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Supercritical Flow Flumes for Measuring Sediment-Laden Flow

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Supercritical Flow Flumes for Measuring Sediment-Laden Flow

R. E. Smith, D. L. Chery, Jr., K. G. Renard,
and W. R. Gwinn

Acknowledgments

The authors wish to express their appreciation to the many people who helped develop these measuring flumes, only a few of whom can be mentioned here.

Special thanks to D. A. Woolhiser, Fort Collins, Colo., whose cooperation made possible the 1:5 outdoor hydraulic model tests leading to development of the control dikes and a confident rating of the flume's lower flows. Howard Larson with his staff of technicians at Tombstone, Ariz., for many years provided the much needed help and understanding in building and evaluating these flumes. Gary Smillie carried on calibration measurements at the Walnut Gulch watershed after the principal investigators left.

We thank D. G. DeCoursey, Oxford, Miss., F. W. Blaisdell, St. Anthony Falls, Minn., and J. W. Ruff, Fort Collins, Colo., for their many helpful suggestions in the preparation of this publication.

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Abstract

Smith, R. E., D. L. Chery, Jr., K. G. Renard, and W. R. Gwinn. 1981. Supercritical flow flumes for measuring sediment-laden flow. U.S. Department of Agriculture Technical Bulletin No. 1655, 72 p., illus.

A general type of supercritical flow flume has been developed over many years of experience and testing in discharge measurements at the Walnut Gulch experimental watershed, Tombstone, Ariz. The design and experience with the original type flume, called the Walnut Gulch flume, is discussed and its features and application difficulties are described. Methods have been developed to analyze flows that exhibited lateral asymmetry in cross sectional profile, and porous dikes have been developed to considerably reduce asymmetry in the alluvial approach section to these flumes. Rating relations have been developed by both experimental and theoretical means. The experience with the Walnut Gulch flumes has led to an improved design of supercritical flume, called the Santa Rita flume. The Santa Rita flume design is presented in several sizes, along with a discussion of design requirements for stilling well intakes to minimize sediment inundation, record lag interpretation, and construction methods.

Keywords: Open channel, flow, flume, stilling well, sediment transport, measurement, supercritical, alluvial, sonar, pressure transducer, instrumentation, design, intake.

Contents

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List of Symbols

Symbol	Description	Units
a	Exponent for R in velocity relation	
A	Cross-section area of flow	L^2
A	Length of flume entrance	L
A_o	Orifice area	L^2
A_w	Surface area at recorded depth	L^2
b	Parameter in a discharge rating relation	
C	Friction coefficient, or design parameter for entrance shape of Santa Rita flume	
C_D	Discharge coefficient	
C_S	Orifice coefficient	
C_W	Dimensioned weir or flume coefficient	
D	Hydraulic depth = A/T	L
Fr	Froude number	
g	Gravitational acceleration	L/T^2
h	Measured flow depth	L
h_e	Eddy loss head	L
h_f	Flow head in the flume	L
h_n	Notch depth	L
h_o	Head at the orifice exit	L
h_w	Stilling well depth	L
i	Distance increment number in solution in finite difference expression	
j	Subscript of incremental flow velocity and cross section area	
K_e	Eddy loss coefficient	
L	Scale ratio, or length from beginning of throat section of a flume to the measuring section	
L_W	Width of weir	L
m	Subscript indicating model	
n	Manning roughness coefficient	
p	Subscript indicating prototype	
q_o	Flow through an orifice	L^3/T
Q	Flow discharge	L^3/T
R	Hydraulic radius	L
r	Subscript indicating rougher case	
r_c	Scaling ratio for roughness	
s	Subscript indicating smoother case	
S_c	Natural channel slope	L
S_f	Friction slope of flume	L
S_o	Bottom slope of flume	L
t	Width of water surface in flume at measuring point when depth is h	L
t	Time	T
T	Top width of free surface of a channel flow	L
T	Length of throat of flume	L
V	Velocity	L/T
$\overline{V}$	Average velocity	L/T
V_o	Velocity through the orifice	L/T
w	Dimensionless parameter representing relative value of Fr	
W	Top width of a flume throat	L
WF	Projected width of floor section of a flume	L
x	Distance along flume	L
X	Horizontal coordinate positive in upstream direction	L
x_p	Distance from critical section to the measuring section	L
y	Flow depth	L
y_m	Flow depth at the measuring section of a supercritical flume	L
Y	Vertical coordinate	L
z	Horizontal coordinate normal to and measured from the center of the flume	L
ZF	Horizontal dimension per one unit vertical dimension in the side slope of a flume floor	
ZW	Horizontal dimension per one unit vertical dimension in the side slope of a flume wall	
α	Open channel velocity distribution energy coefficient	
$\triangle h$	Head difference across the orifice	L
$\triangle t$	Time step	T
σ	Measurement sensitivity	T/L^2

Supercritical Flow Flumes for Measuring Sediment-Laden Flow

By R. E. Smith, D. L. Chery, Jr., K. G. Renard, and W. R. Gwinn[1]

Introduction

This publication describes a general type of flume particularly suited for measuring discharge in streams with high velocities and high sediment concentrations. This kind of flow is common in many areas where runoff results from high-intensity rainstorms and the channels have a steep gradient (0.5 to 2.0 percent) and alluvial bed. This situation is common in the American West and Southwest, northern Mexico, certain areas of Australia, North Africa, the Middle East, and arid areas of Asia. Often, the regional ground water level lies considerably below the channel surface and, consequently, there is no base flow.

The laboratory and field experience involved in developing and evaluating the original flume, called the Walnut Gulch flume, is described here. In addition, we present an improved supercritical flume design that grew out of this experience and several typical designs covering a wide range of flows.

Hydraulic Classification of Flumes

To measure water discharge in an open channel, we apply our knowledge of the distribution of the energy of flowing water. For accurate measurements, we also need a location where hydraulic control exists. Hydraulic control occurs when a local flow condition exists such that the relation between discharge and depth is reasonably independent of changes in upstream or downstream conditions.

Flowing water has both potential and kinetic energy. When the total energy for a given discharge is minimum, critical flow is said to occur. Velocities greater than critical flow velocity, and therefore having a higher proportion of kinetic energy, define supercritical flow. Conversely, velocity lower than critical occurs in subcritical flow. Most discharge measuring structures in open channel flow depend on the fact that a contraction can cause subcritical flow to accelerate through critical flow. This constitutes a form of hydraulic control, and the known relation between kinetic and potential energy is then used to derive a presumed invariant relationship between flowing water depth and discharge. This relationship is the structure's rating.

Most open channel flow is subcritical, whether in canals or natural streams, with velocities well below critical, and flumes are usually designed to measure depth upstream of a contraction where critical flow is caused to occur.

High natural velocities and sediment concentrations in many locations prohibit the use of ordinary subcritical flow (often referred to as critical depth) flumes because these flumes require such low approach velocities that sediment is deposited. The deposited sediment causes a shift in the rating or a loss of hydraulic control.

The flumes described in this report have been developed to provide flow measurement for conditions of heavy sediment load where ordinary flumes do not perform satisfactorily. Such adverse conditions often coincide with situations of critical hydrologic interest. This may be the case when ephemeral flow represents a scarce water resource or when flash floods in otherwise dry channels are potentially damaging to local agriculture.

Hydrologic Properties of Ephemeral Alluvial Streams

The supercritical flumes discussed in this publication have application in a wide variety of hydrologic conditions, but measurement of ephemeral flows is a major one. Therefore, a summary of the peculiar hydraulic problems of this type of hydrology is presented. Ephemeral streamflow in alluvial channels usually originates in the uplands where slopes are relatively steep. Streamflow velocities are therefore typically high.

Closely related to the occasional flow and steepness of slope is a typically high sediment load. Were these streams to flow more or less continuously, erosion processes would quickly develop a meandering, mild slope stream with the finer material flushed from the basin. Ephemeral flows are characterized by an imbalance between sediment load and carrying capacity, and often carry a large volume of sediment. Under these circumstances, the sedimentation processes are almost never in equilibrium and are either eroding or depositing sediment at any point along the stream. Ephemeral flow implies rapidly changing discharge so that the interrelated hydraulic processes of flow, bed forms, and sediment concentration are truly dynamic.

In channel networks, other factors complicate these dynamics. The movement and spatial variability of runoff-producing storms, as well as topography, cause surface runoff to enter the channel network at different times at different places. The convergence of these time-displaced hydrographs determines the pattern of flow at any given point along the channel. The sediment load in the water entering an ephemeral stream at any point affects the amount of alluvial materials picked up from the channel bed. If the sediment load of water entering

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the channel is high, the channel may aggrade locally; however, if the sediment load is low, the channel can degrade extensively, depending on the armoring action of bed materials.

The effect of the local flow history on (a) the nature of the alluvial material at the beginning of any flow event and (b) the channel shape complicates the interrelation between flow and sediment. A larger, longer flow event will leave the channel in a different condition than would a series of smaller flows; it will leave different materials at the surface and will shape a different longitudinal and lateral channel configuration.

Problems of measuring the discharge in ephemeral streams are derived primarily from (a) the high velocities, (b) the sediment-carrying capacity of high velocities, and (c) temporal variations of the streambed shape and local flow direction, resulting from the ephemeral nature of the watershed.

Accurate flow measurement requires a flume that causes repeatable hydraulic conditions defining a unique predesigned relation between the flow depth and the discharge at some measuring point (or points). This means that the flume must exercise hydraulic control for a large range of upstream and downstream conditions. Most importantly, where there are heavy sediment loads, flumes cannot provide hydraulic control by reducing the velocity because the sediment load will likely deposit in the flume throat and hydraulic control will be lost. Flumes for these condtions must be designed with some understanding of the dynamic nature of the alluvial bed, the sediment load, and hydraulic parameters.

Failure to account for large sediment loads, for example, has caused severe measurement problems on very small watersheds in New Mexico and Arizona where weirs were installed to measure flow. Sediment quickly filled the upstream ponds and depositional bars covered some weirs, destroying the control and severely reducing their effectiveness as measuring structures. This condition is illustrated in figure 1. This is a small watershed measuring station near Safford, Ariz. A weir causes hydraulic control by reducing the upstream kinetic energy to a negligible value, creating a pond of tranquil flow. The head measurement at a point above the weir is then an indication of the total specific energy involved (that is, there is no appreciable velocity head). The severe deceleration of flowing water makes a weir a very effective sediment trap, and, therefore, inappropriate where sediment load is high.

Many watershed research locations in the United States have used broadcrested *V*-notch weirs (developed by the Soil Conservation Service) to measure runoff, and found that continuous maintenance is required to remove sediment deposited in the pond above the weir. Figure 1 illustrates the type of wide deposition bar typical of stable conditions at these weirs.

The broad-crested *V*-notch weir may be used in channels that do not form a true tranquil pond above the weir. Rating data for these weirs are presented in USDA Agriculture Handbook 224 (USDA *1979*),[2] including corrections for upstream velocity at the measuring point. This correction is valid for a limited range of velocities and assumes the weir notch elevation to be above the channel bottom elevation.

[2]The year in italic, when it follows the author's name, refers to Literature Cited, p. 39.

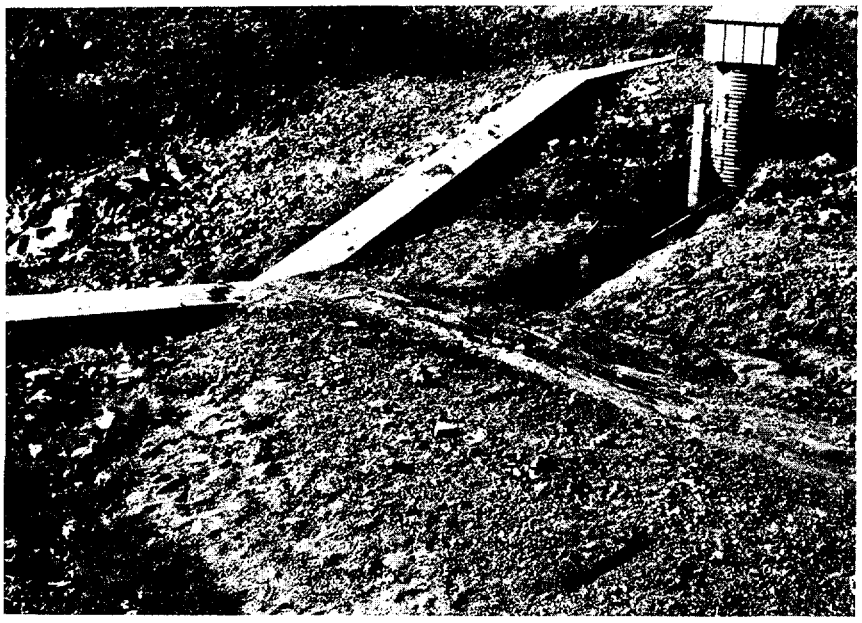


FIGURE 1.—V-notch concrete weir filled with bedload near Stafford, Ariz. BN-48648

Recent field studies at the Walnut Gulch watershed and laboratory experiments by Ruff et al. (*1977*) at the Hydraulics Laboratory at Colorado State University have quantified the effects of the deposition on the rating of the weir. Sometimes, the structure no longer acts as a weir but rather loses total control of the low to moderate flows.

An experiment that demonstrated this was performed in the 1.22-m (4-ft) wide tilting flume at the USDA Water Conservation Laboratory, Tempe, Ariz. The 3:1 weir at Walnut Gulch location 63.113 was hydraulically modeled at a 1:5 scale. The prototype upstream bed had aggraded to near preinstallation grade of some 2 to 3 percent, with aggraded material just upstream of the weir some 3 cm (0.10 ft) higher than the weir notch. At all but a limited range of flow, the weir acted more as a free overfall than as a hydraulic control. Figure 2 is a photograph of the model test after one experiment. Figure 3 is a photograph of the prototype weir. Model deposition patterns were similar to those observed in the field.

Work by Ruff et al. (*1977*) and field measurements at an experimental tandem-flume location (supercritical flume immediately below a sediment-filled weir) are shown in figures 4 and 5 to illustrate the effect of sediment deposition on weir ratings. The standard ratings used in the comparisons of figures 4 and 5 include the velocity correction in USDA Agriculture Handbook 224 (U.S. Department of Agriculture *1979*) tables, based on the flow cross section area at the measuring point. In each of these cases, upstream channel slope was sufficiently mild to retain weir control at higher water levels where the narrowing effect of the *V*-notch could slow the flow and exert control. This was not the case, however, for the weir in figure 3 (Walnut Gulch Weir No. 63.113) where the upstream grade was so steep and the channel so narrow that essentially all control was lost. Obviously, the weir is not a suitable measuring device for these conditions.

Another structure that has more potential than a weir for measuring sediment-laden flow in small channel applications is the venturi flume, designed for higher but still mild flow conditions. Although this flume has application where sediment sizes and concentrations are relatively low, when used in ephemeral streams of southeast Arizona where considerable sediment moves as bedload, it failed to pass the sediment carried by the flow (fig. 6). In this example, sediment was deposited through the flume, including the throat where supercritical flow was designed to occur. In one experiment, turbulence-generating vanes were added in the approach section walls, but this still failed to prevent bottom sedimentation in the flume.

Such field observation clearly indicates the need for measuring flumes that will maintain a velocity sufficient to transport the sediment entrained in the flow.

FIGURE 2.—Photograph looking downstream at 1:5 model of Weir 63.113 at Walnut Gulch , after large simulated flow event. The weir exercises no control for lower flows as a result of alluvial filling. BN-48649

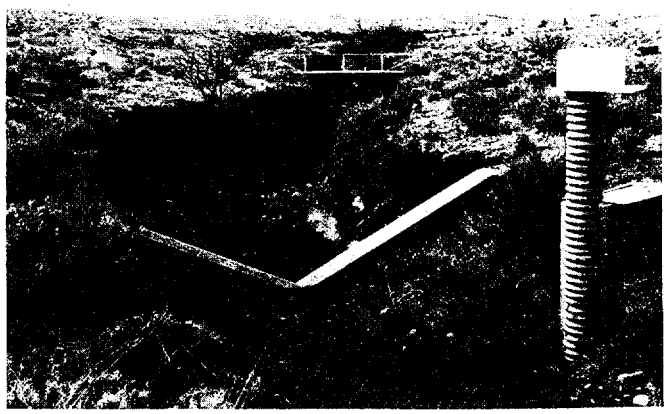


FIGURE 3.—Weir location 63.113 at Walnut Gulch, near Tombstone, Ariz. The channel bed above the weir has achieved a new aggraded stable condition. BN-48650

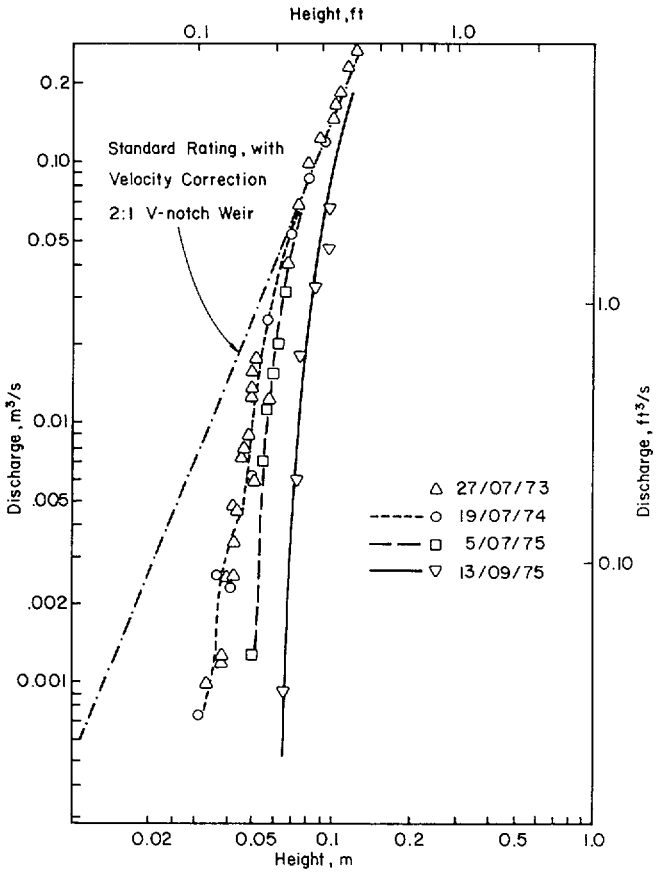


FIGURE 4.—Measured rating of an alluvial filled 2:1 V-notch weir, Walnut Gulch Location 63.102. Only at depths greater than 0.4 to 0.6 m (1.5 to 2 feet) does the weir exert control.

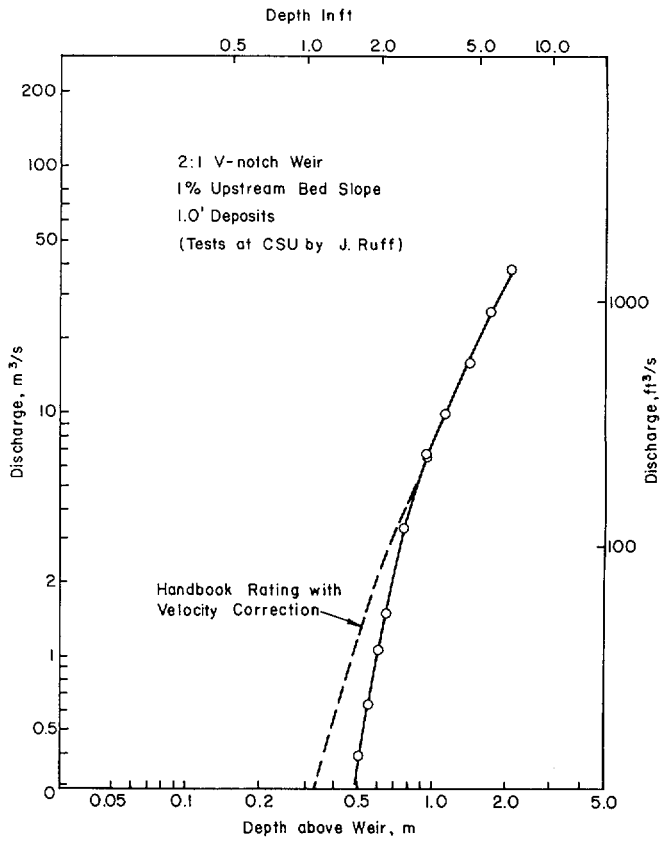


FIGURE 5.—Laboratory model simulation of the rating of a 2:1 V-notch weir with 0.3 m (1.0 foot) of upstream deposit depth (data from Ruff *et al.*, 1977).

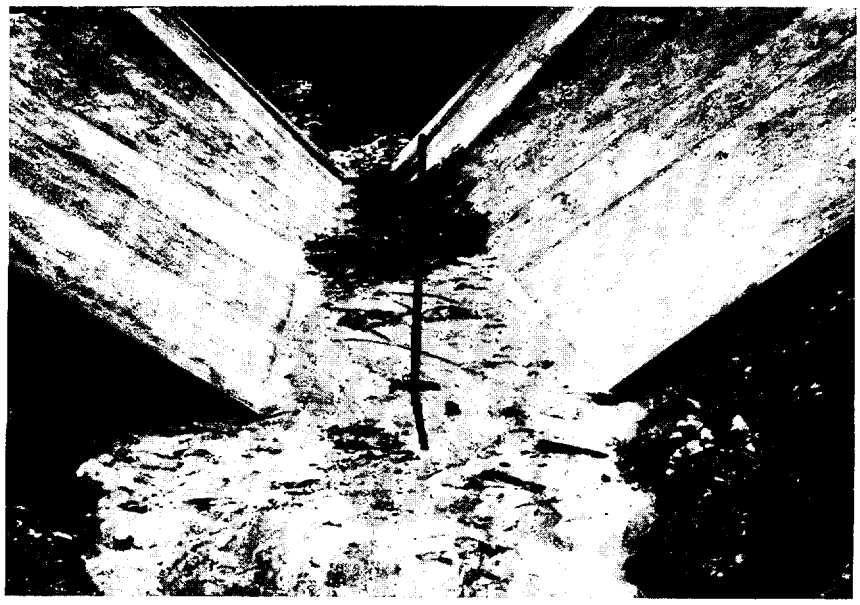


FIGURE 6.—Sediment deposited in a venturi flume after a summer storm flow at Walnut Gulch watershed, Arizona. BN-48651

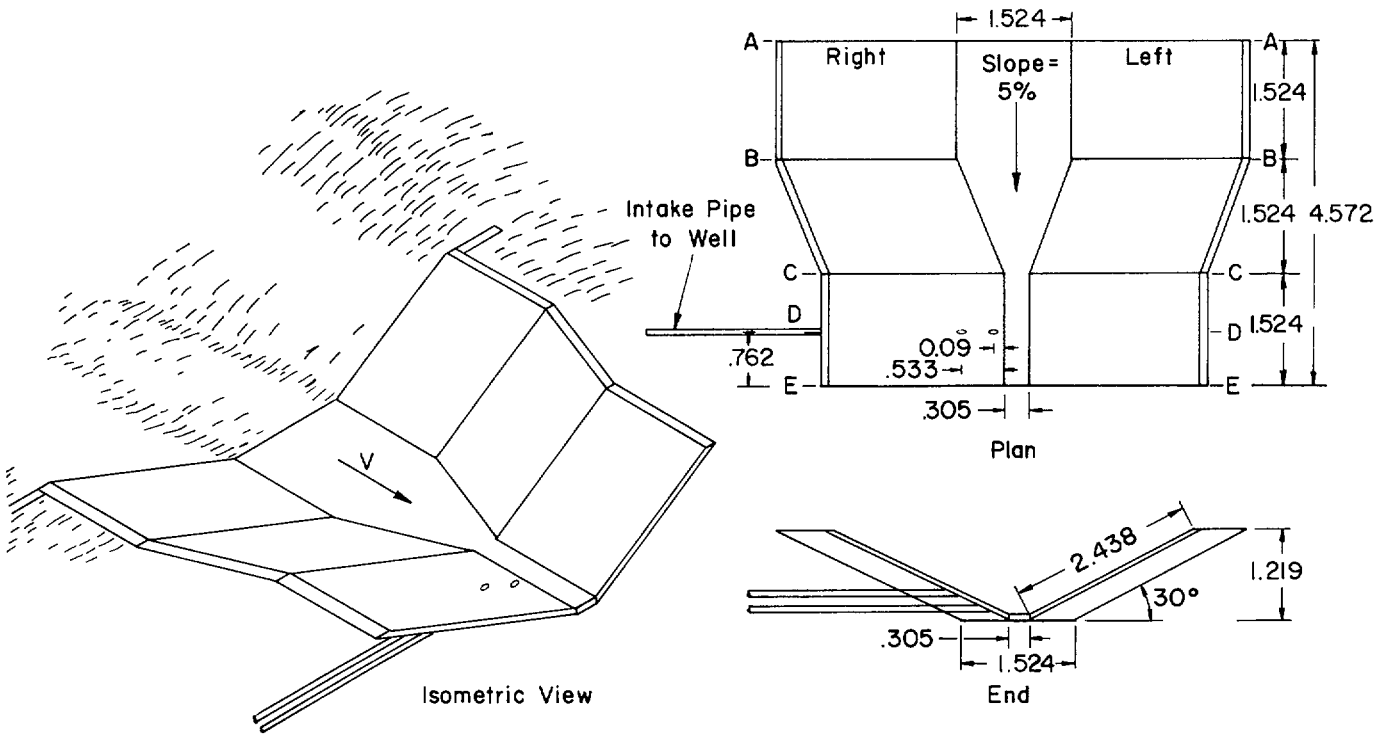
Background and History

Use of supercritical flow flumes for flow measurements in the field began at several places in the late 1950's. Between 1956 and 1961, Colorado State University developed a supercritical flume for use by the Rocky Mountain Forest and Range Experiment Station at Beaver Creek, Ariz. The work was performed by Chamberlain (*1957*) and Robinson (*1961*). The flume developed is trapezoidal in cross section and is similar to a venturi flume, with a straight approach section, a rectilinear transition region, and a narrow throat section. Figure 7 illustrates the flume geometry. Unlike venturi flumes, the Beaver Creek flume is sloped 5 percent longitudinally to induce supercritical flow. At lower flows, the flow is supercritical throughout, but flow in the approach section is subcritical at higher flows. These trapezoidal flumes were installed in the Beaver Creek watershed, and most are still in use.

Several supercritical measuring flumes in Switzerland, of individually varying design, are described by Ree (*1965*). These are all long throated, 15 to 17 m (49.2 to 55.8 ft), with a complex cross section to concentrate low flows but provide capacity for larger spring flows. Approach transitions are all quite short and slopes are relatively mild, 0.5 to 1.0 percent. The Swiss flumes were individually rated with current meters.

The supercritical flumes discussed in this report were first developed in conjunction with hydrologic studies on watersheds in southwest United States by the USDA Agricultural Research Service (ARS). Construction of flumes for flow measurement began in 1953 on the Walnut Gulch area near Tombstone, Ariz., and in 1954 on the upper Alamogordo Creek area near Santa Rosa, N. Mex.

In the first effort at flow measurement at the Walnut Gulch watershed, five critical flow measuring stations were constructed by July 1954. The first five flumes built at Walnut Gulch were simply smooth flow constrictions that contracted the flow sufficiently to cause critical flow at a smooth overfall, but created some backwater. They measured runoff from the outlet of the 149-km^2 (57.7-mi^2) study area and from four interior subwatersheds, varying in size from 2.3 to 114 km^2 (0.88 to 43.9 mi^2).



Note: Dimensions Shown In Meters, Original Design In Dimensions of Feet, 1.0 m = 3.28 ft

FIGURE 7.—Trapezoidal supercritical measuring flume for flow measurement on streams with steep slopes designed by Robinson (1961).

Figure 8 shows the structure at the Walnut Gulch outlet shortly after its completion. Later that summer, the structure failed as shown in figure 9. The failure occurred because it was (1) structurally inadequate to carry the weight of water involved, (2) hydrologically too small, and (3) hydraulically inadequate with resulting downstream scour undermining the concrete.

By the end of 1954, the only original structure left intact was the flume on the 2.3-km^2 (0.88-mi^2) watershed. It had been seriously overtopped, however, and was replaced in 1967. The flume at the 22.3-km^2 (8.61-mi^2) watershed, called subwatershed 5, has been extensively undermined and damaged below the critical section. A new supercritical flume was built downstream in 1966.

A structure similar to those described above at Walnut Gulch was built at the outlet of a 73.5-km^2 (67-mi^2) watershed at Alamogordo Creek. This structure remains intact today, although extensive repairs have been required to prevent the hydraulic jump at the lower edge of the flume from undermining the structure. Sheet piling and large boulders have been anchored below the flume to protect against undercutting.

FIGURE 8.—Critical flow flume originally installed for flow measurement at the Walnut Gulch Watershed outlet, 1954. BN-48652

FIGURE 9.—The first structure for Watershed 1 was seriously damaged by the first large flows of the first season of use, 1954. The sidewalls and floor were badly undermined and inundated as shown here, and were competely washed out by the end of the season. BN-48653

As a result of these early failures, a series of hydraulic model investigations began in 1957 at the ARS Stillwater Hydraulic Laboratory, Stillwater, Okla. From these tests evolved the measuring device known today as the Walnut Gulch supercritical flume (Gwinn *1964*), with the largest of 11 such structures on Walnut Gulch having a peak measuring capacity of over 623 m^3/s (22,000 ft^3/s) (fig. 10).

The design of this flume came from a study of earlier supercritical flumes, especially the San Dimas flume (Wilm et al. *1938*), which had a supercritical throat with vertical sides, and the trapezoidal flume of Robinson (*1961*), discussed above. It was felt necessary to (a) contract the flow, (b) pass it through a throat section at supercritical velocity, and (c) measure the depth within this throat where hydrostatic pressure exists. The cross-sectional shape was chosen as a compromise, considering (a) the need to pass large floods, (b) the efficiency in matching flume shape to channel shape, and (c) the desire to measure low, moderate, and high flows.

Figure 11 shows the design geometry of a typical Walnut Gulch flume. The flume has a 4.57-m (15-ft) curved entrance approach to a 6.10-m- (20-ft) long straight section having a shallow V-shaped floor and sidewalls with one-to-one slope.

The curved entrance approach has a cylindroid surface (coordinate origin shown in fig. 12) defined by the equation:

$$y = 0.03x + \frac{z - 0.09842x^2}{0.0287x^2 + 1}$$

where:

x = horizontal coordinate positive in the upstream direction, in meters

y = vertical coordinate, in meters

z = horizontal coordinate normal to and measured from the centerline of the flume, in meters

or

$$y = 0.03x + \frac{z - 0.03x^2}{0.00267x^2 + 1}$$

where x, y, and z are in feet.

An isometric view of this surface is shown in figure 12. The floor of the flume has a slope of 0.03 in the downstream direction parallel to the centerline to insure movement of sediment through the flume. This is the same slope used in the San Dimas flume (Wilm et al. *1938*).

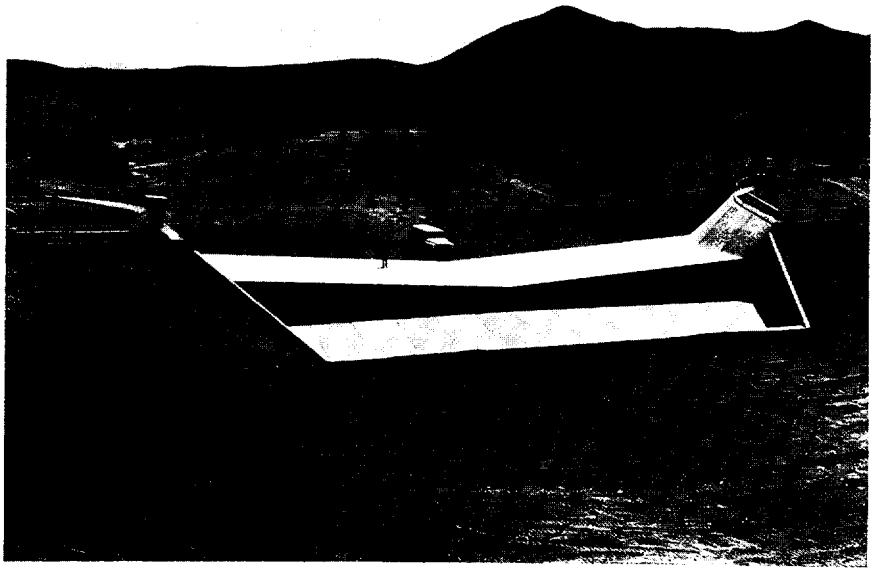


FIGURE 10.—The finished structure at the outlet of Walnut Gulch is considerably larger than the earlier one shown in figure 9 (4-21-64). BN-48654

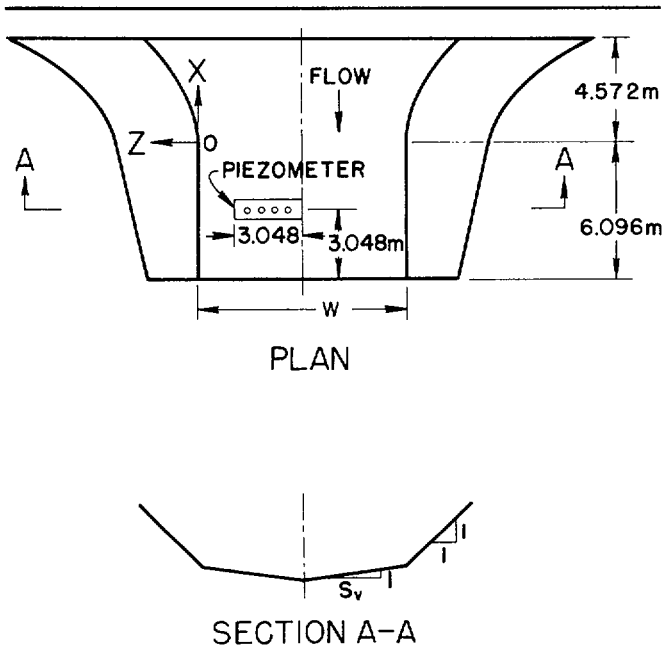


FIGURE 11.—Walnut Gulch supercritical measuring flume.

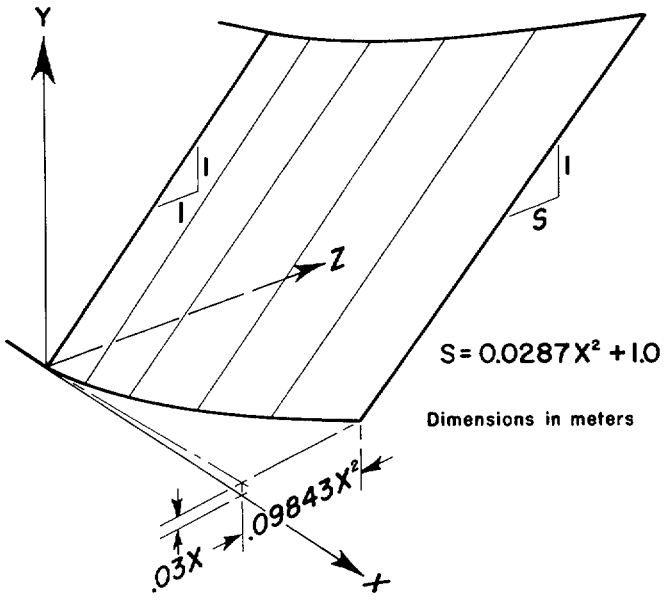


FIGURE 12.—Approach cylindroid surface, Walnut Gulch flume.

Before discussing the experimental development and field performance of this flume, it is useful to understand some of the hydraulic theory that deals with the measurements of flow and the development of a flume's rating by use of hydraulic models. In the following section, we present a brief explanation of the measurement sensitivity that may be reduced in order to pass heavily sediment-laden flows. We also discuss the hydraulic theory of flow through a supercritical flume and the theory that governs the similitude of model and prototype. Both mathematical and hydraulic models have played an important role in the development and analysis of this type of flume.

When natural stream velocities are sufficiently high, common flumes, which depend on measuring head upstream of a critical flow control section, are not suitable for reasons discussed above. In this case, we may still use a critical flow control section, but we measure depth below the critical section as the flow is accelerating in the supercritical region. The insurance that no deposition will take place in the flume itself is obtained at the cost of some sensitivity.

Measurement Sensitivities

Measurement sensitivity σ may be defined for our purpose here as:

$$\sigma(Q) = \frac{dh}{dQ} \qquad (1)$$

where Q is discharge, and h is measured depth. Thus, equation 1 states that sensitivity is a measure of the relative change in depth with a unit change in discharge. Typically, for a weir or flume that forces the flow to pass through a critical depth section and measures a rating depth, h, above or below critical depth, the discharge is:

$$Q = C_w h^b$$

in which b is a parameter; or

$$h = \left[\frac{Q}{C_w}\right]^{1/b}$$

where C_w is a dimensioned weir or flume coefficient that includes the effect of flow area geometry. Sensitivity is thus

$$\sigma(Q) = \frac{dh}{dQ} = \frac{C_w^{-1/b}}{b} Q^{\frac{1-b}{b}}. \qquad (2)$$

Although velocities are widely different, both flumes and weirs have a value for b of 1.5, if width is constant. The value of b may be greater than 2 if width varies with depth.

Sensitivity for a particular discharge is then a function of C_w and b, and since flumes with high velocities have large C_w, they exhibit a lower sensitivity than measuring devices with low velocity and small C_w.

Flow Equations

Since almost all flumes or weirs use a critical flow section as a control for measurement, critical, subcritical, and often supercritical flow are experienced. All three forms of flow are defined in reference to the Froude number, Fr;

$$\mathrm{Fr} = \frac{V}{\sqrt{gD}} \tag{3}$$

where V is velocity,
g is gravitational acceleration, and
D is hydraulic depth, defined as the cross section area of the flow, A, divided by the width of the free surface, T.

Weirs use a free overfall where critical flow, Fr = 1, occurs near the brink, downstream from the measuring point. Parshall or venturi flumes have transition sections that force flow through critical to supercritical flow for a short distance and then resume subcritical flow at or before the flume exit.

Supercritical flow flumes force flow through a critical section **above** the depth measuring point; depth is measured in the throat where flow is accelerating to normal depth for the supercritical slope within the flume throat.

Flow in this section is described by the same steady nonuniform flow equations that apply to other flumes. These are:

$$\frac{\partial Q}{\partial x} = 0 \tag{4a}$$

$$\frac{V}{g}\frac{\partial V}{\partial x} + \frac{\partial y}{\partial x} = S_o - S_f \tag{4b}$$

in which Q = discharge = AV
A = cross section area = $A(y)$
y = depth
x = distance along flume
V = velocity
g = gravitational acceleration
S_o = bottom slope of flume
S_f = friction slope of flume.

S_f is calculated from the friction relation defining uniform flow. For the Chezy or Manning relationship,

$$V = CR^a S_f^{1/2} \tag{5}$$

in which R is hydraulic radius and C is the friction coefficient. For the Chezy roughness relation, $a = 1/2$. If a Manning relation is used, $a = 2/3$, and $C = \frac{1.49}{n}$, (English units) where n is the Manning roughness coefficient. Solving equation 5 for S_f, we have

$$S_f = \frac{V^2}{C^2} R^{-2a}. \tag{6}$$

To calculate flow depth at any point within the flume, equation 4 is employed, starting with the critical depth section as a boundary condition. The transition region is divided into arbitrarily small increments, as illustrated in figure 13, and equation 4 is used in a finite difference expression. The equation thus becomes a Bernoulli equation for flow between the two sections i and $i + 1$:

$$y_i + \alpha\frac{V_i^2}{2g} = y_{i+1} + \alpha\frac{V_{i+1}^2}{2g} + h_e + \Delta x(\overline{S_f} - S_o), \tag{7a}$$

with

$$V_i A_i = V_{i+1} A_{i+1}. \tag{7b}$$

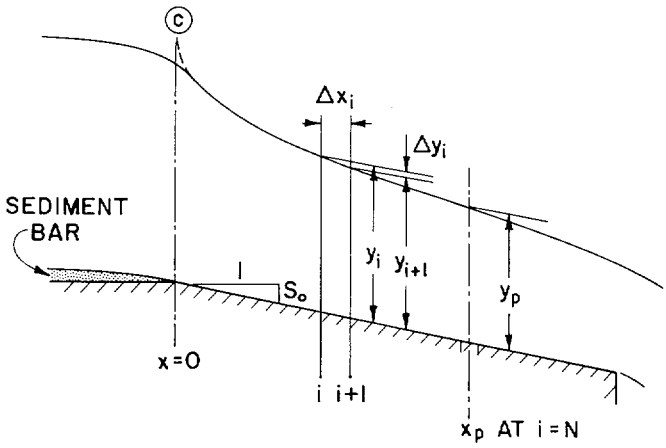


FIGURE 13.—Definition sketch of flow in supercritical transition.

Here α is the open channel energy coefficient (Chow *1959*). $\overline{S_f}$ is taken from equation 6 using a mean value of V and R in the length Δx. The eddy loss head (h_e) is defined by Chow (*1959*) as

$$h_e = K_e\alpha\left|\frac{V_i^2 - V_{i+1}^2}{2g}\right|$$

in which K_e is eddy loss coefficient. Chow (*1959*) gives typical upper limits of 0.1 and 0.2 for K_e in gradually converging and diverging reaches, respectively (English units). Section *C* (fig. 13) is about where critical depth occurs. Mathematically, this is a singular point, where the surface water slope is undefined. Practically, in alluvial channels with moving beds, the channel bed material will often form a region of transition of bottom slope from natural channel slope, S_c, to the imposed flume slope, S_o, where $S_c < S_o$.

In applying equation 7 to a specific flume, we use the actual geometry at each section to define:

$$R = R(x,y) \tag{8}$$

$$A = A(x,y). \tag{9}$$

Computationally, the distance from the critical section to the measuring section, x_p, is divided into $N-1$ increments. Equations 7, 8, and 9 are solved between successive sections $i = 1$ through $i = N$.

The boundary condition upstream at $i = 1$ (critical section) specifies that for a given Q, the Froude number is (nominally) 1.0, so that, by definition

$$V = \frac{Q}{A_i} = \left[\frac{gy_1}{\alpha_1}\right]^{1/2}. \tag{10}$$

Thus, y_1 and $A(Q,y_1)$ may be found from the geometry of the flume. Newton iteration is used to calculate y_{i+1} from equation 7 in sections 1 through $N-1$, and therefore to calculate y_p for any given discharge Q. A computer program developed for the simulation described herein is listed in Appendix A.

This numerical method provides a mathematical model for flow within a supercritical flume of any specified geometry. The same model will provide simulation of a subcritical flume, such as a venturi or Parshall flume, where depth is measured above a critical section, provided the numerical steps move upstream from the critical condition rather than downstream.

The analysis of the Walnut Gulch and similar supercritical flumes described below depend on both theoretical and experimental studies. Hydraulic models were an important part of the rating of the Walnut Gulch flumes. The transfer of model ratings to prototype ratings depends on proper use of hydraulic similitude, discussed below.

Model Similitude

Laws of similitude must be considered in using a hydraulic model to predict the rating of a larger flume. The most appropriate similitude criterion for open channel flow is the Froude number, which implies equality of the ratio of inertia to gravity forces for both model and prototype. Another important criterion is the Reynolds number, which implies equality of the ratios of inertia forces to viscous or friction forces in model and prototype. Both criteria cannot be met simultaneously, but for fully turbulent open channel flow with a high Reynolds number, the friction changes little with the Reynolds number. Therefore, the Froude number is commonly the governing similitude criterion.

When model scales in the horizontal and vertical are the same, the model is referred to as undistorted. When they are different, it is a distorted model. An undistorted scale model uses identical scale ratios in all three spatial dimensions, providing geometrical similarity. Using an undistorted model, with a scale ratio of L (using subscript p for prototype and m for model):

$$y_p = Ly_m, \tag{11}$$

$$A_p = L^2A_m. \tag{12}$$

The Froude number is defined as

$$\mathrm{Fr} = \frac{V}{\sqrt{gD}} \tag{13}$$

where D is hydraulic depth. Thus, if $\mathrm{Fr}_m = \mathrm{Fr}_p$,

$$\frac{V_m}{\sqrt{gD_m}} = \frac{V_p}{\sqrt{gD_p}}$$

or, from equation 11,

$$V_p = V_m\left(\frac{D_p}{D_m}\right)^{1/2} = V_m\sqrt{L} \tag{14}$$

where D_m and D_p are hydraulic depth of model and prototype, respectively. From equation 12,

$$Q_p = V_pA_p = Q_mL^{5/2} \tag{15}$$

Thus, equations 11 and 15 allow us to estimate a prototype rating from a hydraulic model rating.

Original Model Studies

The initial supercritical flume design was studied in the laboratory using a 1:32 scale model of a flume whose geometry was as shown in figure 11, with floor width of 9.14 m (30 ft), as in Walnut Gulch flume No. 3 (63.003). Piezometers flush with the surface were located in the downstream half of the straight section, both in the *V*-shaped floor and sides of the flume. The purpose of these measurements was to determine the best location to measure the head. Results of some of these measurements are shown in figure 14. Station 10+75 was the outlet end of the flume. The pressure on the floor at the measuring section of the flume was found to be approximately hydrostatic when the depth of the flow is less than the distance to the downstream edge of the flume. The midpoint of the narrow, straight portion of the flume was selected as the best point to measure the head. For Walnut Gulch flume No. 3, the total width (4.57 m; 15 ft) of one side of the floor was used to measure the head and acted as the intake to the stilling well. For larger flumes, the length of the intake was limited to 3.05 m (10 ft) as shown in figure 11. The development of the flumes and model techniques used in these studies were reported by Gwinn (*1964, 1970*). The various flume dimensions were chosen solely to match existing channel geometry and reflect a compromise between desire for contraction and need for peak flow capacity. A summary of the flume dimensions and scales used in the model studies is given in table 1.

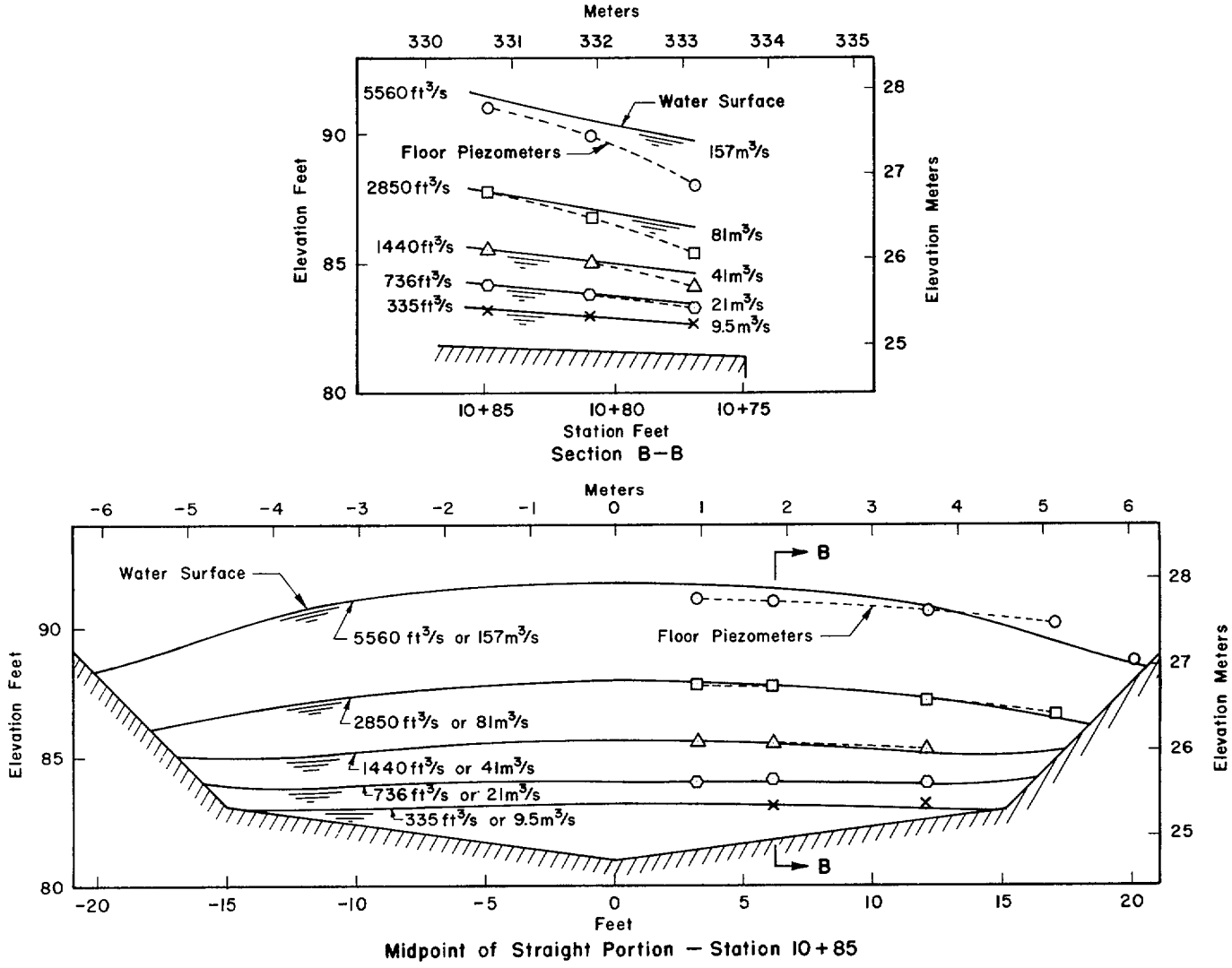


FIGURE 14.—Water surface profiles and floor piezometer measurements for the initial design of Walnut Gulch Flume No. 3.

TABLE 1.—*Summary of laboratory-calibrated Walnut Gulch flumes*

Flume No.	Floor cross slope (S_v)	Depth at sidewall interesection		Flume width		Maximum discharge		Model length scale
		Meters	*Feet*	*Meters*	*Feet*	M^3/s	Ft^3/s	
1	15	1.22	4	36.58	120	740	26,000	1:40
2	[1] 5	.61	2	24.38	80	560	19,700	1:40, 1:20
2	[2] 5 15	.90	2.95	24.96	81.9	560	19,700	1:20
3	7.5	.61	2	9.14	30	170	6,000	1:32
4	10	.08	.25	1.52	5	34	1,200	1:30
6	10	1.07	3.50	21.33	70	470	16,500	1:30
7	10	0.61	2	12.19	40	244	8,600	1:30
8	10	.61	2	12.19	40	244	8,600	1:30
11	10	.46	1.50	9.14	30	170	6,000	1:30
15	10	.61	2	12.19	40	235	8,300	1:30

[1]Original floor, combination slope 1 on 5 for 3 m (10 ft) and horizontal for 9.14 m (30 ft), (fig. 15).
[2]Revised floor, combination slope 1 on 5 for 0.503 m (1.65 ft) and 1 on 15 for 11.98 m (39.3 ft).

Prototype ratings were originally obtained from the small model studies by a scaling that used a discharge coefficient, C_d. Model results were used to obtain a value of C_d in the expression

$$Q_m = C_D \frac{t_m}{2} \sqrt{2g}\, h_m^{1.5} \qquad (16)$$

in which t is width of water surface at the measuring point when depth is h. C_D is dimensionless and is assumed to apply, therefore, to a prototype scale relationship for Q_p, using equation 16 where $h_p = Lh_m$ and $t_p = Lt_m$. C_D includes a distortion factor for flows above the sidewall-floor intersection, since $\frac{t}{2} h_m$ represents area for floor region geometry only. Model scale L, in these cases, is from 20 to 40, as in table 1. Model roughness was assumed properly scaled in comparison with prototype roughness, and no correction was applied. Selected data from these model ratings in prototype dimensions are given in Appendix B. A more complete discussion of the model results has been given by Gwinn (*1970*).

Colorado State University 1:5 Model Rating of the Floor Section

The small scale model studies at the Water Conservation Structure Laboratories in Stillwater, Okla., were unable to accurately evaluate low flow ratings. Subsequent knowledge of the distribution of flow depths indicated that this was an important range of flow in the ephemeral type of hydrologic regime of the southwestern U.S. watersheds. Figure 15 shows the sample distribution of flume flow depths for flumes 1 and 6 (code numbers 63.001 and 63.006, respectively), indicating some 96 to 93 percent peak flow depths, respectively, occur in the floor region. The floor region of these flumes refers to that portion of the flume cross section below the intersection of the bottom V-shaped region and the 1:1 sidewalls. To better define low flow ratings, a larger scale 1:5 model study was initiated in a 6-m (20-ft) flume at the Colorado State University Engineering Research Center. The model consisted entirely of the flume floor section of the Walnut Gulch flumes with a sandbed approach. It had a cross-channel slope of 1:10 with a longitudinal slope of 3 percent. It was constructed of epoxy-coated plywood with an estimated Manning roughness of 0.011. Figure 16 shows the flume and sandbed approach.

Discharge rates were measured downstream from the model by a well-calibrated knife edge rectangular weir. Water was supplied by a pump from a lake below the flume. Upstream bed topography was simulated by sand placed and maintained at approximately the 1-percent slope of the natural channel. The model scale was not distorted. Head was measured by manometer at several points across the measuring section, located 7.62 m (25 ft) (in prototype dimensions) below the entrance edge of the floor.

Figure 17 summarizes results of the tests over a wide range of flows encompassing the capacity of the floor section.

Scaling of the 1:5 Model Data

Equation 15 neglects differences in friction coefficient C between model and prototype. More rigorously, equations 14 and 15 are independent of friction and friction law at a critical flow control where Fr = 1 in both model and prototype. If friction is dissimilar between model and prototype and flow is at normal depth as described by equation 5, scaling following the Froude number criterion depends upon which hydraulic friction relation is taken to apply (Murphy *1950,* Chapter 8). The Chezy friction law satisfies the Froude number criterion if the velocities are scaled by the geometric scale ratio, $L^{1/2}$, (equation 14) multiplied by the ratio of roughness coefficients, C. The Manning friction law requires geometric scale ratio for velocity to be multiplied by $\frac{C_p}{C_m} L^{1/6}$. For example, equation 14, for differing surface roughness, C_m and C_p, for Chezy's law becomes

$$V_p = V_m \frac{C_p}{C_m} \sqrt{L} \qquad \text{(normal flow)} \qquad (17)$$

and, for Manning's law, becomes

$$V_p = V_m \frac{C_p}{C_m} L^{1/6} \sqrt{L} \qquad \text{(normal flow)} \qquad (18)$$

in which case, C is $\frac{1.49}{n}$, n = Manning's n.

The hydraulic conditions of this model study introduce a special problem in model scaling. The flume floor surfaces were significantly different. For the model, it was polished epoxy coating; for the prototype, it is finished concrete. Also, the flow is not critical at the measuring point. Therefore, roughness and geometric scaling are necessary. Moreover, flow conditions at the measuring point for which scaling is necessary are between critical flow, Fr = 1 (independent of roughness), and supercritical normal flow. Thus, scaling will lie somewhere between that required for critical flow and that required for normal flow, where roughness must be scaled as in equations 17 or 18.

The scaling procedure used here employs the numerical model (equations 7a and b for flow through the flume) to characterize scaling for the different roughness in the flume throat. In this region, flow is neither normal nor critical. We rewrite equations 17 and 18 as

$$V_p = V_m r_c \sqrt{L} \qquad (19)$$

in which r_c is the scaling ratio for roughness. As noted above, r_c = 1 at critical depth, and is defined by equations 17 or 18 for normal depth.

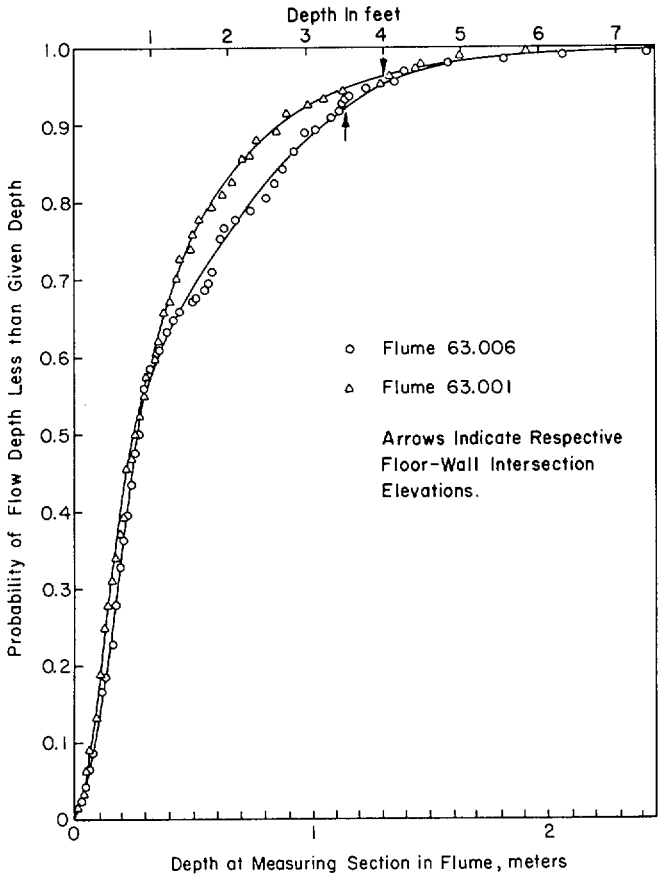


FIGURE 15.—Sample distribution of flow depths for flumes 63.001 and 63.006 at Walnut Gulch. Floor-wall intersection occurs at 1.07 m (3.5 ft) for flume 63.003, and at 1.22 m (4.0 ft) for flume 63.001.

FIGURE 16.—1:5 scale model of floor section of the Walnut Gulch supercritical flume, looking downstream along the sand approach. BN-48655

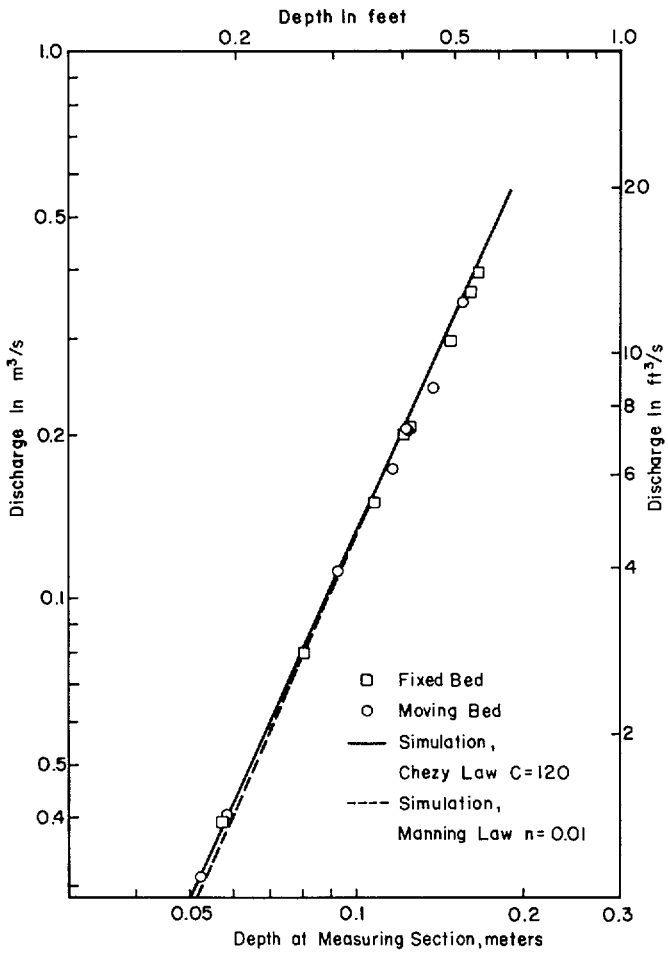


FIGURE 17.—Plot of experimental 1:5 model rating from experiments at Colorado State University and from computer simulation.

The following procedure was developed to evaluate r_c for the transition flow in the supercritical throat. If we assume flumes are identical except for roughness, with the same discharge in each flume, by the Chezy law,

$$C_r h_r^{2.5} = C_s h_s^{2.5}$$

where subscripts r and s refer to the rougher and smoother case, respectively. Scaling ratio, r_c, due to the Chezy roughness alone, is found as

$$r_c = \frac{C_r}{C_s} = \left(\frac{h_s}{h_r}\right)^{2.5} \tag{20a}$$

since the prototype is rougher than the model in this case. For the Manning roughness law, similarly,

$$r_c = \frac{C_r}{C_s} = \left(\frac{h_s}{h_r}\right)^{2.67} \tag{20b}$$

The Froude number at normal depth in supercritical flow, Fr_n, is always greater than one, and the Froude number at the measuring point $X = X_p$ (fig. 13), Fr_p, is always less than Fr_n. Expressed in equation form,

$$\mathrm{Fr}_n > \mathrm{Fr}_p > 1.$$

We define a dimensionless parameter, w, to represent the relative value of Fr_p, within its limits. Let

$$w = \frac{\mathrm{Fr}_p - 1}{\mathrm{Fr}_n - 1} \tag{21}$$

so that $1 > w > 0$, with $w = 1$ at normal flow and $w = 0$ at the critical section.

Simulations were performed over the range of model discharges 0.0003 to 0.57 m^3/s (0.01 to 20 ft^3/s) for a ratio of roughness coefficients of 1.2 (Chezy C = 113 and 134 or Manning's n = 0.013 to 0.011). The relation between r_c and the parameter w is found from this numerical simulation, and the results are expressed graphically as shown in figure 18. This graphical relation is then used to find r_c given w for a particular model test whose relative roughness parameters have the same ratio. Here, w is for the model from which scaling is to be done. The results in this figure should not be taken as general, even though both variables are dimensionless. For example, w at x = 1m in the smoother case is not the same as w for x = 1m in the rougher case. It does provide a more accurate estimate of prototype scale rating for conditions in the supercritical drawdown region.

Analysis of Model Ratings

Scaled values are computed in table 2 and plotted in figure 19 along with scaled results from the 1:30 model test at Stillwater and the computed rating from equations 7a and b. Within the respective ranges of applicability of the two model studies, rating relations for the supercritical flumes from the Stillwater and Colorado State University model tests are in excellent agreement.

Figure 19 also shows the computer simulation for the same conditions, indicating the ability to simulate ratings by using the mathematical model. Either the Manning or Chezy friction relation may be used for higher flows, and sufficient data at the very lowest flows are not available to discriminate categorically between the two relationships.

TABLE 2.—*Scaling of Colorado State University flume model results*

Q_m		h_m		F_{pm}	w_s	r_c[1]	Q_p[2]		h_p[2]	
					Fixed bed					
M^3/s	(*Cfs*)	*Centimeters*	*Feet*				M^3/s	Ft^3/s	*Meters*	*Feet*
0.0374	1.32	5.85	0.192	2.28	0.242	0.973	2.03	71.8	0.29	0.96
.0402	1.42	5.97	.196	2.28	.242	.973	2.19	77.2	.30	.98
.079	2.79	8.23	.27	2.01	.19	.981	4.33	153.0	.41	1.35
.150	5.30	10.97	.36	1.86	.16	.985	8.27	292.	.55	1.80
.199	7.01	12.47	.409	1.79	.15	.986	10.93	386.	.625	2.05
.2033	7.18	12.5	.41	1.79	.15	.986	11.21	396.	.625	2.05
.244	8.61	13.9	.457	1.69	.13	.989	13.48	476.	.695	2.28
.297	10.49	15.0	.493	1.69	.13	.989	16.42	580.	.750	2.46
.364	12.87	16.4	.538	1.65	.12	.990	20.16	712.	.820	2.69
.395	13.96	17.0	.558	1.64	.12	.990	21.89	773.	.85	2.79
					Moving bed (upstream)					
.031	1.09	5.33	.175	2.32	.25	.972	1.68	59.2	.265	.87
.111	3.92	9.45	.310	2.0	.19	.981	6.09	215.	.47	1.55
.174	6.15	11.89	.390	1.77	.15	.986	9.60	339.	.59	1.95
.205	7.25	12.74	.418	1.75	.14	.987	11.3	400.	.637	2.09
.244	8.62	14.05	.461	1.63	.12	.990	13.5	477.	.700	2.30
.349	12.32	15.8	.52	1.72	.13	.989	19.3	681.	.792	2.60

[1]For all data, $Q_p(m^3/s) = 30.83\ h_p^{2.2}$, h_p in meters, with $r^2 = 0.9993$, or $Q_p(ft^3/s) = 79.75\ h_p^{2.2}$, h_p in feet.
[2]From figure 13 with w_s from model Froude numbers.

The computer model may also be used to demonstrate the sensitivity of rating at the control section to changes in the actual location of the critical flow section. The hypothesis is that in many streams steep slopes may cause critical flow to occur above the specified section, and the Froude number at the presumed critical flow point may be somewhat larger than 1.0. Table 3 shows computer simulation results, indicating that small changes in Froude number at the flume entrance do not have a large effect on the rating relationship, especially at lower flows.

Using the best fit regression line given below table 2, one may evaluate relative sensitivity of these flumes. For example, one may compare sensitivity of a typical supercritical flume of 1:10 cross-channel floor slope, at approximately 30 m^3/s, with that of a weir of the same approximate width, L_w. The weir rating is $Q = 3.3\ L_w h^{1.5}$, approximately. One may apply equation 2 and find that the supercritical flume is only some 40 percent less sensitive than the weir.

TABLE 3.—*Effect of upstream Froude number on flume rating simulation data for 21.34-m-(70-ft) wide flume*

Discharge (m^3/s)	Froude number at $x = 0$			
	1.0	1.1	1.2	1.4
	Depth, m			
0.057	0.0578	0.0578	0.0577	0.0577
0.28	.1175	.1175	.1173	.1168
2.8	.3289	.3284	.3273	.3236
28	.9096	.9068	.9007	.8818

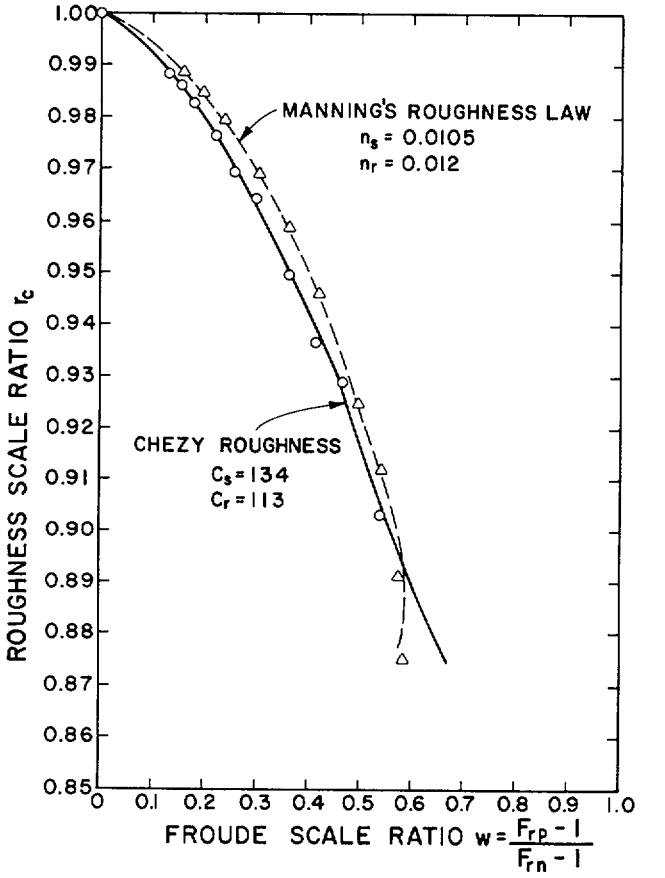


FIGURE 18.—Graphical procedure developed from computer simulation to estimate roughness scaling for transition flow.

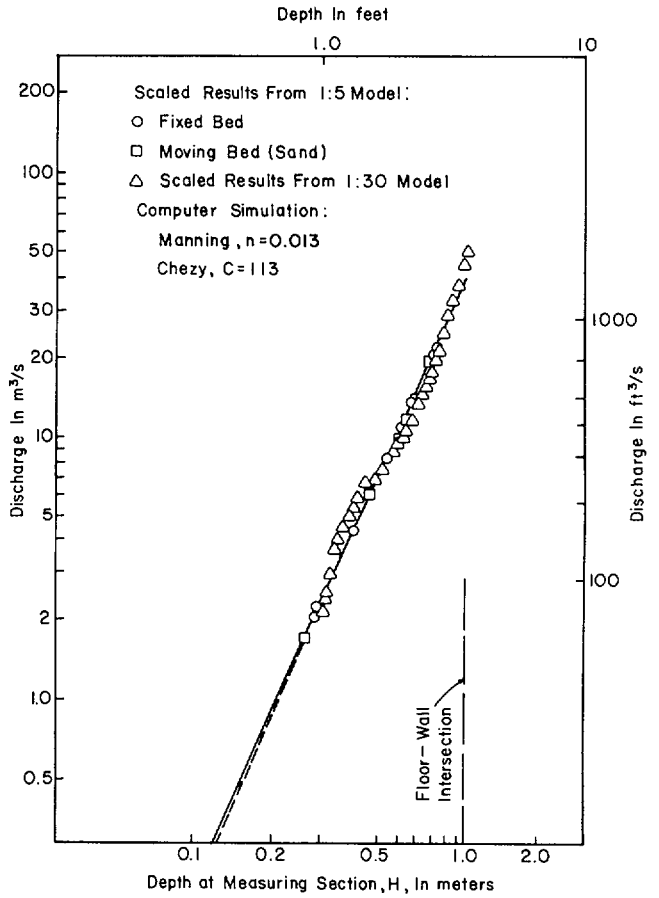


FIGURE 19.—Prototype rating for flume 63.006 developed from 1:5 model tests, scaled as in equation 19. Also shown are scaled results from the 1:30 model tests at Stillwater, Okla.

Prototype Evaluations

Velocity measurements for evaluation of prototype rating for the Walnut Gulch flumes are difficult if not impractical using ordinary stream current metering methods. Flow discharges at any point and, for that matter channel cross sections, change quite rapidly during the flows in this hydrologic regime. Moreover, flows are ephemeral and unpredictable, velocities are often above most current meter ranges, and the amount of sediment and other suspended matter make using current meters impractical in most cases. The limited amount of prototype verification information is presented below. This includes one special measurement in the early flume history and a major ongoing instrumentation effort at another flume. Field performance of the Walnut Gulch flumes was briefly summarized by Smith and Chery (*1974*).

Velocity Measurements at Flume 63.002

When Walnut Gulch flume No. 2 (code number 63.002) was built, a railroad adjacent to the measuring site restricted the flume geometry and overall head loss available for use by the measurement structure. Thus, the geometry at the cross section was made to include a floor section with no cross slope, but it did include a 0.61-m- (2-ft) deep × 6.1-m- (20-ft) wide notch (fig. 20) to provide additional sensitivity for measuring the base flow, which often occurs at this station.

On July 31, 1961, current meter measurements were taken in the "notch" of this flume using a Price current meter. The measurements were made from a rigid temporary bridge, which was positioned across the notched section of the flume near the flume entrance. Although the velocities at this point were high, flow depths were not changing rapidly enough to affect the measurements, and debris did not collect on the

meter. Thus, we have considerable confidence in the accuracy of these discharge measurements, although a few measured velocities required extrapolation of the meter calibration curve.

The results of the measurements, shown in figure 20, appear to agree with the laboratory-determined values from the model studies. The slight departure of the measured data from the small model rating and the bias in comparison with the computer model may be partly associated with the need to extrapolate the current meter calibration for this measurement. Perhaps more important in evaluating these results is the apparent movement of the critical flow section to several feet upstream from the flume entrance. This movement of control was likely the result of the streambed narrowing in response to the flume notch. The general agreement among the three estimated ratings, however, added confidence to the scaled (1:30) model rating at higher discharges. At lower flow depths, water viscosity was a potential problem in the small scale model.

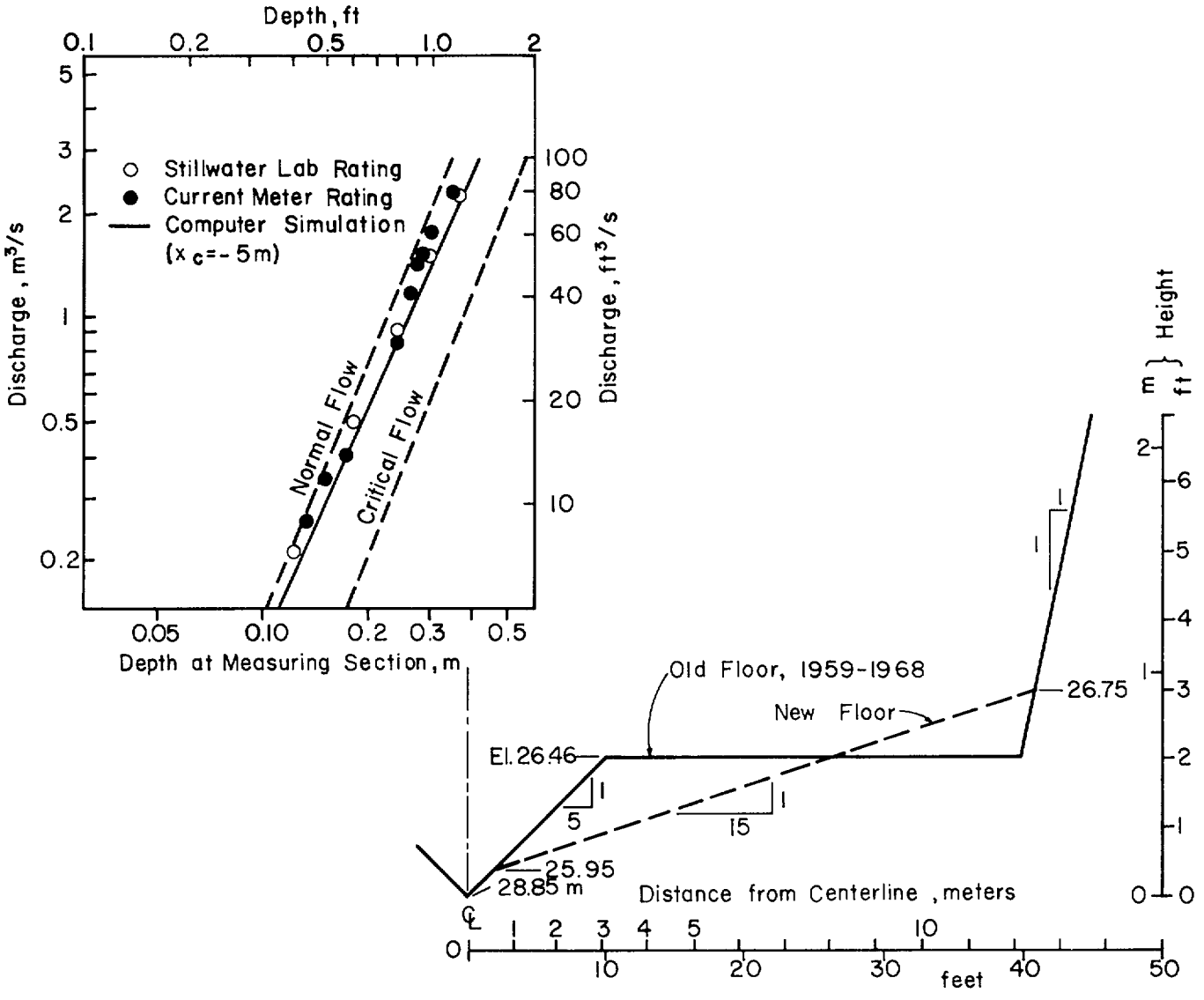


FIGURE 20.—Sketch of early cross-section at flume 63.002, and plot of current meter results obtained in 1961. Numerical model comparison indicates strongly that critical flow was occurring some distance above the flume during this test.

After the railroad was abandoned in 1970, the flat cross-sloped floor was replaced with the sloping floor shown in figure 20. The new floor affords greater sensitivity at depths exceeding 0.6 m (2 ft) than was possible with the shallow flow over the flat floor.

Prototype Data at Flume 63.006

In response to the need for prototype information on hydraulics of the Walnut Gulch flumes, a special study of flume 6 (station 63.006) was initiated in June 1973. An instrumentation footbridge 1.2 m (4 ft) wide and 30.5 m (100 ft) long was installed across this flume at approximately the measuring section, so that sampling across the flow would be possible. A movable carriage was mounted on the bridge to allow sampling at selected locations in both x and y dimensions. The instrument carriage and bridge are illustrated in figure 21.

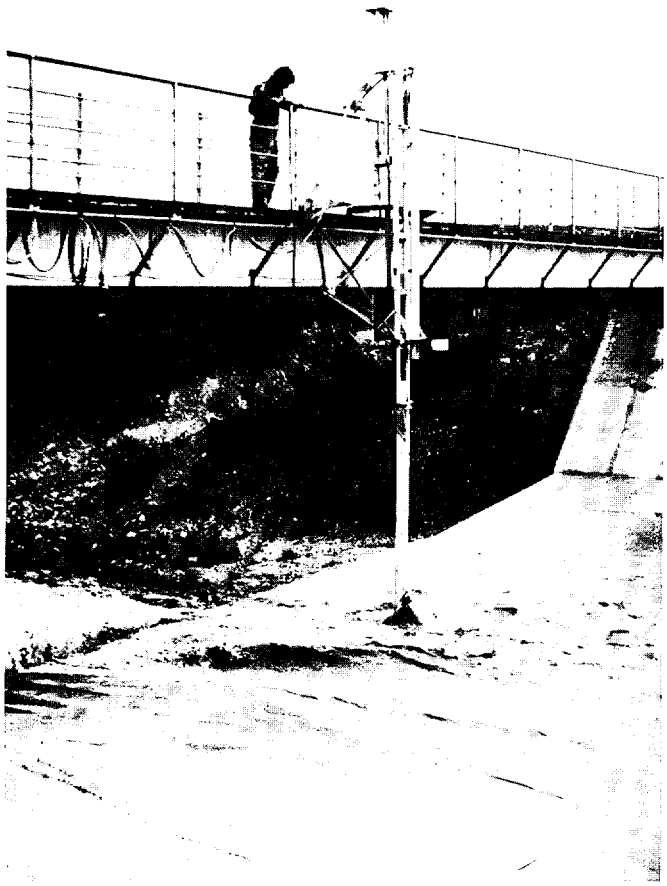


FIGURE 21.—Instrument carriage and velocity meter probe in use at flume 63.006, August 10, 1976. BN-48656

A streamlined electromagnetic velocity meter was mounted at the bottom of a vertical, movable probe, at a 45° incline to pass trash without damage or interference with meter readings. The meter has range capability that matches the high velocities in the flume throat. Depth of flow is recorded by using two sonar depth gages, one fixed at the center of the flume and a second moving with the sampling carriage. Potentiometers automatically record the x and y positions of the velocity probe. All hydraulic data are recorded digitally for automatic processing.

Measurements were made during 15 flow events in 1974-77 with varying degrees of success. From these measurements, four scans from four events, shown in figures 22 through 25, were selected as valid representations of the range of flow depths observed to date. These measured flows ranged in average depth from 0.38 m (1.25 ft) to 0.89 m (2.92 ft) with discharges of about 1.81 to 29.3 m^3/s (64 to 1,036 ft^3/s).

Velocity contours were estimated from the point measurements of the velocity probe. At each point, at least two samples of velocity were recorded. After the data were obtained, it was observed that after moving the probe the first measurement was usually low because of the time constant of the instrument response. Thus, the low first values were disregarded. Of all the cross sections, only scan No. 12 on September 4, 1975, has two points in which there is considerable confidence. Both were obtained by placing the probe in one position for several minutes, just before the scan and then just after the scan. The average velocity for both points was 0.25 m^3/s (8.8 ft^3/s) (fig. 23).

The area between each contour was measured and multiplied by its representative velocity to determine the discharge and calculate the velocity distribution (energy) coefficient, α. This coefficient was calculated by the following relation:

$$\alpha = \frac{\sum_j V_j^3 \triangle a_j}{\overline{V}^3 A} \qquad (22)$$

where v_j = point velocity between contours;
$\overline{V}$ = average velocity
a_j = incremental cross section area, contour j
A = total cross section area.

The discharges calculated by these computations are plotted at both the centerline depth and the mean depth in figure 26. The line on this plot is the flume rating from 1:5 model tests. Also shown in symbols is the laboratory rating prepared from the early 1:30 scale model tests. For a given depth, less than 1 m (3 ft), the measured discharge is less than the rating relation indicates. This difference decreases as the depth increases until the measured values agree with the laboratory ratings at about 1 m (3 ft).

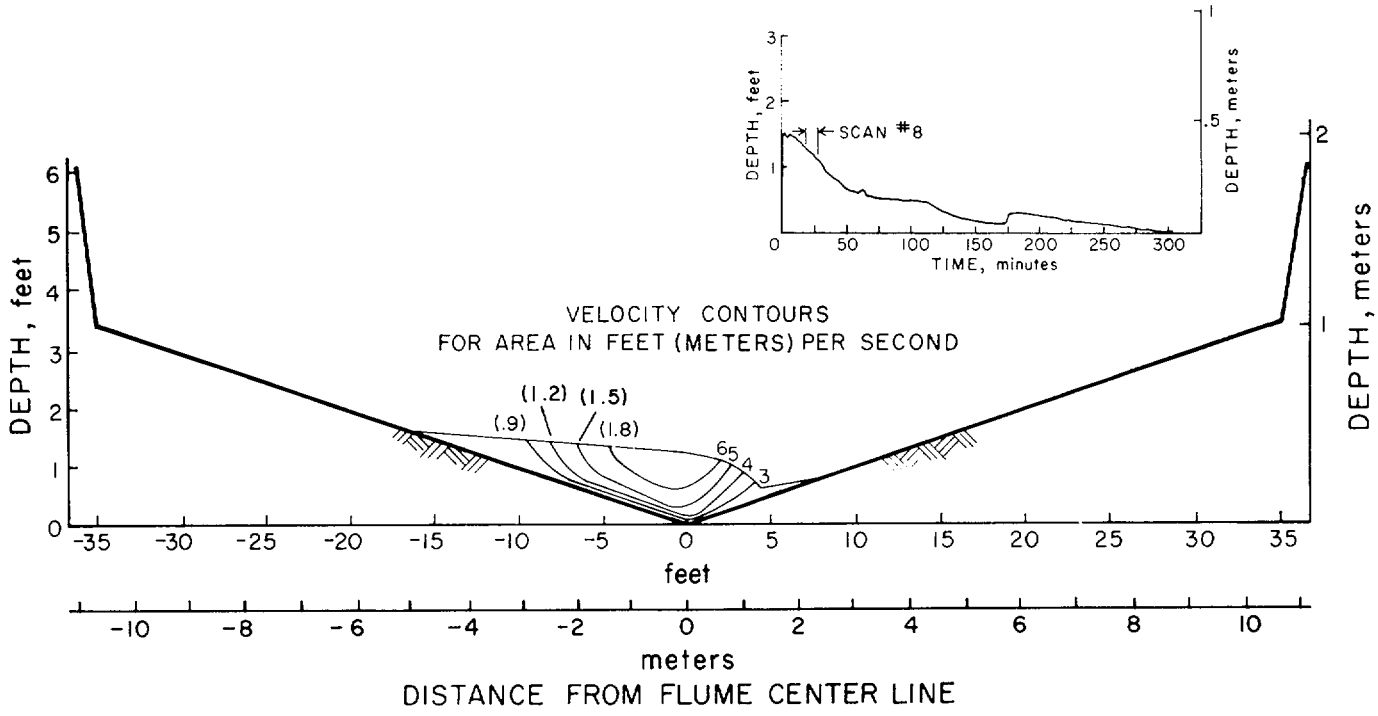


FIGURE 22.—Flume 63.006, flow profile and velocity contours for flow of September 6, 1974, scan No. 8.

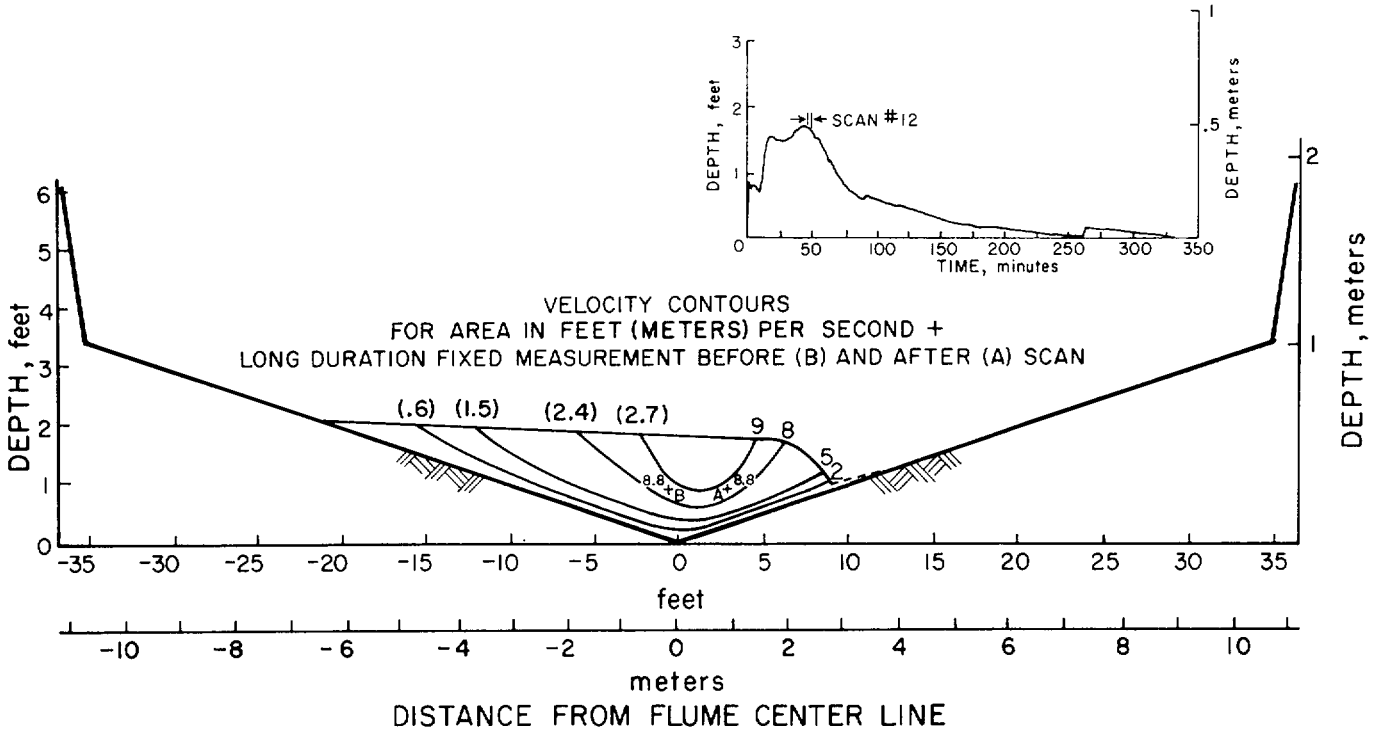


FIGURE 23.—Flume 63.006, flow profile and velocity contours for flow of September 5, 1975, scan No. 12.

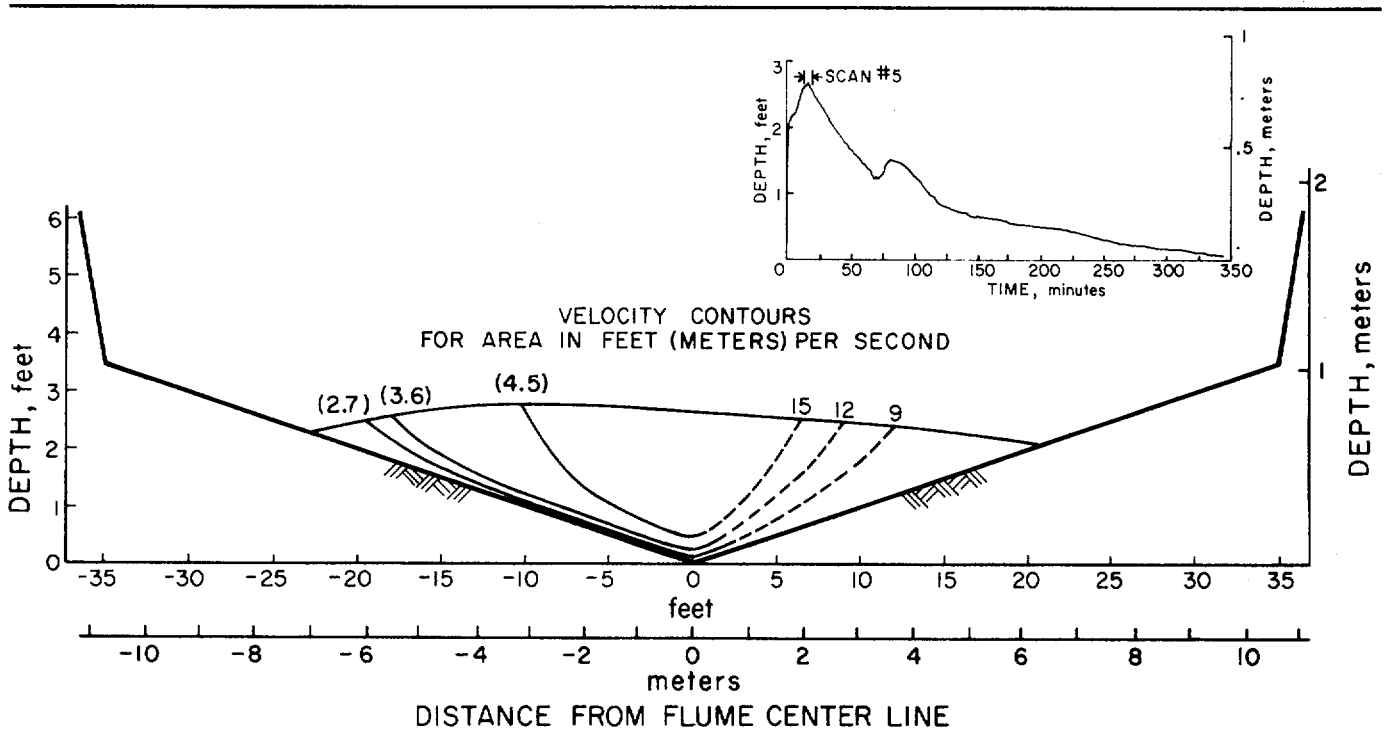


FIGURE 24.—Flume 63.006, flow profile and velocity contours for flow of July 22, 1975, scan No. 5.

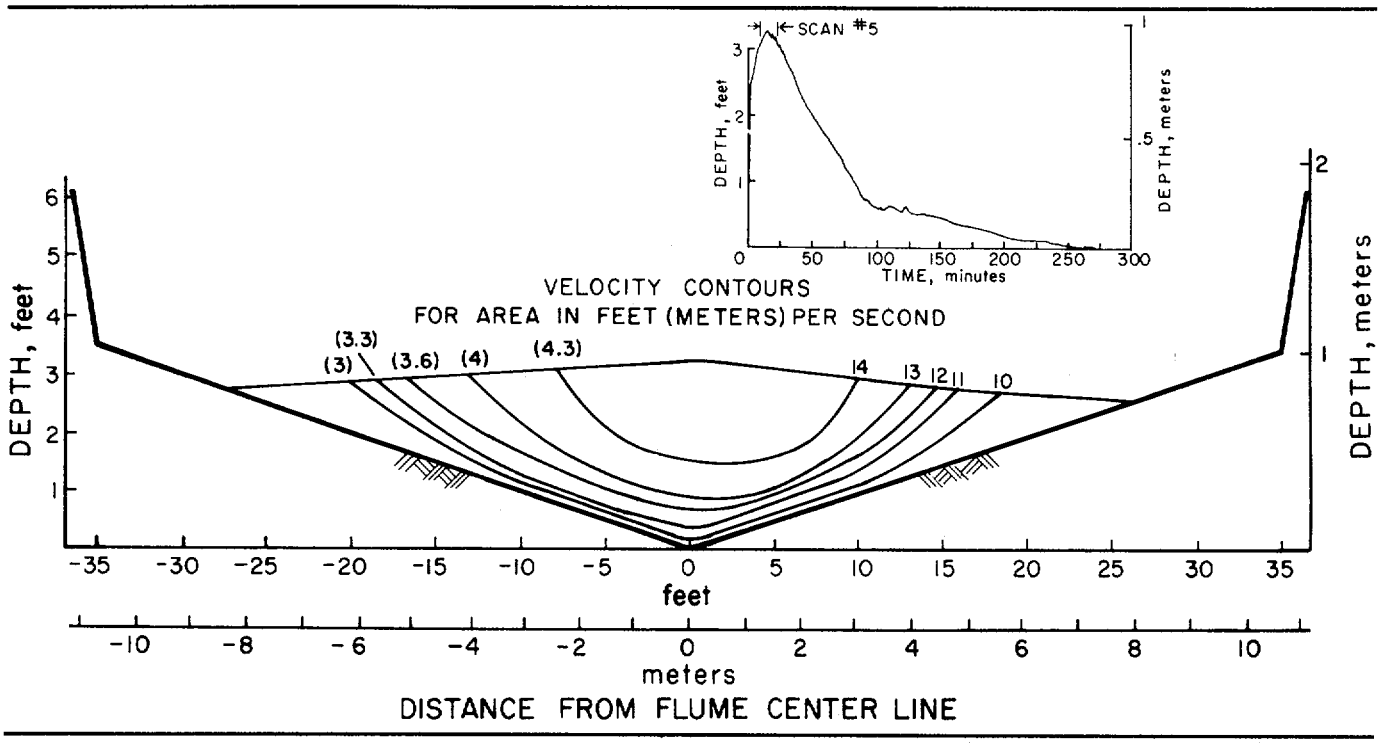


FIGURE 25.—Flume 63.006, flow profile and velocity contours for flow of September 2, 1974, scan No. 5.

These departures from the rated discharge are believed to be a result of the flow entering the flume at an angle and having an asymmetrical cross section at the measuring point of the flume. The profiles in figures 22 through 24 show the various surface configurations of this asymmetrical flow. Flow depths must exceed about 1 m (3 ft) at this location before the flow will aline itself enough that the flume geometry will control the flow in the measuring section, and produce a nearly symmetrical cross section, as is seen in figure 25.

The high-water traces on the flume floor, resulting from a medium depth flow of 0.46 and 0.61 m (1.5 to 2 ft) entering the flume at an angle, are shown in figure 27. This shows clearly that the path of the flow riding up on the left side of the flume and then turning back toward the centerline causes a standing wave to form about halfway through the flume and to extend to the outlet of the flume. This wave causes the stepped profile at the measuring section seen in the cross section plots of figures 22 and 23. Photograph of a small wave occurring in the flow of August 10, 1976, is shown in figure 21.

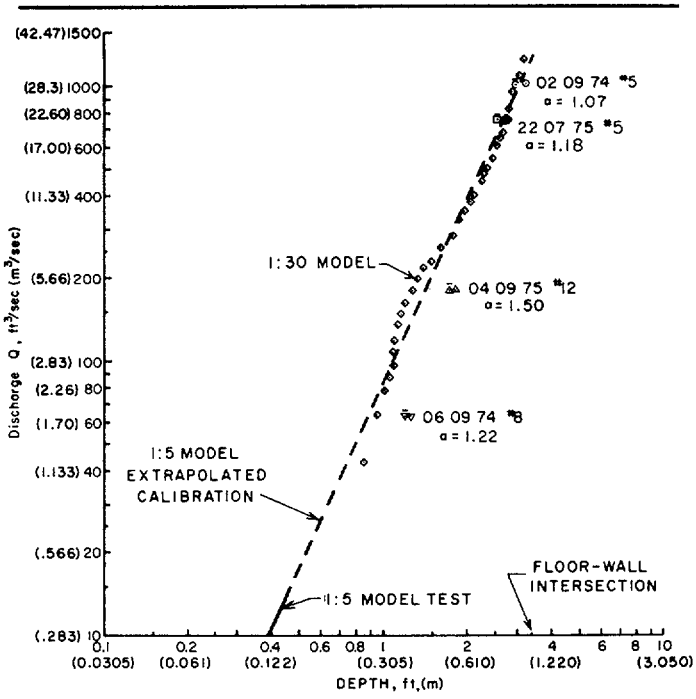


FIGURE 26.—Laboratory rating relation for flume 63.006 compared with measured values.

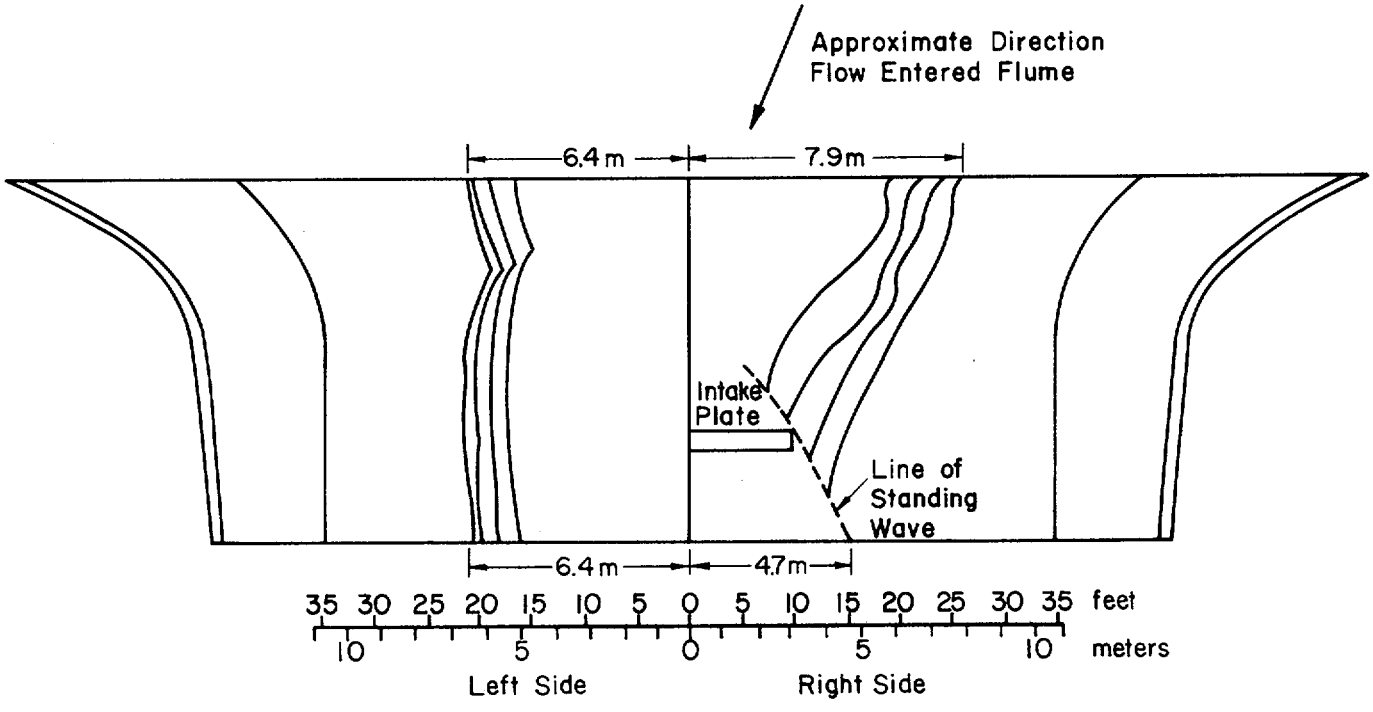


FIGURE 27.—High water traces for flow of August 17, 1976.

Stabilization of Approach Channels

Compounding the alluvial flow measurement problems outlined above, which led to the development of the Walnut Gulch flume, is the instability of thalweg location in the ephemeral flows of wide alluvial channels such as Walnut Gulch. Extremely asymmetrical entrance conditions have been observed in these flumes many times (Smith and Chery *1974*). Figure 28 is a dramatic illustration of the asymmetry of the alluvial bar formed during flow through flume No. 63.007. Not only are flow centroids off center, but also mean flow direction at the flume entrance is often at a significant angle to the centerline direction of the flume. Methods to correct this misalinement had been generally unsatisfactory up to the time of modeling the flume floor at Colorado State University in 1971.

Experimental arrangement.—The 6.1-m (20-ft) wide outdoor flume at Colorado State University used for the 1:5 model rating of the floor section was also used to conduct tests on methods to stabilize the alluvial approach channel conditions. Views of the experimental arrangement are shown in figure 11. Sand was placed to a 60-cm (2-ft) depth for 15 m (50 ft) upstream from the 1:5 scale model of the flume floor section. The sand was chosen to closely model the mean grain size of sand found at Walnut Gulch; however, rocks and large gravel were not present, although slope and general bed shape were duplicated. Water was introduced into the sand approach severely off center to test the adequacy of a number of possible arrangements of dikes and fences for moving flow to a centered, alined position. Sand was introduced at the upstream end of the alluvial section to maintain a bedload without

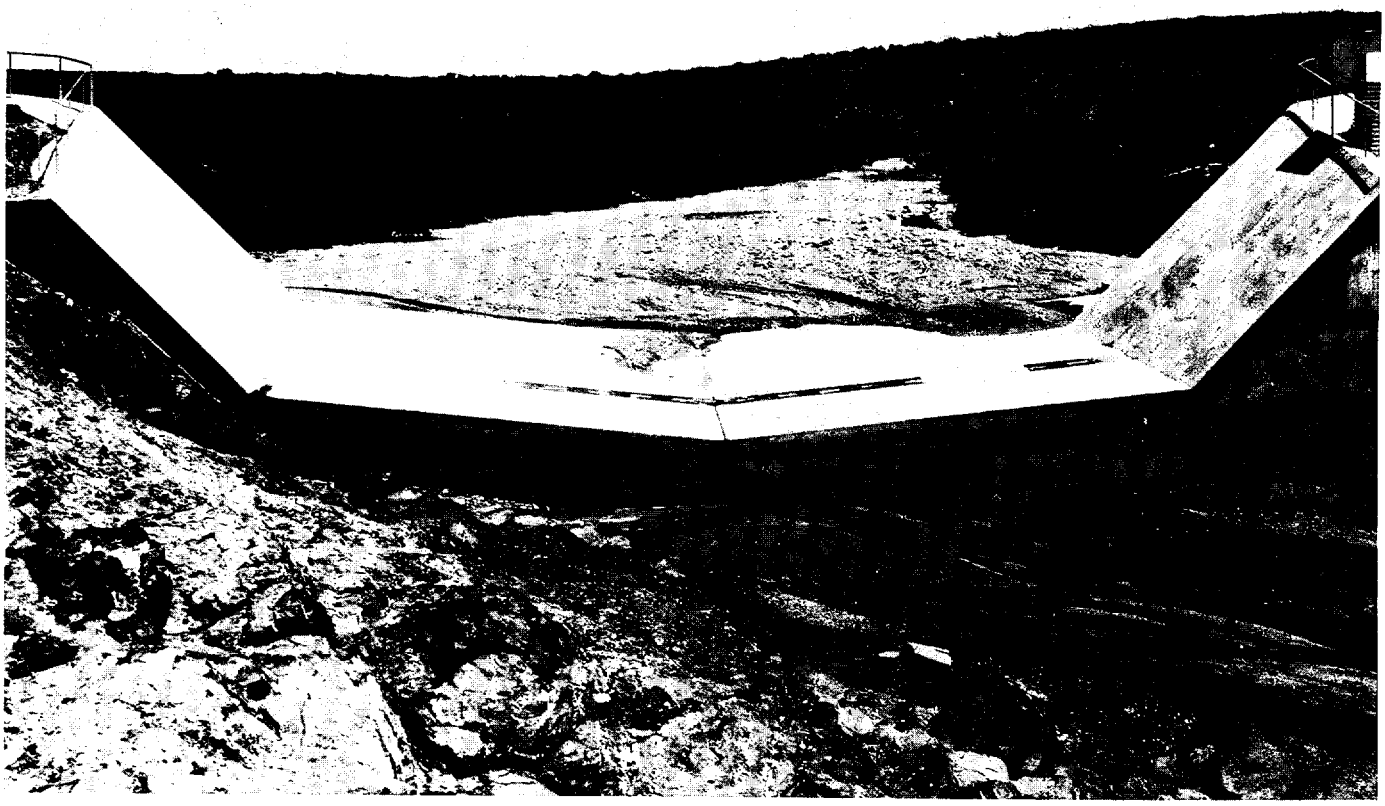


FIGURE 28.—View of flume 63.007 looking upstream, showing the large alluvial bar at the flume entrance which indicates a strongly asymmetrical entrance condition. BN-48657

appreciable net scour. Figure 29 shows how the model duplicated standing waves common at flume 6 (and others), and affords a view of the asymmetry of the bedload.

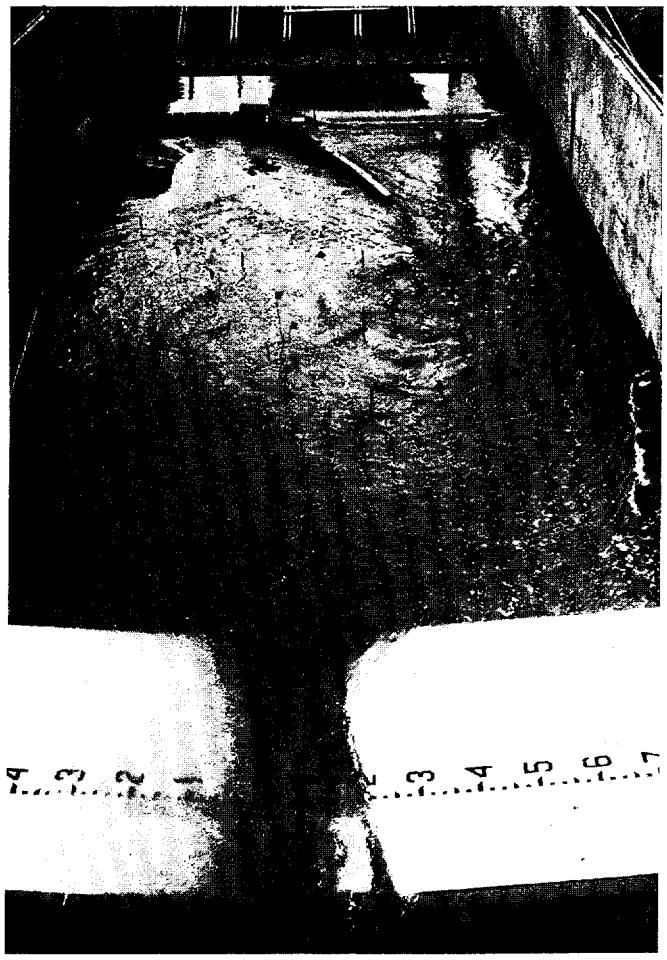


FIGURE 29.—Standing wave produced in the measuring section of the 1:5 model at Colorado State University. The pattern of asymmetrical flow produced by flow entering the flume floor from the upper right is reflected in the pattern of bed load, which is easily seen through the relatively clear water. BN-48658

Model results.—The first series of tests used 0.64-cm (1/4-inch) mesh hardware cloth "fences" or porous dikes placed in the alluvium with their tops at the desired bed elevation, assuming the bed steepens as measured, with a transition from the 1-percent natural slope flat bed to the 3-percent slope 1:10 floor of the flume. These fences were placed in equally spaced pairs at a 45° horizontal angle to the centerline, with each pair allowing a 5-m (15-ft) (prototype) open space in the channel center.

These fences or porous dikes did help to center the flow, but the control was inefficient. A centered bar developed, and the thalweg that developed during recession was to one side of this bar. Also, a slight wave formed on either side within the flume probably due to the contraction around the center bar. The fences proved inadequate to control the sand elevation without more positive control of the water.

The next series of tests used metal sheets acting as low, impervious dikes. In contrast to the porous sand dikes, these were placed to model a 0.3-m (1-ft) extension above expected minimum bed elevation, and the dike tops maintained a 1:10 cross-channel slope and a 1-percent upstream slope. These dikes provided positive control of the flow as would be expected; however, small scour areas formed at the center end of the dikes during recession, causing low flows to favor a position at one side of the open center section. Some scour behind these impervious dikes was also noted in every test.

The last series of tests used porous dikes (0.64-cm) (1/4-inch) (hardware cloth) on the same pattern as the impermeable dikes tested previously. These dikes are distinguished from the fences previously described in that they are higher and act to direct the flow rather than attempt to stabilize the sand bar location alone. This arrangement appeared to be a satisfactory method of control, with a minimum amount of scour and no appreciable asymmetry at the flume entrance, as illustrated in figure 30.

Prototype trials.—Existing Walnut Gulch flume No. 3, code No. 63.003, 12.2 m (40 ft) wide, was used as a prototype test site, and porous dikes constructed of surplus aircraft landing mat were installed early in the summer of 1971. One previously installed dike pair was incorporated into the arrangement as the upstream pair. The resulting pattern consisted of two pairs of dikes, at approximately a 45° angle to the centerline, with a 4.9-m (16-ft) center opening and the upstream pair of dikes at approximately a 30° angle to the centerline. Figure 31 shows the condition of the alluvium and dikes after a large flow at this site soon after dike installation. Several conclusions from this first prototype test were:

a) Scour at the higher flows requires strong supports to prevent possible overturning of the dikes. Also, the dikes may accelerate scour and undermine the supports.

b) Trash in the flow makes it imperative that the mesh openings be larger than 2.5 cm (1 inch) and the fences be alined no more than 30° to centerline.

c) Sand replaced around the fences after installation must be carefully compacted.

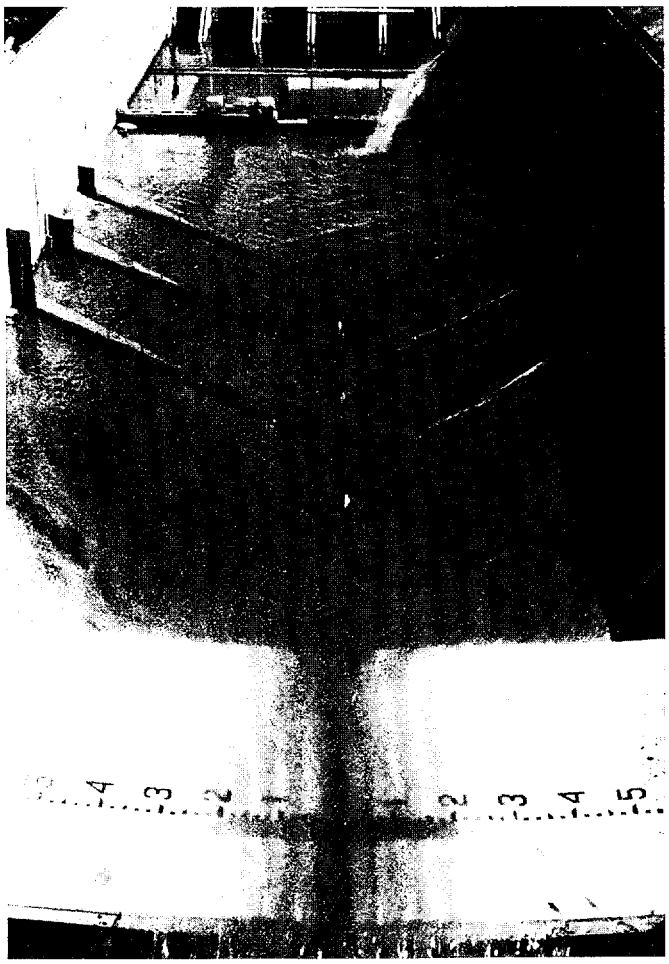


FIGURE 30.—The 1:5 model was used to develop the porous control dikes shown here. The porous dikes are effective in preventing asymmetry upstream of the flume entrance. BN-48659

FIGURE 31.—Prototype control dikes at flume 63.003 after large storm flow in early dike evaluation at Walnut Gulch Watershed. BN-48660

Porous flow control dikes at an angle of 30° to the flow have been placed above all but the smallest (site 63.004) Walnut Gulch flumes. As a result, maximum observed flow asymmetry has been limited to approximately 10 percent occurring at the widest flume (63.001). Figure 32 shows the controlled alluvial channel above flume 63.011. Flume 63.006 was left uncontrolled to obtain additional data on hydraulic conditions in asymmetrical flow into the flume. Control dikes will be installed there when experimentation is completed.

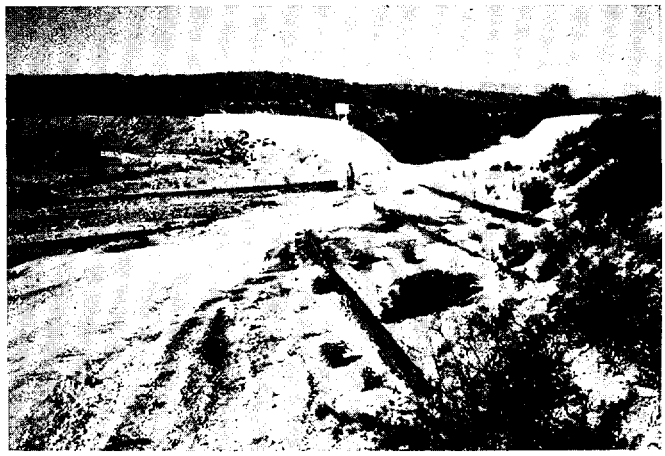


FIGURE 32.—Control dikes in place above flume 63.011, Walnut Gulch Watershed. BN-48661

Flows to date have shown little or no tendency to undercut or modify the dikes' performance. Exceptions have occurred at flume 63.001 where incoming flows were severely off center (an additional dike was installed), and at flume 63.002 where considerable damage occurred to some portions of the dikes and large holes were scoured downstream of the dikes after the large (1.5-m, or 5-ft peak depth) flow of July 17, 1975. Periodically, debris needs to be cleaned from dike openings to prevent excessive hydraulic roughness at the dike locations. Upstream dikes, which have the largest hydraulic forces acting upon them, must be anchored against overturning because the larger flows cause the bed to become fluid to a greater depth, thus maximizing the hydraulic overturning pressure.

Since many years of early records at Walnut Gulch include flows with asymmetries ranging in extent from mild to severe, a method was devised to estimate discharge from such biased depth readings, based on the Colorado State University model tests. The operating assumption is that the area of flow at the measuring point for an asymmetrical cross section is roughly equal to the area for the discharge flowing symmetrically.

Figure 33 indicates that the assumption of conservation of area under asymmetry is apparently reasonable, based on the model test results at Colorado State University. Flow depths across the flow for asymmetrical flow conditions in the model were taken by a series of manometers during tests that imitated asymmetrical entrance into the flume.

For many flows at Walnut Gulch, there were observer notes giving elevations of each edge of the flow at the measuring point, in addition to the stilling well record of the mean depth over the stilling well intake plate. From this information and the study of typical asymmetrical cross section shapes from the model tests (fig. 34), the cross section areas were estimated from records and observer notes. The equivalent uniform flow depth may then be calculated from this area, and the rating is then calculated for that depth at that particular flume.

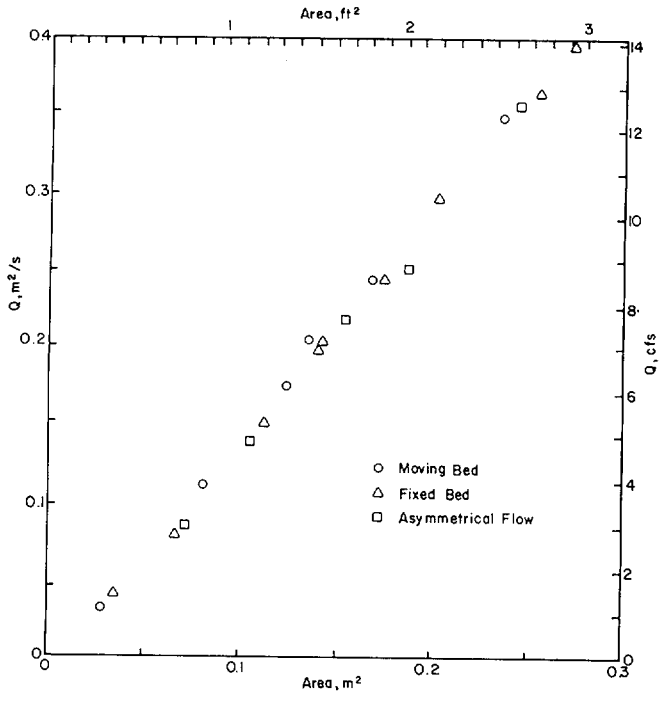


FIGURE 33.—Discharge-area relations; symmetrical and asymmetrical conditions.

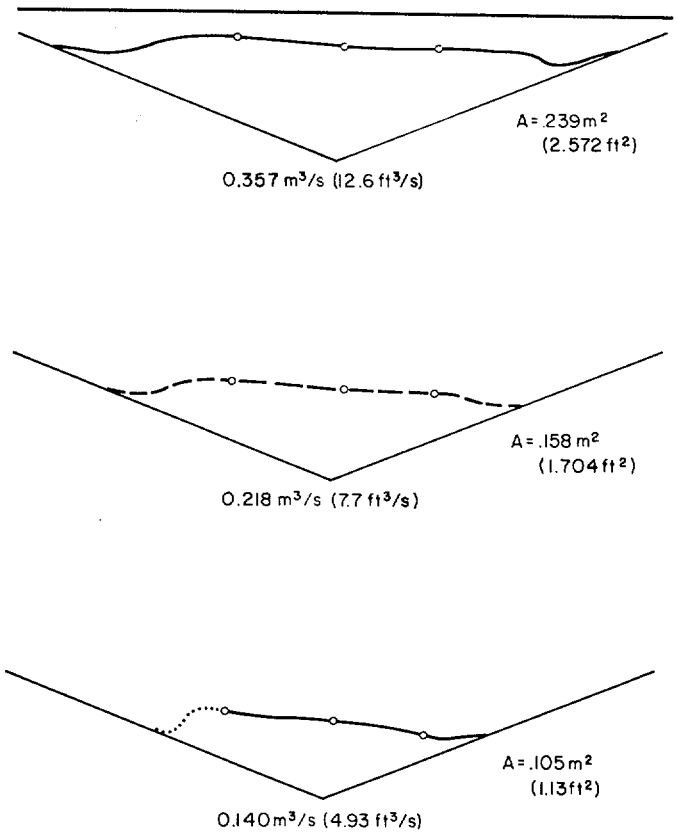


FIGURE 34.—Cross-section shapes at measuring section in Colorado State University 1:5 model flume. (Drawings not to scale.)

Design Modifications

Use and observation of the Walnut Gulch flumes to date have indicated some modifications and design limits for better performance.

The original effort in designing and testing these flumes emphasized their ability to pass and measure the peak flows of large floods. Indeed, the experience in 1954 with the first flumes used in the Walnut Gulch watershed was with a series of unusually large and, in some cases, disastrous flash floods. As a result, flows that were confined to the portion of the flume below the 1:1 sidewalls, called the floor region, were not given much consideration. Experience since 1954, as illustrated in figure 10, has revealed that flow depth distribution places the overwhelming majority of flows in the floor region of most of the flumes. Mean flume depth could be increased by a narrowing of flume geometry, which would force a significant contraction from the channel shape. This might, however, increase low-flow upstream control problems discussed above, and would require more attention to entrance transition conditions.

As originally designed, the Walnut Gulch flumes exhibit the characteristic of a shifting location of critical section as flow depth increases above the floor-wall intersection. At some flow depth above the intersection, control is expected to pass from the floor slope to the wall contraction. The depth at which this occurs depends on the particular flume geometry, as well as local upstream conditions. Unfortunately, the large width-to-length ratios of the larger Walnut Gulch flumes make wall control poor, unless flow depths are well above floor-wall intersection height. At the great widths and velocity of flows, with the relatively shallow depths encountered, contraction at the walls cannot change conditions at the center of the flume within the relatively short flow length of the flume.

Only a small number of flows have occurred that were deep enough to submerge the 1:1 flume sidewall, and no observations or measurements have been obtained to indicate precisely when and if control is exerted on the flow by the sidewall contractions. Figure 35, for example, taken at flume 6, seems to indicate that the drawdown at the curved wall is local and does not appear to be affecting flow in the flume center. "Humped" water surface cross sections, such as shown in figure 14, plus similar ones noted in field observation may reflect this condition.

Curved wall contractions or other flow redirection at higher flows and approach velocities typically generate surface waves (Wilm et al. *1938*), which would affect flow within a supercritical flume; however, model tests and field observations indicate that channel conditions at higher flows usually produce relatively lower velocities at the channel sides. In fact, large standing waves, indicative of local supercritical flow conditions, are often observed in the channel thalweg. Under these conditions, a flume should not contract the natural channel width excessively, since it could generate waves affecting the flume measurement.

These observations indicate that the wall contraction (curved) region should be located to produce the critical flow control section at the same place that the sloping floor (throat) of the flume begins, so that the control section location will not change at increasing discharge. These observations also indicate careful judgment be exercised in extent of contraction used, with consideration given to length-to-width ratio of flumes. For very wide channels, it seems reasonable not to expect control from wall contractions.

These considerations were incorporated in the design of the Santa Rita flume, developed primarily for somewhat smaller capacity channels, which are described in the following chapter.

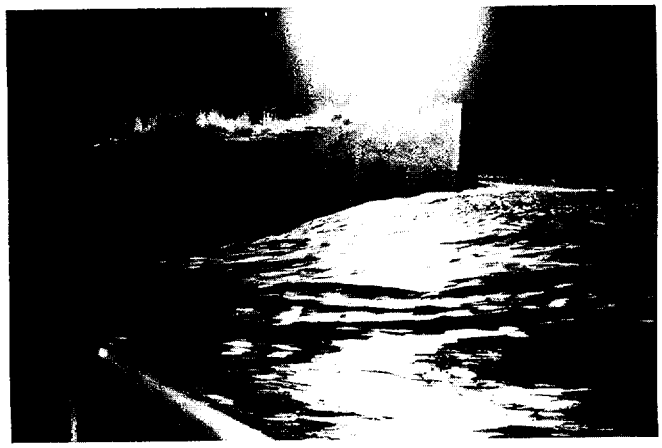


FIGURE 35.—Photograph of flow at night through Walnut Gulch Flume 63.006 (date unknown). BN-48662

The previously described difficulties with weirs and venturi flumes and favorable experience with the Walnut Gulch supercritical flume in measuring sediment-laden flow prompted the development of an improved design for a supercritical flow flume. This flume was intended for use in small channel flow measurement, generally less than 4 m^3/s (100 ft^3s); however, there seems to be no reason for limiting the size of the flume as long as certain proportions are maintained. The basic features and geometry of this flume are shown in figure 36. The name, Santa Rita, comes from the experimental area, south of Tucson, where the flume was first extensively used.

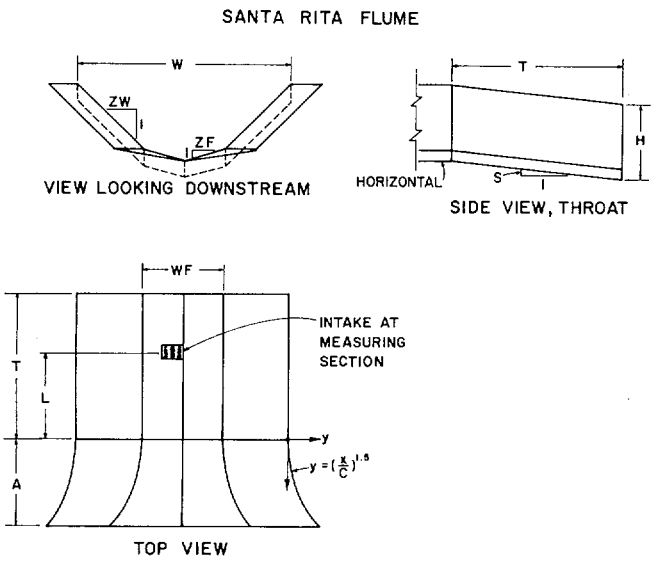


FIGURE 36.—Generalized design features of the Santa Rita supercritical flume.

The flume design incorporates two improvements over the original Walnut Gulch flume. The slope of the floor breaks at the entrance to the throat defined by the walls, rather than at the entrance to the curved approach. No movement of the measuring section should occur, therefore, for flows over the entire measurable range. Also, curvature of the approach wall has been reduced somewhat to decrease the tendency of waves to develop in the throat when flow direction changes too rapidly at the entrance walls for high entrance velocities. The length of the throat of such a flume is arbitrary, although the designs illustrated here are intended to insure that the supercritical drawdown profile is well developed upstream from the measuring section for the largest flow expected. Another important design consideration is that the distance below the measuring point to the flume exist be longer than the deepest flow expected. This is necessary to prevent modification of the hydrostatic pressure distribution at the measuring section by the downstream overfall. As a rule of thumb, the measuring section should be at approximately two maximum depths downstream of the critical section, and the flume exit should be at least one and one-half depths beyond the measuring section.

One flume was rated at full scale in the 2.44-m (8-ft) wide tilting recirculating flume at the Engineering Research Center, Colorado State University. The data are shown in figure 37, along with computer simulation using both Manning's and Chezy's roughness relationships.

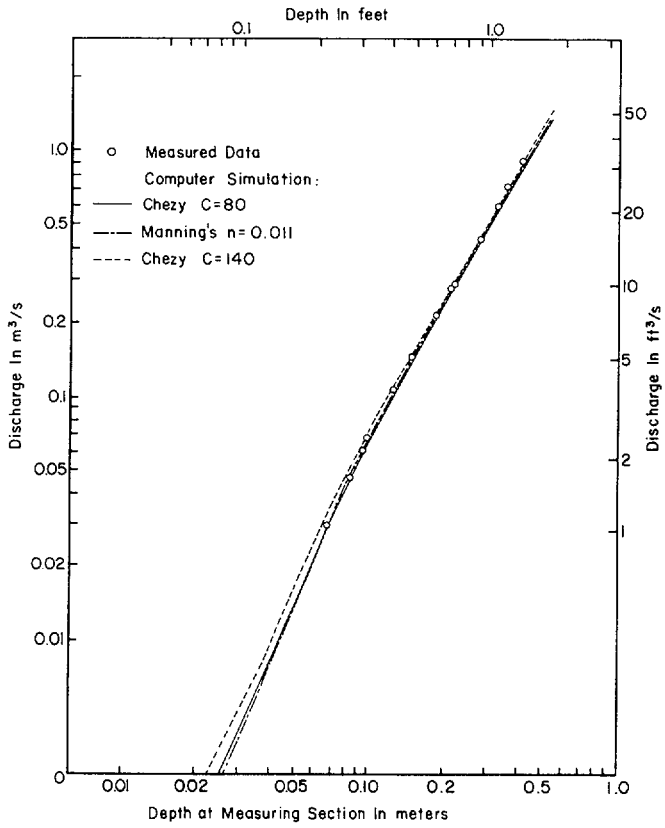


FIGURE 37.—Rating by hydraulic tests and by computer simulation for a 2.1 m^3/s (75-ft^3/s) Santa Rita flume.

Dimensions and ratings for several sizes of the Santa Rita flume are presented in Appendix C in both English and metric dimensions. The ratings assume that the Santa Rita flumes with capacity smaller than 5 m^3/s (100 ft^3/s) are made of metal, and all others are assumed made of concrete. Our experience suggests that after construction, a single rating point should be taken during low flow by volumetric measurement of discharge at the downstream overfall. At low flows, if flow at the measuring point is nearly at normal depth, the

rating gives a value of roughness for use in the computer rating. Then, a more accurate rating can be simulated. Small errors in estimation of flume roughness do not significantly affect higher flow ratings.

Some judgment is appropriate in designing the length and expansion of the approach. This depends on the width and slope of the stream channel at the site. Suggested approach curvatures and lengths are shown. Since the approach floor has no longitudinal slope, the alluvium will form a bar on the approach floor, which will taper out before the critical section where flow accelerates. This has been observed to occur at all installations to date.

The first such flume was made of concrete, using gunite and metal screeds to control the finished shape; however, most of the other early flumes were made of steel, constructed in the shop to be moved to the measurement site. Appendix D shows typical fabrication plans for a metal Santa Rita flume.

If $ZF \rightarrow \infty$ (fig. 36) or the floor section is flat, the Santa Rita flume becomes trapezoidal, recalling the early design of Robinson (*1961*). In this version, the stilling well intake is located in the sloping wall adjacent to the floor intersection. This design, while sacrificing some lower flow sensitivity (depending on the floor width), has the advantage of a stilling well intake that traps far less sediment than the flumes with a *V*-shaped floor, where much of bedload flows directly over the intake slots. The trapezoidal shape, with $ZF \rightarrow \infty$, cannot be simulated by the computer program in appendix A without minor modifications.

A method for handling the high exit velocities needs to be devised. A rock-lined (preferably cement-mortared) depression in the bed, into which the flume can discharge, has worked well at one location. In other sites in the Santa Rita range, the channel is sufficiently rocky that artificial downstream protection has not been required. Natural pools developed and apparently became stabilized, providing an energy dissipation pool during flow periods. Several large Santa Rita flumes are presently (1979) being installed in a watershed in Mississippi. These are located in erodible sand material; therefore, downstream scour protection is being designed by model testing.

In general, channels where sediment and stream velocities require supercritical flumes have relatively steep channel slopes and do not have tailwater inundation problems. After flume installation, a transition period of upstream bed accretion and limited downstream degradation should be expected. The original Walnut Gulch flumes were installed nearly a meter above the channel bottom; however, from experience gained since installation, this now seems unnecessary. Considerable upstream accretion has taken place at each flume over a period of several years, and the drop to streambed at the downstream edge of the flume is now a potential safety hazard.

Weir Extension Flume

One method developed to reclaim measurement stations where weirs are being inundated with bedload sediment is construction of a supercritical V-shape or triangular flume just below the weir. The former weir becomes the upstream cutoff wall for the flume, as shown in figure 38. As in the Santa Rita flume, the length of the measuring section should lie at least one and one-half depths upstream of the flume exit to reduce the effects of the free overfall on the pressure distribution at the measuring section.

Appendix C includes dimensions and rating table for two sample triangular flumes. Local entrance flow conditions will determine whether it is necessary to construct approach walls for a site where a weir is being converted.

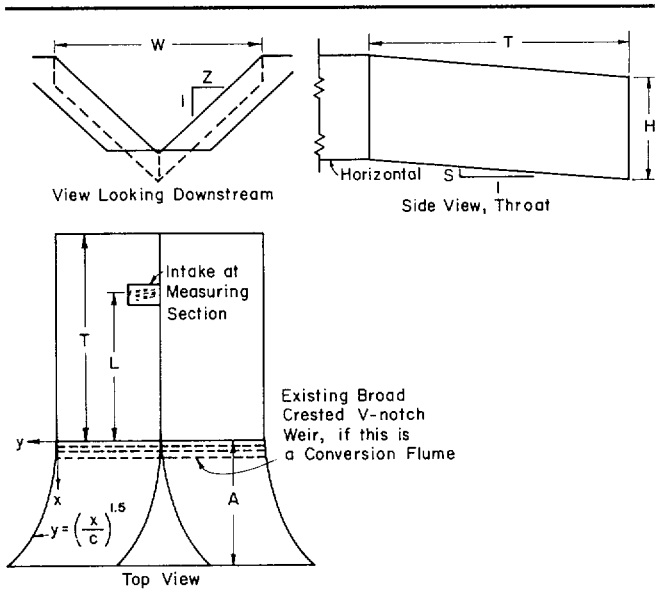


FIGURE 38.—A supercritical flume design which uses an existing V-notch weir for an upstream cutoff wall.

Sensing Water Level in Heavy Sediment Conditions

Problems arise in sensing water level in a supercritical flume, not due to the velocity of the flow, but due to the nature of the flow. These are (a) rapid rise in flow depths, (b) high sediment loads, and (c) long periods of no flow. These conditions can occur also in subcritical flow flumes.

The first and third conditions constitute a hydraulic problem in stilling well design; for each event, the stilling well must first be filled from water flowing through the flume before flow depths can be recorded. Times to peak flow of only a few minutes are not uncommon even for flows of more than 100 m^3/s, and the recorded level will lag the flume level by as much as several minutes. Large sediment traps need to be provided under the intake in the flume to deal with the high sediment leads. Records obtained from the stilling wells originally constructed in the Walnut Gulch flumes are affected by all of the above conditions. Flows are often obscured by sand deposition in the stilling well after only an hour or two of flow. Figure 39 shows the extent of sand accumulation in the original intakes. This typically must be removed after each flow event.

An ideal water-level sensor would operate without taking on any volume of water from the low. The nitrogen bubble gage, with a servomanometer follower, has this feature; however, there are serious problems in locating a bubble orifice in the boundary layer of a high-velocity heavily sediment-laden flow such that it will (a) not disturb the flow so as to record a biased value of the local hydrostatic pressure, and (b) not allow sediment to enter and clog the gasline. It remains to be demonstrated how this can be accomplished for reliable operation in field situations. In order to sense low flows, the bubble outlet must be located at the center notch of a V-shaped floor, where exposure to bedload sediment is most extensive. An experimental installation of a bubbler gage in a Santa Rita flume is presently (1979) being evaluated. The outlet tube for nitrogen is installed near the notch of the floor but at a slight upward angle. The resultant tube opening, cut flush with the flume floor, presents an elongated elliptical orifice to the floor.

Two electronic depth-sensing devices, a sonar and pressure transducer, have been used at flume 63.006 to measure stage. Both have the ideal characteristic of not taking any volume of water from the flow and, thus, have no intake and stilling well problems.

The sonar transducer functions by measuring the time that a high-frequency audible sound wave travels to a reflecting surface and returns to the transducer. Care must be used in positioning the transducer with respect to the flow surface, but, in general, the instrument has proved to be quite reliable and durable. It has given resolutions of stage sufficient to plot all the flow profiles illustrated in figures 17 through 20.

The pressure transducers used have had a measuring surface about 2 cm in diameter on which the strain is measured flush with the flume floor. These are commercially available models. Pressure transducers with small tubes leading to the measuring surface were not used because sediment could plug the tubes, and air could easily be trapped inside. These transducers measure the stage very accurately when functioning; however, the types used to date have not proved durable for the rigorous demands of stage measurement in the field conditions. Damage of the measuring surfaces occurred quite readily, and shifts in the zero calibration were a common

FIGURE 39.—Technicians removing sand from beneath the intake plate at Walnut Gulch Flume 6. Sand completely seals the intakes, often after a few hours of flow. BN-48663

occurrence. With improved design of the mounting (later pressure transducers were mounted inside a small intake box to protect them) and associated electronics, a pressure transducer may prove to be most useful for fixed-position measurement of stages.

Design of Stilling Well Intake for Sediment Conditions

The most common water-level sensor is a float located in a stilling well. An optimum design would (a) take on a minimum volume of water (considering an intial dry or unfilled system), (b) respond quickly to water surface elevation changes, and (c) trap all sediment (other than wash load) prior to entering the stilling well. The first two requirements are compatible, but the large amount of sediment entering the intakes obviates the desired condition of minimum volume of water in the stilling well system. In addition, reduction of sediment entry by using a small surface area for intake slots reduces the dynamic response of the stilling well considerably. The effect of differences in fluid density between flow and stilling well on the recording accuracy has been discussed previously by Ree and Garton (*1971*).

Good water-level measurements in heavily sediment-laden flows are difficult to obtain because the lowest point, where intakes should be located, is also the point most exposed to the bedload movement. A reasonable compromise design would include the following:

1. A minimum diameter of stilling well, and, thus, a narrow but possibly deeper float, so that the float volume is large enough to overcome drag within the recorder mechanism.

2. A pipe connecting the intake with the stilling well that is as short as practical and of a diameter that balances head loss with minimization of volume. This pipe should also have no interior high points where air can be trapped.

3. As much volume for deposited sand storage below the intake slots as practicable. The expected flow duration, bedload, and intervals between servicing should be considered in sizing.

4. A slot area that allows water entry at a tolerable head loss for no more than the expected average rate of rise.

In addition, the intake should be constructed so as to allow convenient servicing and cleanout.

Figure 40 is a sketch of the generalized design being used for several supercritical flumes. Stilling wells are used that have a 15-cm (6-inch) or 20-cm (8-inch) diameter. This restricts the float size and may prevent use of some of the larger water-level recorders. The float counterweight may have to be located outside the stilling well.

Tapered slots are milled into the intake plate perpendicular to the flow, with larger slot width on the inside. This helps reduce lodging of particles in the slots during inflow. Provisions are made at each installation to facilitate sediment trap cleaning and flushing. Little sediment deposition in the stilling well bottom has been observed, but cleanout must be provided.

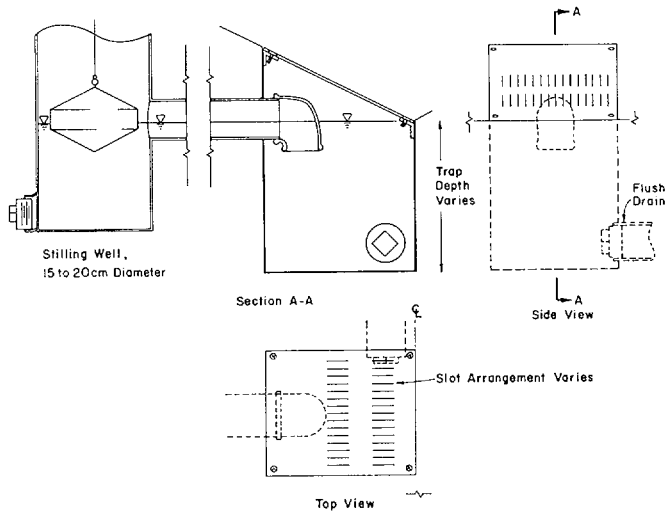


FIGURE 40.—Example design of stilling well entrance for trapping sediment during ephemeral flows.

Analysis of Lag in Stilling Well Records

Stilling well systems may exhibit considerable lag in the recorded depth as compared with actual flow depth in the flume when rapid rates of rise occur. This is caused by both the volume of the intake system that must be filled and the resistance to flow through the intake plate orifices. For example, the original Walnut Gulch intakes with 0.9-m-diameter stilling wells, 0.61-m × 3-m (2-ft × 10-ft) intake boxes, and large connecting tunnels, altogether comprise several cubic meters of water (and sediment) storage volume.

Flow records can easily be analyzed to estimate actual hydrograph rise, which may be necessary if intake slot area is restricted to prevent excessive sand influx. Consider the intake slot as a simple orifice. The equation for head loss through the orifice is written as:

$$\triangle h = h_f - h_o = \frac{1}{C_s^2}\frac{V_o^2}{2g} \qquad (22)$$

where $\triangle h$ is the head difference across the orifice
h_f is the head in the flume, L,
h_o is the head at the orifice exit, L,
C_s is the orifice coefficient,
V_o is the velocity through the orifice, L/T,
g is the gravitational acceleration, L/T^2.

We can also write continuity equations as follows:

$$V_o A_o = q_o \tag{23}$$

$$q_o = A_w \frac{dh_w}{dt} \tag{24}$$

in which A_o is the orifice area, L^2,
q_o is the flow through the orifice, L^2/T,
A_w is the surface area at recorded depth, L^2,
h_w is the stilling well depth, L, and
t is time.

Two cases may be distinguished. In the first case, the recorded depth h_w is less than the notch depth h_n, such that $h_o = h_n$ and equation 22 becomes

$$h_f - h_n = \frac{1}{2g}\left(\frac{V_o}{C_s}\right)^2. \tag{25}$$

Combining equations 23 and 24 into 25, and considering the change over time step $\triangle t$, we have

$$h_f(t) = h_n + \frac{1}{2g}\,\frac{A_w}{A_o}\,\frac{[h_w(t) - h_w(t-\triangle t)]^2}{C_s \triangle t}. \tag{26}$$

In the second case, $h_w > h_n$ and $h_o = h_w$ so that

$$h_f(t) = \frac{h_w(t) + h_w(t+\triangle t)}{2} + \frac{1}{2g}\,\frac{A_w}{A_o}\,\frac{[h_w(t) - h_w(t-\triangle t)]^2}{C_s \triangle t}. \tag{27}$$

Equations 26 and 27 are applied to the recorded sequence of flow depths of h_w divided into several $\triangle t$ elements to calculate $h_f(t)$. A table of values for $A_w(h_w)$ describes the inlet system. One may also use equations 26 and 27 to solve for the entrance coefficient C_s if one or two observations of actual values of h_f and h_w are available. C_s may be estimated from hydraulic handbooks and is about 0.65.

Except in truly exceptional cases, lag correction for head loss through the intake plate and storage delay in the intake system is only necessary on the rising portion of a rapidly rising hydrograph, and thus will ordinarily be necessary only up to the point where $h_f \leq h_w$.

The position of a flume in a channel is important. Among the items to be considered are (1) channel width, (2) channel depth, (3) channel slope, (4) channel sinuosity, and (5) foundation material. Following selection of the drainage area desired, the site must be selected and the measuring device designed to fit the limitations of the stream site. For example, the flume cross sections at the entrance and at the measuring section must fit reasonably in the channel.

As previously discussed, some flow contraction is useful for control, but should be limited so that flow is not retarded to the extent that excessive ponding and deposition occur.

Geologic conditions at the measuring site are very important. The presence or absence of foundation material that can support the structure and its dynamic loads greatly affects the structural design and the costs. Where good foundation material is close to the channel surface, construction is relatively simple.

The presence of geologically resistant material also eliminates the need for scour protection downstream of the flume; thus, the scour pool can form in the resistant material without constructing an energy dissipator. At one location where the geologic material was insufficient to resist the anticipated scour, construction of an energy dissipator from reinforced concrete doubled the cost of the measuring station.

Finally, the presence of cohesive material at the measuring site enables the structure to intercept flow that might exist along the axis of the stream, below the surface of the channel alluvium. It is then possible to measure the total runoff or yield of the watershed with the exception of losses to deep ground water. Locations where flumes have been "keyed" into the bedrock assure minimum seepage under the structure.

When locating a flume, care must also be taken to avoid bends in the channel immediately upstream. In straight channel sections, the flow can be expected to enter the flume fairly symmetrically. Our experience suggests that the measuring station be located at least 10 flow widths downstream from channel bends.

Numerous construction techniques can be used for the flumes, depending upon the foundation, structural, and economic conditions prevailing. On small stations, it has been most satisfactory to construct the measuring device from sheet metal. In these instances, the flume was shop-assembled on a rigid rectangular wide-flange steel support frame, which was subsequently transported to the field site and positioned into the channel with a backhoe or small crane. The upstream end of the wide-flange support frame was then set on a concrete footing. The downstream support was made of large, threaded bolts anchored in concrete. The upstream footing serves as a cutoff, and the bolts in the downstream footing enable accurate leveling to obtain the desired slope. Small flumes such as this can also be made of reinforced concrete or gunite.

Concrete has proved to be an economical construction material on many streams because of the availability of suitable aggregate in the streambeds. The bed material often has a size distribution that is desirable for concrete aggregate and does not require screening and washing; thus, concrete costs can be low.

With the low costs of concrete construction, the larger measuring devices have been made exclusively of concrete in such a way as to minimize forming costs. The earliest stations consisted of concrete cells with support walls 3 m (9 ft) on centers and 30 cm (12 inches) thick rising from the bedrock to a point about 30 cm (12 inches) below finish grade. The honeycomb-type substructure cells were backfilled with the channel alluvium to provide additional structural support. The floor and walls were poured over the substructure cells. On some of the more recently constructed stations, upstream and downstream cutoff walls were connected by longitudinal and transverse walls to form a T-beam floor support. The openings were backfilled and compacted for additional support.

The earliest large measuring stations were made by pour forming the 1:1 sidewalls and the entrance parabolic sections. Control was obtained by stretching piano wire between extensions of the slope above and below the final grades. All layout control was made to ± 3 mm (± 0.01 ft) in the x-y-z planes. Forms were reused numerous times. The last few flumes were constructed by "shooting" gunite into place. Steel screeds were fixed at the finish plane slope, and the gunite concrete was then applied and struck off to the finish grades. This technique proved superior to ordinary forming methods.

On the floor sections, screeds were installed at the final grade, and the concrete pours were finished to the grade of the screeds. Control accuracy of ± 3 mm (± 0.01 ft) was maintained in the x-y-z planes.

A series of construction photographs for the large flume at the outlet of Walnut Gulch are shown in figures 41 to 43.

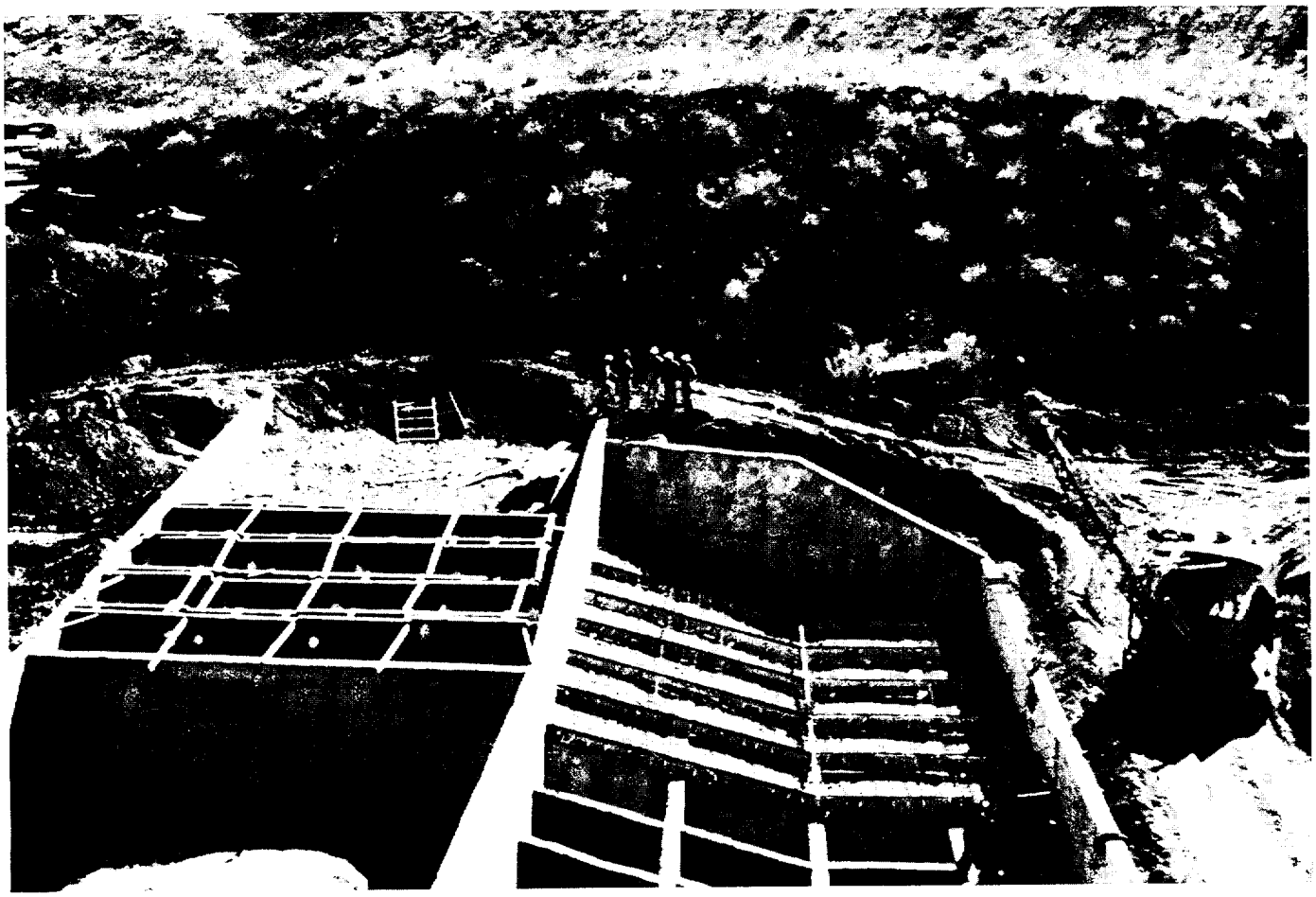


FIGURE 41.—Flume construction in early stages with form construction just beginning. BN-48664

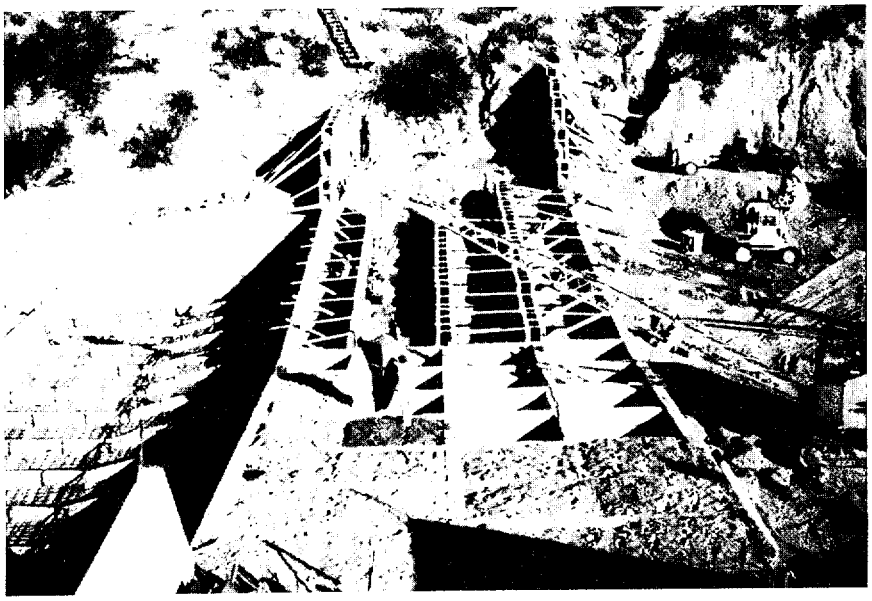


FIGURE 42.—Flume construction showing honeycomb floor support structure. Foreground honeycombs have been backfilled with gravel in preparation for floor pouring. BN-48665

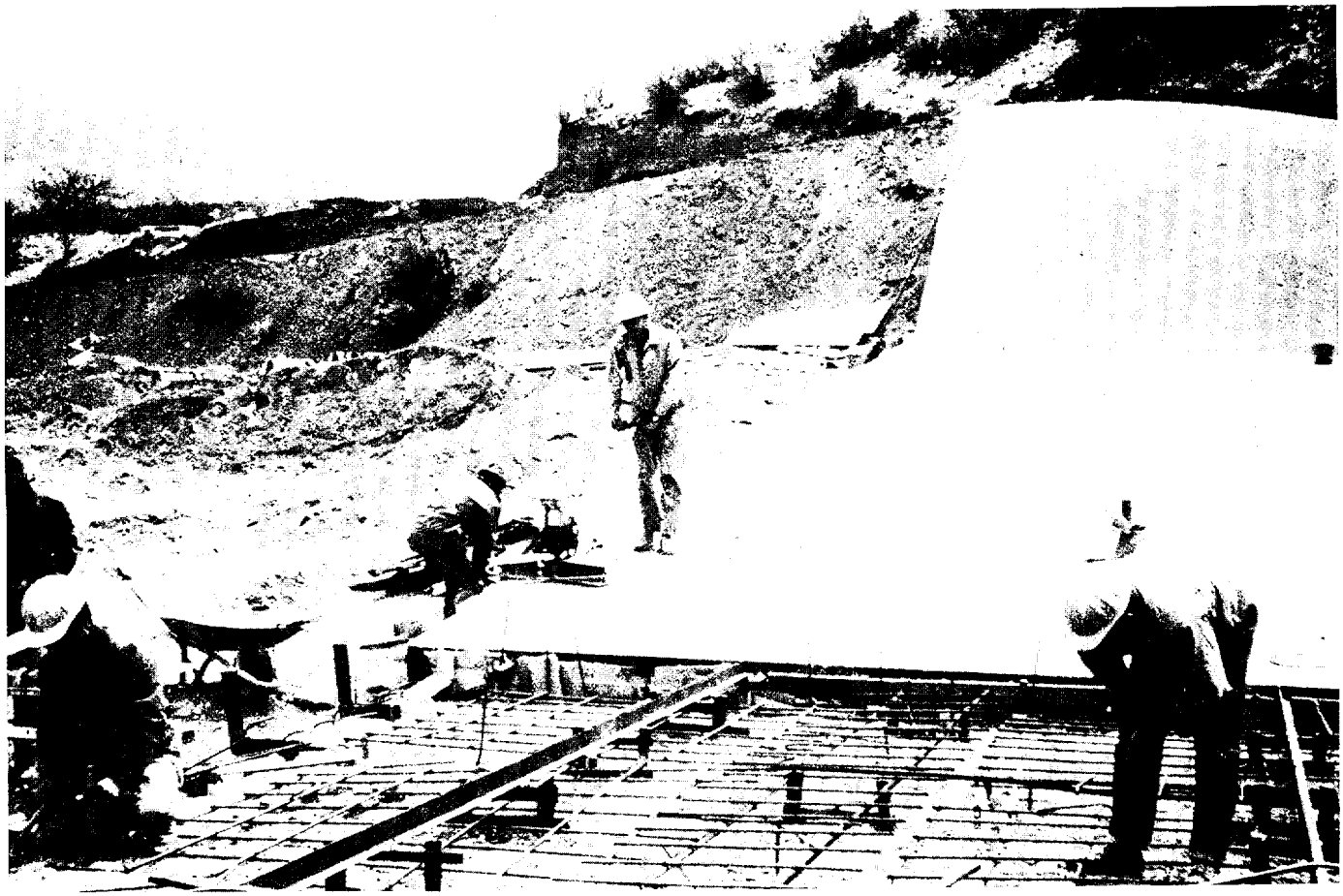


FIGURE 43.—Flume construction showing pouring of the floor area with screed controls in the foreground.

BN-48666

Summary

No flume can be expected to solve hydraulic measurement problems for all conditions at any one site, much less to be ideal for a range of flow conditions at different locations. Nevertheless, for many heavily sediment-laden flows at high velocities, the flumes described in this bulletin have proved superior to presently available alternatives. For many conditions supercritical flow is necessary to transport the sediment load through any measuring device, and no other commonly used flume would be a suitable measuring facility.

Flumes made exceptionally wide to fit the geometry of channels require extra control measures, such as the porous control dikes in the approach channel described in this bulletin, to insure good flow control. Where the length of the flume is sufficiently greater than its width, extra control measures are not necessary. Ratings for several flume sizes are given in Appendix C.

Results of laboratory model and computer simulation experiments reported here indicate that Santa Rita flumes constructed with a wide variety of proportions may be rated with reasonable accuracy by computer simulation. The computer program is listed in Appendix A.

Extra care in design and maintenance is required to insure that the high sediment concentrations associated with high-velocity ephemeral flows do not interfere with measurement of depths by conventional float methods. Corrections for lag between the flume and stilling well water levels are necessary if the stage change is rapid. Use of bubble-type water level sensors may be a practical method, although present experience is insufficient to evaluate this technique as applied to the special conditions of high velocities and high sediment concentrations. Sonic transducers are useful devices for sensing sediment-laden water level where power is available, but require a permanent mounting above the flow.

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Appendix A

Computer program to determine measuring section depth (y_m) for any given discharge, q, for a supercritical depth flume.

```
      PROGRAM STARITA (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      DIMENSION H(11),X(11),A(11),W(11),V(11),E(11),R(11),SF(11),Z(11),
     1TITLE(8)
      COMMON ZF,WF,ITZ,ZW
    1 READ(5,101) TITLE
    2 READ(5,106) WF,ZF,DX,EPS,OPT,FR,SZ,XMS
C  * WF = WIDTH OF FLOOR SECTION
C  * ZF = LATERAL SLOPE OF FLUME FLOOR SECTION =COTAN OF ANGLE TO HORIZON
C  * DX = ROUTING INTERVAL DISTANCE IN FT.
C  * EPS = CONVERGENCE RELATIVE ERRO CRITERION
C  * OPT = PRINT OPTION, PRINTS RESULTS AT EACH SECTION AND Q IF .GT. 0.
C  * FR = FROUDE NO. AT UPSTREAM SECTION
C  * SZ = LONGITUDINAL FLUME SLOPE = TAN OF ANGLE TO HORIZON
C  * XMS = DISTANCE FROM UPSTREAM CONTROL SECTION TO MEASURING SECTION
      READ(5,106) QMAX,DLQ,ALPH,CRF,ER,EDDY,HMX,ZW
C  * QMAX = MAX. DISCHARGE IN CFS FOR WHICH RATING IS DESIRED
C  * DLQ IS STARTING MINIMUM VALUE OF DISCHARGE IN RATING
C  * ALPH IS ENERGY COEFFICIENT FOR OPEN CHANNEL FLOW
C  * CRF IS ROUGHNESS COEFFICIENT = 1.49/N FOR MANNING LAW IN FT.-SEC. SYSTEM
C                                 = CHEZY C IN FT.-SEC. SYSTEM FOR ER=.5
C  *                 METRIC CHEZY CRF = 0.552*(ENGLISH CHEZY)
C  *                 METRIC MANNING CRF = 1./MANNING N
C  * ER IS ROUGHNESS LAW EXPONENT ON HYDRAULIC RADIUS, R.
C  * EDDY IS EDDY LOSS COEFFICIENT
C  * HMX = MAXIMUM DEPTH AT CONTROL TO BE CONSIDERED. MUST BE GT. DEPTH
C        CORRESPONDING TO QMAX
C  * ZW = SLOPE OF FLUME SIDEWALL = COTAN OF ANGLE TO HORIZON
      READ(5,110) IPW,IMET
C  * IMET = 0 FOR FT. - SEC. SYSTEM, IMET GT. 0. FOR METER - SEC. SYSTEM
C  *** IPW IS PARAM. FOR FLOOR AND WALL CONFIG.(=1) OR FLOOR ONLY(=0).
      WRITE(6,102)
      WRITE(6,103) TITLE
      IF(IMET) 23,23,21
   21 G = 9.80665
      UNITL = 3HM.
      UNITQ = 3HCMS
      WRITE(6,115)
      GO TO 24
   23 G = 32.172
      UNITL = 3HFT.
      UNITQ = 3HCFS
      WRITE(6,116)
   24 GG = G*2.
      WRITE(6,114) SZ,XMS
      WRITE(6,104) ALPH,CRF,EDDY,ZF,WF,QMAX,DLQ,EPS,DX,FR,ER
      IF(IPW.GT.0) WRITE(6,117) ZW
      IFLAG = 0
      XCR = 0.
      CNM = 1./CRF/CRF
      WC1 = WF
      HC1 = 0.5*WC1/ZF
      AC1 = HC1*HC1*ZF
      QTR = SQRT(AC1**3*G/ALPH/WC1)
      WRITE(6,109) QTR,HC1
      Q = DLQ
      HH = (2.*ALPH/G*(Q/FR/ZF)**2)**0.2
    9 CONTINUE
    5 IF(Q.GT.QTR.AND.IPW.LT.1) GO TO 150
      CALL CRLOC(Q,QTR,XC1)
      IF(XC1.GT.XCR) XCR = XC1
C     ---- HH IS CRITICAL DEPTH AT XCR
C ******* FIND FLUME CRITICAL DEPTH AT CONTROL XCR ******
      ITZ = 1
      ITRC = 0
   10 IF(HH.LE.0.) HH = .001
      ITRC = ITRC + 1
```

Computer program to determine measuring section depth (y_m) for any given discharge, q, for a supercritical depth flume—Con.

```
      IF(ITRC.LT.25) GO TO 16
      WRITE(6,119) HH
      GO TO 200
   16 IF(HH - HMX) 12,12,11
   11 HH = HMX - .5
   12 IF(HH - HC1) 13,13,14
   13 ZZ = ZF
      GO TO 15
   14 ZZ = ZW
   15 CALL AREA(HH,XCR,AC,WC,HTR,RC,APH,RPH)
      DHY = AC/WC
      PC = Q/FR/SQRT(G/ALPH) - AC*SQRT(DHY)
      PCP = DHY**1.5*ZZ - 1.5*SQRT(DHY)*APH
      DC = PC/PCP
      HH = HH - DC
      IF(ABS(DC) - EPS) 18,10,10
   18 H(1) = HH
      CALL AREA(H(1),XCR,A(1),W(1),HTR,R(1),APH,RPH)
   25 CC = 1./CRF/SQRT(SZ)
      HS = 0.75*HH
C     ---- FIND NORMAL SUPERCRITICAL DEPTH, HS
      ITNC = 0
   26 CALL AREA(HS,XCR,AS,WS,HT,RS,APH,RPH)
      ITNC = ITNC + 1
      IF(ITNC - 20) 28,28,29
   29 WRITE(6,112) Q
      GO TO 100
   28 FS = AS*RS**ER - CC*Q
      FSP = APH*RS**ER + AS*ER/RS**(1.-ER)*RPH
      IF(ITNC.GE.10) WRITE(6,113) HS,FS,FSP
      TEP = FS/FSP
      HS = HS - TEP
      IF(HS.GE.HH) HS = .9*HH
      IF(HS.LE.0.) HS = 0.1
      IF(ABS(TEP) - EPS) 27,27,26
   27 WRITE(6,108) XCR,UNITL,HH,UNITL,HS,UNITL,ITRC
      FRN = Q/AS/SQRT(G*.5*AS/WS/ALPH)
      WRITE(6,118) FRN
   30 ITCR = 0
      V(1) = Q/A(1)
      Z(1) = 10. - SZ*XCR + H(1)
      E(1) = Z(1) + ALPH*V(1)**2/GG
      SF(1) = CNM*V(1)*V(1)/R(1)**(2.*ER)
      LDX = XCR/DX
      DX1 = DX*(LDX + 1) - XCR
      M = INT(XMS/DX) - LDX + 1
      IF(DX1 - .1) 31,31,33
   31 M = M - 1
      DX1 = DX + DX1
   33 X(1) = XCR
      TS = XMS - XCP
      DO 50 I = 2,M
      DIFF = 0.
      IF(I-2) 35,35,36
   35 DXX = DX1
      GO TO 37
   36 DXX = DX
C     ----- ESTIMATE DEPTH AT NEXT SECTION
   37 X(I) = X(I-1) + DXX
      DS = X(I) - XCR
      PR = 0.70 + 0.28*DS/TS
      P = PR + (1. - PR)*(1. - DXX/DX)
      H(I) = H(I-1)*P
      ITER = 0
      IIZ = 1
   40 ITER=ITER+1
```

Computer program to determine measuring section depth (y_m) for any given discharge, q, for a supercritical depth flume—Con.

```
      IF(ITER - 20) 44,44,99
   99 WRITE(6,111) Q
      GO TO 100
   44 HIM = 1.001*HS
      H(I) = AMAX1(HIM,H(I))
      H(I) = AMIN1(H(I),H(I-1))
      Z(I) = 10. - SZ*X(I) + H(I)
      CALL AREA(H(I),X(I),A(I),W(I),HTR,R(I),APH,RPH)
      IF(R(I)) 42,42,41
   41 IF(ITER-10) 43,43,42
   42 WRITE(6,105) X(I),H(I),UNITL,DIFF,ITER
   43 V(I)=Q/A(I)
      SF(I) = CNM*V(I)*V(I)/R(I)**(2.*ER)
      VP = -Q*APH/A(I)/A(I)
      E(I) = Z(I) + ALPH*V(I)*V(I)/GG
      F = E(I-1) - .5*DXX*(SF(I-1) + SF(I)) - EDDY*ALPH*(ABS(V(I)**2 - V
     1(I-1)**2))/GG   - E(I)
      FP = -.5*CNM*DXX*(2.*V(I)*VP/R(I)**(2.*ER) - V(I)*V(I)*2.*ER*RPH/R
     1(I)**(1.+2.*ER)) - ALPH*(1. + EDDY)*V(I)*VP/ G   - 1.
      DIFF = F/FP
      H(I) = H(I) - DIFF
      IF(ABS(DIFF) - EPS) 480,40,40
   61 XCK = XCK + 1.
      GO TO 10
  480 IF(OPT)50,50,49
   49 FRX = V(I)/SQRT(G*A(I)/W(I)/ALPH)
      IF(I.LT.M) WRITE(6,105) X(I),H(I),UNITL,FRX,DIFF,ITER
   50 CONTINUE
      FRX = V(M)/SQRT(G*A(M)/W(M)/ALPH)
      WRITE(6,107) Q,UNITQ,H(M),UNITL,FRX
  100 QL = Q
      B = ALOG10(Q) + 1.E-07
      IF(B)3,4,4
    3 J = INT(B)
      JCH = J-1
      GO TO 6
    4 JCH = INT(B)
    6 VMAN = B-FLOAT(JCH)
      DCQ = 1.0
      IF(VMAN.LT.0.301) DCQ = .2
      DQ = DCQ*10.0**(JCH)
      Q = Q + DQ
      IF(Q - QMAX) 9,9,7
    7 IF(QL-QMAX* .0001) 8,150,150
    8 Q = QMAX
      GO TO 9
  150 READ (5,110) ICONT
      IF(ICONT)200,200,1
  200 CONTINUE
  101 FORMAT(8A10)
  102 FORMAT(1H1,10X*THIS PROGRAM HYPOTHESIZES RATING CURVE FOR W R GWIN
     1N FLUME BY DIRECT STEP COMPUTATION OF FLOW PROFILE,*/9X,*ASSUMING
     2TRANSITION OF CRITICAL SECTION FOR FLOW ABOVE FLOOR WALL INTERSECT
     3ION*/10X*AS SPECIFIED BY SUBROUTINE CRLOC*/)
  103 FORMAT(1H0,20X,8A10/)
  104 FORMAT(1H ,5X*ALPHA = *F6.4,*, ROUGHNESS = *F8.3,*, EDDY LOSS RATI
     1O = *F6.4/10X* SIDE SLOPE OF FLOOR =1/*F4.1,*, FLOOR WIDTH = *F5.2
     2,*,  QMAX = *F10.3,*, DELTA Q = *F6.3/10X,*CONVERGENCE CRITERION =
     3 *F8.6,*, DELTA X ALONG FLOOR = *F6.2/15X,*F NO. AT FLOOR CONTROL
     415 *F8.5/20X,* HYDRAULIC EXPONENT IS *F10.6/)
  105 FORMAT(1H ,4X,*AT X= *F5.2,*, H= *F8.5,A3,*, FR= *F8.6,*, DIFF= *F
     18.6,*, AFTER *I3,* ITERATIONS*)
  106 FORMAT(8F10.0)
  107 FORMAT(1H ,5X,*FOR FLOW OF *F9.3,A3,* , DEPTH AT MEASURING SECTION
     1 IS *F7.4,A3,*, FR =*FR.6)
  108 FORMAT(1H0,20X,*AT *F5.2,A3,* FROM START OF FLUME, CRITICAL DEPTH
```

Computer program to determine measuring section depth (y_m)
for any given discharge, q, for a supercritical depth flume—Con.

```
     1IS *F6.4,A3,* , NORMAL DEPTH IS *F6.4,A3,* , ITCR =*I5)
  109 FORMAT(1H0,5X,*CALCULATED Q FOR TRANSITION FROM FLOOR TO WALL CONT
     1ROL IS *F8.3,*, TRANSITION DEPTH IS *F8.4/)
  110 FORMAT(2I5)
  111 FORMAT(* PASSED TO NEXT Q WHEN ITER G.T. 20 AT Q =*F12.4)
  112 FORMAT(*   NORM DEPTH NOT FOUND AT 20 ITER FOR Q = *F12.2)
  113 FORMAT(1H ,3(5X,F12.7))
  114 FORMAT(1H ,10X*FLUME SLOPE IS *F7.5,*  DIST. TO MEASURING SECTION
     1IS *F6.2)
  115 FORMAT(/40H ALL DIMENSIONS IN METERS, Q IN M**3/SEC/)
  116 FORMAT(/33H ALL DIMENSIONS IN FEET, Q IN CFS/)
  117 FORMAT(22H WALL SIDE SLOPE IS 1/,F7.4)
  118 FORMAT(*  NORMAL FLOW FROUDE NO. =*F8.6)
  119 FORMAT(50H CRITICAL DEPTH NOT CONVERGED AT 25 ITERS. TRIAL= ,F9.5)
      END
      SUBROUTINE CRLOC(Q,QTR,XCR)
      COMMON ZS,WF,ITZ,ZW
C ****** THIS VERSION APPLIES TO FLUMES WHICH HAVE FIXED CONTROL AT X = 0.
      XCR = 0.
      RETURN
      END
      SUBROUTINE AREA(H,X,A,W,HTR,R,APH,RPH)
      COMMON ZF,WF,ITZ,ZW
C ** THIS VERSION GOOD FOR FLUMES WITHOUT CRITICAL SECTION SHIFT (K38)
      APH = 0.
      RPH = 0.
      WT = WF
      ZQ = SQRT(1. + ZF*ZF)
      HTR = WF*.5/ZF
C  ----- HTR IS DEPTH OF FLOOR WALL INTERSECTION AT X
      IF(HTR - H) 30,20,20
   20 A = H*H*ZF
      W = 2.*H*ZF
      R = A/(2.*H*ZQ)
      IF(ITZ) 29,29,21
   21 APH = 2.*H*ZF
      RPH = ZF/ZQ - A/(2.*H*H*ZQ)
   29 CONTINUE
      RETURN
   30 A = HTR*HTR*ZF + (H-HTR)*(WT+ (H-HTR)*ZW)
      W = WT +(H-HTR)*2.*ZW
      ZQW = SQRT(1. + ZW*ZW)
      R = A/(2.*HTR*ZQ +(H-HTR)*ZQW*2.)
      IF(ITZ) 39,39,31
   31 HPM = H - HTR
      APH = WT + 2.*HPM*ZW
      P = 2.*HTR*ZQ + HPM*2.*ZQW
      RPH = APH/P - A/P/P*2.*ZQW
   39 CONTINUE
      RETURN
      END
```

Appendix B

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.

WALNUT GULCH FLUME NO. 1, LOCATED NEAR TOMBSTONE, ARIZONA
EXP. NO. 1, FILLED APPROACH, DISCHARGE COEFFICIENTS FOR 1 TO 40 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
1	19904	10.30	1.132	1033.3	11.03	0.0630
2	18351	9.87	1.121	975.7	10.39	0.0601
3	16928	9.50	1.101	927.4	9.42	0.0200
4	15026	8.83	1.101	840.3	8.78	0.0200
5	13858	8.48	1.086	794.3	8.40	0.0200
6	13384	8.35	1.075	777.8	8.43	0.0494
7	12103	7.87	1.070	716.7	7.95	0.0459
8	11052	7.54	1.048	674.4	7.47	0.0449
9	10060	6.99	1.078	604.6	7.10	0.0443
10	8581	6.26	1.098	512.9	6.53	0.0453
11	9406	6.66	1.089	563.4	6.86	0.0451
12	8754	6.30	1.107	518.9	6.63	0.0455
13	8118	6.14	1.071	498.5	6.25	0.0464
14	7370	5.97	1.016	477.6	5.81	0.0492
15	7002	5.88	0.988	466.7	5.59	0.0512
16	6164	5.58	0.947	429.2	5.11	0.0517
17	5468	5.30	0.911	394.8	4.66	0.0484
18	4778	5.07	0.855	366.4	4.05	0.0369
19	3961	4.74	0.787	326.9	3.56	0.0390
20	22531	11.10	1.133	1139.0	12.01	0.0666
21	23376	11.54	1.148	1199.3	12.67	0.0678
22	26172	12.01	1.153	1262.7	13.28	0.0693
23	20145	10.44	1.121	1051.4	11.10	0.0635
24	21432	10.77	1.132	1095.6	11.61	0.0654
25	3674	4.49	0.797	295.9	3.42	0.0373
26	3405	4.40	0.763	284.7	3.20	0.0369
27	3060	4.20	0.735	261.6	2.97	0.0375
28	2738	4.02	0.709	239.9	2.75	0.0376
29	2436	3.84	0.711	218.1	2.54	0.0393
30	1967	3.48	0.735	179.0	2.16	0.0367
31	1600	3.12	0.780	144.6	1.92	0.0384
32	1352	2.91	0.789	125.3	1.65	0.0359
33	1796	3.28	0.773	159.8	2.06	0.0369
34	1296	2.82	0.817	117.8	1.62	0.0357
35	1155	2.73	0.791	110.3	1.44	0.0351
36	1000	2.57	0.797	97.7	1.30	0.0352
37	875	2.43	0.801	87.4	1.23	0.0345
38	787	2.32	0.812	79.5	1.14	0.0345
39	633	2.04	0.892	61.9	1.08	0.0402
40	498	1.82	0.934	49.3	1.02	0.0424
41	445	1.72	0.959	44.0	1.21	0.0439
42	408	1.68	0.932	42.0	1.15	0.0455
43	359	1.60	0.926	38.1	1.08	0.0465
44	300	1.46	0.987	31.4	1.01	0.0486
45	254	1.32	1.058	26.0	0.94	0.0505
46	208	1.17	1.180	20.4	0.90	0.0546
47	184	1.08	1.268	17.4	0.86	0.0554
48	145	0.92	1.484	12.6	0.83	0.0605
49	99	0.71	1.952	7.5	0.87	0.0700

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 2, LOCATED NEAR TOMBSTONE, ARIZONA, EXP. NO. 1
DISCHARGE COEFFICIENTS FOR 1 TO 20 MODEL, OLD FLOOR, SMOOTH CHANNEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
1	2782	4.59	0.825	4940.9	2.27	0.0210
2	2580	4.44	0.808	4857.8	2.10	0.0188
3	2263	4.15	0.788	4703.5	1.80	0.0165
4	2004	3.81	0.802	4516.9	1.63	0.0162
5	1765	3.47	0.819	4333.7	1.55	0.0170
6	1504	3.23	0.780	4207.7	1.51	0.0185
7	1280	2.93	0.773	4046.2	1.38	0.0172
8	1092	2.68	0.808	5745.4	2.88	0.0229
9	990	2.55	0.799	5622.0	2.78	0.0226
10	826	2.33	0.820	5393.7	2.59	0.0209
11	716	2.19	0.859	5158.1	2.47	0.0203
12	602	2.04	0.854	5005.7	2.34	0.0207
13	444	1.75	1.121	5564.6	2.20	0.0217
14	358	1.38	0.879	5908.9	3.09	0.0208
15	205	1.26	0.935	5840.7	3.15	0.0214
16	628	2.22	0.840	5820.1	2.99	0.0232
17	248	0.84	0.792	3997.7	1.34	0.0161
18	146	0.78	0.751	3966.5	1.28	0.0157
19	009	0.70	0.694	3921.3	1.24	0.0167
20	846	0.58	0.624	3857.8	1.14	0.0153
21	717	0.49	0.560	3808.4	1.04	0.0156
22	547	0.30	0.482	3707.3	0.95	0.0177
23	482	0.24	0.443	3676.1	0.90	0.0185
24	424	0.18	0.407	3642.8	0.85	0.0190
25	366	0.10	0.371	3599.8	0.76	0.0174
26	300	0.00	0.327	3543.9	0.68	0.0169
27	256	1.91	1.253	4497.7	0.72	0.0232
28	148	1.82	1.189	5063.9	0.72	0.0274
29	172	1.70	1.126	7038.9	0.70	0.0306
30	150	1.61	1.129	8601.2	0.66	0.0309
31	133	1.54	1.118	9787.8	0.67	0.0350
32	116	1.48	1.069	10769.0	0.63	0.0370
33	102	1.44	1.016	11480.0	0.62	0.0384
34	93	1.41	0.967	11953.0	0.60	0.0367
35	73	1.35	0.845	12965.0	0.55	0.0379
36	78	1.38	0.866	12527.0	0.57	0.0380
37	72	1.35	0.842	12965.0	0.55	0.0377
38	65	1.30	0.833	13841.0	0.53	0.0381
39	57	1.25	0.809	14650.0	0.52	0.0393
40	50	1.21	0.771	15423.0	0.49	0.0403
41	42	1.13	0.766	16665.0	0.47	0.0433
42	37	1.07	0.763	17671.0	0.46	0.0472
43	31	0.99	0.778	19010.0	0.42	0.0484
44	25	0.91	0.783	20347.0	0.39	0.0505
45	20	0.81	0.840	22014.0	0.36	0.0532

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 2, LOCATED NEAR TOMBSTONE, ARIZONA
EX. # 4, NEW FLOOR, 1969 SURVEY, DISCHARGE COEFFICIENTS FOR 1:20 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
144	7120	6.64	1.158	437.9	4.17	0.0304
145	8400	7.53	1.111	518.0	4.94	0.0317
146	7887	7.06	1.161	475.5	4.60	0.0310
147	6365	5.91	1.253	373.1	3.60	0.0270
148	5991	5.75	1.233	359.0	3.43	0.0257
149	6950	6.49	1.175	422.4	4.06	0.0302
150	7080	6.60	1.165	433.4	4.16	0.0308
151	6597	6.31	1.167	408.5	3.91	0.0298
152	6166	5.93	1.209	374.5	3.61	0.0281
153	5672	5.52	1.244	336.5	3.34	0.0257
154	5154	5.07	1.218	343.0	3.20	0.0253
155	4696	4.65	1.234	312.0	3.18	0.0262
156	3994	4.07	1.193	255.6	2.84	0.0246
157	3576	3.75	1.246	230.8	2.55	0.0226
158	3392	3.63	1.286	219.5	2.46	0.0291
159	2955	3.29	1.314	186.2	2.18	0.0273
160	2368	2.80	1.364	145.2	1.83	0.0252
161	1907	2.43	1.427	114.4	1.59	0.0257
162	1637	2.19	1.448	94.0	1.42	0.0244
163	1327	1.93	1.445	76.5	1.27	0.0233
164	1048	1.71	1.428	58.6	1.13	0.0251
165	837	1.58	1.363	47.7	1.03	0.0242
166	776	1.48	1.092	44.8	1.00	0.0248
167	998	1.33	0.989	51.7	1.05	0.0230
168	819	1.17	0.972	38.6	0.80	0.0209
169	662	1.07	0.934	30.3	0.74	0.0191
170	515	1.01	0.878	25.4	0.58	0.0169
171	384	0.93	0.837	18.8		
172	227	0.75	0.834	10.4		
173	106	0.64	0.837	5.4		
174	001	0.61	0.779	3.2	1.00	0.0182
175	842	0.38	0.838	6.3	0.88	0.0157
176	751	0.07	1.080	6.8	0.83	0.0149
177	648	0.98	1.050	5.7	0.78	0.0152
178	596	0.97	0.979	5.1	0.77	0.0146
179	539	0.95	0.910	5.0	0.82	0.0123
180	477	0.89	0.868	4.2	0.77	0.0125
181	500	0.93	0.865	4.1	0.72	0.0142
182	397	0.83	0.790	4.0	0.73	0.0147
183	335	0.69	0.827	3.1	0.71	0.0164
184	293	0.63	0.804	2.2	0.78	0.0231
185	233	0.53	0.756	2.1	0.78	0.0285
186	214	0.51	0.720	2.3	0.76	0.0287
187	200	0.49	0.696	2.3	0.72	0.0281
188	174	0.46	0.645	2.2	0.68	0.0294
189	153	0.39	0.649	2.7	0.66	0.0314
190	137	0.33	0.653	2.7	0.61	0.0300
191	117	1.26	0.642	2.5	0.54	0.0282
192	100	1.19	0.651	1.2	0.52	0.0292
193	79	1.10	0.643	1.6	0.50	0.0333
194	72	1.07	0.636	1.8	0.48	0.0337
195	65	1.04	0.620	1.1	0.48	0.0360
196	58	1.01	0.601	1.5	0.47	0.0383
197	50	0.97	0.583	1.5	0.43	0.0377
198	44	0.93	0.578	1.6	0.39	0.0357
199	38	0.89	0.573	1.8	0.37	0.0375
200	34	0.85	0.596	1.0	0.36	0.0388
201	29	0.81	0.579	9.3	0.34	0.0410
202	24	0.78	0.536	8.8	0.32	0.0436
203	19	0.75	0.479	8.3	0.29	0.0461

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 3, LOCATED NEAR TOMBSTONE, ARIZONA
DISCHARGE COEFICIENTS FOR 1 TO 32 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM HYDRAULIC RADIUS (FT.)	APPROACH MANNING N
1	7680	11.20	1.067	385.1	9.53	0.0389
2	6990	10.60	1.082	356.7	9.17	0.0373
3	5731	9.51	1.095	307.1	8.39	0.0355
4	4611	8.43	1.112	259.6	7.61	0.0333
5	5340	9.16	1.098	291.4	8.13	0.0333
6	4941	8.78	1.102	274.9	7.85	0.0334
7	4532	8.39	1.100	258.5	7.54	0.0340
8	4085	7.91	1.109	238.5	7.19	0.0345
9	3585	7.38	1.109	216.7	6.77	0.0367
10	2991	6.67	1.118	188.6	6.12	0.0333
11	2406	5.87	1.137	158.2	5.37	0.0285
12	1965	5.24	1.136	135.5	4.77	0.0289
13	2087	5.37	1.157	139.0	5.29	0.0487
15	1769	4.98	1.123	125.9	4.76	0.0461
16	1575	4.61	1.146	113.0	4.17	0.0285
17	1395	4.33	1.134	103.4	3.83	0.0276
18	1177	3.85	1.173	87.3	3.30	0.0228
19	955	3.43	1.159	73.7	2.94	0.0255
20	784	3.15	1.105	64.5	2.68	0.0221
21	751	3.13	1.068	64.0	2.61	0.0243
22	692	2.99	1.066	59.5	2.50	0.0252
25	571	2.75	1.012	52.0	2.01	0.0267
26	499	2.59	0.977	47.1	1.86	0.0287
27	416	2.36	0.948	40.3	1.63	0.0284
28	340	2.14	0.910	33.7	1.46	0.0308
29	275	1.91	0.919	27.0	1.27	0.0297
30	251	1.80	0.973	23.9	1.24	0.0306
31	235	1.75	0.980	22.6	1.19	0.0307
32	215	1.71	0.957	21.5	1.13	0.0324
33	194	1.64	0.951	19.9	1.03	0.0316
34	170	1.58	0.921	18.4	0.94	0.0320
35	141	1.51	0.848	16.9	0.80	0.0313
36	116	1.45	0.774	15.5		
37	95	1.39	0.710	14.2		
38	76	1.32	0.643	12.9		
39	57	1.24	0.565	11.4		
40	47	1.16	0.547	9.9		

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 4, LOCATED NEAR TOMBSTONE, ARIZONA
EXP. NO. 1, UNFILLED APPROACH, DISCHARGE COEFFICIENTS FOR 1:30 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
1	2287	9.76	0.784	138.4	6.39	
2	2088	9.35	0.790	128.8	5.99	
3	1904	8.96	0.794	120.0	5.58	
4	1691	8.46	0.805	109.0	5.06	
5	1513	7.97	0.824	98.9	4.57	
6	1985	9.16	0.787	124.4	5.76	
7	1286	7.38	0.833	87.2	4.26	
8	1171	7.03	0.848	80.6	4.05	
9	1064	6.70	0.858	74.6	3.21	
10	999	6.48	0.867	70.8	3.16	
11	892	6.13	0.878	64.7	3.16	
12	804	5.78	0.900	59.0	3.13	
13	707	5.39	0.923	53.0	3.02	
14	600	4.94	0.949	46.2	2.70	
15	508	4.50	0.981	40.2	2.41	
16	468	4.30	1.000	37.4	2.28	
17	187	2.61	1.127	18.2	1.78	
18	431	4.10	1.015	34.9	2.14	
19	398	3.95	1.027	32.8	2.00	
20	374	3.80	1.036	31.2	1.89	
21	341	3.62	1.047	29.1	1.73	
22	312	3.44	1.066	27.0	1.59	
23	275	3.23	1.073	24.6	1.37	
24	235	2.96	1.097	21.7	1.13	
25	203	2.72	1.123	19.3	0.91	
26	155	2.35	1.160	15.6	0.51	
27	142	2.23	1.186	14.5	0.40	
28	127	2.09	1.196	13.4	0.25	
29	112	1.94	1.221	12.1	0.09	
30	100	1.83	1.229	11.1	1.94	
31	86	1.68	1.235	9.9	1.74	
32	66	1.45	1.263	8.1	1.46	
33	50	1.22	1.302	6.5	1.18	

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 4, LOCATED NEAR TOMBSTONE, ARIZONA
EXP. NO. 2, PREDICTED FILL, DISCHARGE COEFFICIENTS FOR 1 TO 30 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
34	155	2.35	1.157	15.6	1.17	
35	141	2.21	1.189	14.4	1.05	
36	127	2.08	1.210	13.2	0.93	
37	114	1.95	1.228	12.1	0.81	
38	100	1.81	1.247	11.0	0.70	
39	85	1.65	1.266	9.7	0.59	
40	65	1.42	1.299	7.9		
41	50	1.21	1.332	6.4		
42	219	0.59	0.765	134.4	5.65	
43	184	0.04	0.773	121.7	4.10	
44	167	8.50	0.789	109.9	4.55	
45	147	7.99	0.798	99.2	3.96	
46	150	8.05	0.803	100.4	4.07	
47	128	7.51	0.850	85.8	4.67	
48	116	6.95	0.864	79.2	4.60	
49	107	6.67	0.876	74.0	4.54	
50	99	6.41	0.889	69.5	4.44	
51	87	6.01	0.901	62.7	4.35	
52	79	5.71	0.919	57.9	4.27	
53	69	5.29	0.942	51.4	4.06	
54	57	4.78	0.974	44.1	3.65	
55	50	4.45	0.994	39.4	3.36	
56	19	2.72	1.105	19.2	1.53	
57	45	4.21	1.017	36.7	3.17	
58	41	4.01	1.042	33.7	3.01	
59	38	3.80	1.063	31.1	2.82	
60	34	3.59	1.081	28.7	2.62	
61	30	3.34	1.098	25.8	2.34	
62	25	3.04	1.132	22.5	2.03	
63	22	2.81	1.158	20.1	1.77	

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 6 , LOCATED NEAR TOMBSTONE, ARIZONA
DISCHARGE COEFICIENTS FOR 1 TO 30 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM HYDRAULIC RADIUS (FT.)	APPROACH MANNING N
139	733	2.81	0.683	79.9	1.65	0.0272
140	655	2.70	0.671	74.0	1.51	0.0274
143	638	2.63	0.702	70.0	1.49	0.0271
144	590	2.57	0.685	67.0	1.35	0.0246
145	541	2.49	0.683	62.7	1.32	0.0277
146	487	2.40	0.673	58.2	1.23	0.0282
147	443	2.28	0.697	52.5	1.11	0.0257
148	399	2.16	0.719	47.1	1.04	0.0260
149	379	2.10	0.732	44.6	1.01	0.0264
150	349	1.99	0.767	40.2	0.92	0.0238
151	324	1.91	0.787	37.1	0.87	0.0237
152	287	1.74	0.880	30.8	0.78	0.0206
153	262	1.64	0.942	27.1	0.75	0.0212
154	235	1.52	1.011	23.4	0.73	0.0222
155	638	2.64	0.694	70.6	1.49	0.0271
156	467	2.34	0.686	55.5	1.14	0.0252
157	485	2.37	0.692	56.8	1.18	0.0251
158	215	1.41	1.124	20.1	0.72	0.0213
159	199	1.35	1.164	18.4	0.70	0.0219
160	183	1.31	1.146	17.4	0.65	0.0227
161	164	1.24	1.190	15.5	0.57	0.0191
162	151	1.20	1.181	14.5	0.58	0.0238
163	138	1.17	1.153	13.8	0.54	0.0245
164	121	1.14	1.082	13.1		
165	110	1.11	1.043	12.5		
166	98	1.10	0.958	12.1		
167	89	1.08	0.910	11.8		
168	80	1.05	0.873	11.1		
169	65	0.99	0.818	9.9		
170	44	0.89	0.813	7.3		
171	146	1.23	1.077	15.5		

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 6 , LOCATED NEAR TOMBSTONE, ARIZONA
DISCHARGE COEFFICIENTS FOR 1 TO 30 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
22	10597	9.27	1.144	562.7	9.46	0.0260
23	9882	9.02	1.118	542.6	9.14	0.0245
24	9028	8.57	1.116	505.9	8.72	0.0236
25	8174	8.07	1.118	466.3	8.28	0.0231
26	7391	7.65	1.107	433.1	7.82	0.0251
27	6878	7.40	1.092	412.9	7.47	0.0236
28	6375	7.11	1.083	390.5	7.15	0.0247
29	5915	6.86	1.066	371.3	6.83	0.0247
30	5417	6.58	1.046	350.1	6.47	0.0247
31	4950	6.28	1.035	326.8	6.12	0.0238
32	4404	5.78	1.056	289.5	5.75	0.0221
33	4684	6.01	1.054	306.4	5.95	0.0228
34	5708	6.75	1.057	363.0	6.68	0.0247
35	4567	5.91	1.055	299.4	5.86	0.0217
36	4214	5.52	1.090	270.1	5.69	0.0223
37	3680	5.08	1.091	237.7	5.29	0.0211
38	3326	4.68	1.127	208.6	5.16	0.0218
39	2991	4.42	1.115	189.3	4.86	0.0191
40	2726	4.44	1.009	190.8	4.32	0.0181
41	2885	4.40	1.083	188.0	4.67	0.0181
42	2610	4.31	1.011	181.9	4.24	0.0191
43	2266	4.20	0.916	173.7	3.72	0.0202
44	1991	4.01	0.867	160.2	3.38	0.0217
45	3163	4.55	1.122	199.2	4.02	0.0208
46	1755	3.69	0.876	137.3	3.15	0.0171
47	1583	3.57	0.833	128.8	2.89	0.0166
48	1380	3.42	0.785	118.5	2.60	0.0195
49	1191	3.21	0.796	104.2	2.31	0.0164
50	1046	3.12	0.750	98.4	2.01	0.0164
51	922	3.00	0.728	91.2	1.82	0.0182
52	799	2.89	0.695	84.4	1.63	0.0200
53	18953	12.23	1.257	814.5	9.95	0.0200
56	16116	11.39	1.214	740.6	8.47	0.0308
57	15091	10.95	1.218	703.3	7.91	0.0287
58	14334	10.61	1.222	674.8	7.50	0.0279
59	13248	10.13	1.226	633.6	6.90	0.0273
60	12054	9.45	1.259	577.4	7.56	0.0251
61	11175	8.85	1.305	528.9	8.08	0.0250
62	10124	8.09	1.381	467.5	8.40	0.0258
63	8938	8.50	1.119	500.6	8.69	0.0251
64	9505	8.65	1.111	528.7	8.96	0.0257
65	10131	9.14	1.121	551.9	9.27	0.0255
266	11100	9.42	1.164	575.5	8.94	0.0255

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 7, LOCATED NEAR TOMBSTONE, ARIZONA
EXP. NO. 4, PREDICTED FILL, DISCHARGE COEFFICIENTS FOR 1 TO 30 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM HYDRAULIC RADIUS (FT.)	APPROACH MANNING N
80	9080	11.28	1.030	495.1	9.00	0.0576
81	8470	10.77	1.047	465.9	8.51	0.0562
82	7470	9.89	1.082	416.1	8.10	0.0507
83	8009	10.38	1.060	443.8	8.12	0.0533
84	6589	9.16	1.097	376.6	7.76	0.0491
85	6014	8.52	1.143	342.4	7.50	0.0450
86	5057	7.65	1.169	296.9	6.88	0.0433
87	5518	8.14	1.141	322.1	7.19	0.0436
88	4512	7.32	1.127	280.3	6.44	0.0428
89	4000	6.85	1.125	256.7	6.06	0.0417
90	3472	6.34	1.118	232.0	5.94	0.0426
91	3070	5.93	1.111	211.2	5.71	0.0423
92	2592	5.39	1.107	186.7	5.33	0.0445
93	1999	4.78	1.048	158.6	4.63	0.0428
94	1715	4.41	1.032	141.9	4.28	0.0415
95	1530	4.16	1.014	130.9	4.00	0.0413
96	1354	3.91	0.996	119.8	3.72	0.0409
97	1202	3.67	0.982	109.5	3.47	0.0374
98	1002	3.38	0.939	96.9	3.12	0.0372
99	806	3.05	0.897	82.7	2.75	0.0385
100	635	2.69	0.867	67.6	2.40	0.0415
101	571	2.56	0.847	62.2	2.26	0.0422
102	500	2.42	0.811	56.5	2.08	0.0429
103	417	2.23	0.770	48.8	1.86	0.0442
104	323	2.00	0.721	39.5	1.60	0.0461
105	250	1.73	0.795	29.7	1.42	0.0479
106	200	1.47	0.959	21.4	1.34	0.0474
107	176	1.36	1.019	18.4	1.28	0.0479
108	140	1.22	1.074	14.7	1.14	0.0479
109	99	1.08	1.036	11.5	0.86	0.0494
110	68	0.98	0.913	9.4	0.61	0.0496
111	51	0.89	0.861	7.8	0.52	0.0574

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 8, LOCATED NEAR TOMBSTONE, ARIZONA
DISCHARGE COEFICIENTS FOR 1 TO 30 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
51	1779	4.43	1.063	142.1	3.76	0.0211
52	1641	4.28	1.040	135.4	3.56	0.0217
53	1493	4.07	1.030	126.1	3.34	0.0221
54	1314	3.41	1.214	97.8	2.77	0.0225
55	1096	3.16	1.150	87.0	2.84	0.0225
56	883	3.02	1.000	81.0	2.47	0.0283
57	603	2.60	0.873	63.5	1.91	0.0296
58	1386	3.65	1.147	107.8	3.27	0.0214
59	736	2.75	0.972	69.7	2.20	0.0290
60	1460	3.94	1.063	120.5	3.50	0.0213
61	1214	3.16	1.275	86.9	2.80	0.0215
62	990	3.17	1.037	87.2	2.64	0.0266
63	804	2.82	1.016	72.8	2.39	0.0301
64	10782	12.04	1.076	540.1	9.07	0.0390
65	9684	11.41	1.069	502.8	8.38	0.0373
66	8614	10.78	1.058	466.3	7.97	0.0347
67	7369	9.81	1.078	411.9	7.40	0.0329
68	6246	8.90	1.093	362.3	6.80	0.0300
69	5234	8.23	1.058	326.4	6.66	0.0328
70	4349	7.88	1.096	277.7	6.66	0.0288
71	6240	8.90	1.094	361.8	6.58	0.0344
72	4621	7.63	1.071	295.4	6.48	0.0301
73	4280	7.33	1.066	280.1	6.32	0.0296
74	3893	6.72	1.132	249.6	6.15	0.0281
75	3378	6.27	1.109	227.8	5.81	0.0274
76	3003	5.96	1.078	212.8	5.38	0.0267
77	2683	5.56	1.086	194.0	5.01	0.0254
78	2376	5.18	1.088	176.1	4.63	0.0244
79	2076	4.80	1.059	161.6	4.22	0.0238
80	639	2.69	0.874	67.4	1.96	0.0288
81	586	2.57	0.863	62.3	1.87	0.0299
82	532	2.46	0.839	58.0	1.80	0.0318
83	468	2.36	0.792	53.7	1.65	0.0322
84	403	2.24	0.740	49.0	1.48	0.0319
85	352	2.15	0.693	45.1	1.39	0.0336
86	310	2.03	0.671	40.5	1.28	0.0338
87	262	1.91	0.661	35.8	1.13	0.0326
88	218	1.79	0.647	31.4	1.00	0.0327
89	198	1.73	0.638	29.4		
90	178	1.67	0.630	27.5		
91	155	1.58	0.630	24.5		
92	129	1.46	0.637	20.9		
93	106	1.31	0.685	16.8		
94	81	1.16	0.716	13.2		
95	70	1.06	0.768	11.0		
96	53	0.92	0.825	8.3		

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 11, LOCATED NEAR TOMBSTONE, ARIZONA
EXP. NO. 1, FILLED APPROACH, DISCHARGE COEFFICIENTS FOR 1 TO 30 MODEL

TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	APPROACH MANNING N
1	5948	9.86	1.021	344.5	7.26	0.0801
2	6323	10.26	1.004	363.7	7.63	0.0800
3	5664	9.57	1.028	331.2	6.91	0.0694
4	5399	9.27	1.044	317.0	6.60	0.0625
5	5089	8.89	1.064	300.0	6.25	0.0579
6	4616	8.65	1.017	289.5	5.59	0.0562
7	4343	8.36	1.020	276.5	5.22	0.0525
8	3987	7.94	1.032	258.5	4.74	0.0511
9	3767	7.68	1.038	247.0	4.42	0.0489
10	3479	7.28	1.058	230.3	4.20	0.0468
18	1781	4.96	1.085	139.0	3.09	0.0200
19	1651	4.75	1.087	131.2	2.90	0.0200
20	1472	4.39	1.114	118.0	2.62	0.0200
11	1264	3.97	1.139	103.3	2.30	0.0200
12	1084	3.47	1.229	86.1	2.08	0.0200
13	987	3.28	1.231	79.7	1.91	0.0200
14	854	3.05	1.207	71.9	1.65	0.0200
15	727	2.71	1.251	60.9	1.28	0.0102
16	868	6.56	1.056	100.9	3.99	0.0400
17	924	6.72	1.066	107.2	4.10	0.0423
28	866	7.02	1.062	119.6	4.33	0.0449
29	675	6.27	1.072	89.0	3.78	0.0410
30	558	6.13	1.066	83.7	3.61	0.0394
31	518	5.81	1.064	71.3	3.50	0.0200
32	999	5.90	1.075	75.5	3.59	0.0200
33	639	5.59	1.185	57.0	1.15	0.0150
34	96	4.49	1.185	53.6	1.10	0.0162
35	440	4.36	1.175	49.4	1.04	0.0183
36	392	4.20	1.158	46.0	0.97	0.0192
37	332	4.11	1.144	42.2	0.87	0.0201
38	266	1.95	1.084	36.8	0.80	0.0211
39	206	1.86	1.046	33.9	0.73	0.0217
40	95	1.80	0.999	32.1		
41	55	1.74	0.910	30.2		
42	16	1.65	0.845	27.4		
43	00	1.62	0.800	26.6		
44	78	1.56	0.760	24.8		
45	56	1.50	0.707	23.1		
46	42	1.44	0.692	21.3		
47	30	1.42	0.660	20.6		
48	13	1.34	0.662	18.5		
49	00	1.27	0.674	16.4		
50	85	1.20	0.656	14.7		
51	76	1.14	0.665	13.3		
52	65	1.07	0.656	11.8		
53	48	0.95	0.661	9.4		
54	41	0.88	0.692	7.9		
55	25	0.69	0.788	4.9		
56	33	0.79	0.724	6.4		

Selected model ratings of discharges for Walnut Gulch flumes 1, 2, 3, 4, 6, 7, 8, 11, and 15 in prototype dimensions.—Con.

WALNUT GULCH FLUME NO. 15, LOCATED NEAR TOMBSTONE, ARIZONA
EXPERIMENT NO. 2, FILLED APPROACH DISCHARGE COEFFICIENTS FOR 1 TO 30 MODEL

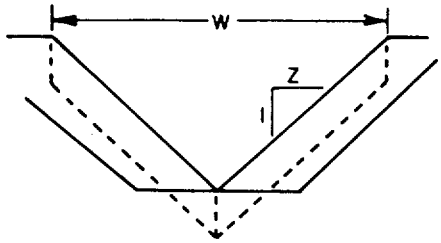
TEST NO.	PROTOTYPE DISCHARGE (C.F.S.)	FLUME HEAD (FT.)	DISCHARGE C	CONTROL AREA (SQ.FT.)	UPSTREAM APPROACH HYDRAULIC RADIUS (FT.)	MANNING N
50	10545	11.62	1.127	515.5	8.19	0.0625
51	9741	11.35	1.089	499.5	7.50	0.0629
52	8790	10.98	1.045	478.3	6.70	0.0635
53	7983	10.58	1.018	455.9	6.52	0.0627
54	7271	10.06	1.018	425.9	6.12	0.0652
55	6539	9.42	1.033	390.8	5.59	0.0663
56	6064	9.04	1.034	369.8	5.24	0.0685
57	5398	8.44	1.046	337.4	4.72	0.0690
58	4932	7.99	1.052	314.5	4.32	0.0687
59	4554	7.65	1.050	297.2	4.01	0.0696
60	4264	7.36	1.054	282.8	3.80	0.0697
61	3789	6.87	1.059	257.8	3.40	0.0694
62	3338	6.38	1.061	234.0	3.03	0.0684
63	2907	5.88	1.068	209.6	2.65	0.0661
64	2547	5.44	1.070	188.9	2.66	0.0626
65	2239	5.05	1.070	170.8	2.44	0.0580
66	1956	4.66	1.071	153.1	2.31	0.0549
67	1759	4.42	1.056	142.1	2.39	0.0510
68	1588	4.15	1.059	130.2	2.35	0.0496
69	1431	3.85	1.081	117.1		
70	1255	3.61	1.057	106.6	2.10	0.0390
71	1062	3.28	1.049	92.46	2.46	0.1200
72	905	2.95	1.065	78.44		
73	778	2.72	1.047	68.88		
74	657	2.56	0.973	62.33	1.97	0.0304
75	999	3.15	1.054	87.70		
76	642	2.56	0.953	62.22	1.95	0.0304
77	602	2.47	0.943	58.88	1.91	0.0307
78	555	2.41	0.907	56.22	1.83	0.0301
79	501	2.28	0.892	51.12	1.77	0.0303
80	434	2.14	0.859	45.22	1.69	0.0304
81	382	2.02	0.833	40.64	1.61	0.0295
82	347	1.93	0.846	36.99	1.48	0.0258
83	306	1.84	0.839	33.55	1.38	0.0246
84	267	1.74	0.842	30.05	1.31	0.0253
85	229	1.64	0.840	26.60	1.21	0.0245
86	216	1.63	0.803	26.66	1.16	0.0245
87	200	1.59	0.787	25.66		
88	182	1.53	0.791	23.60		
89	155	1.42	0.811	20.00		
90	134	1.36	0.780	18.33		
91	112	1.33	0.694	17.44		
92	103	1.29	0.692	16.44		
93	80	1.21	0.624	14.55		
94	66	1.15	0.584	13.11		
95	50	1.06	0.542	11.11		

Appendix C

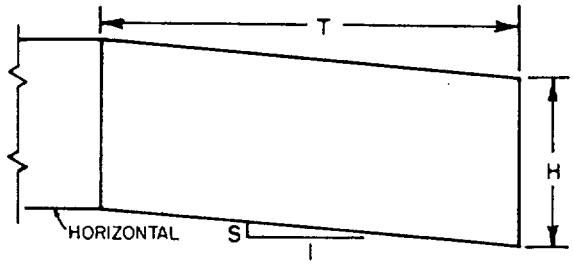
Design dimensions of Santa Rita flumes for a range of flow capacities.

SANTA RITA FLUME - TRIANGULAR SECTION

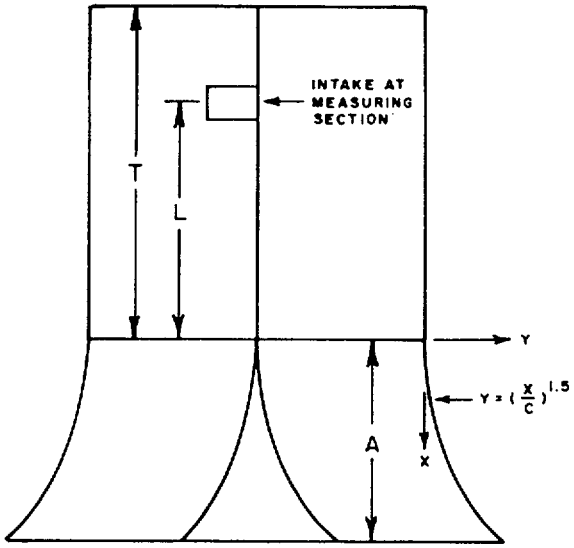
DESIGN CAPACITY 0.5 m^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, METERS

L	2.0
T	2.7
H	0.5
W	2.0
S	0.03
Z	2.0
A	≥ 1.0
C	2.0

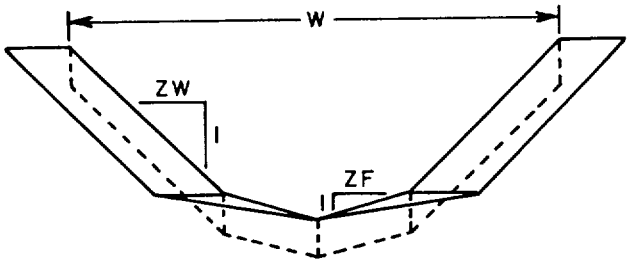
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN m^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.063	0.176				
1.2	0.069	0.190				
1.4	0.074	0.203				
1.6	0.078	0.216				
1.8	0.082	0.227				
2.0	0.086	0.238				
3.0	0.103	0.284				
4.0	0.117	0.321				
5.0	0.129	0.354				
6.0	0.140					
7.0	0.150					
8.0	0.159					
9.0	0.168					

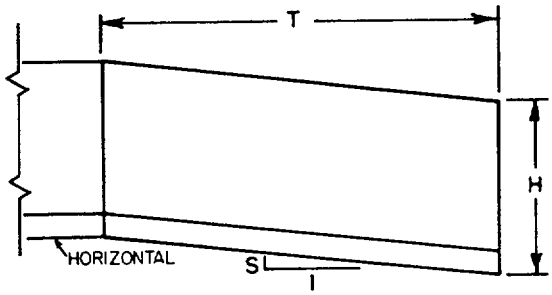
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

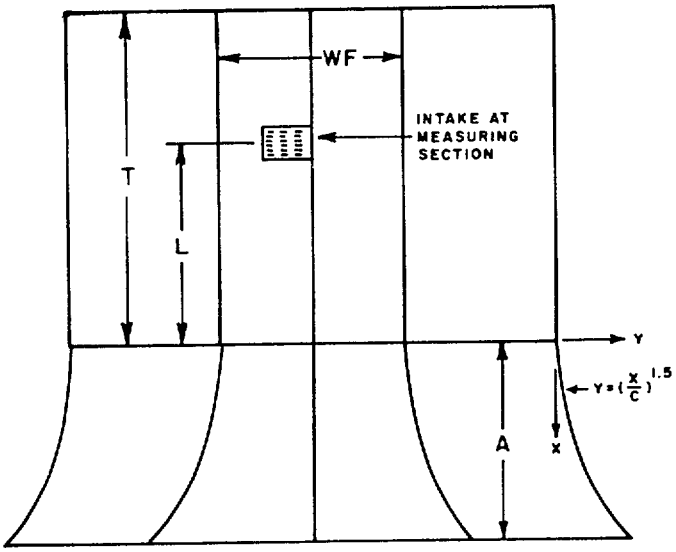
DESIGN CAPACITY 1.0 m^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, METERS

L	2.0	ZW	1.0
T	2.7	ZF	4.0
H	0.8625	S	0.03
W	2.0	A	$\geq$ 1.0
WF	0.50	C	2.0

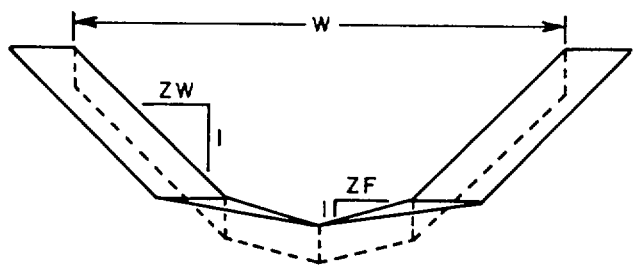
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN m^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.046	0.139	0.493			
1.2	0.050	0.154				
1.4	0.053	0.168				
1.6	0.057	0.181				
1.8	0.059	0.193				
2.0	0.062	0.205				
3.0	0.075	0.257				
4.0	0.086	0.302				
5.0	0.096	0.341				
6.0	0.105	0.376				
7.0	0.114	0.409				
8.0	0.123	0.439				
9.0	0.131	0.467				

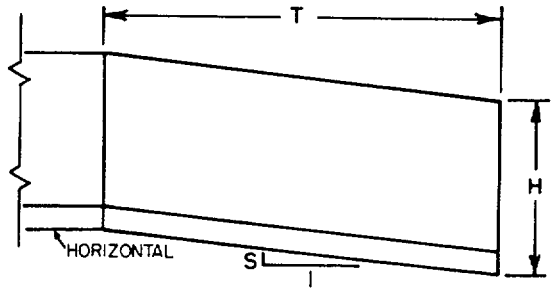
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

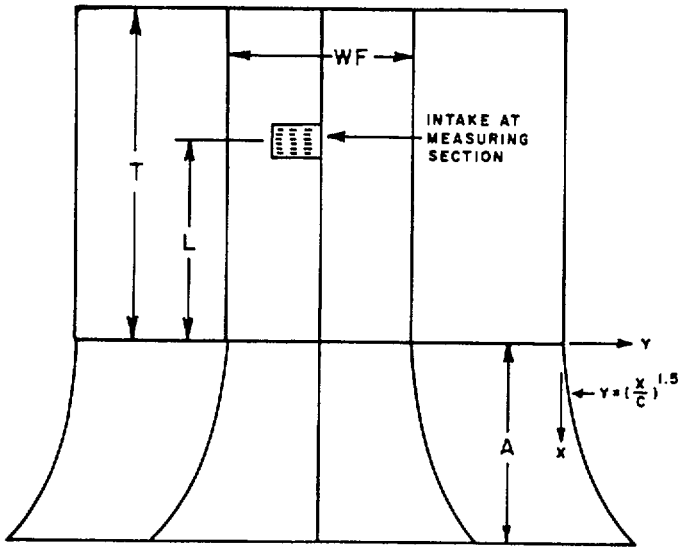
DESIGN CAPACITY 2.0 m^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, METERS

L	2.0	ZW	1.0
T	2.80	ZF	4.0
H	0.90	S	0.03
W	2.60	A	$\geq$ 1.0
WF	1.0	C	2.0

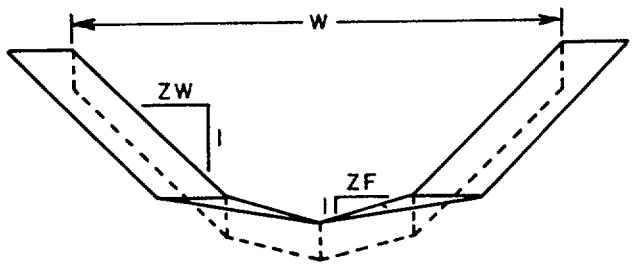
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN m^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.046	0.127	0.411			
1.2	0.050	0.138	0.454			
1.4	0.054	0.147	0.494			
1.6	0.057	0.157	0.531			
1.8	0.060	0.168	0.567			
2.0	0.063	0.175	0.600			
3.0	0.075	0.214				
4.0	0.086	0.249				
5.0	0.095	0.281				
6.0	0.102	0.310				
7.0	0.110	0.338				
8.0	0.116	0.368				
9.0	0.122	0.388				

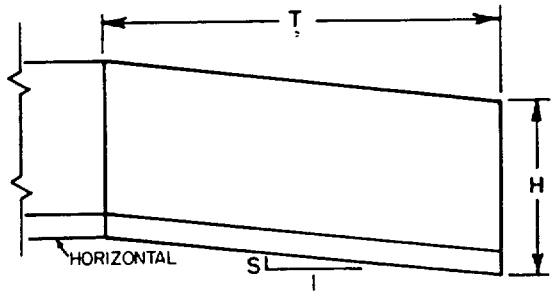
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

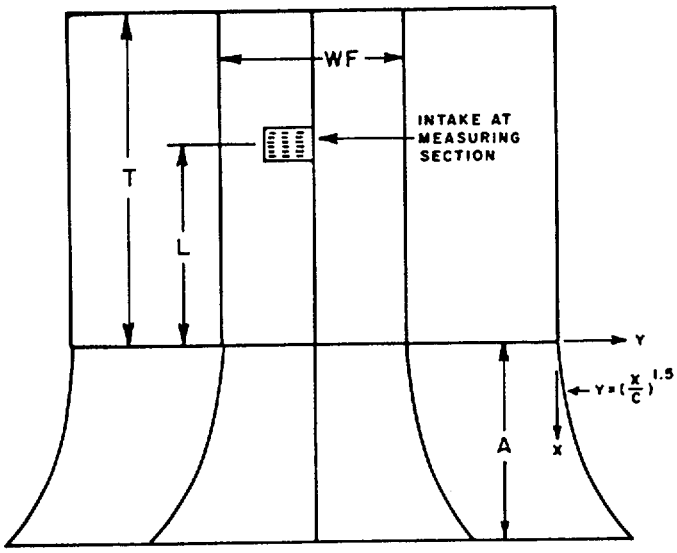
DESIGN CAPACITY 5.0 m^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, METERS

L	2.50	ZW	1.0
T	3.50	ZF	4.0
H	1.25	S	0.03
W	3.7	A	≥1.50
WF	1.50	C	2.0

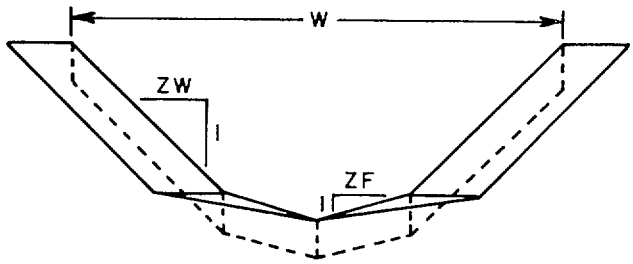
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN m^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.045	0.125	0.359			
1.2	0.048	0.135	0.396			
1.4	0.052	0.145	0.430			
1.6	0.055	0.153	0.462			
1.8	0.058	0.161	0.493			
2.0	0.061	0.169	0.522			
3.0	0.073	0.200	0.651			
4.0	0.083	0.227	0.762			
5.0	0.092	0.253	0.860			
6.0	0.099	0.276				
7.0	0.106	0.298				
8.0	0.113	0.319				
9.0	0.119	0.340				

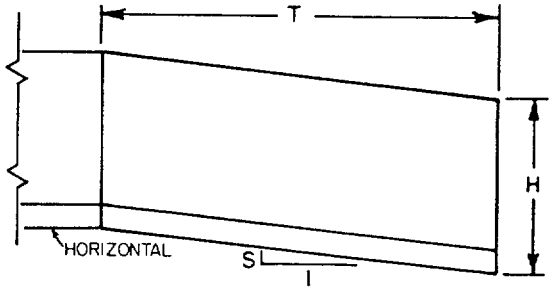
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

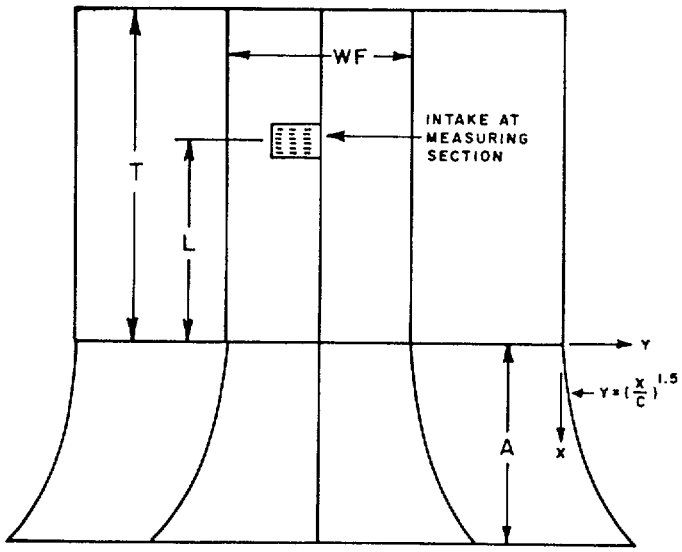
DESIGN CAPACITY 10 m^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, METERS

L	2.5	ZW	1.0
T	4.5	ZF	5.0
H	1.7	S	0.03
W	5.0	A	≥ 1.5
WF	2.0	C	2.0

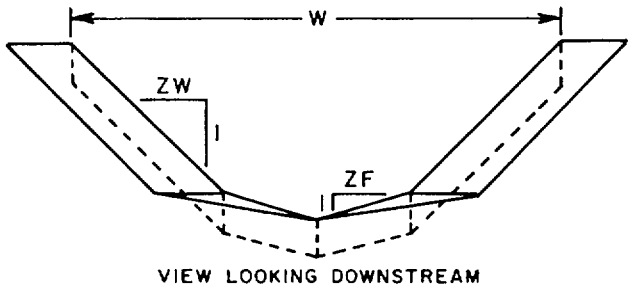
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN m^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.040	0.113	0.316	1.119		
1.2	0.044	0.122	0.347			
1.4	0.047	0.131	0.376			
1.6	0.050	0.139	0.404			
1.8	0.053	0.147	0.431			
2.0	0.055	0.154	0.457			
3.0	0.066	0.183	0.572			
4.0	0.075	0.206	0.672			
5.0	0.083	0.227	0.761			
6.0	0.090	0.246	0.843			
7.0	0.096	0.265	0.919			
8.0	0.102	0.282	0.990			
9.0	0.108	0.299	1.056			

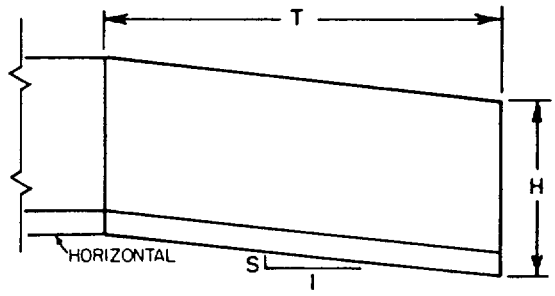
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

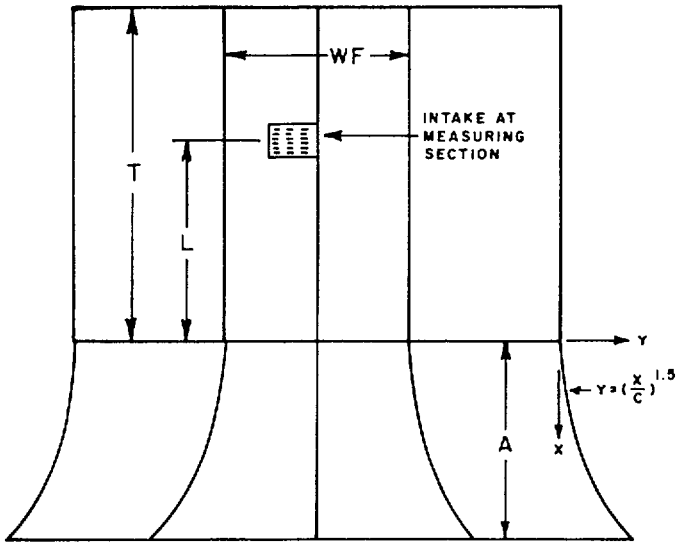
DESIGN CAPACITY 50 m^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, METERS

L	4.0	ZW	1.0
T	7.50	ZF	5.0
H	3.0	S	0.03
W	9.2	A	≥ 2.5
WF	4.0	C	2.0

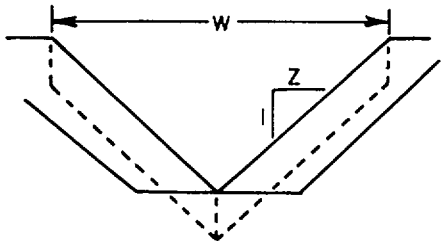
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN m^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.039	0.107	0.298	0.871		
1.2	0.042	0.116	0.323	0.963		
1.4	0.045	0.124	0.345	1.048		
1.6	0.047	0.132	0.365	1.128		
1.8	0.050	0.139	0.383	1.204		
2.0	0.052	0.146	0.400	1.277		
3.0	0.063	0.175	0.476	1.601		
4.0	0.071	0.198	0.544	1.878		
5.0	0.079	0.219	0.606	2.124		
6.0	0.085	0.238	0.665			
7.0	0.091	0.254	0.720			
8.0	0.097	0.270	0.773			
9.0	0.102	0.284	0.823			

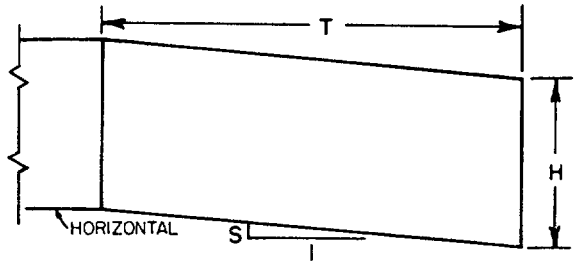
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME - TRIANGULAR SECTION

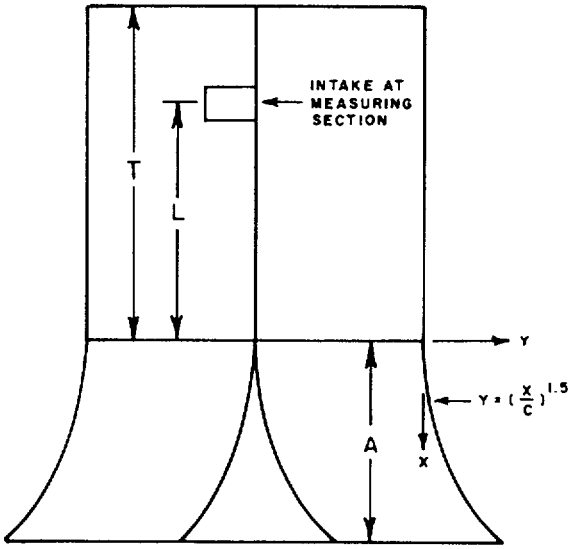
DESIGN CAPACITY 12 ft^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, FEET

L	6.0
T	8.0
H	1.4
W	5.4
S	0.03
Z	2.0
A	4.0
C	4.0

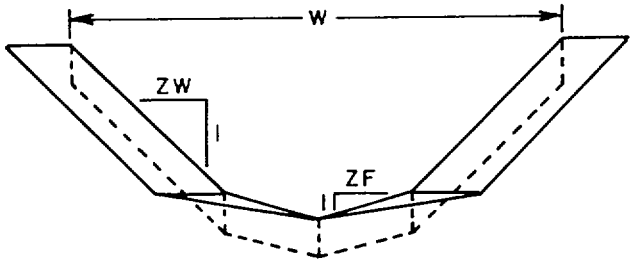
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN FEET AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN ft^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.044	0.119	0.330	0.906		
1.2		0.129	0.358	0.981		
1.4		0.138	0.383			
1.6		0.146	0.406			
1.8		0.154	0.428			
2.0	0.058	0.161	0.448			
3.0	0.070	0.193	0.536			
4.0	0.079	0.220	0.608			
5.0	0.087	0.243	0.670			
6.0	0.095	0.263	0.726			
7.0	0.101	0.282	0.776			
8.0	0.107	0.299	0.822			
9.0	0.113	0.315	0.866			

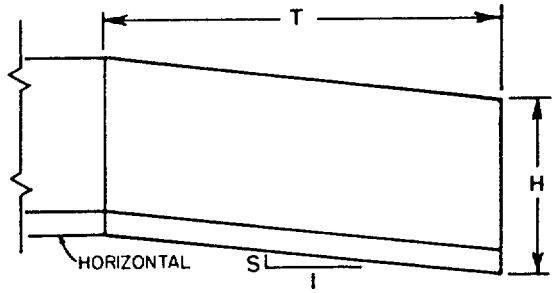
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

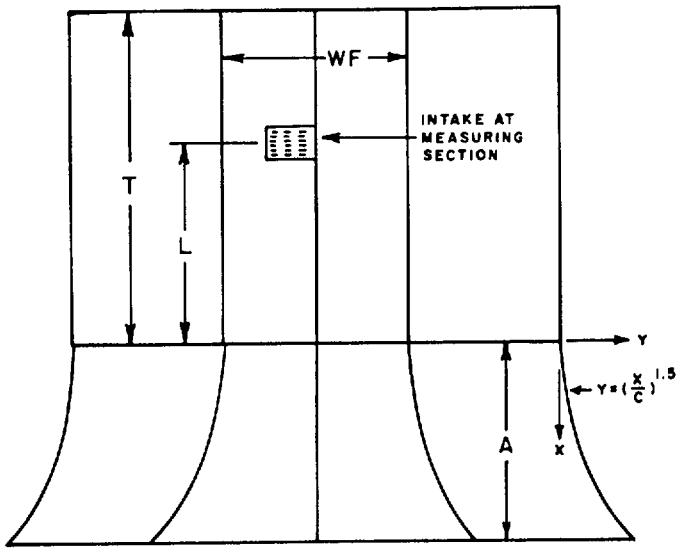
DESIGN CAPACITY 20 ft^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, FEET

L	6.0	ZW	1.0
T	8.0	ZF	4.0
H	2.0	S	0.03
W	4.75	A	≥ 4.0
WF	1.0	C	4.0

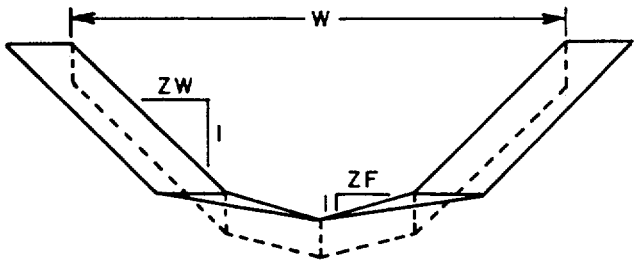
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN ft^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.031	0.086	0.257	0.930		
1.2		0.093	0.284	1.025		
1.4		0.099	0.310	1.111		
1.6		0.106	0.335	1.191		
1.8		0.111	0.358	1.266		
2.0	0.042	0.116	0.380	1.336		
3.0	0.050	0.139	0.479			
4.0	0.057	0.159	0.563			
5.0	0.063	0.177	0.638			
6.0	0.068	0.195	0.705			
7.0	0.073	0.211	0.767			
8.0	0.078	0.227	0.825			
9.0	0.082	0.242	0.879			

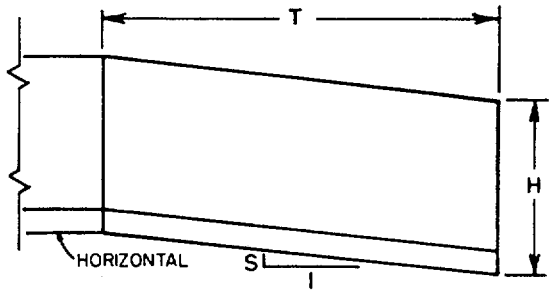
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

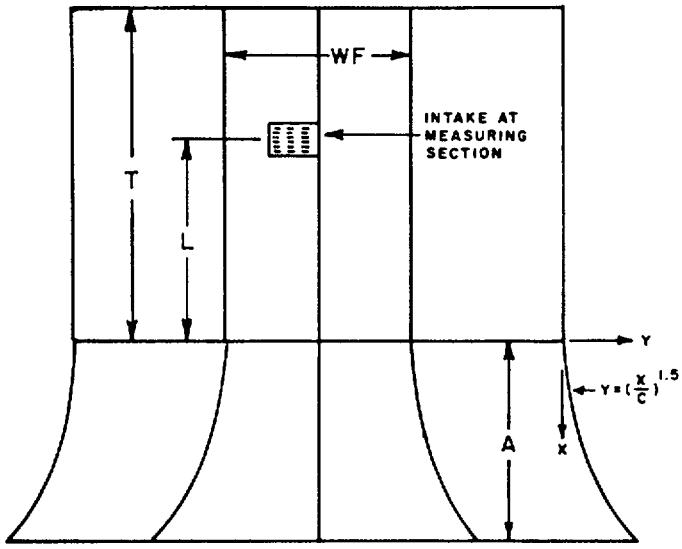
DESIGN CAPACITY 50 ft^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, FEET

L	7.0	ZW	1.0
T	9.0	ZF	4.0
H	2.375	S	0.03
W	7.0	A	≥ 5.0
WF	3.0	C	4.0

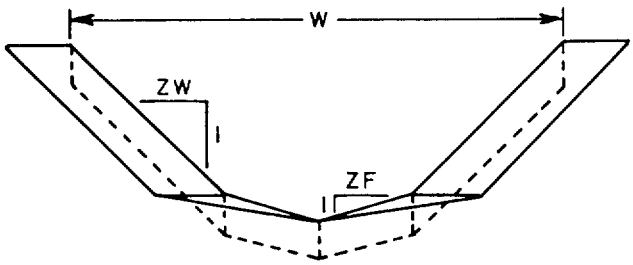
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN ft^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.031	0.084	0.236	0.682		
1.2		0.091	0.256	0.752		
1.4		0.098	0.274	0.817		
1.6		0.104	0.291	0.879		
1.8		0.110	0.307	0.937		
2.0	0.042	0.115	0.321	0.994		
3.0	0.050	0.138	0.381	1.244		
4.0	0.056	0.157	0.433	1.458		
5.0	0.062	0.173	0.480	1.648		
6.0	0.067	0.188	0.524			
7.0	0.072	0.201	0.566			
8.0	0.076	0.214	0.607			
9.0	0.080	0.225	0.645			

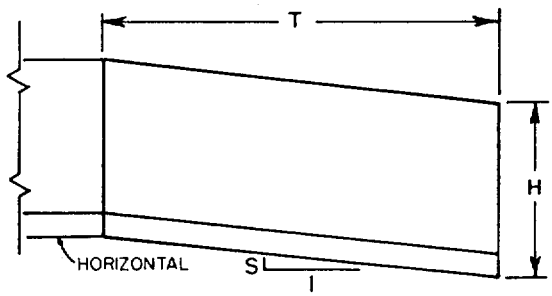
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

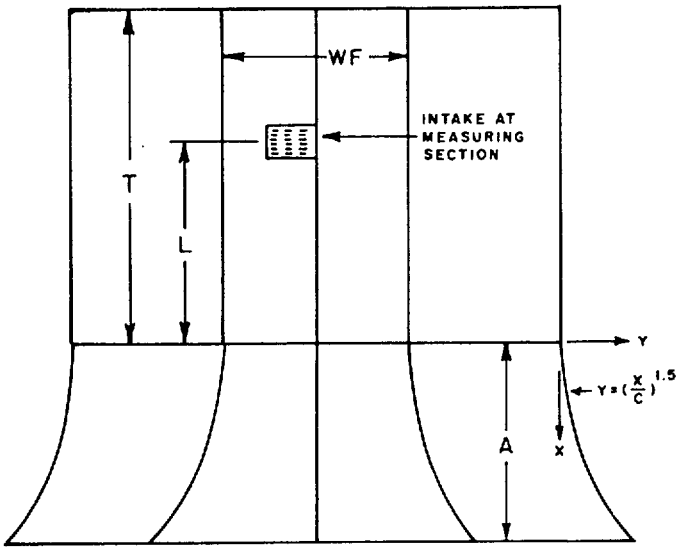
DESIGN CAPACITY 100 ft^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, FEET

L	8.0	ZW	1.0
T	12.0	ZF	4.0
H	3.125	S	0.03
W	10.0	A	≥ 6.0
WF	5.0	C	4.0

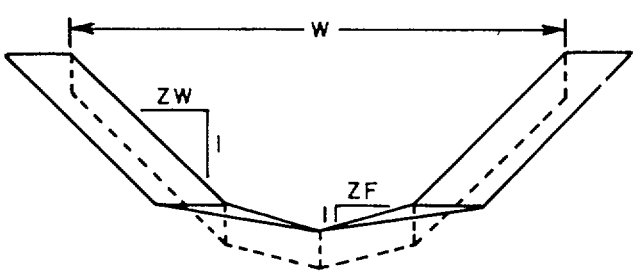
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN ft^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.032	0.084	0.234	0.643	2.066	
1.2		0.091	0.254	0.695		
1.4		0.098	0.272	0.745		
1.6		0.104	0.289	0.792		
1.8		0.109	0.304	0.837		
2.0	0.042	0.114	0.319	0.881		
3.0	0.050	0.137	0.382	1.081		
4.0	0.057	0.156	0.434	1.257		
5.0	0.062	0.172	0.479	1.416		
6.0	0.067	0.186	0.519	1.563		
7.0	0.072	0.200	0.554	1.700		
8.0	0.076	0.212	0.586	1.829		
9.0	0.081	0.223	0.616	1.950		

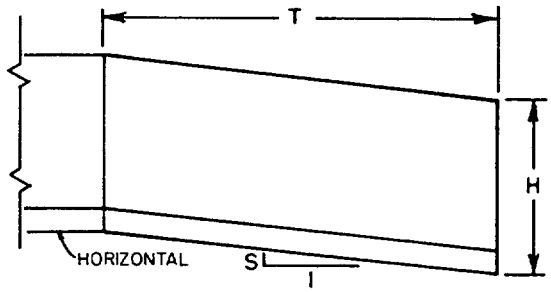
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

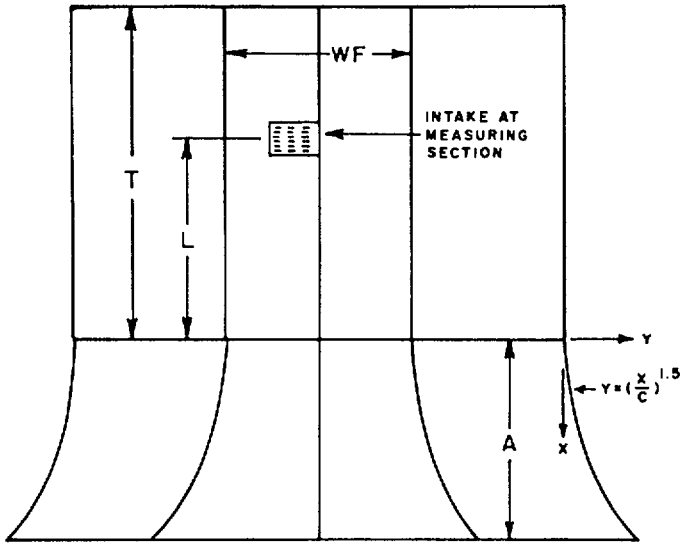
DESIGN CAPACITY 500 ft^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, FEET

L	10.0	ZW	1.0
T	18.0	ZF	4.0
H	6.0	S	0.03
W	18.0	A	≥ 6.0
WF	8.0	C	4.0

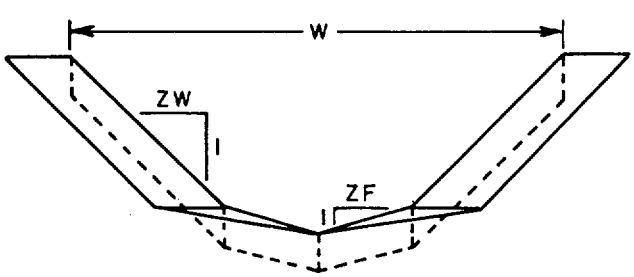
FLUME RATING

Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN ft^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.031	0.083	0.228	0.635	1.797	
1.2		0.089	0.247	0.689	1.977	
1.4		0.096	0.265	0.737	2.144	
1.6	0.038	0.101	0.281	0.782	2.303	
1.8		0.107	0.296	0.823	2.453	
2.0	0.042	0.112	0.311	0.862	2.597	
3.0	0.049	0.134	0.372	1.020	3.235	
4.0	0.056	0.152	0.423	1.154	3.782	
5.0	0.061	0.168	0.467	1.277	4.267	
6.0	0.066	0.182	0.507	1.392		
7.0	0.071	0.195	0.543	1.501		
8.0	0.075	0.206	0.576	1.604		
9.0	0.079	0.218	0.607	1.703		

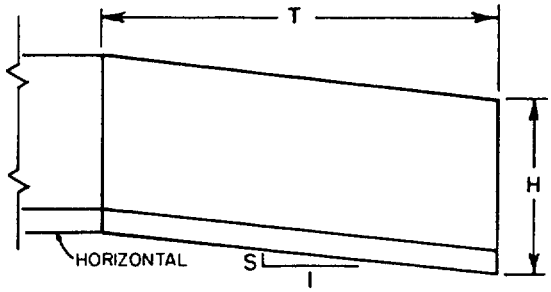
Design dimensions of Santa Rita flumes for a range of flow capacities—Con.

SANTA RITA FLUME

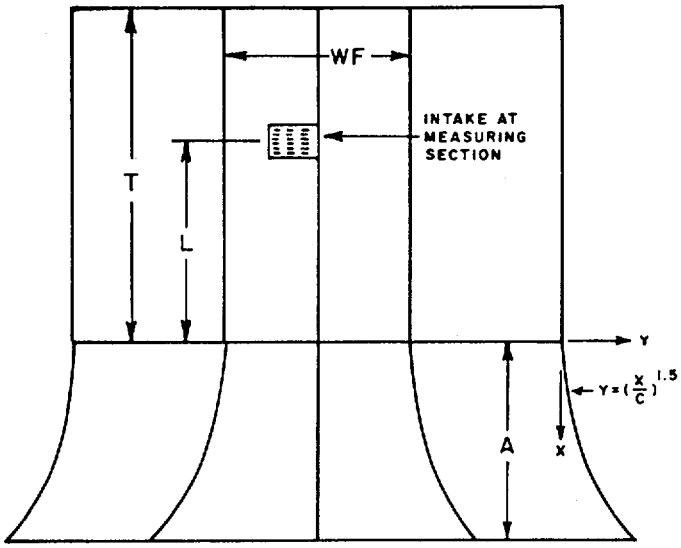
DESIGN CAPACITY 1000 ft^3/s



VIEW LOOKING DOWNSTREAM



SIDE VIEW, THROAT



TOP VIEW

FLUME DIMENSIONS, FEET

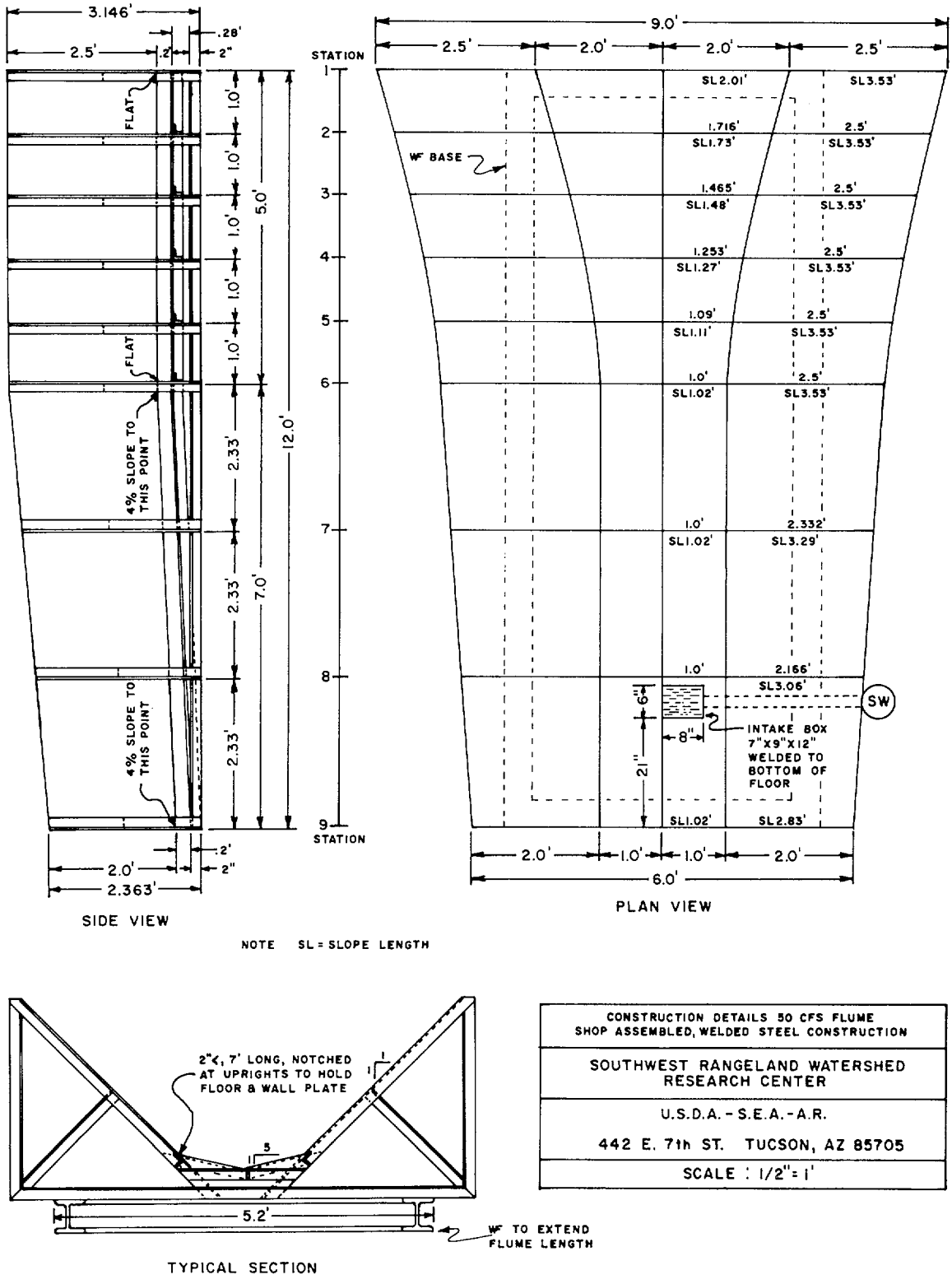
L	15.0	ZW	1.0
T	25.0	ZF	5.0
H	8.0	S	0.03
W	24.0	A	≥ 9.0
WF	10.0	C	4.0

FLUME RATING

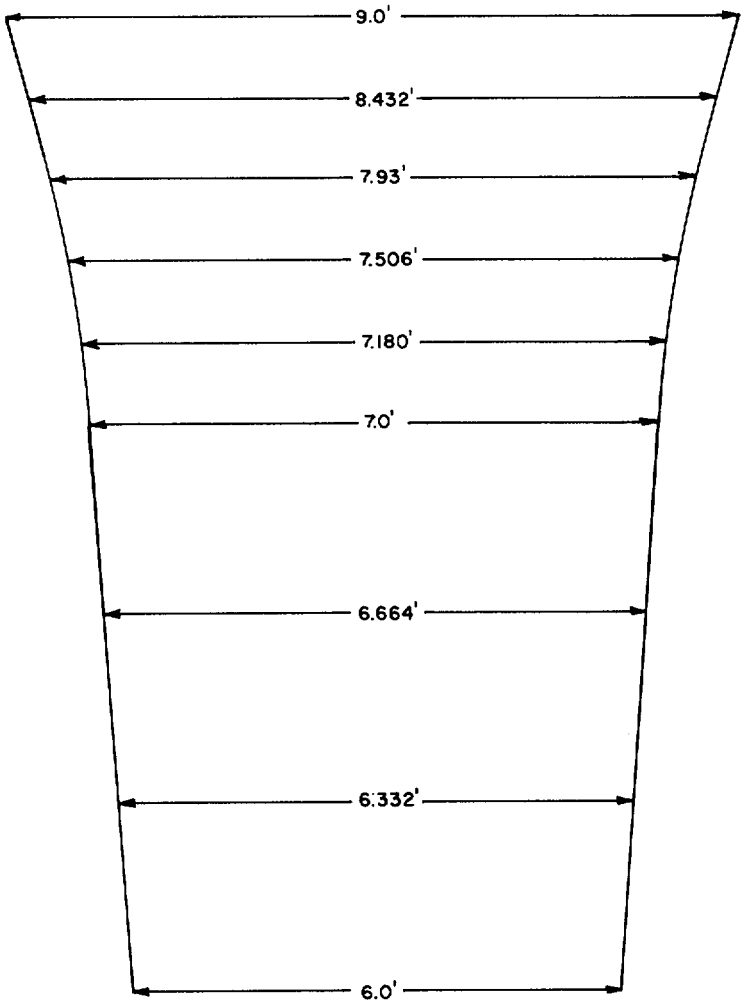
Q RANGE MULTIPLIER	HEAD IN METERS AT MEASURING SECTION					
	DISCHARGE, Q, RANGE IN ft^3/s					
	0.01	0.1	1.0	10	100	1000
1.0	0.029	0.073	0.197	0.55	1.539	5.485
1.2		0.079	0.214	0.597	1.691	
1.4		0.084	0.229	0.639	1.835	
1.6		0.089	0.243	0.678	1.971	
1.8		0.094	0.256	0.715	2.102	
2.0	0.038	0.098	0.268	0.749	2.227	
3.0	0.044	0.117	0.321	0.893	2.791	
4.0	0.050	0.132	0.366	1.007	3.281	
5.0	0.055	0.146	0.404	1.107	3.722	
6.0	0.059	0.158	0.438	1.201	4.124	
7.0	0.063	0.169	0.469	1.291	4.497	
8.0	0.067	0.179	0.498	1.377	4.846	
9.0	0.070	0.188	0.525	1.460	5.174	

Appendix D

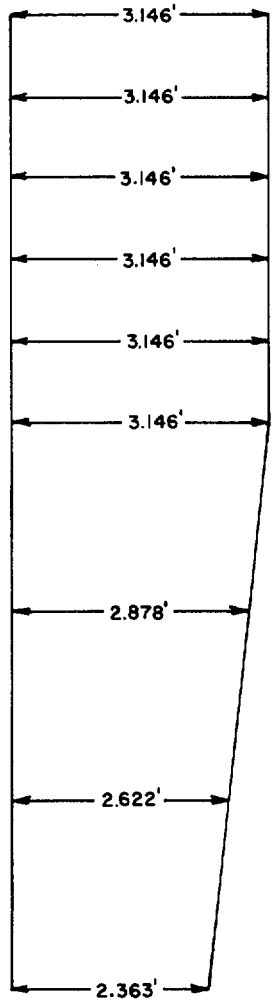
Construction drawings for a 1.5-m^3/s metal Santa Rita flume.



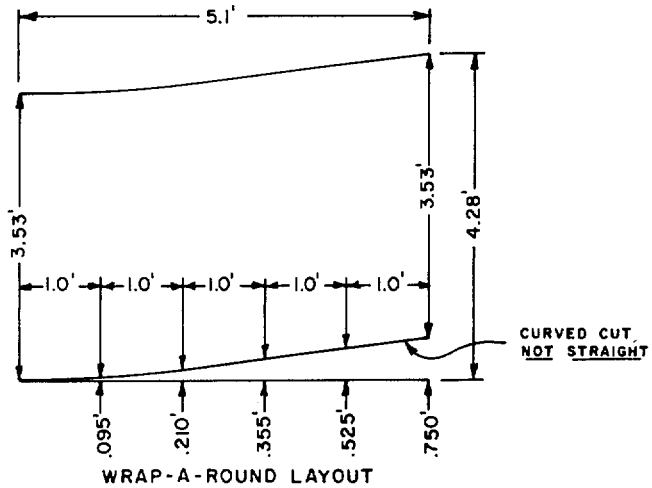
Construction drawings for a 1.5−m³/s metal Santa Rita flume—Con.



STATION WIDTHS FINISHED AT FLUME TOP



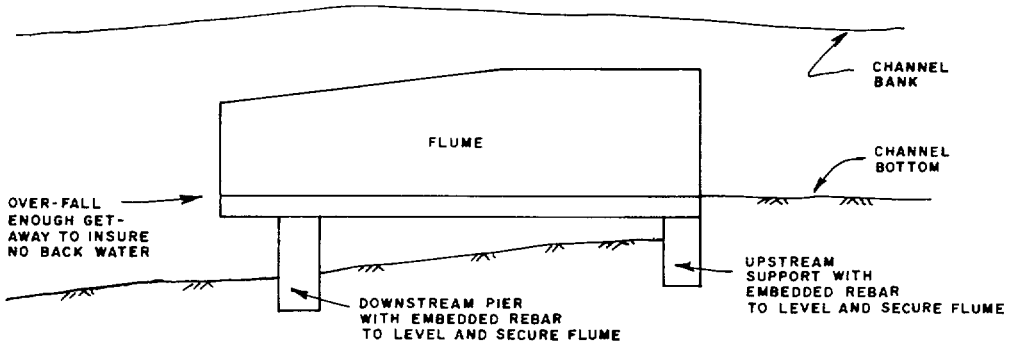
STATION HEIGHTS FINISHED FROM LEVEL FRAME



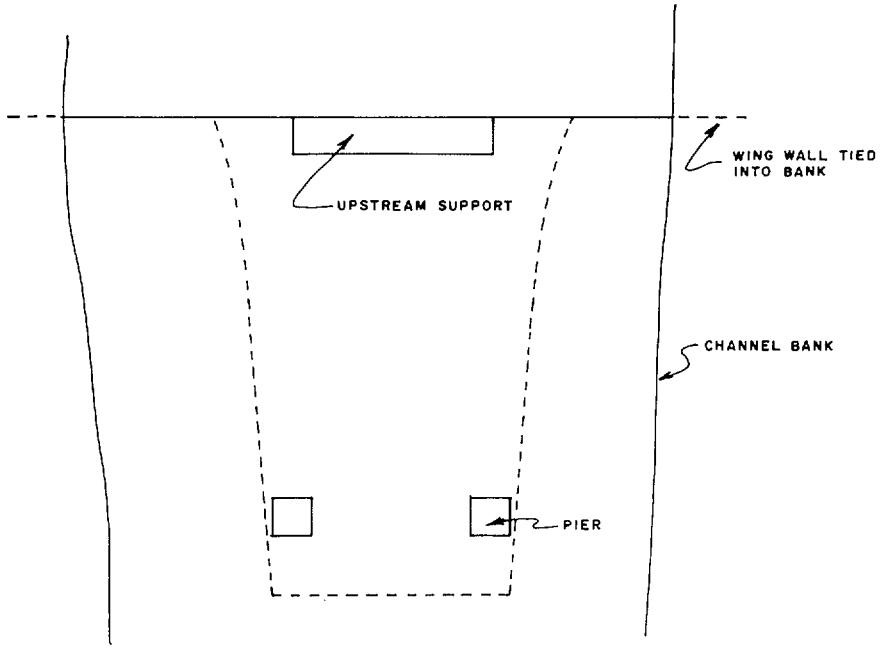
WRAP-A-ROUND LAYOUT

CONSTRUCTION DETAILS 50 CFS FLUME SHOP ASSEMBLED, WELDED STEEL CONSTRUCTION
SOUTHWEST RANGELAND WATERSHED RESEARCH CENTER
U.S.D.A. - S.E.A. - A.R. 442 E. 7th ST. TUCSON, AZ 85705
SCALE : 1/2" = 1'

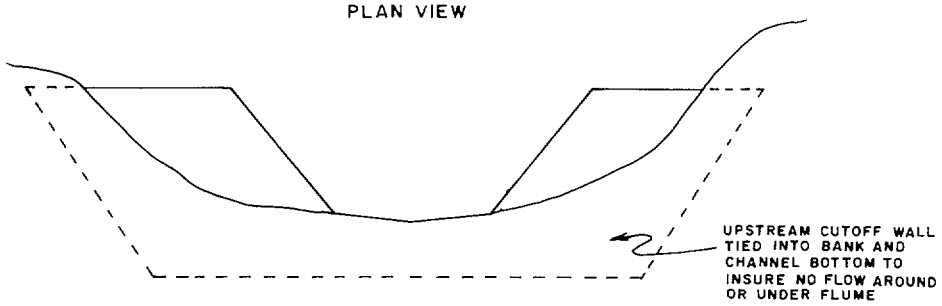
Construction drawings for a 1.5−m^3/s metal Santa Rita flume—Con.



PROFILE VIEW



PLAN VIEW



DOWNSTREAM VIEW

SCHEMATIC OF A
TYPICAL SITE INSTALLATION
NOT TO SCALE