Stream Gaging Control Structure for the Rio Grande Conveyance Channel Near Bernardo New Mexico

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1369-E

W-1369-E

Stream Gaging Control Structure for the Rio Grande Conveyance Channel Near Bernardo New Mexico

By D. D. HARRIS *and* E. V. RICHARDSON

RIVER HYDRAULICS

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1369-E

Design of a control structure

UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1964

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

Thomas B. Nolan, *Director*

The U.S. Geological Survey Library has cataloged this publication as follows:

Harris, David Dell, 1925–

Stream gaging control structure for the Rio Grande conveyance channel near Bernardo, New Mexico, by D. D. Harris and E. V. Richardson. Washington, U.S. Govt. Print. Off., 1964.

iv, 123–154 p. illus., maps, diagrs., profiles, tables. 24 cm. (U.S. Geological Survey. Water-Supply Paper 1369–E)

River hydraulics.

Bibliography: p. 154.

1. Stream measurements—New Mexico—Rio Grande. 2. Hydraulic engineering. I. Richardson, Everett V. 1924– joint author. II. Title. (Series)

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C. 20402

W-1369-E

CONTENTS

ILLUSTRATIONS

W.1369-E

TABLES

W-1369-E

RIVER HYDRAULICS

STREAM GAGING CONTROL STRUCTURE FOR THE RIO GRANDE CONVEYANCE CHANNEL NEAR BERNARDO, NEW MEXICO

By D. D. Harris and E. V. Richardson

ABSTRACT

A stream gaging control structure to stabilize and increase the sensitivity of the stage-discharge relation of the Rio Grande conveyance channel near Bernardo, N. Mex., was designed on the basis of model studies and observed field conditions. The structure is designed to eliminate the effect of changes in bed configuration and bed elevation on the stage-discharge relation. These changes have shifted the water-surface elevation at the gaging station as much as 5 feet without a change in discharge. In addition to improving the stage-discharge relation, the structure provides a section where the concentration of the total sediment discharge of the channel can be measured.

INTRODUCTION

A stream gaging control structure for the Rio Grande conveyance channel near Bernardo, N. Mex., was designed on basis of model studies and field observations of the channel characteristics. The control structure is intended to stabilize the stage-discharge relation and to facilitate measurement of total sediment discharge at a site 5 miles below the channel-head gates. This report describes the problems and site conditions for the conveyance channel, the recommended control design and position in the channel, and the details of the model study.

The conveyance channel, which is about 80 feet wide and has bed and banks composed of fine sand (median diameter of 0.24 mm), is designed to transport all riverflows up to about 2,000 cfs (cubic feet per second); the old river channel is now used as a floodway. Flows greater than 100 cfs occupy the full width of the channel; lesser flows occupy only the streambed. Median flow is 280 cfs, and flows exceed 1,000 cfs 16 percent of the time (fig. 29). The bottom slope ranges from about 0.00055 foot per foot in the upper 1½ miles to about 0.0008

W-1369-E

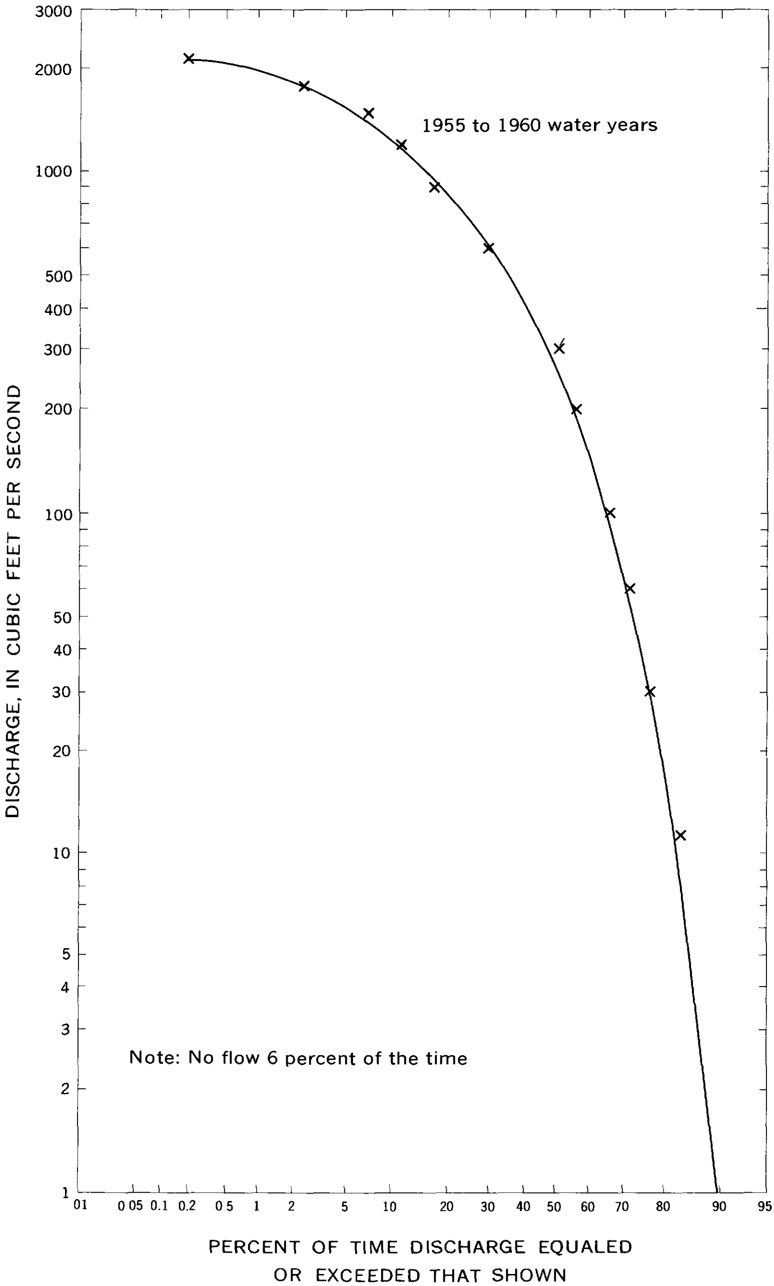


FIGURE 29.—Duration curve of daily flow, Rio Grande conveyance channel near Bernardo, N. Mex.

W-1369-E

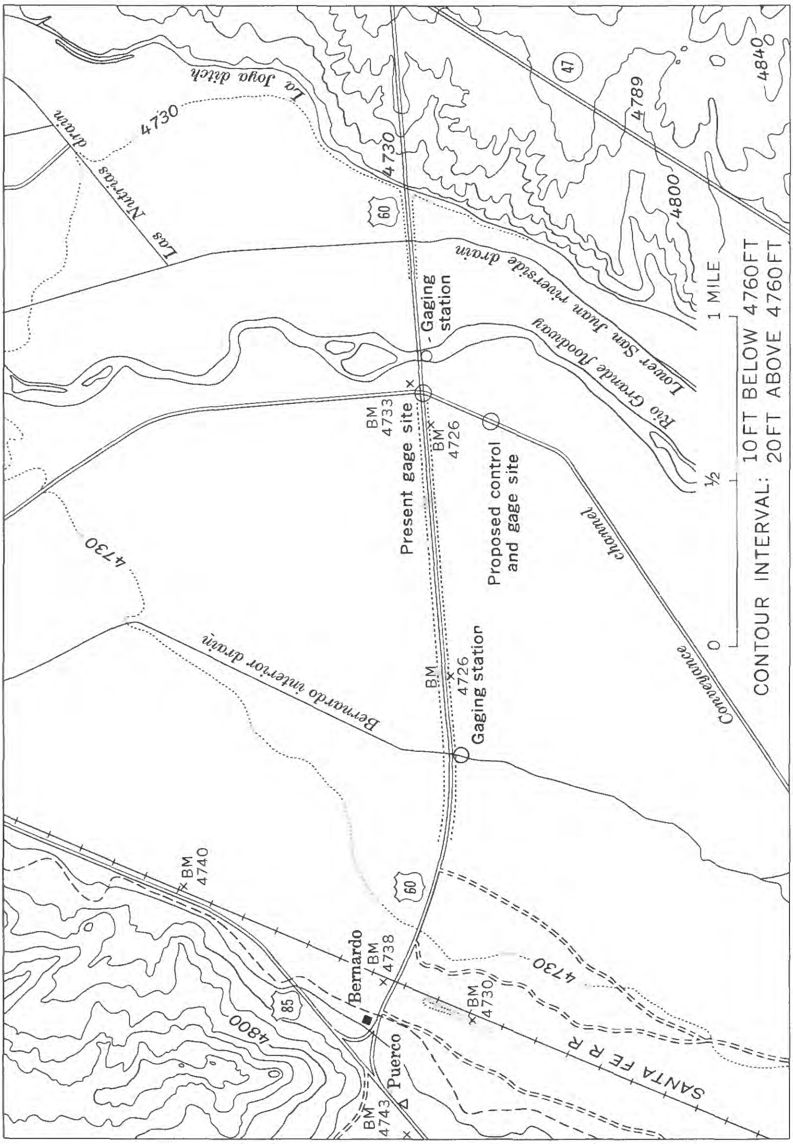


FIGURE 30.—Map of Bernardo area.

foot per foot in the lower reaches. The location of the present gaging station and the site selected by engineers of the U.S. Geological Survey and the U.S. Bureau of Reclamation for the control structure are given in figure 30. Photographs of the control-structure site are shown in figures 31 and 32.

Both the configuration and mean elevation of the channel bed change with discharge. As shown in figure 33, two different depth-discharge relations occur. During low flows dune-bed configuration generally occurs, changing to plane-bed configuration at discharges between 500 and 1,200 cfs. However, a dune bed has been observed at flows as large as 2,000 cfs and a plane bed, at flows as small as 200 cfs. The change in velocity associated with the change in bed configuration is given in figure 34. Channel conditions for 1961 are indicated on figures 33 and 34 by measurements numbered 968 to 975. The stage-discharge relation, in addition to being affected by all the factors of variability of the depth-discharge relation, is affected by the changes in bed elevation. The mean bed elevation at the gage has varied between a maximum gage height of about 5 feet and a minimum gage height of slightly less than 1 foot during the period 1955–61. This variation of 4 feet in mean bed elevation may occur in a single water year. (See fig. 36.) An increase in discharge is generally accompanied by an increase in bed elevation, and a decrease in discharge is usually accompanied by a decrease in bed elevation; however, exceptions have occurred, as shown in figures 35 and 36. Bankfull stage, about 9.5 feet, occurs at discharges of about 2,000 cfs, accompanied by a combination of high bed elevation and a dune-bed configuration.

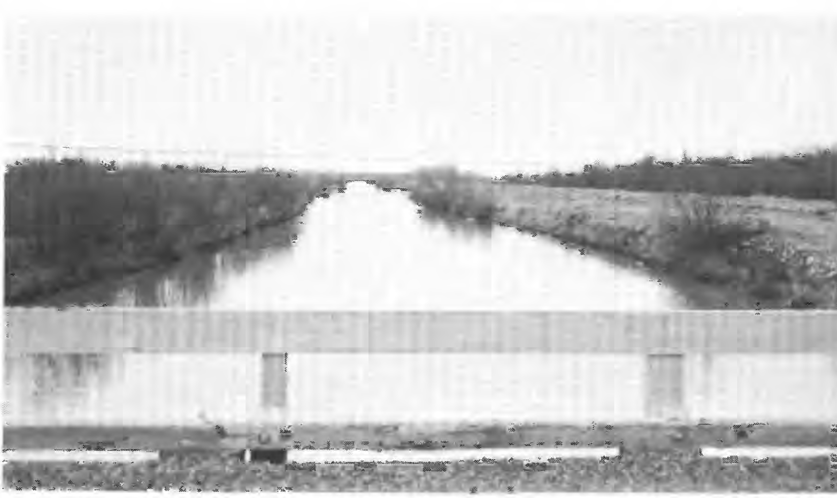


FIGURE 31.—Rio Grande conveyance channel, near Bernardo, with a flow of 700 cfs. View is downstream toward proposed control site.

FIGURE 32.—Rio Grande conveyance channel, near Bernardo, without flow. View is downstream toward proposed control site.

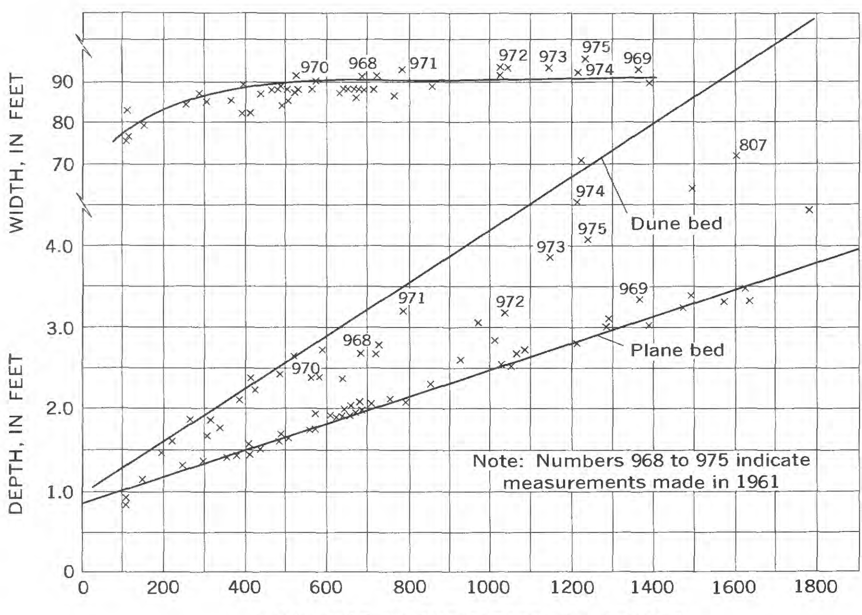


FIGURE 33.—Relation of width and depth to discharge on the Rio Grande conveyance channel near Bernardo, N. Mex.

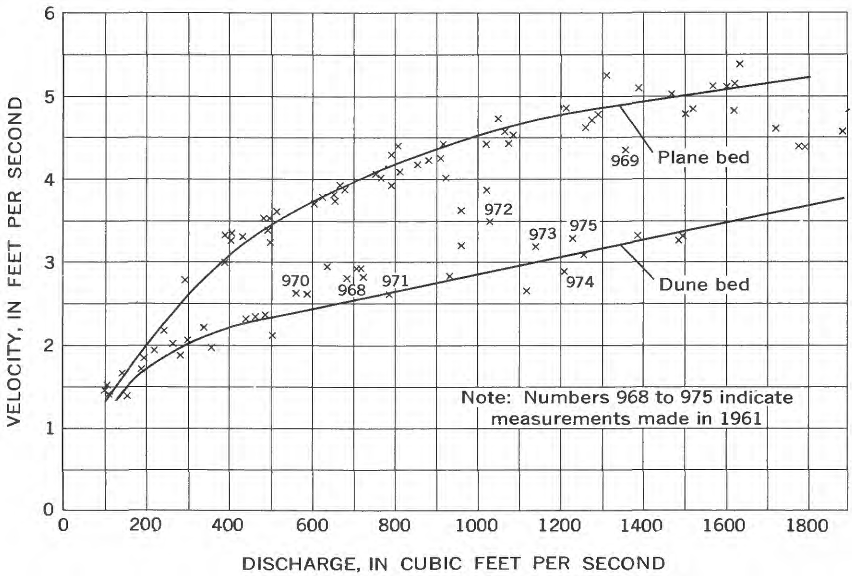


FIGURE 34.—Relation of velocity to discharge on the Rio Grande conveyance channel near Bernardo, N. Mex.

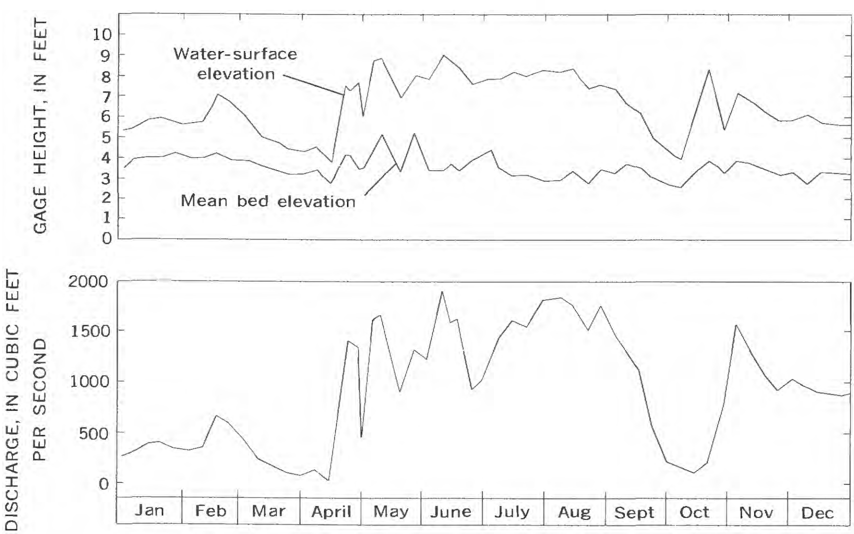


FIGURE 35.—Variation in water-surface elevation, bed elevation, and discharge during 1957.

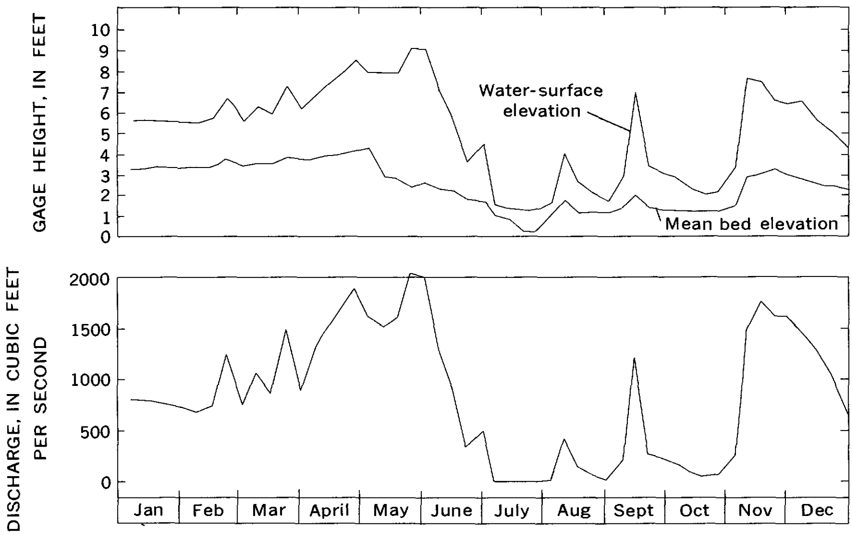


FIGURE 36.—Variation in water-surface elevation, bed elevation, and discharge during 1958.

An auxiliary gage was operated at the proposed site during the summer of 1961 to determine prototype conditions. The data for this gage is given in table 1. The auxiliary gage was 1,000 feet downstream from the present gage, and the differences in mean sea level elevation are consistent with the slope of the channel. The hydraulic characteristics of the channel at the site of the proposed control structure are probably the same as those at the present gage, on the basis of channel appearance and a comparison of observations at the auxiliary gage with the record at the present gage.

The purpose of the model study was to determine a control-structure design that would increase the accuracy of the water-discharge

TABLE 1.—*Observations of gage height and mean sea-level elevations, in feet, of the water surface at the present and auxiliary gage*

Date	Discharge (cfs)	Present gage		Auxiliary gage	
		Gage height	Mean sea level	Gage height	Mean sea level
5-19-61	0	0	4, 719. 23	0	4, 722. 14
Do	1, 210	8. 44	4, 727. 67	4. 70	4, 726. 84
5-22-61	1, 170	8. 30	4, 727. 53	4. 66	4, 726. 80
5-26-61	1, 230	8. 29	4, 727. 52	4. 72	4, 726. 86
6- 2-61	1, 090	7. 95	4, 727. 18	4. 25	4, 726. 39
6- 5-61	----------	8. 08	4, 727. 31	4. 46	4, 726. 60
6- 9-61	932	7. 47	4, 726. 70	3. 86	4, 726. 00

records from 10–15 percent (at present) to 5 percent and to decrease the number of discharge measurements needed to obtain the record (at present discharge measurements are made weekly). Also, the control structure should provide for the measurement of the total sediment discharge of the channel.

The objectives of the model study were as follows:

1. Determination of a suitable control-structure design that would stabilize and increase the sensitivity of the stage-discharge relation. The structure would have to eliminate, or at least minimize, the effects of changes in bed configuration and changes in bed elevation on the stage-discharge relation. The desired sensitivity of the control structure would be a stage-discharge relation in which the change in discharge would be less than 1.5 percent for a 0.01-foot change in stage.
2. Determination of the maximum elevation of a control crest which would not create sufficient backwater to interfere with the maximum design capacity of the channel (2,000 cfs); also, determination of the backwater characterists of the control under various degrees of submergence. This information, along with the prototype stage-discharge relation, would be used in determining the recomended elevation of the control crest.
3. Selection of a section or sections where accurate measurements of water discharge and sediment discharge can be made.
4. Design an energy dissipator to eliminate adverse scour downstream from the control structure.

The site conditions present many problems in meeting the objectives of the control study. The major problems are (1) the variation in the bed elevation, (2) the changing bed form, (3) the low bankfull stage, (4) the proximity of the head gates, and (5) the scouring nature of the fine-grained bed material.

The control structure must be located above the maximum bed elevation to be stable and free of sediment; however, the low bankfull stage governs the maximum elevation of the crest. Also, the changing bed form creates a problem in determining the crest elevation because a control crest that is too high may cause a rougher dune bed form, resulting in greater resistance to flow and consequent larger flow depths. Larger flow depths would restrict the capacity of the channel, either because of overbank flow or backwater at the diversion structure. If the control crest is too near the maximum bed elevation, submergence is a problem because changing bed form and elevation downstream will result in variable submergence.

The fine-grained bed material scours easily, and probably requires laying down some riprap for protection. Also, the horizontal constriction desired for control sensitivity is limited by the resulting sand erosion.

The authors gratefully acknowledge the advice and assistance given by D. B. Simons, D. W. Hubbell, W. L. Haushild, and W. L. Heckler of the U.S. Geological Survey, E. L. Pemberton of the U.S. Bureau of Reclamation, and R. V. Asmus and R. Garza of Colorado State University.

PROPOSED CONTROL STRUCTURE

RECOMMENDED CONTROL-STRUCTURE DESIGN

On the basis of the model studies (see Experimental procedures and equipment) conducted in the hydraulic laboratory at Colorado State University, the following control structure (type K) is proposed for installation at the Rio Grande conveyance channel near Bernardo, N. Mex.

The recommended control-structure design is given in figures 37 and 38. Model studies indicated that a control surface, 16 feet long, having a longitudinal slope of 16 : 1 and a transverse slope of 35 : 1, an approach apron slope of 2 : 1, and a downstream apron slope of 3 : 1 would provide a control structure with a stable stage-discharge relation under all but the most severe conditions. The transverse crest slope forms a low V-notch to provide a higher degree of rating sensitivity and to concentrate the low flows. A V-notch having a 40 : 1 lateral slope was found to change discharge 1.5 percent or less for a 0.01-foot change in stage at 400 cfs or more (fig. 39). At less than 400 cfs, the change in discharge for a 0.01-foot change in stage increases gradually to 7 percent at 30 cfs. A transverse slope of 35 : 1 is recommended for the proposed control structure. Additional convergence or installation of a sharper V-notch in the control structure tends to create an irregular water surface and causes scour.

A set of baffles mounted on the upstream edge of the crest apron (fig. 10) served to keep the low part of the V-notch clear of sand under all model conditions. A small teardrop-shaped mounting provided a means of keeping the bubbler-gage orifice above any layers of sand that encroached downstream toward the crest. The "teardrop" was located with its top 0.05 foot below the crest at a point 4 feet upstream from the crest (fig. 38). Water-surface profiles (fig. 40) showed that auxiliary orifices could be placed at points 5 feet and 8 feet upstream from the crest. Orifices placed any further than 8 feet upstream from the crest could be affected by a small surface wave or by sand encroachment. Sand intrusion on the control surface outside the baffle zone did not seem to create any unusual water-surface conditions or to affect the rating adversely.

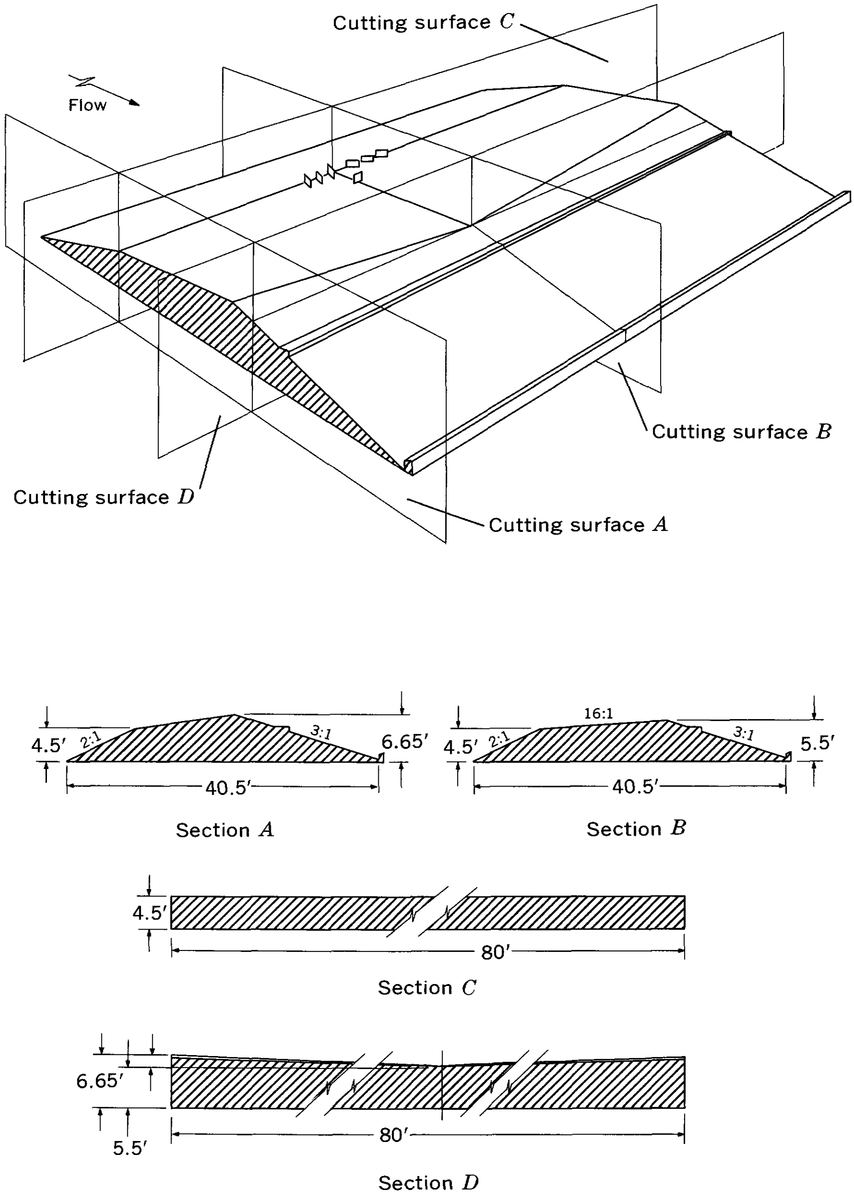


FIGURE 37.—Details of proposed control structure.

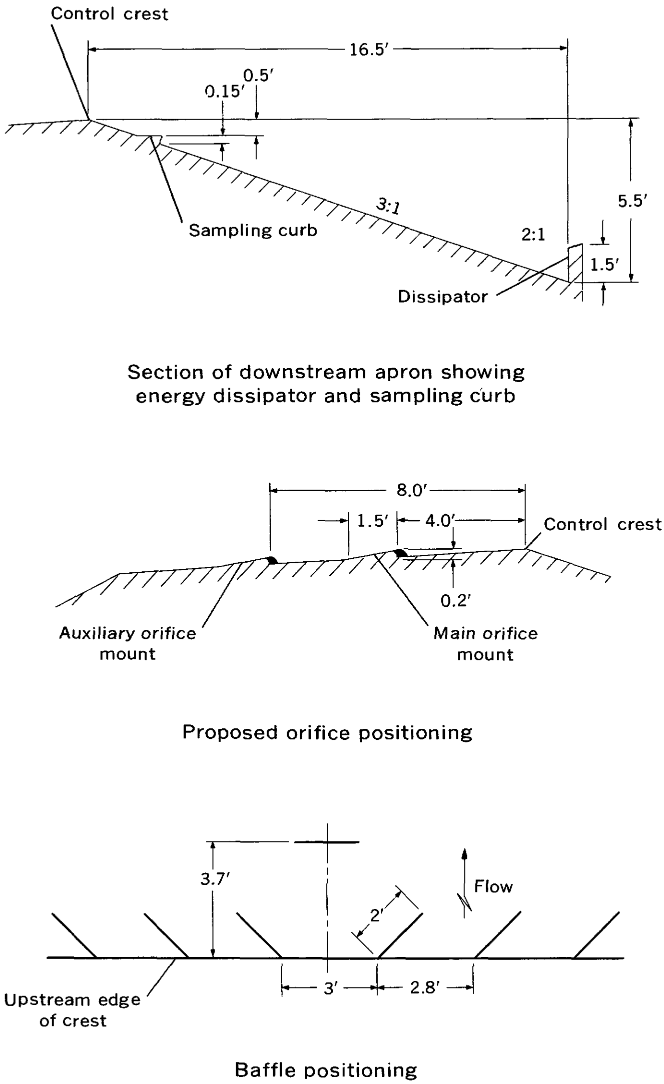


FIGURE 38.—Accessories of proposed control structure.

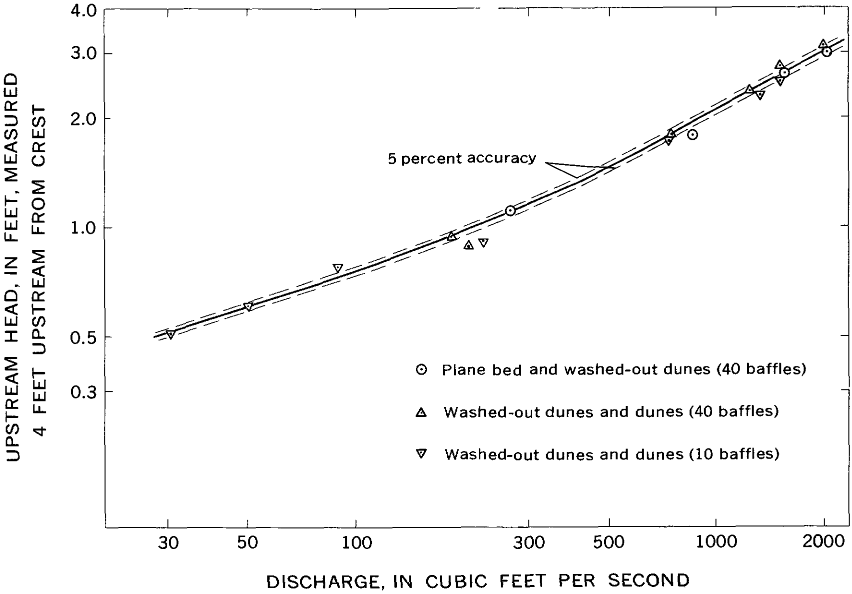


FIGURE 39.—Stage-discharge relation for control K.

An energy dissipator sill, 1.5 feet high, at the toe of the downstream face of the control prevented excessive downstream scour. However, for protection against scour, riprap may be needed on the bed and along banks for a short distance downstream from the dissipator. Riprap along banks in the vicinity of and upstream from the control structure may also be necessary. The elevation of the top of the sill should be the same as the low bed elevation. This elevation would be 4,720.2 feet at the present gage or 4,719.5 feet at the auxiliary site, if one assumes that a change in bed elevation would be equal to the change of the water-surface elevation given in table 1.

RECOMMENDED CONTROL-STRUCTURE POSITION

It is recommended that the elevation of the crest of the control structure be located 4.50 feet below bankfull stage, which is equivalent to a mean sea level elevation of 4,724.2 feet at the present gage and 4,723.5 feet at the auxiliary gage (proposed control-structure site).

Discharge measurements, made in the spring of 1961 and plotted on figures 30, 31, and 41, indicate lower regime flow having great depth, low velocity, and maximum gage height. Therefore, elevations given in table 1 should indicate maximum water-surface conditions for the given water discharge. The crest, elevation 4,723.5 feet, would be approximately 1.5 feet above the mean bed elevation. However, the crest of the control structure would be only 0.5 foot above the

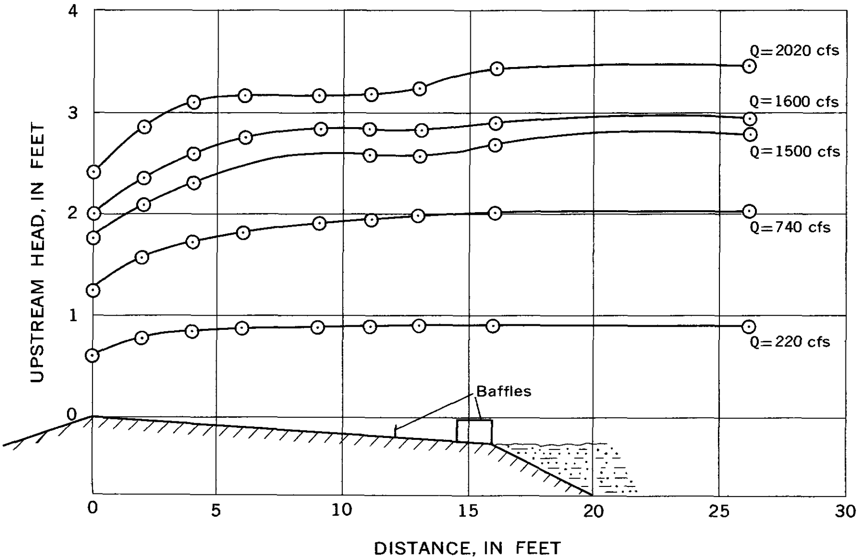


FIGURE 40.—Water-surface profiles for control K. (Free-fall conditions.)

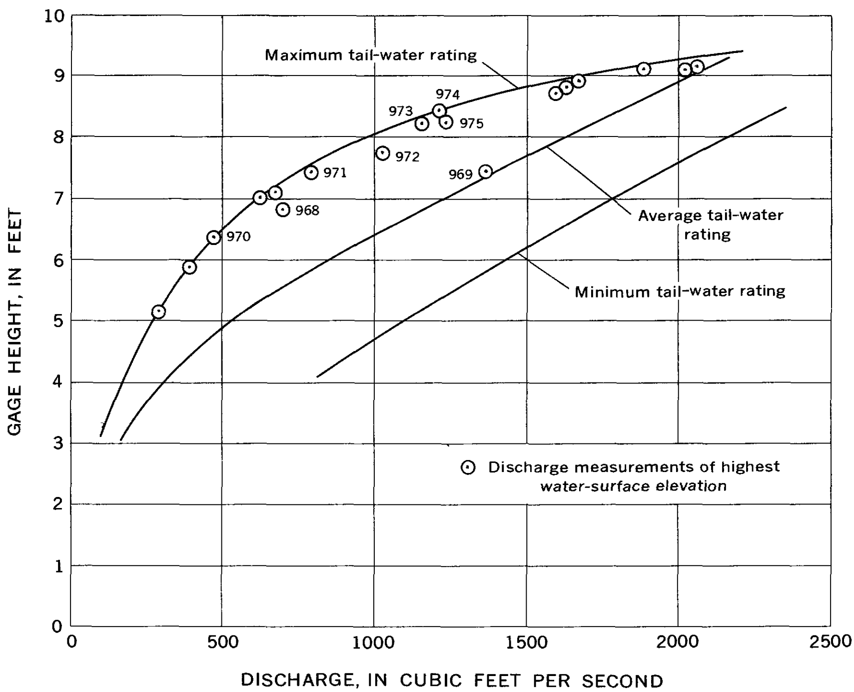


FIGURE 41.—Stage-discharge relations for maximum and average tail-water levels. (Based on field data.)

maximum bed elevation measured in 1958 and would be 0.25 foot lower than the maximum bed elevation measured in 1957. A higher crest elevation is not recommended because the effects on the bed configuration are not known; a change in the bed configuration could result in higher resistance to flow and in greater depths, which would cause the channel not to operate at design capacity.

The elevation of the control was determined from the model study, which indicated that at 2,000 cfs and 100 percent submergence the upstream water-surface elevation would be 4.0 feet above the crest of the control (fig. 42). If 4.0 feet is subtracted from the maximum tail-water elevation at 2,000 cfs (fig. 41) the crest is computed to be 4.25 feet below bankfull stage. However, if 0.5 foot of freeboard from bankfull stage is allowed, the control crest is placed 4.50 feet below bankfull stage or elevation 4,724.2 feet at the present gage. If the change in water-surface elevation between the present gage and the auxiliary gage is assumed to be the same for 2,000 cfs as for the lower discharges, the crest elevation is 4,723.5 feet.

If the crest is placed at 4.50 feet below bankfull stage, the data from the present gaging station indicate that free fall would occur at flows of less than 300 cfs under maximum tail-water conditions. However, under average tail-water conditions, free fall would occur at flows as great as 1,100 cfs. The rating curves shown in figures 43 and 44 were constructed to indicate possible measurement scatter

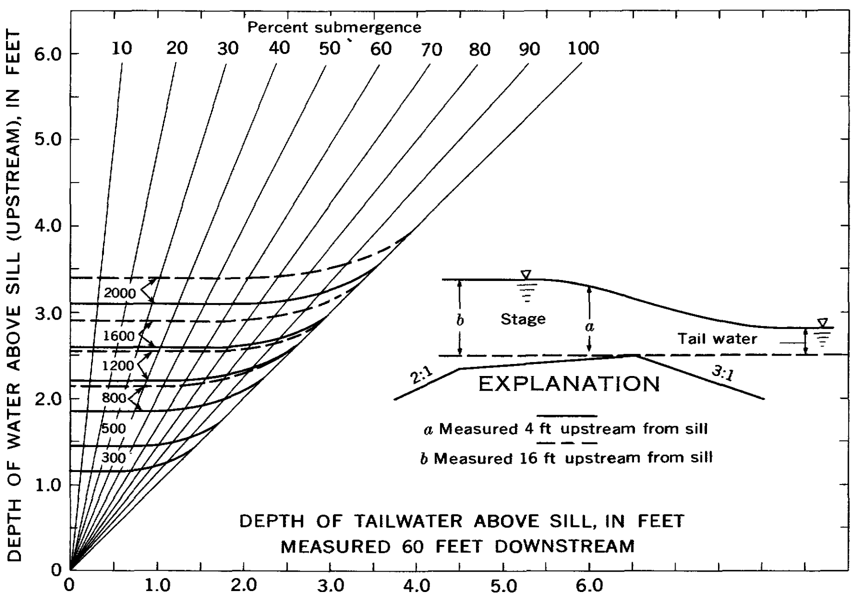


FIGURE 42.—Submergence curves for control K.

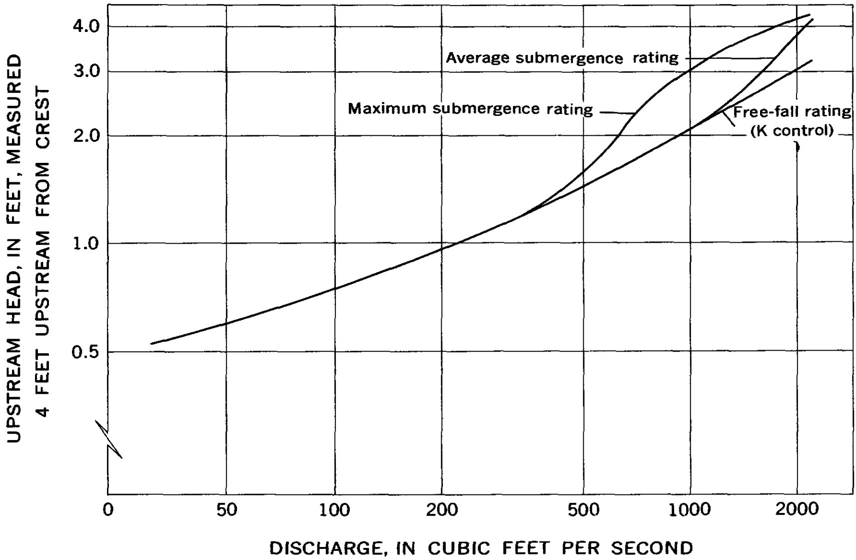


FIGURE 43.—Comparison of free fall—average submergence and maximum submergence ratings for a control crest 4.5 feet below bankfull stage. (Based on field and model data.)

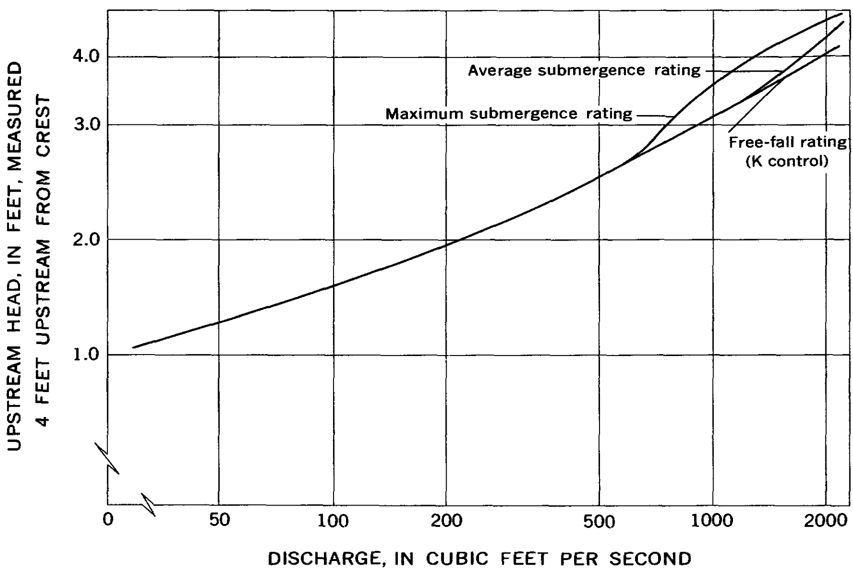


FIGURE 44.—Comparison of free fall—average submergence and maximum submergence ratings for a control crest 4.0 feet below bankfull stage. (Based on field and model data.)

resulting from variable submergence. Figure 43 was constructed under the assumption that the crest was 4.5 feet below bankful stage. Figure 44 indicates the improvement in the rating, if the control-structure height is raised 0.5 foot. The curves show the maximum scatter that could be anticipated as a result of variable submergence. A comparison of the flow-duration curve (fig. 29) and figure 43 indicates that free-fall conditions could be expected 50 percent of the time under maximum submergence conditions and 87 percent of the time under average submergence conditions.

DISCHARGE MEASUREMENTS ON THE CONTROL STRUCTURE

Vertical-velocity curves for a cross section 9 feet upstream from the control-structure crest are shown in figures 45 and 46. Slight velocity irregularities are indicated directly downstream from the baffles. However, reasonably consistent mean velocities could be obtained by using the three-point method in the baffle area (Corbett and others, 1945). Outside the baffle area or from 8 feet either side of the crest to the banks, indications are that 0.6-foot depth readings would yield a velocity as close or closer to the true mean than 0.2-foot and 0.8-foot depth readings. Small horizontal angles may exist downstream from the baffles, although the model study indicates that the angles are negligible.

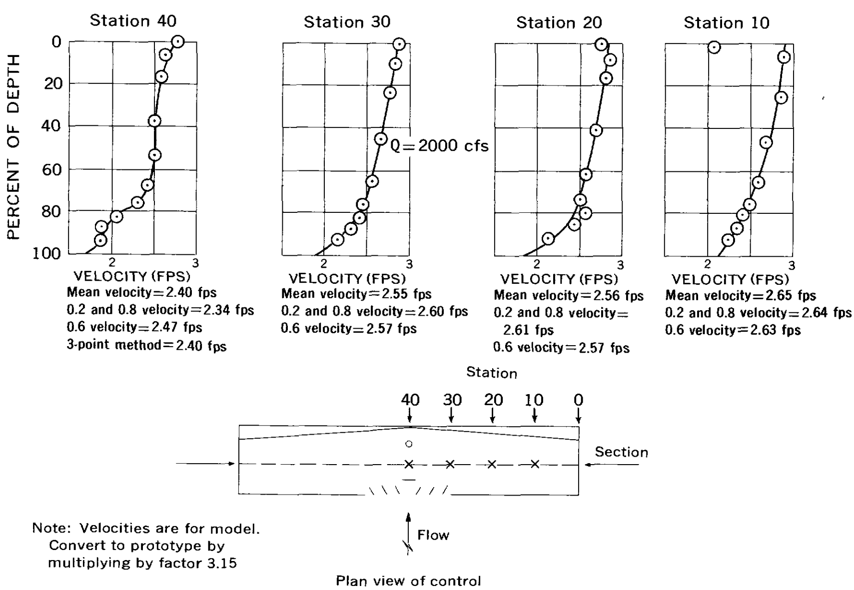


FIGURE 45.—Vertical-velocity curves based on data taken 9 feet upstream from the crest. (Free-fall conditions.)

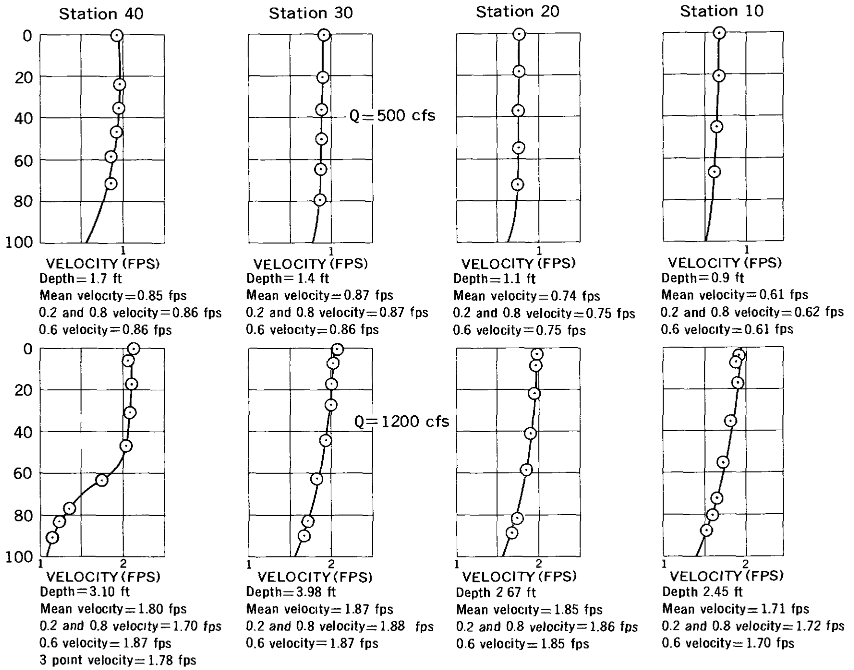


FIGURE 46.—Vertical-velocity curves based on data taken 9 feet upstream from control crest. (Free-fall conditions.)

The vertical-velocity curves are actually plotted to model velocities. By multiplying by the factor 3.15 to convert to prototype velocities, it is evident that velocities exceeding 8 fps (feet per second) are possible at 2,000 cfs under free-fall conditions. Actually, the control structure should be submerged at the higher flows. If so, it is unlikely that the velocities would exceed 6 fps. Because of the longitudinal and transverse crest slope, care must be taken in sounding and positioning the meter in order to ensure that the true depth and velocity are measured. This procedure may require heavier sounding weights than are normally used.

SEDIMENT SAMPLING FROM THE CONTROL STRUCTURE

A curb, 0.2 foot high, installed at a position 0.5 foot vertically below the crest on the downstream apron (fig. 10) provides a means of taking total sediment-load samples. The design and location of the curb were determined by a study made in conjunction with the energy dissipator. The horizontal fillet upstream from the curb eliminated excessive water-surface disturbance on the apron and improved the vertical distribution of the sediment. The curb, in addition to improving the sediment distribution, provided a satisfactory means of sampling the total flow depth by allowing the sample nozzle to touch the apron floor. The curb was positioned 0.5 foot below the crest to eliminate any possibility of affecting the depth-discharge relation. If it had been positioned any lower, there would have been a possibility of the sand bed covering the sampling point under adverse conditions. Samples could be obtained from the curb under low-flow conditions by wading on the crest of the control structure and reaching downstream to sample the flow. Naturally, care would have to be taken when sampling by this method so as not to disturb the flow upstream from the sampler. Sampling under high-flow conditions would have to be accomplished by using a guide to position the sampler. It is suggested that a channel iron be bolted to the apron, and the area between the apron and the upstream leg be filled with cement. The area between the upstream and downstream legs of the channel should be left open to serve as guide-rod supports. Another possibility would be to use the angle between the apron and the horizontal fillet as guide-rod supports.

A pump sampler developed by the Federal Inter-Agency Sedimentation Project at the St. Anthony Falls Hydraulic Laboratory (H. H. Stevens, Jr., oral commun.) could be used to measure the sediment concentration. Accomodations could be provided for the piping leading from the pump sampler to be mounted either in the curb fillet or downstream from the lower leg of the channel iron. If a pump sampler is used, two nozzles should be installed: one nozzle downstream from the low point in the control and the other nozzle downstream from the lateral, one-third position. The clearing effect of the baffle in the center of the control should be considered when obtaining pump samples. By proper calibration of the two nozzles, accurate total sediment-load samples could be obtained.

SUGGESTIONS FOR CONSTRUCTION AND OPERATION

A control structure built of loosely grouted rock to a depth 10 feet below bankfull stage or 1.5 feet below low bed elevation should provide a stable structure. Cutoff walls of sheet piling at or near the upstream and downstream toes of the weir would add to the stability. A 3- to 5-inch concrete cap should be placed at the top of the structure. A smooth crest surface would decrease both the amount of backwater and the possiblity that large sand deposits might accumulate, as indicated in baffle studies. Possible exposure of the bare rocks on the downstream apron may be useful for additional dissipation of energy.

David Hubbell, associated with the Dunning flume (Benedict and others, 1953), indicates that baffles constructed of sheet steel welded to angle irons would be superior to the mounting used for the Dunning installation. Bolts could be set in the proper position on the control apron, and the welded angle iron baffles could be installed or removed as needed.

Installation of a simple tail-water gage at a point 60 feet downstream from the control crest (this distance provided representative tail-water readings in the model studies) would provide submergence information. A small well and shelter containing a continuous recorder would be adequate. The low-water record at this tail-water gage would be unnecessary because submergence would occur only at the higher flows.

EXPERIMENTAL PROCEDURES AND EQUIPMENT

The model study was made in three phases. First, two-dimensional models were tested in the 2- by 60-foot flume to determine the most suitable design. This design was then checked and modified in a three-dimensional model in the 8- by 150-foot flume. The third phase was the development of an energy dissipator to decrease downstream scour.

In order to simulate the different bed forms that occur in the prototype in the 2-foot flume, the slope of the flume was varied. At a slope of 1.0 percent, a plane bed existed for all model discharges. At a slope of 0.7 percent, a plane bed existed for discharges larger than about 1,000 cfs, and a dune-bed form existed for discharges less than 1,000 cfs. At a slope of 0.3 percent, a dune-bed form existed for all discharges. Ratings were established for each control at the different flume slopes.

Slope was not changed in the 8-foot flume. Instead, the vertical position of the model control structure was changed to create the various bed conditions.

FLUMES

The 2-foot-wide flume used in the two-dimensional study was 60 feet long and 2.5 feet deep. Its walls were of plexiglass, and its bottom was of metal. The flume was equipped with a 12-inch centrifugal pump that recirculated sediment and flows up to 7.5 cfs. The discharge was regulated by a gate valve and was measured by a calibrated orifice in the discharge line. The slope of the flume could be changed automatically.

The 8-foot-wide flume used for the three-dimensional study was 150 feet long and 2 feet deep. The flume was equipped with a 12-inch and a 19-inch recirculating pump system having a discharge capacity of up to 21 cfs. The walls were of plexiglass and plywood, and the bottom was of plywood. The discharge was regulated by valves and measured by calibrated orifice meters in discharge lines from each pump. In this study either the 12-inch or the 19-inch recirculating pump was used, but they were never used together. The model control structure was installed 12 feet from the downstream end. To study the effect of submergence, an adjustable gate at the downstream end of the flume was used to create backwater. To determine the most effective energy dissipator, the limits of scour were determined; the control structure was moved 60 feet upstream to obtain a sufficient length of sand bed.

SEDIMENT

The sediment used in the 2-foot flume was quartz sand having a median fall diameter of 0.33 mm and a specific gravity of 2.65. The material in the 8-foot flume was quartz sand having a median fall diameter of 0.19 mm. and a specific gravity of 2.65. The sediment in both flumes was about 0.5 foot deep.

WEIR MODELS

Most of the models were made of ¾-inch plywood, although some were made of sheet metal and molding plaster. Various models were tried, but all had the basic dimensions found most favorable in the Del Rio studies (Karaki, 1961). All models had a downstream apron slope of 3 : 1 and an upstream apron slope of 2 : 1.

MODEL SCALES

A scale of 1 : 8 was chosen for the two-dimensional studies. This scale was chosen for the 8-foot flume to amplify the detail of the basic structure. This scale related only to the structure and to the fluid flow and not to the sand grain size or dune heights. The following relationships apply:

$$L_r = \frac{L_p}{L_m} = 8$$

$$q_r = \frac{q_p}{q_m} = (L_r)^{3/2} = 22.6$$

$$Q_p = 22.6 q_m \times 70$$

$$V_r = \frac{V_p}{V_m} = (L_r)^{1/2} = 2.83$$

A scale of 1 : 10 was used for the 8-foot flume. This ratio was convenient because the average prototype channel width is 80 feet; hence, 0.1 on the model represents 1 foot on the prototype. The following relationships apply to the 8-foot flume models:

$$L_r=\frac{L_p}{L_m}=10$$

$$q_r=\frac{q_p}{q_m}=(L_r)^{3/2}=31.6$$

$$Q_p=31.6q_m\times 80$$

$$V_r=\frac{V_p}{V_m}=(L_r)^{1/2}=3.16$$

EXPERIMENTAL RESULTS AND DISCUSSIONS

WEIR A

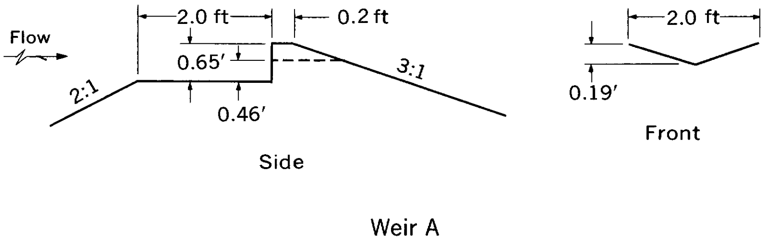
Weir A, figure 47, was a modification of Karaki's (1961) type G control. A transverse slope was incorporated into the downstream control lip to form a V-notch, whose purpose was to increase the accuracy of results and to confine low flows. Adverse waves occurred, however, just upstream from the lip at flows of more than 900 cfs. After a short period, the sand moved in and covered the apron upstream from the control lip. This encroachment eliminated this control structure from consideration.

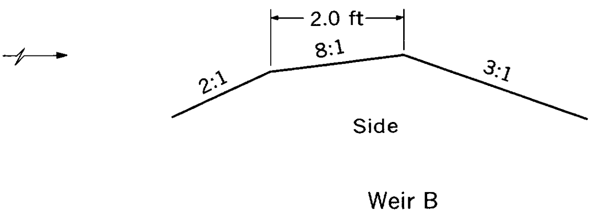
WEIR B

Weir B used a longitudinally sloping crest similar to Karaki's (1961) type F control (fig. 47). During tests of high flows the crest stayed clear, but at flows of less than 800 cfs fingers of sand encroached to within 2 feet of the crest.

WEIRS C, D, E, AND F

Weirs C, D, E, and F (fig. 47) were tested using the same basic shape as weir B, but their longitudinal crest slope was 18:1. Also, various forms of transverse sloping crests were superimposed on the basic 18:1 longitudinal slope to concentrate the flow, to improve the rating, and to increase movement of sand over the control. Weirs C, D, and E represented varying degrees of convergence. The sand was swept across the apron for all flows of 500 cfs and greater. At flows of less than 500 cfs, the sand encroached to within 1 foot of the crest and was concentrated in the middle of the apron. Convergence of the flow to the center was noticeable on all three weirs. Adverse waves were also noted. Weir F was of the same design as weir E, except that baffles were mounted on the upstream edge.





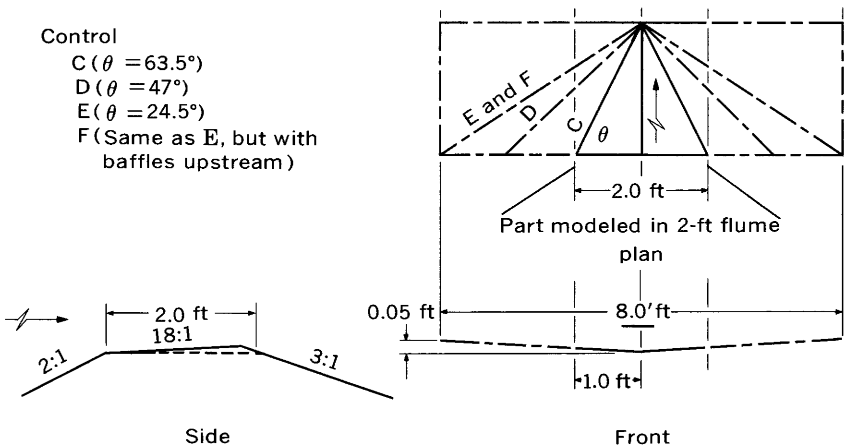


FIGURE 47.—Design of model weirs A, B, C, D, E, and F.

Three rows of baffles having spacing and heights similar to the turbulence flume on the Middle Loup River were used (Benedict and others, 1953; see also "Baffle studies", this report). The three rows of baffles were not effective in keeping the weir clear of sand, and an irregular water surface resulted. Because these control structures had some undesirable characteristics, further modification to obtain a more suitable design seemed desirable. Stage-discharge curves are shown in figure 48.

WEIR G

Weir G used a V-shaped apron having a longitudinal crest slope of 18:1 and transverse crest slope of 35:1 (fig. 49). Water-surface conditions were relatively smooth at all flows. At flows below 500 cfs there was a tendency for the sand to collect in the low part of the control. Deflecting baffles were used upstream, and by proper positioning the sand was eliminated.

WEIR H

Because of the success of weir G, a new basic control structure, weir H, was modeled for further and more detailed study. This control had a longitudinal crest slope of 16:1 but no transverse slope to converge the flow (fig. 49). Rating for weir H is shown in figure 50.

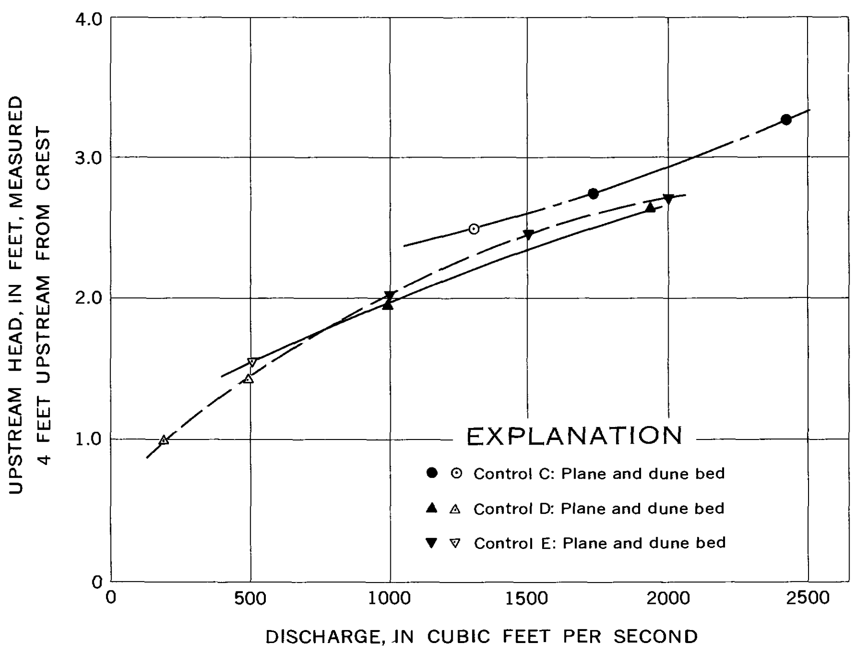


FIGURE 48.—Stage-discharge relations for controls C, D, and E.

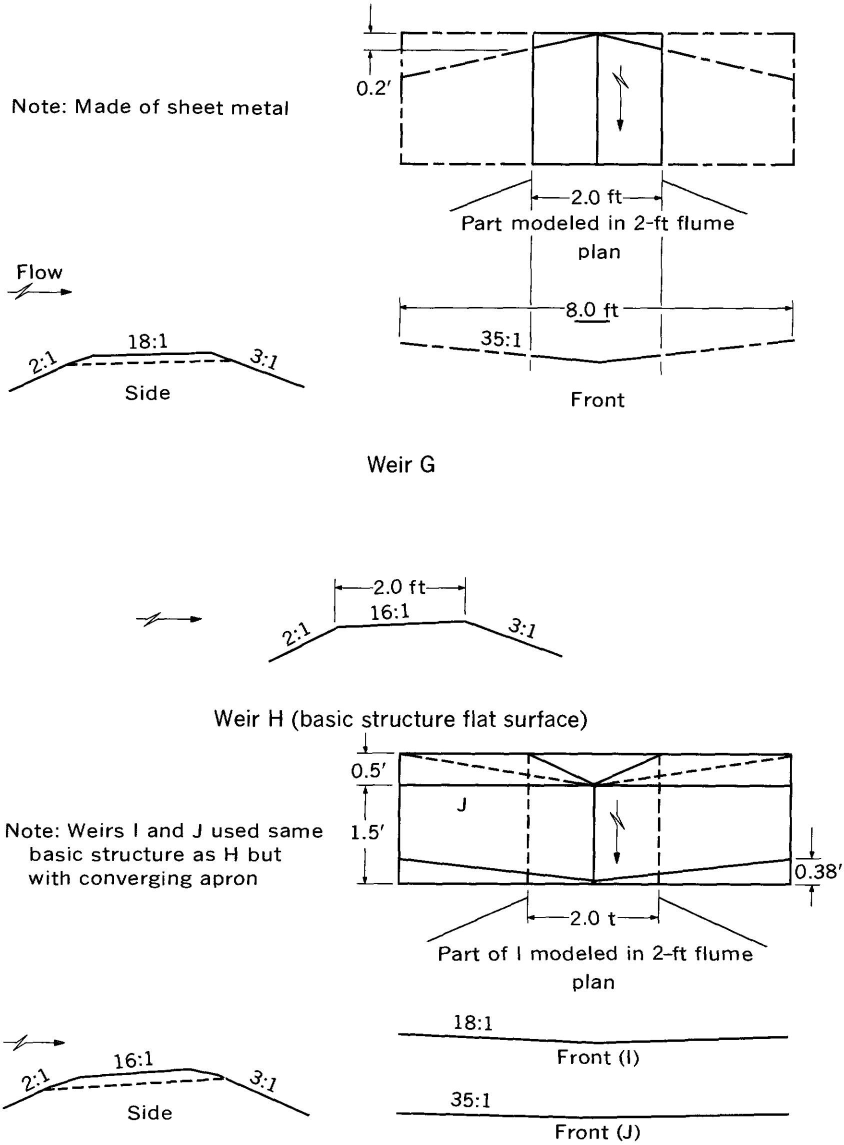


FIGURE 49.—Design of model weirs G, H, I, and J.

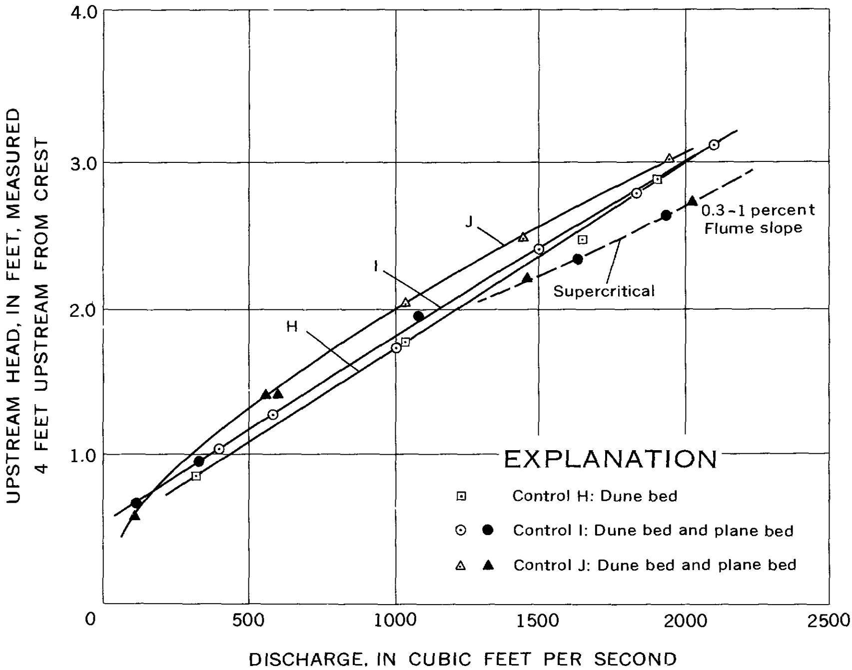


FIGURE 50.—Stage-discharge relations for controls H, I, and J.

WEIR I

Weir I was a modification of weir H, to which a transverse crest slope was added. The longitudinal crest slope was 16:1, and the transverse slope was 18:1 (fig. 49). This model control structure gave a smooth and even rating (fig. 23), except that at the 1-percent slope there was a shift in critical flow point between 1,000 and 1,500 cfs. This shift probably resulted from the modeling technique used to obtain a plane bed, and so will not occur in the prototype. Sand encroached on the low part of the V-notch to within 2 feet of the crest. A set of deflecting baffles, installed at the upstream edge of the crest, cleared off the low part of the crest. The transverse crest slope resulted in an undesirable convergence of the flow.

WEIR J

Weir J was of the same design as weir I, except that the transverse crest slope was changed to 35:1 (fig. 49). The rating for this control had the same shape as the rating for control I at the 1 percent slope (fig. 50). Baffle arrangements 2 and 9 (fig. 53) were effective in keeping the crest free of sand.

Because of the favorable results obtained in the two-dimensional model study, control J was selected for study in the 8-foot flume. In the 8-foot flume the longitudinal crest slope was 16:1, the transverse crest slope was 40:1, the approach apron slope was 2:1, the downstream apron slope was 3:1, and there was variable slope in the transition from the approach apron to the crest of the weir. A comparison of the basic control structure (control H) and the control J as modeled in the 8-foot flume is given in figure 51. The figure also includes a comparison of control J as modeled in the 2 and 8-foot flumes. Obviously the two-dimensional model study did not take into account the converging effect that the transverse slope had on the flow.

A series of 40 baffles was installed at the upstream edge of the transition from the approach apron to the crest. The transition slope on the upstream end of the apron rendered the outside baffles ineffective. Also, small bits of debris collected on the closely spaced baffles.

WEIR K

Weir K was a modification of weir J and was designed to eliminate unfavorable water-surface conditions and to simplify the shape for easier construction. This design was tested and recommended for installation at the proposed site. (See "Proposed control structure.")

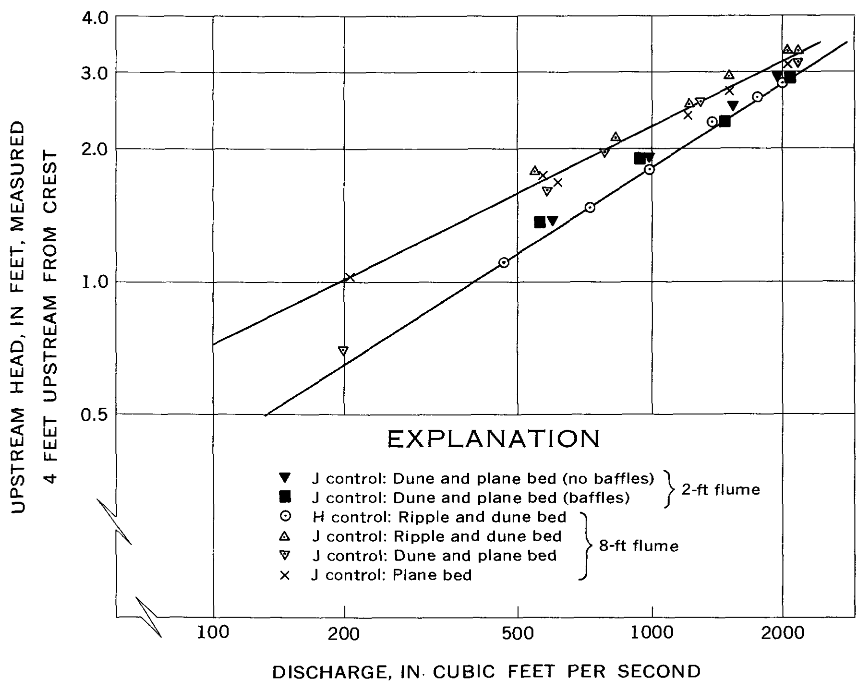


FIGURE 51.—Comparison of stage-discharge relations for basic controls H and J, modeled in 2- and 8-foot flumes.

A comparison of the ratings for controls J and K modeled in the 8-foot flume is given in figure 52.

BAFFLE STUDY

A study was made to determine the effectiveness of baffles located along the upstream edge of a control structure in keeping the structure clear of sand. Position 9 (fig. 53) more than any other baffle arrangement modeled kept the crest clear of sand, brought about less water-surface disturbance, and lessened the occurrence of horizontal angles on the crest. The baffles in the upstream row were of the same dimensions as those used in the Dunning flume; that is, 1.0 foot high and 2.0 feet wide. The downstream baffle was 0.5 foot high and 2.0 feet wide. The baffles brought about less water-surface disturbance and were still effective when the top of the upstream row of baffles was set 0.1 foot (prototype distance) below the level of the control crest. However, if the baffles were set at too great a distance below crest elevation they were less effective. The baffle study is summarized in figure 53.

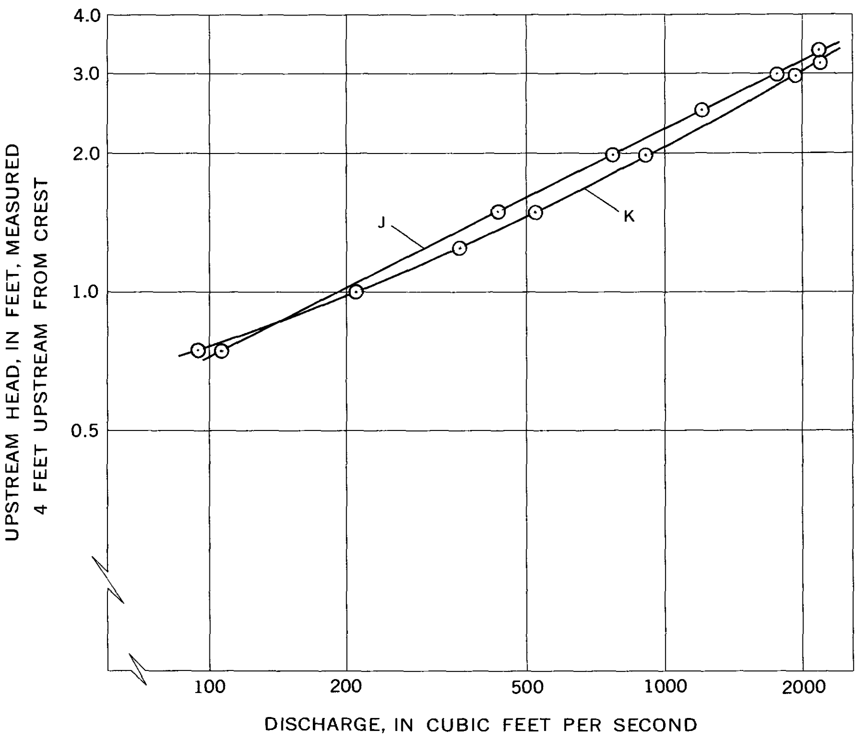


FIGURE 52.—Comparison of stage-discharge relations for controls J and K, modeled in 2- and 8-foot flumes.

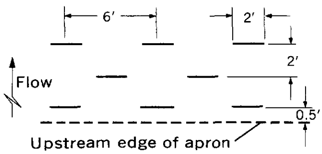
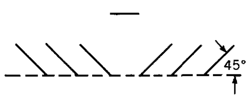
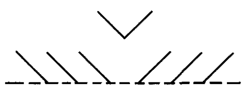
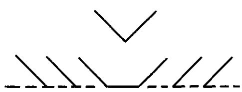
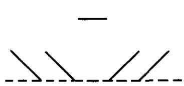
Baffle positioning	Water surface	Clearing effectiveness
 Position 1	Extremely rough for all flows	Not too effective; slows down velocity so that sand will deposit on apron
 Position 2	*HF: A little choppy **LF: Pretty choppy	Very good, but would make angular flow for measuring
 Position 3	HF: Fairly smooth LF: Still a little choppy, but smoother than in position I	Clears sand out at high flows (1000 and up). "V" baffle not as effective at low flow
 Position 4	HF: High hump at upstream baffles LF: Creates bad wave in front of front baffles	Not as effective at high or low flow because it seems to slow down velocity and allow sand to deposit
 Position 5	Same as in position 2	Middle 6 to 8 feet of apron is kept clean
*HF = high flow **LF = low flow		

FIGURE 53.—Baffle model study.

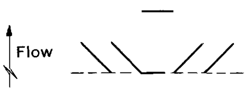
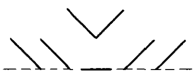
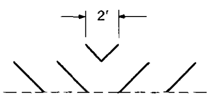
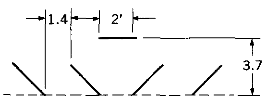
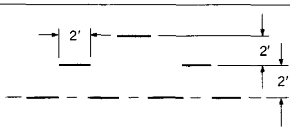
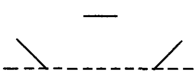
Baffle positioning	Water surface	Clearing effectiveness
 Position 6	Same as position 3	Same as position 3
 Position 7	Same as position 5	Same as position 5
 Position 8	Same as position 2, except does not cause as much turbulence	Does not spread sand out quite as much as position 2
 Position 9	Same as position 8 except slightly more turbulence but less than position 2	Less angular flow than position 2. Does not spread sand out quite as much as position 2 More effective than position 8
 Position 10	Very rough	Same as position 1
 Position 11	Smoother than positions 2 or 9	Fair. Not quite as good as positions 2 or 9

FIGURE 53.—Continued.

ENERGY DISSIPATOR STUDY

A model study was conducted to determine the most effective energy dissipator for controlling and minimizing downstream scour. It must be emphasized that model studies of scour can give only qualitative results. For example, the model study indicates that a reverse roller keeps sand piled against the downstream toe of the structure. Presumably, the design that performs best in the laboratory will perform best in the field. For this reason, all the results of the test are reported for the model except the discharge, which is reported for the prototype.

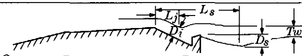
The study involved the use of sills located at various positions on the downstream apron whose slope was 3:1. Also, baffles upstream from the sills were tried in an attempt to determine if they could improve the jump action.

Two tail-water conditions were considered in conducting the study. One, termed "theoretical minimum tail-water elevation" was the tail-water elevation that resulted from adding the depth of flow (fig. 33) for the discharge being modeled to the low bed elevation. The other condition, termed "low tail-water elevation," was taken from the low gage-height curve (fig. 41). Low bed elevation from a study of the field data was 4 feet below the low point on the control crest. These tail-water elevations, in feet above low bed elevation, are given in table 2.

TABLE 2.—*Tail-water depths, in feet, above low bed elevation*

Q_p (cfs) (Prototype)	Minimum tail-water elevation		Low tail-water elevation	
	Prototype	Model	Prototype	Model
500	1.64	0.16	2.7	0.27
1,000	2.47	.25	3.7	.37
1,500	3.29	.33	5.2	.52
2,000	4.09	.41	6.6	.66

The study of the various sill heights and locations and the results of the study are summarized in figure 54. Models 1 and 2 had sills that were located too high above low bed elevation. This situation resulted in excessive scour downstream, as compared with scour in other models. The top of the sill in model 3 was located at low bed elevation, and as indicated in figure 26, scour was lessened at 500 and 1,000 cfs but not at 1,500 and 2,000 cfs when tail-water elevation was at a minimum or lower. Under low tail-water conditions, however, scour was lessened. The purpose of model 4 was to decrease scour by extending the apron length without increasing the height of the sill. At minimum or lower tail-water elevation this model worked well at the two lower



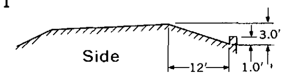
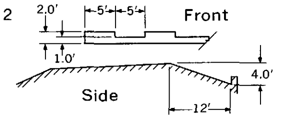
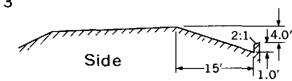
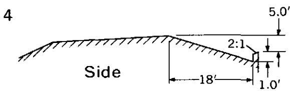
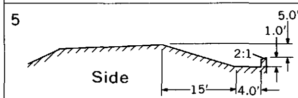
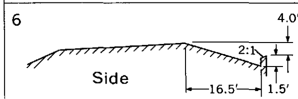
Model	Q_p	L_j	D_i	L_s	D_s	T_w	Submergence (percent)	Remarks: 1. Sand deposit against sill, 2. Extent of scour, 3. Jump conditions } Referred to by number
1 	730 1,400			2.8 2.8	0.383 .423	0.244 .344	0 14.9	
2 	1,200 1,500 2,000			3.8 4.0	0.428 Floor Floor	0.300 .293	0 0	1. Good at 1,200 cfs. Poor at 2,000 cfs 2. Severe at 2,000 cfs 3. Jump contained at 1,200 cfs Passed over sill at 2,000 cfs
3 	500 1,000 1,500 1,500 2,000 2,000	1.10 1.0 1.2 .40 .9 .40	0.04 .06 .13 .19 .18 .23	2.7 3.2 2.75 3.2 4.1 4.2	0.220 .375 Floor .287 Floor .357	0.164 .212 .329 .52 .409 .60	0 0 0 41 62	1. Good sand deposit 2. Scour to floor. Extends all across flume. Scour not excessive at high flows with submergence 3. Jump contained, all flows
4 	500 1,000 1,500 2,000	0.9 .9 1.2 1.2	0.05 .09 .12 .15	2.6 3.2 4.5	0.1 .20 Floor Floor	0.169 .370 .400 .455	0 0 0	1. In all flows- good 2. Scour severe except at 500 cfs 3. Jump contained, all flows
5 	500 2,000	0.9 1.0	0.05 .15	2.3 4.9	0.1 Floor	0.211 .409	0	1. Good at both flows 2. Good at 500 cfs. Severe at 2,000 cfs 3. Hydraulic jump well contained
6 	1,500 1,500 2,000 2,000 1,000	0.7 .45 .9 .30 .75	0.14 .18 .15 .25 .06	3.15 3.85 3.05 3.85 2.65	0.271 .362 Floor .307 .116	0.329 .52 .409 .66 .37	0 40.6 10.75 84.5 0	1. Good all flows 2. Scour not excessive except at 2,000 cfs with 10% submergence 3. Jump contained all flows

FIGURE 54.—Energy dissipator study.

flows, but excessive scour occurred at the higher flows. Model 5 was an attempt to decrease the scour by using a horizontal basin. In this model also excessive scour occurred downstream from the sill. Model 6 was a slight modification of model 3, in which the sill height was increased 0.5 foot, but in which the sill crest was kept at low bed elevation by extending the apron 1.5 foot. This energy dissipator performed very well under minimum tail-water conditions, except at 2,000 cfs. However, when the tail-water elevation was low, the scour was not excessive. It is questionable if tail-water elevation would ever be at the minimum theoretical elevation.

So that the performance of model 6 might be improved, baffles—0.03 foot high and 0.2 foot long (model distance), spaced the same as those in position 1 (fig. 53)—were placed on the face of the apron. Also placed were three rows of baffles, in which the crest of the lowest row was at the same elevation as the sill crest. The upper row was then removed so that only two rows were on the apron. The upper row was replaced and the lowest row was moved above the other two rows. No matter what the position of the baffles, they resulted in a decrease in efficiency because they caused the flow to leave the face of the apron and to override the sill. Preliminary investigation in which larger baffles were used indicated the same result.

The test data indicate that model 6 will protect the control structure and that excessive scour will not occur downstream unless tail-water elevation at 2,000 cfs becomes much lower than indicated by gage-height records. Then, a moderate amount of riprap may have to be laid down for protection. Model 3, although adequate, was not as effective as model 6.

LITERATURE CITED

Benedict, P. C., Albertson, M. L., and Matejka, P. Q., 1953, Total sediment load measured in turbulence flume: Am. Soc. Civil Engineers Proc., Separate 230.

Corbett, D. M., and others, 1945, Stream-gaging procedure: U.S. Geol. Survey Water-Supply Paper 888.

Karaki, S. S., 1961, Stream gaging control structure in the Rio Grande: Colorado State Univ. Research Found. Rept. CER 61–SSK17.

River Hydraulics

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1369

This water-supply paper was published as separate chapters A–E

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

Thomas B. Nolan, *Director*

W-1369

CONTENTS

[The letters in parentheses designate separately published chapters]