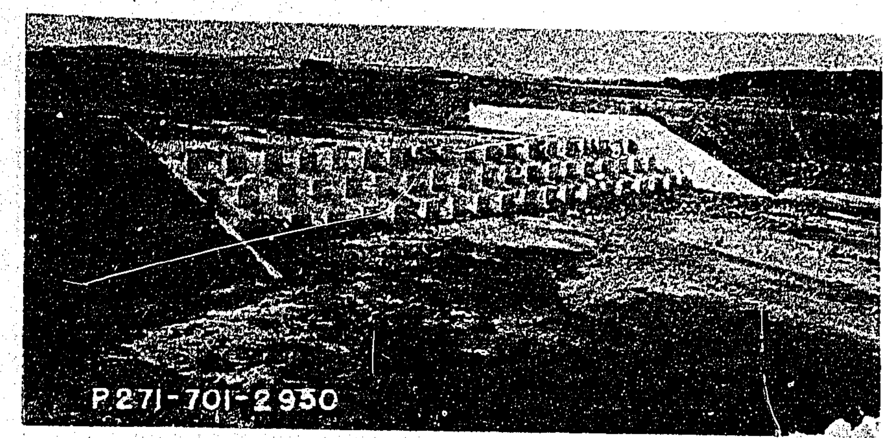
Structure after 4 years of operation; January 1954
Performance has been satisfactory. Design
Discharge 625 second-feet

Bostwick Franklin Canal Drain F-10.1 Sta. 84+68
Spec. No. DC-3720

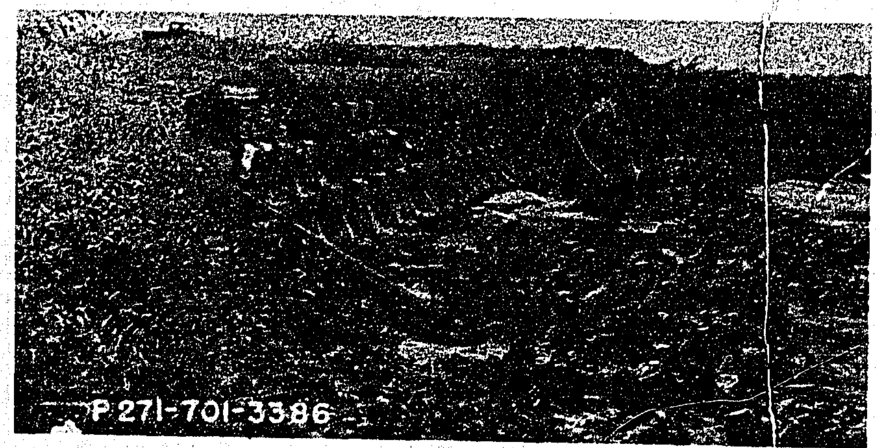
Structure after 5 years of operation; April 1959
Satisfactory performance. Design Discharge
1,100 second-feet

Bostwick Franklin Canal Drain F-14.9
Sta. 5+20 Spec. No. DC-3891

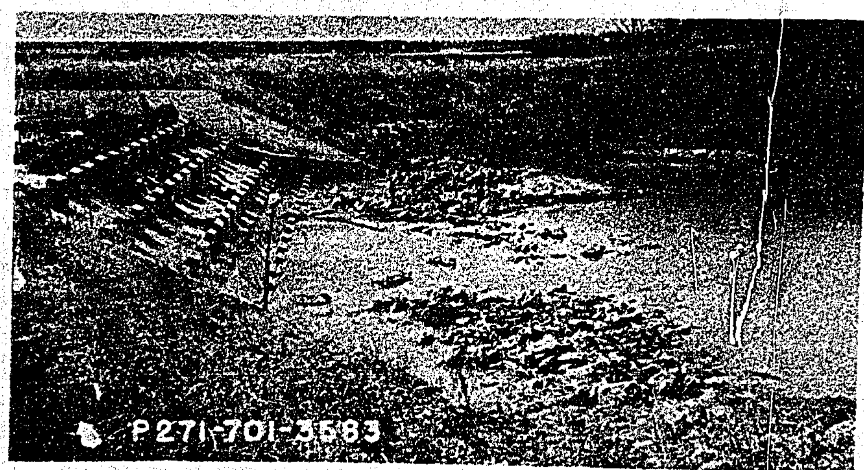
No flow on October 8, 1956



Erosion after a year of operation has exposed 1 more row of blocks. Rocks were sorted from finer material which moved. Note trash. Photo date September 18, 1957

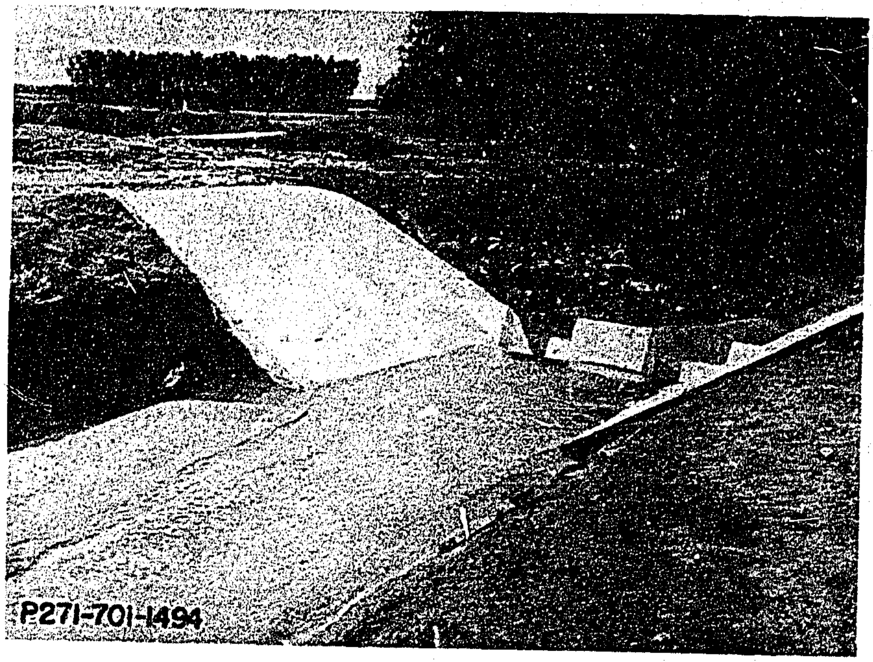


Erosion did not continue at original rate, is no more severe after 2-1/2 years of operation. Photo date April 21, 1959



Progress of Erosion. Bostwick Crow Creek Drain
Sta. 28+90 Design Discharge 2,000 second-feet.
Spec. No. 700C-400

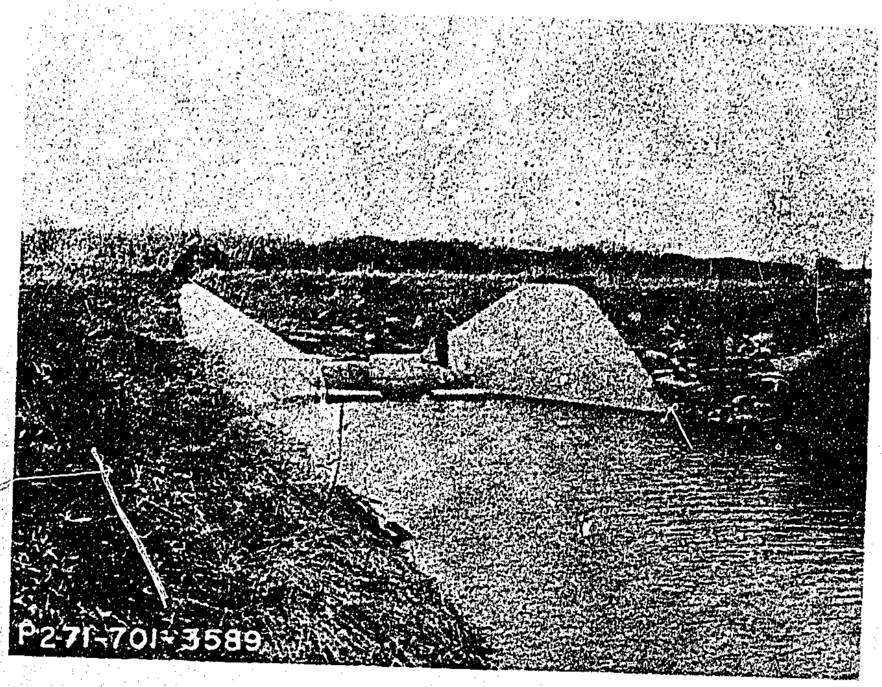
Figure 30



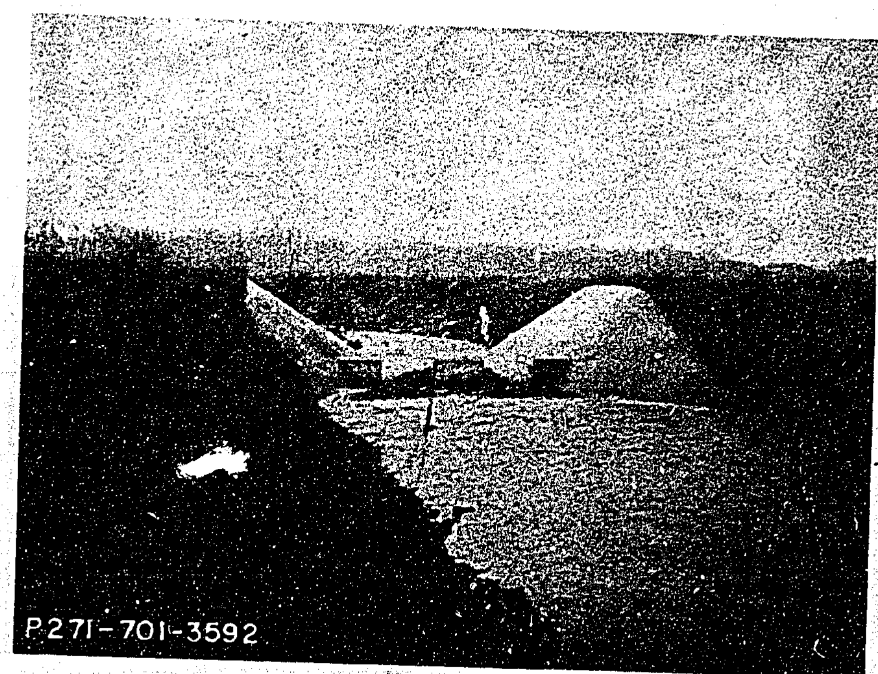
Unstable banks collapsed after only 6 mo. operation.
Note protection afforded by downstream riprap

Upstream banks were badly eroded

Unstable Banks Create Erosion Problem
(See Figure 31) May 28, 1953
Bostwick Superior Canal Drain 2A Sta. 36+82.4



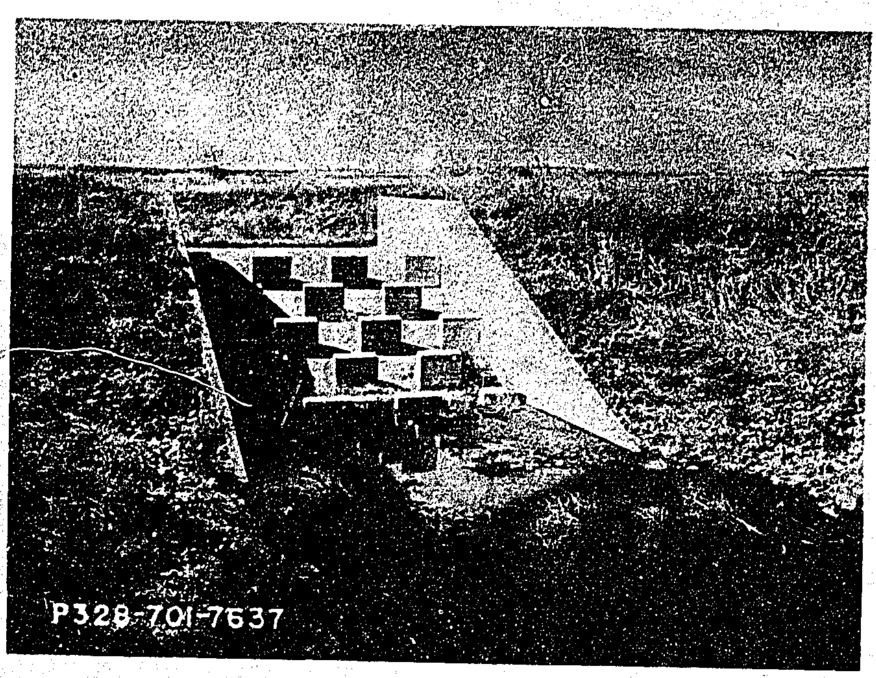
Stablized Banks 6 years later show no evidence of erosion. April 23, 1959.



Performance of structure during rainstorm. Discharge 81 second-feet May 1959. Design Discharge 400 second-feet

Stablished Banks Present No Erosion Problem
(See preceding Figure 30)
Bostwick Superior Canal Drain 2A Sta. 36+82.4

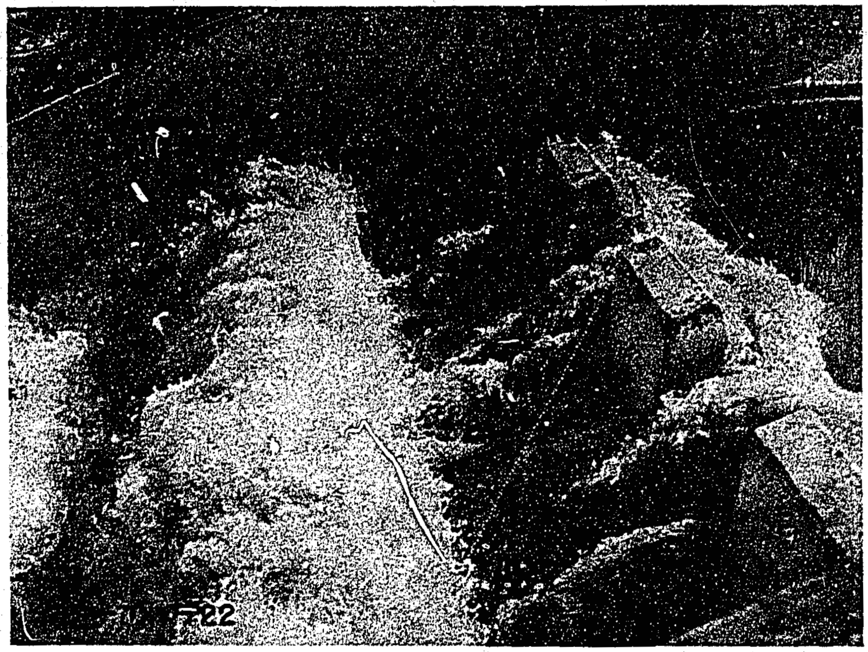
Figure 32



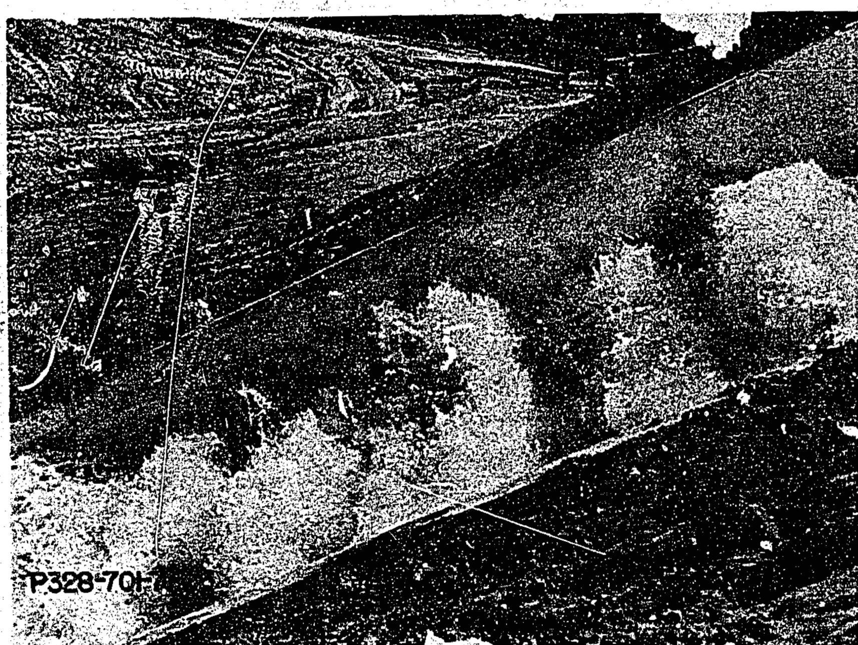
After 4-1/2 years of operation. Excellent performance. Photo date April 14, 1959. Design Discharge 1,000 second-feet. Frenchman-Cambridge - Drain 8-C. Spec. No. DC-3940

Baffled apron discharging 75 second-feet. May 1959. Good performance. Design Discharge 400 second-feet. Spec. No. 5101

Culbertson Canal Wasteway 3.3
Spec. No. 5101

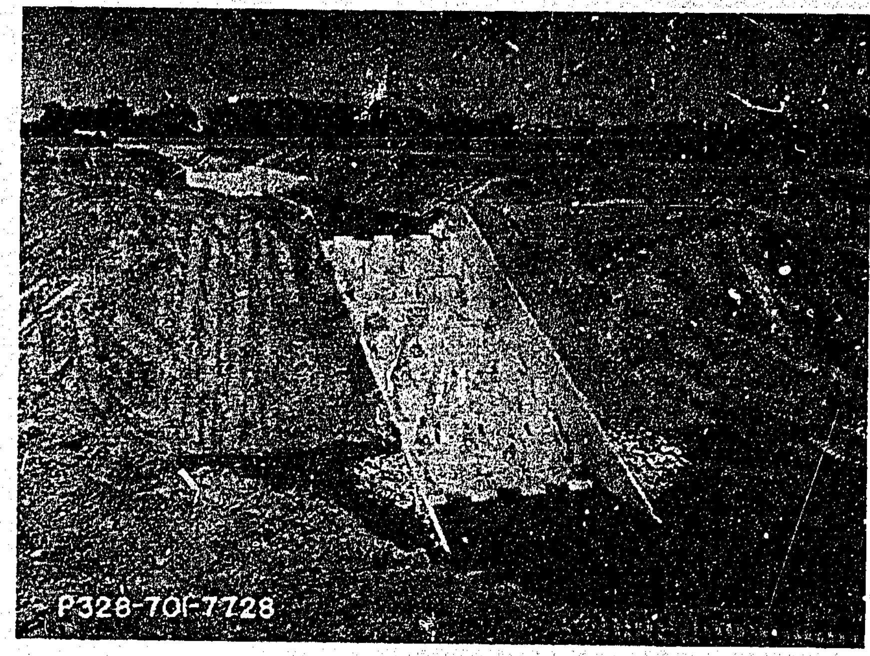


75 sec-ft. at top of baffled apron. Note second row of baffles completely covered because of acceleration of flow between first and second rows.

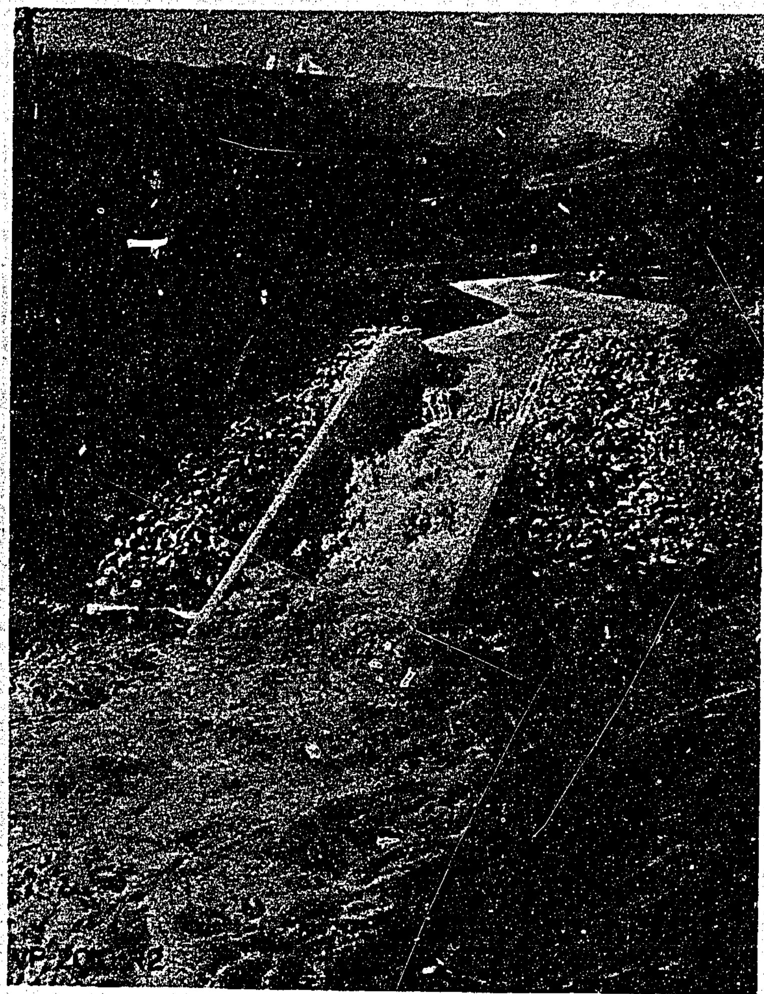


Same flow in mid-portion of chute. (See Figure 32 for general view)

Figure 34

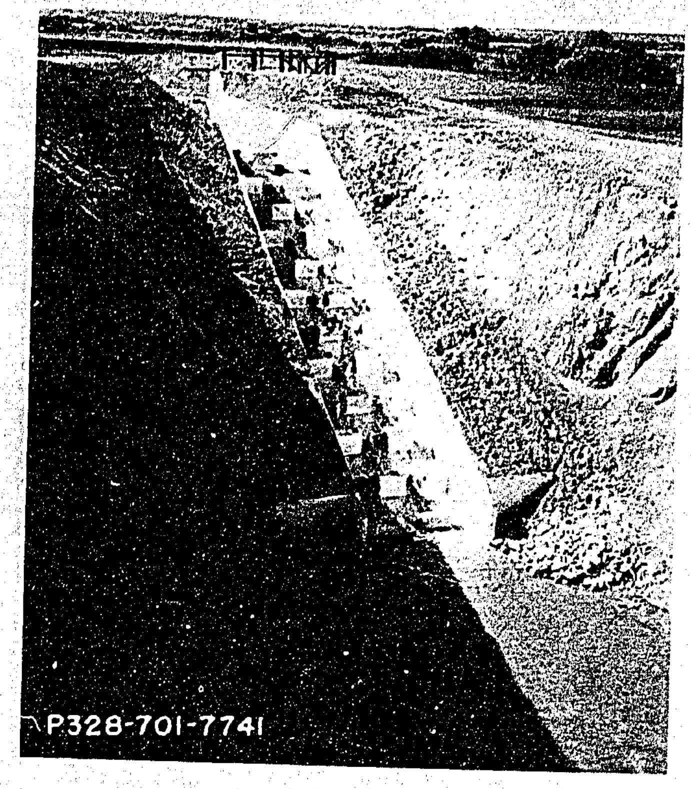


After a discharge of 75 sec-ft. May 1959
Culbertson Canal Wasteway 3.3



Robles-Casitas Diversion Dam and Canal. Canal between station 294 and station 298 with 500 c.f.s. flow of water discharging into Santa Ana Creek at station 298+98. Note that waves in canal section occasionally splash over top of canal concrete lining. February 16, 1959

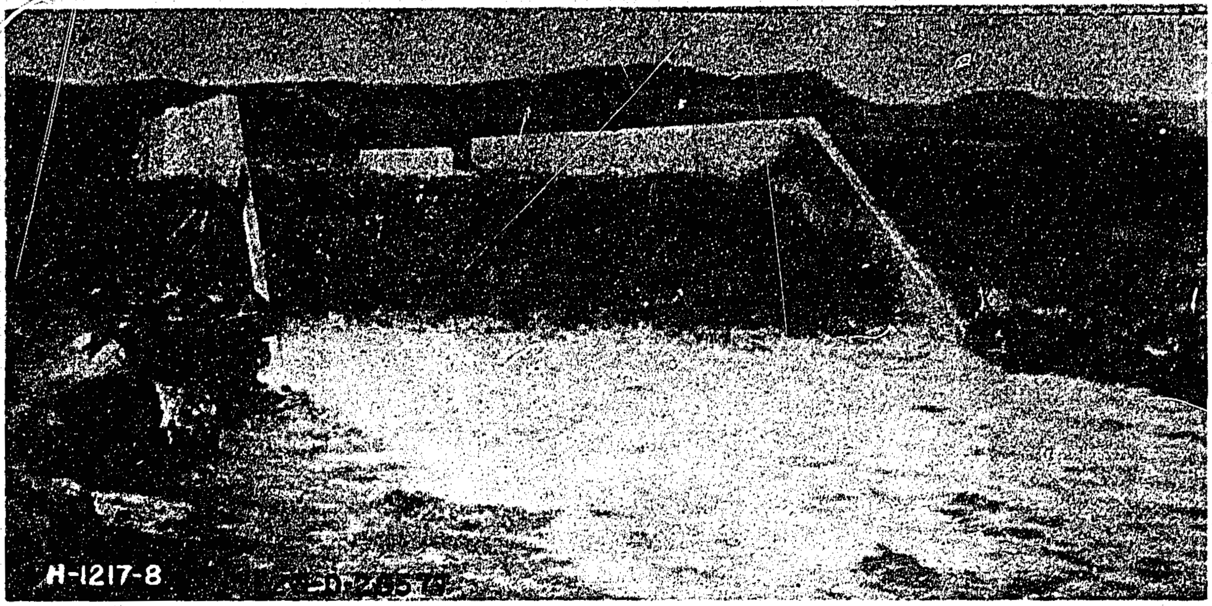
Note excellent use of a small amount of riprap



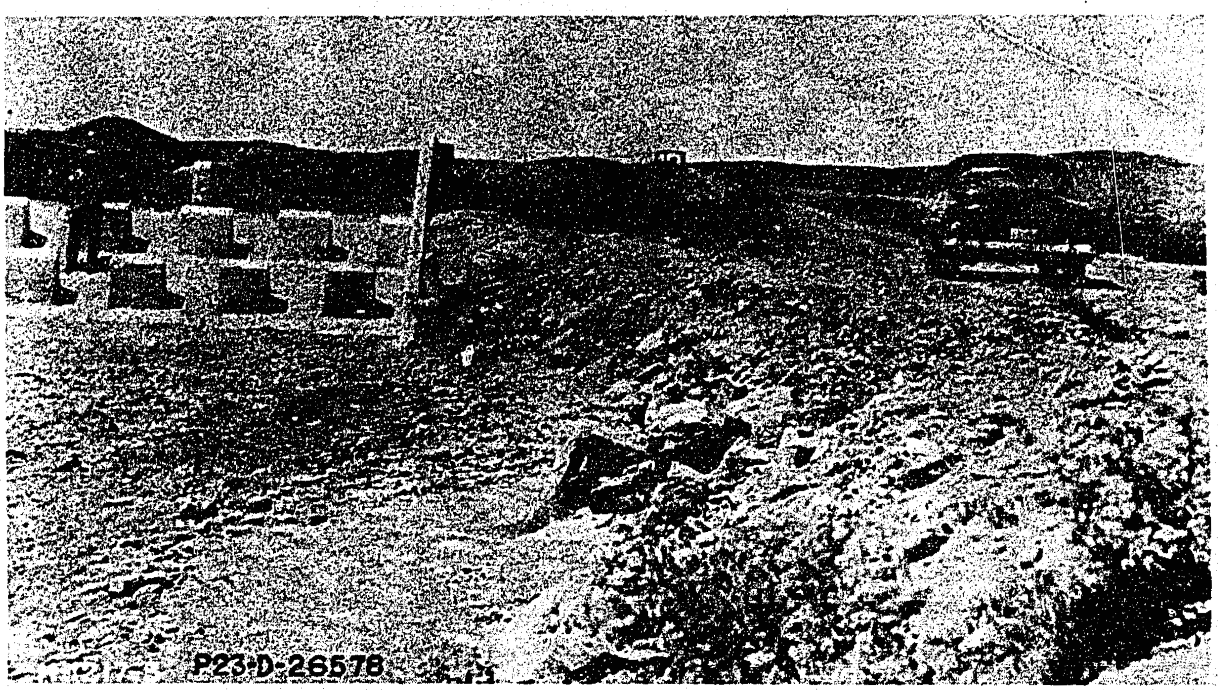
Note stilling action of blocks for small discharge. Spec. No. DC-4972

Frenchman-Cambridge Meeker Extension Canal Wasteway
Sta. 1777+18 May 1959 Discharge about 5 sec. ft.
Design Discharge 269 Sec. ft.

Figure 36



Estimated Discharge 15 cfs/ft (half capacity)



Channel after flood - material was deposited rather than scoured

Rio Grande Project
PICACHO ARROYO SYSTEM
NORTH BRANCH WASTEWAY CHANNEL

GPO 846966