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CHECK-DROP-ENERGY DISSIPATOR STRUCTURES IN IRRIGATION SYSTEMS

Gaylord V. Skogerboe, et al

Colorado State University

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by Gaylord V. Skogerboe,
Venus T. Somoray,
Wynn R. Walker

COLORADO STATE UNIVERSITY
FORT COLLINS, COLORADO
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Water Management Technical Report No. 9

by

Gaylord V. Skogerboe

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College of Engineering
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ABSTRACT

CHECK-DROP-ENERGY DISSIPATOR STRUCTURES IN IRRIGATION SYSTEMS

There is a large volume of available literature regarding small energy dissipator structures. Existing publications present basic information for particular types of structures. This information has been compiled in order to present present-day available design informatio[n] Also, research topics are delineated which would further assist the development of adequate design criteria.

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KEYWORDS - baffles, drops (structures), hydraulic design, *hydraulic structures, *open channel flow, overfalls, riprap, stilling basins.

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The effort reported herein was made possible through the support of the United States Agency for International Development under Contract No. AID/csd-2460, "Improving Capacity of CUSUSWASH Universities for Water Management for Agriculture." This report represents a portion of a much larger program aimed at consolidating existing literature regarding small irrigation structures, accomplishing some of the research necessary to fill gaps in present-day knowledge, and compiling the existing literature and on-going research into design-type manuals which can be used both in the classroom and by field personnel. As a part of the 211 (d) funding received by the Agricultural Engineering Department, a new course titled, "Irrigation Structures," was initiated winter quarter, 1971. An earlier draft of this report constituted a portion of the hand-out material used in this class

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GENERAL NOMENCLATURE

Symbol	Definition	Dimensions
A	Cross-sectional flow area in ft^2	L^2
b	bottom width of channel in ft	L
C_d	coefficient of discharge	
D_s	stone diameter	L
D_{15}	particle size for which 15 percent of the material is finer	L
D_{50}	particle size for which 50 percent of the material is finer	L
D_8	particle size for whicn 85 percent of the material is finer	
E	specific energy of flow	
F	Froude number (dimensionless)	
g	gravitational acceleration	L/T^2
h	drop height in feet	L
L_j	length of the hydraulic jump in feet	L
n	Manning's roughness coefficient	$L^{1/6}$
P	wetted perimeter in feet	L^3/T
Q	discharge in cfs	L^2/T
q	unit discharge in cfs/ft	L
R	hydraulic radius in ft	
S_b, S_o	bed slope in feet per foot	(dimensionless)
S_c	critical slope in feet per foot	(dimensionless)

Symbol	Definition	Dimensions
S_o	slope of the energy line	
S_s	specific gravity of stone particles	
U_{ec}	competent edge velocity	
U_{mc}	mean edge velocity	
V	velocity	
y_b	brink depth, or end depth for horizontal approach channel	
y_c	critical flow depth	
y_e	end depth	
y_s	depth of scour	
y_1	flow depth at bottom of drop	
y_2	sequent flow depth	
ρ	density of water	
ρ_s	density of particle	
τ_c	critical boundary shear	

Chapter 1

INTRODUCTION

The Irrigation and Drainage Research Conference (American Socity of Civil Engineers, 1964) conducted in 1964 delineated many of the research needs regarding "Small Low-Cost Hydraulic Structures for Conveyenace and Distribution Systems." This topic was one of six considered at the conference. Much literature exists throughout the world regarding small water management structures; however, much of the literature is concerned with particular structures for special uses rather than general information. The material of a general nature is scattered in numerous laboratory and project reports or design manuals. Because of the cost of an individual small structure, general rules-of-thumb and large factors of safety have been used in design and construction rather than to develop specific design information based on laboratory tests. Laboratory design information has been developed by various organizations when they have had to construct a large number of the same type of structure. Additional research efforts regarding small irrigation structures can be expected to reduce the cost of constructing such structures. When taken in the aggregate, the small saving on a large number of structures can amount to a significant total saving. Furthermore, better design information can result in structures which operate more nearly as intended by the designer.

The intent of an earlier report (Skogerboe, Walker, Hacking, and Austin, 1969) was to sort through the large volume of literature in an attempt to define the specific research needs regarding small water management structures used in irrigation distribution systems.

One of the more important topics frequently encountered in designing irrigation structures is the dissipation of energy.

There is a large amount of available literature regarding small energy dissipator structures. Existing publications present basic hydraulic relationships, as well as specific design information for particular types of structures. The problem, then, is to evaluate existing information to determine what research is necessary in order to develop generalized design procedures. If basic information is still needed, then experimental programs can be designed which will provide this information. Once the necessary basic research is available, various types of structures can be designed, but it will be desirable at least to verify these designs by laboratory model studies.

The three structural components of combination check-drop-energy dissipator structures are: (1) the inlet or approach section; (2) the stilling basin; and (3) the outlet channel. Each of the components are described below, including design procedures for various types of structures.

Chapter 2

CHECKS AND DROPS

In order to control the flow of the water in a canal or ditch, checks and drops are used. The flow characteristics of special interest are the slope and elevation of the water surface.

A check is any structure used to maintain or increase the level of flow (water surface elevation) in an open channel. The check must be designed so that flow needed downstream can pass over or through the structure while maintaining a constant upstream depth. The check should operate like an overflow weir, an orifice, or a combination of both. The basic criteria for design of a check are: sufficient height of the check to maintain the proper water surface for distribution of the water; and an energy dissipating structure to prevent erosion of the downstream area. Example of typical check structures are shown in Figure 2-1.

The flow capacity of the overflow weir can be calculated from the following relationships:

$$Q = CLh^{3/2} \tag{2-1}$$

or

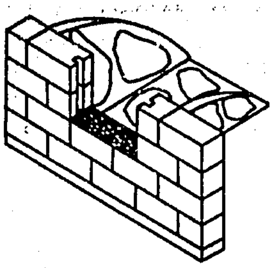
$$Q = C_d Lh(2gh)^{1/2} = CLh^n \tag{2-2}$$

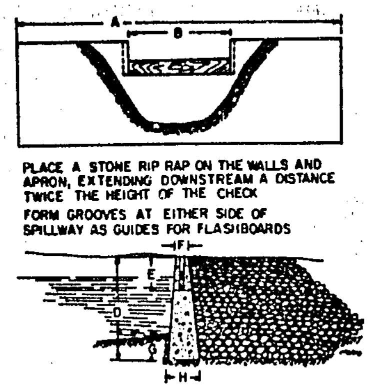
in which

Q = discharge in ft^3/sec

L = width across crest of check in ft

h = upstream depth in ft





A	B	D	E	F	G	H
14'-0"	4'-0"	4'-0"	1'-6"	0'-8"	1'-6"	1'-8"
19'-0"	6'-0"	6'-0"	2'-0"	0'-8"	1'-6"	2'-0"
24'-0"	8'-0"	8'-0"	3'-0"	0'-8"	2'-0"	3'-0"

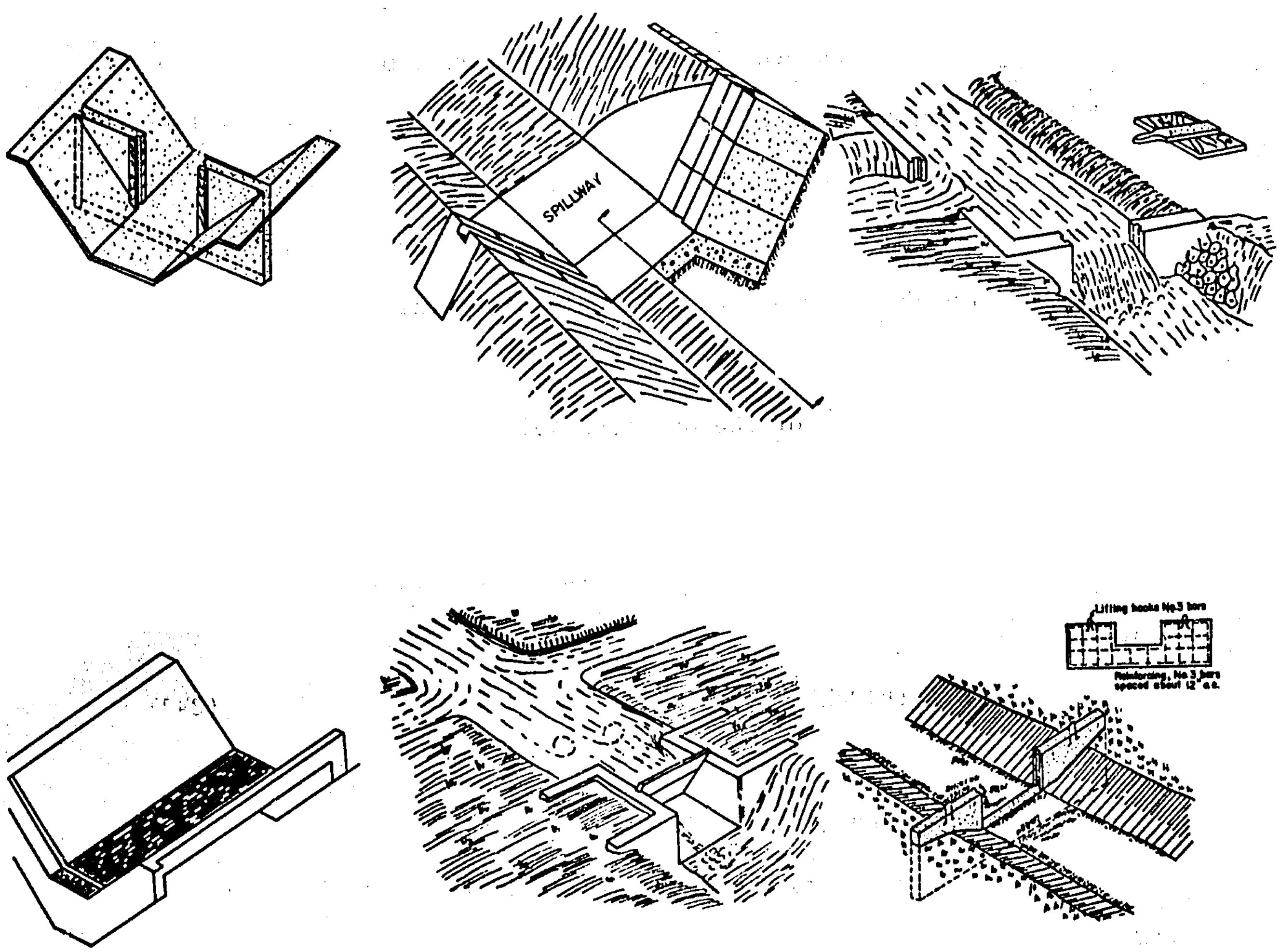


Figure 2-1. Typical check structures.

C_d = coefficient of discharge, dimensionless

$C = C_d(2g)^{1/2}$ = coefficient of discharge in $ft^{1/2}/sec$

L = overflow crest length in ft

h = head or water depth above the crest measured upstream from the check in ft.

The value of the exponent n for most overflow type checks is approximately 1.5. When the crest length L is large, variations in discharge result in relatively small changes in the upstream water level.

For earth lined canals, it is common to use some form of portable prefabricated check dam such as plastic, canvas, steel, cinder blocks, or precast concrete slabs. Where there is erosive soil, it will be necessary to provide the downstream side with an apron, wing wall, cut off, or riprap. In concrete lined canals, precast grooves can be used as gate guides. Another effective check that can be used is a portable steel gate which is held in place by hydrostatic pressure.

Flow through a sluice gate acting as a check is represented by the orifice equation:

$$Q = C_d A(2g\Delta h)^{1/2} = CA(\Delta h)^{1/2} \qquad (2\text{-}3)$$

in which

A is the area of the opening in ft^2

Δh is the difference in water surface elevation upstream

Flow through vertical needles acting as checks is flow through a vertical slot which has similarities to both a weir and an orifice.

Equation 2-3 can be used to measure the flow in which

Δh is the difference in upstream and downsteam water surface elevation

A is the area of the flow section in the slot.

When a check is used the velocity of flow is increased and the excess velocity must be dissipated downstream. The following chapters explain the design of both rigid and alluvial stilling basins.

Drops

Drops are used for gully control and for changing steep slopes to mild slopes. This change is accomplished by a vertical or inclined drop and an energy dissipating device. Drops are used to prevent erosion where the natural foilage will not grow, and to fill in gullies by slowing the flow and allowing the sediment to settle. The location of a drop should be upstream of those areas that have erosion problems. Checks are usually placed just above the drops, in order to raise the water surface sufficient for distribution to the fields.

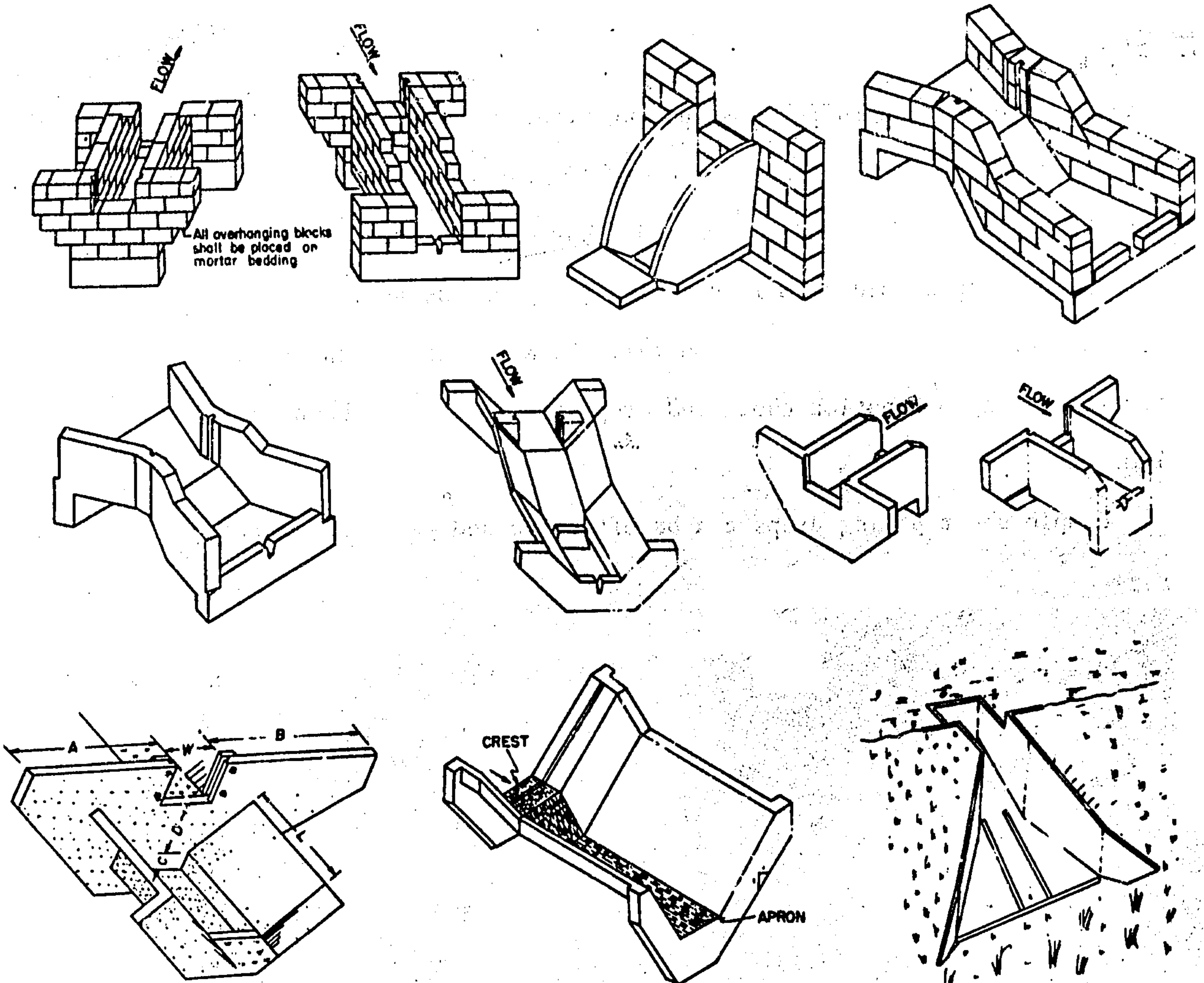
The flow over the top of the drop is given by the weir formula,

$$Q = CLh^{3/2} \tag{2-4}$$

The length L is the sum of the lengths of the three sides of a box inlet, the circumference of an arch inlet, or the crest length of a straight inlet.

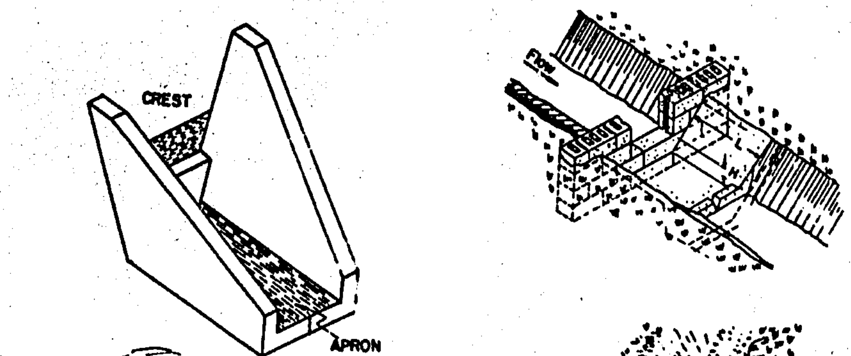
In loose boundary alluvial channels, vertical falls of 12 to 24 inches are commonly used. Riprap is usually placed on the downstream side to prevent erosion. In both rigid and alluvial boundary channels, a stilling pool or an energy dissipating structure should be used. The design of the stilling pool or energy dissipator is dependent upon the height of fall and the unit discharge. In field ditches, the maximum drop should be 6 inches with the grade being 1/2 inch per 100 feet to maintain proper control. Examples of typical drop structures and combination check and drop structures are shown in Figure 2-2.

Simple sheet piling drops can be effective and yet inexpensive.



Capacity of ditch in c.f.s.	Width of opening (W) ft.	(H) in.	(C) in.	(A) (B) ft
2	1	12	6	2
6	2	12	6	2
8	2½	15	6	2
10	3	18	8	2½
14	3½	18	8	3

DROP (D) Feet	LENGTH OF APRON (L) Feet
1	2½
1½	3
2	4
3	6



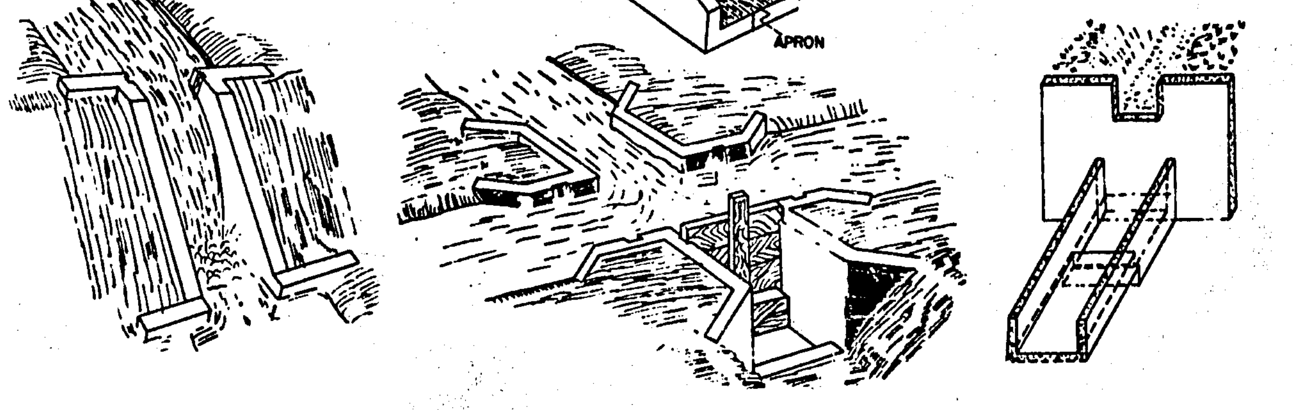


Figure 2-2. Typical drop structures and combination check-drop structures.

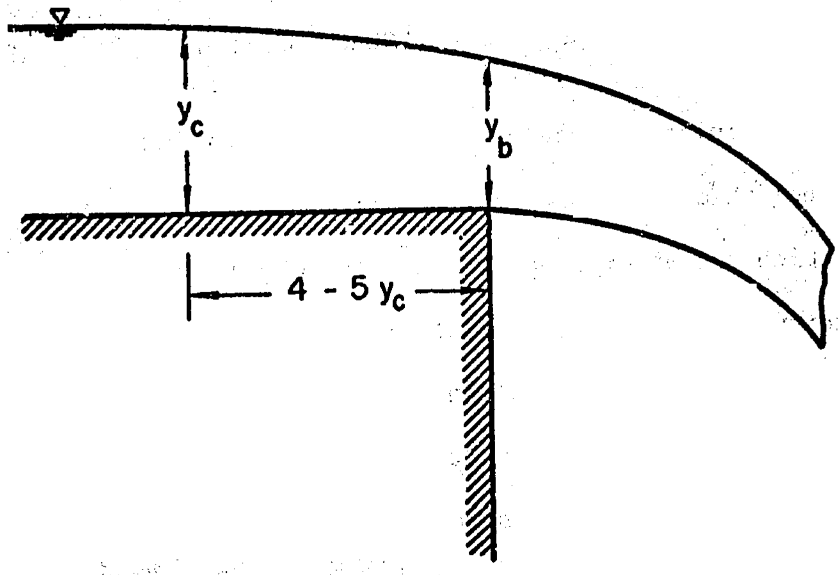
Chapter 3

APPROACH SECTION

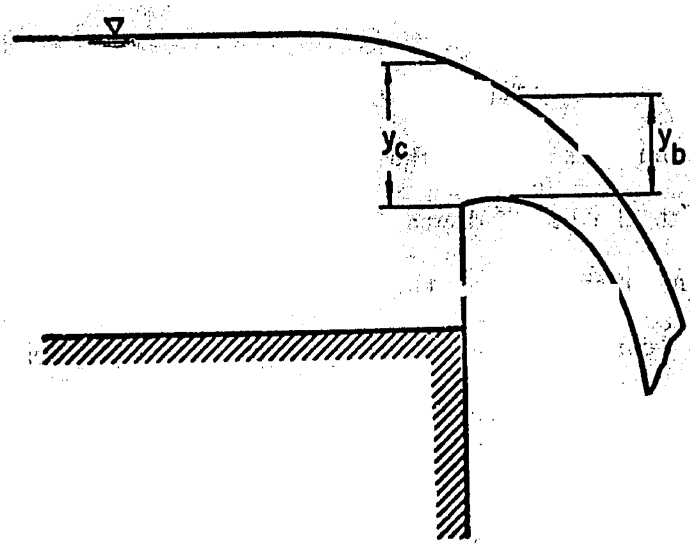
The inlet section of a drop structure can consist of an approach channel which has the same geometry as the irrigation canal or lateral, in which the water drops off the end of the channel, see Figure 3-1. Numerous drop structure inlets in irrigation systems are used as check structures, thereby resulting in a number of possible flow conditions regarding the jet entering the stilling basin. For example, the flow may pass over the check structure as shown in Figure 3-1b. Usually, a turnout structure(s) will be located immediately upstream from the check structure. Thus, the flow passing over the check is usually less than the design discharge for the channel, whereas the check structure might be removed when the full discharge capacity of the canal is to be conveyed downstream, thereby resulting in the flow condition shown in Figure 3-1a.

For the flow condition shown in Figure 3-1a, the flow depth at the end of the channel is called the brink depth Y_b . Rouse (1936) has determined that the brink depth for rectangular channels is equal to 0.715 Y_c (in which Y_c is the critical depth), provided the channel slope S_o is less than the critical slope S_c. Rajaratnam and Muralidhar (1964) have developed brink depth relationships for various channel geometries such as circular (pipes or culverts), trapezoidal, triangular, and parabolic. Thus, the brink depth can be determined for any type of commonly used channel shape.

For the flow condition shown in Figure 3-1b, critical depth occurs in the vicinity of the constriction. Both rectangular and trapezoidal



(a) Free overfall



(b) Constriction (check structure) overfall

Figure 3-1 Inlet or approach section to drop structure check structures are commonly used in irrigation systems. For either shape, the critical depth and its location can be computed with an accuracy sufficient for design purposes.

A more general solution to determining the flow depth at the inlet is obtained letting the approach channel have any bed slope S_o Rajaratnam and Muralidhar (1964) define the flow depth at the end of the approach channel the end depth Y_e Thus, the brink depth Y_b, has the same value as the end depth when the approach channel is horizontal ($S_o = 0$). A definition sketch for the general case is shown in Figure 3-2.

The end depth ratio curves for triangular, parabolic, rectangular, and circular channels are shown in Figure 3-3. The end depth ratio Y_e/Y_c is plotted against the slope ratio S_o/S_c in which S_o is the bed slope and S_c is the critical slope (slope at which the normal flow depth is equal to critical depth). A negative slope ratio means that the bed slope has an adverse gradient. The end depth ratio curves for the trapezoidal channels with different values of the shape parameter my_c/b, in which m is the side slope and b is the bottom width, will lie between the curves for the triangular and rectangular channels.

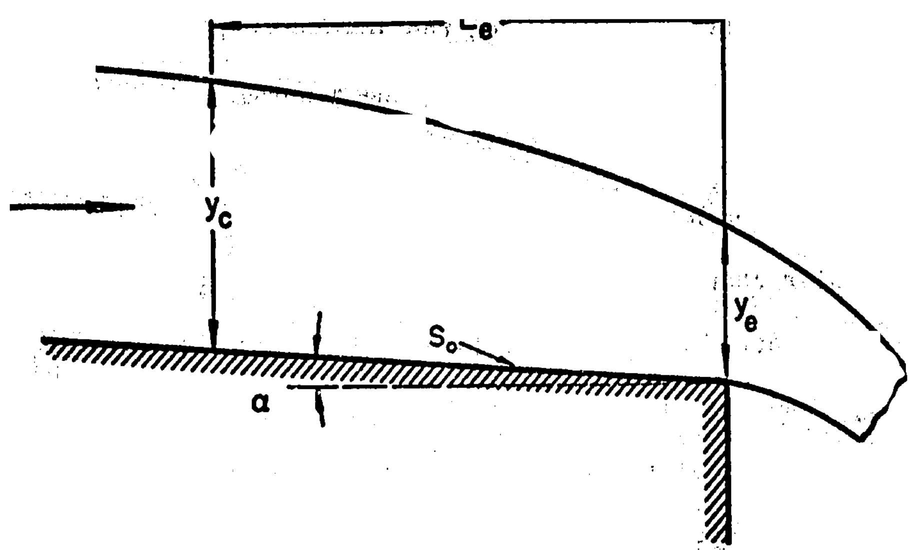


Figure 3-2 Definition sketch for end depth

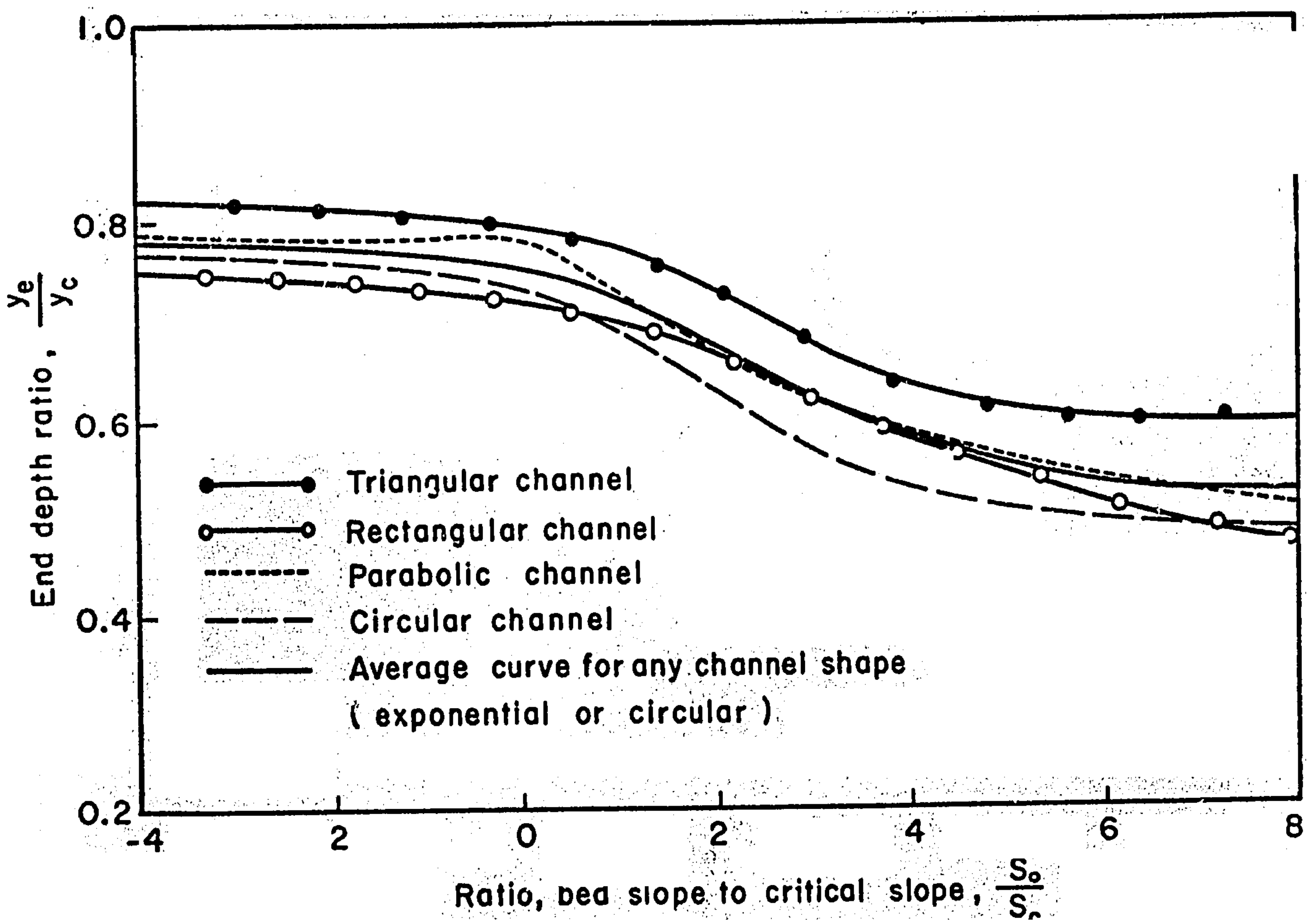


Figure 3-3 End depth ratio for various channel shapes

Chapter 4

INCLINED DROPS

An inclined drop structure is shown in Figure 4-1. Knowing a control point in the approach channel (critical depth, brink depth, or end depth), the design discharge, slope of the chute, and cross-sectional geometry of the chute section, the flow depth at the bottom of the chute y_1 can be computed. Then, using the hydraulic jump equation, the sequent flow depth y_2 can be computed. A number of designs for stilling basin structures have been developed in the past, including the St. Anthony Falls (SAF) stilling basin (Blaisdell, 1948), Stilling Basin II of the USBR (Peterka, 1964), Stilling Basin III of the USBR (Peterka, 1964), and the baffled apron developed by the USBR (Peterka, 1964). A general description of these structures is given below. In addition, there are numerous studies regarding appurtenances, such as sills and blocks, in controlling the hydraulic jump (Blaisdell, 1948; Donnelly and Blaisdell, 1965; Forster and Skrinde, 1950; Katsaitis, 1966; Peterka, 1964; and Rand, 1965,1966).

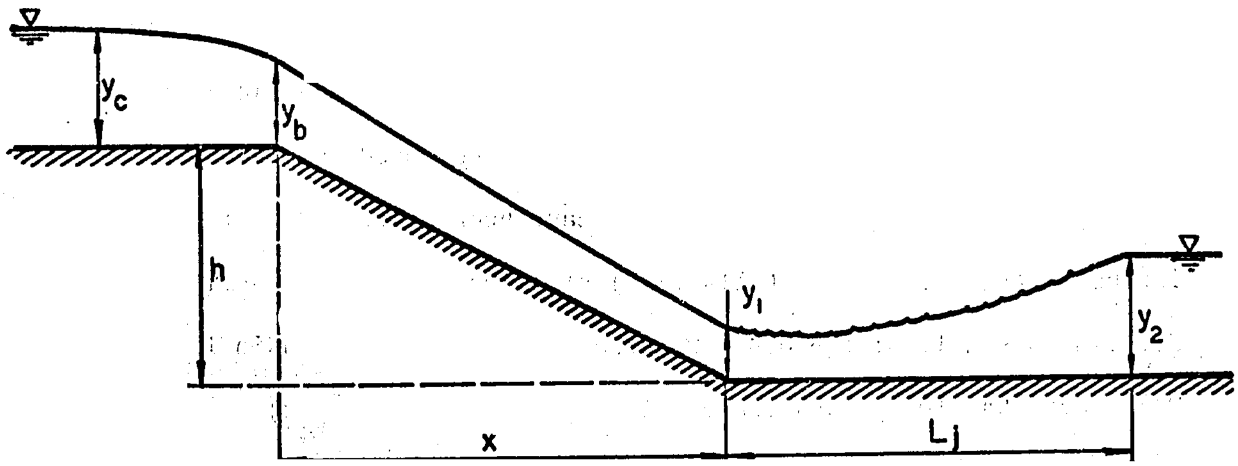


Figure 4-1 Inclined drop structure

Water Surface Profiles

The effective design of an inclined drop is based on the design discharge, depth at the inlet, and channel shape, slope, roughness, and length. The usual case for inclined drops is for the slope of the section to be in the steep range so that the flow control point will be at the inlet. From information concerning these variables either measured or evaluated from the preceeding sections, the next design step is to compute the water surface profile from the inlet to the bottom of the structure or the location of the energy dissipation system.

Starting with the conditions at the control point, the flow profile down the incline can be computed using numerous methods described by most texts on open channel hydraulics. The method described herein was proposed by Prasad (1970) employing a numerical solution to the trial and error procedure. The differential equation of gradually varied flow is used in conjunction with Manning's formula for describing the energy slope. The gradually varied flow equation can be differentiated and expressed as:

$$\frac{dy}{dx} = \frac{S_o - S_e}{1 - \alpha \dfrac{Q^2 T}{g A^3}} \tag{4-1}$$

in which

y is the depth

x is the distance along the channel

S_o is the bed slope

S_e is the energy gradient

α is the velocity head coefficient

Q is the discharge

T is the top width of the channel

g is the acceleration of gravity

and

A is the cross-sectional area of the channel.

The evaluation of S_e when using the Manning formula can be expressed as:

$$S_e = \frac{n^2 V^2}{2.21 R^{4/3}} \tag{4-2}$$

in which

n is Manning's roughness coefficient

R is the hydraulic radius for the section

and

V is the flow velocity.

The numerical solution proposed by Prasad (1970) has been written for a digital computer and the program listing has been included in the Appendix. The procedure is based on the following equation

$$Y_{i+1} = Y_i + \Delta y \tag{4-3}$$

in which the subscript i describes the distance increment along the channel. The value of Δy can be computed from the value obtained in Equation 4-1 multiplied by the incremental value of distance along the channel ΔX. If the value of ΔX is very small, the value obtained in Equation 4-1 can be assumed to vary linearly between stations in the solution. Assuming this condition, Equation 4-3 can be rewritten to form the general basis for the solution:

$$Y_{y+1} = y_i + \left[\frac{\left(\frac{dy}{dx}\right)_{i+1} + \left(\frac{dy}{dx}\right)_i}{2}\right] \Delta X \tag{4-4}$$

The computational procedure, then, is as follows:

(1) Compute dy/dx from Equation 4-1 using the initial value of y or the previous value after the initial step solution;

(2) Assume $(dy/dx)_{i+1} = (dy/dx)_i$;

(3) Evaluate the depth y_{i+1} using the step (2) approximation;

(4) Compute a new value of $(dy/dx)_{i+1}$ based on the value of y_{i+1} obtained in step (3); and

(5) Repeat steps (3) through (5) until the two estimates of $(dy/dx)_{i+1}$ come within the desired degree of accuracy and then using the new value of depth, repeat this process until the last depth is obtained.

It should be noted that the preceeding technique is unstable when the depth approaches critical depth and consequently the problem occurring with critical depth at the inlet should be avoided. Studies have indicated that the inlet depth, the depth at the downstream end of the approach channel, will vary with approach condition and nature of the drop. The inlet depth will approach critical depth when the incline slope is in the very small value of the steep slope. For the general case of the mide approach channel and the very steep incline, a value of between 0.715 times the critical depth y_c and 0.8 y_c will yield good results.

Free Hydraulic Jump

The efficient design of any drop structure employing the hydraulic jump as the means of energy dissipation must rely heavily on the evaluation of the sequent depth ratio y_2/y_1 in which the subscripts refer to the sections after and before the jump respectively. The most common technique used to describe this depth ratio in either the horizontal or the sloping channel is through the use of the force-momentum flux principal,

$$A_1 K_1' y_1 - A_2 K_2' y_2 = \frac{Q^2}{g}\left(\frac{1}{A_2} - \frac{1}{A_1}\right) \qquad (4\text{-}5)$$

in which

A is the cross-sectional areas

K' is the portion of the depth to the center of gravity for the channel shape

Q is the discharge

and

g is the acceleration of gravity.

In this section only the solution to Equation 4-5 concerning the horizontal channels will be presented. Equation 4-5 has been evaluated by numerous authors for the rectangular, triangular, trapezoidal, circular, and parabolic channel shapes. Experimental data have also been gathered to substantiate these equations. Silvester (1964) has probably presented the most complete analysis, along with a substantial literature review and generation of experimental data. The following is a partial summary of this publication.

Rectangular channels. Solution to Equation 4-5 for the rectangular case results in,

$$y_2/y_1 = \tfrac{1}{2}\left(\sqrt{1+8F_1^{\,2}} - 1\right) \qquad (4\text{-}6)$$

in which F_1 is the Froude number at section 1 described by the equation:

$$F_r = \frac{Q^2}{A^2 gy} \qquad (4\text{-}7)$$

Jeppson (1965) provided useful nomographs for the solution to Equation 4-6 as well as for the cases involving triangular and trapezoidal shapes. These nomographs are presented in Figures 4-2 and 4-3.

Trapezoidal channels. With the addition of the side slope as a variable, the analysis of the trapezoidal cross-section is somewhat more complicated than the rectangular. Using the equivalent width, b' Equation 4-5 reduces to:

$$K'_2 \left(\frac{y_2}{y_1}\right)^2 \left(\frac{b'_2}{b'_1}\right) - K'_1 = F_1^{\,2} \left[1 - \frac{b'_1\, y_1}{b'_2\, y_2}\right] \tag{4-8}$$

Massey (1961) represented a shape factor K after algebraic manipulation as

$$K = \frac{b}{ny_1} \tag{4-9}$$

in which n is the side slope and b is the bottom width. Using this relationship, the Froude number can be written as

$$F_1 = \frac{Q}{ng^{\frac{1}{2}}\, y_1^{\,5/2}} \; \frac{1}{K+1} \tag{4-10}$$

Triangular channels. The triangular channel is a special case of the trapezoidal channel in which $b = 0$. Further, it is somewhat simplified by the fact that $K'_1 = K'_2 = 1/3$ and thus $A_1/A_2 = y_1^{\,2}/y_2^{\,2}$ resulting in

$$\left(\frac{y_2}{y_1}\right)^3 - 1 = 3F_1^{\,2} \left[1 - \left(\frac{y_1}{y_2}\right)^2\right] \tag{4-11}$$

Circular channels. Hydraulic jumps in circular channels must be divided into two parts: (1) when y_2 is less than the diameter D and (2) when y_2 is greater than the channel diameter. For the first case,

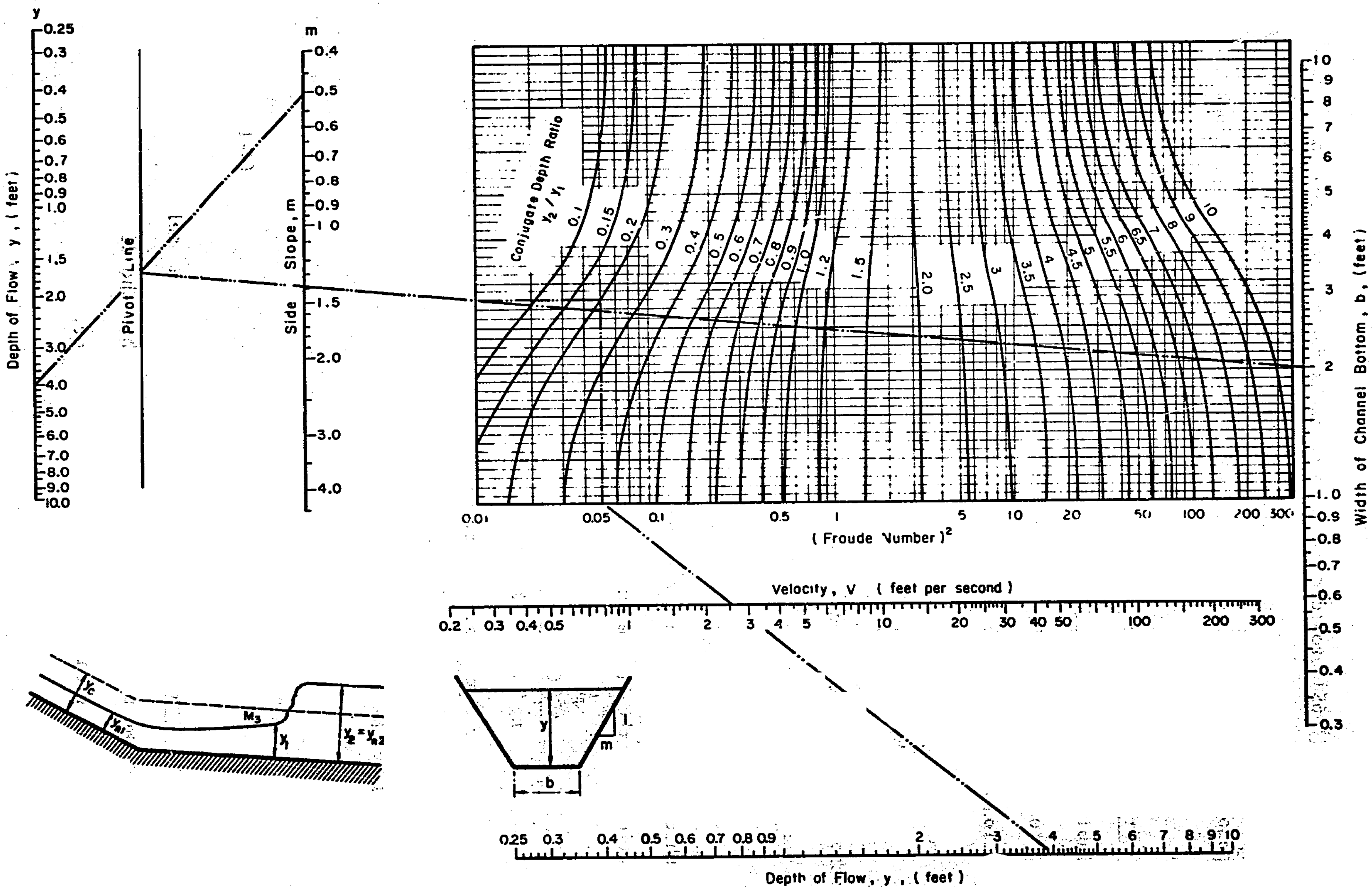


Figure 4-2. Nomogram for the solution of hydrolic jump in rectangular and triangular channels. (Taken from Jeppson, 1965)

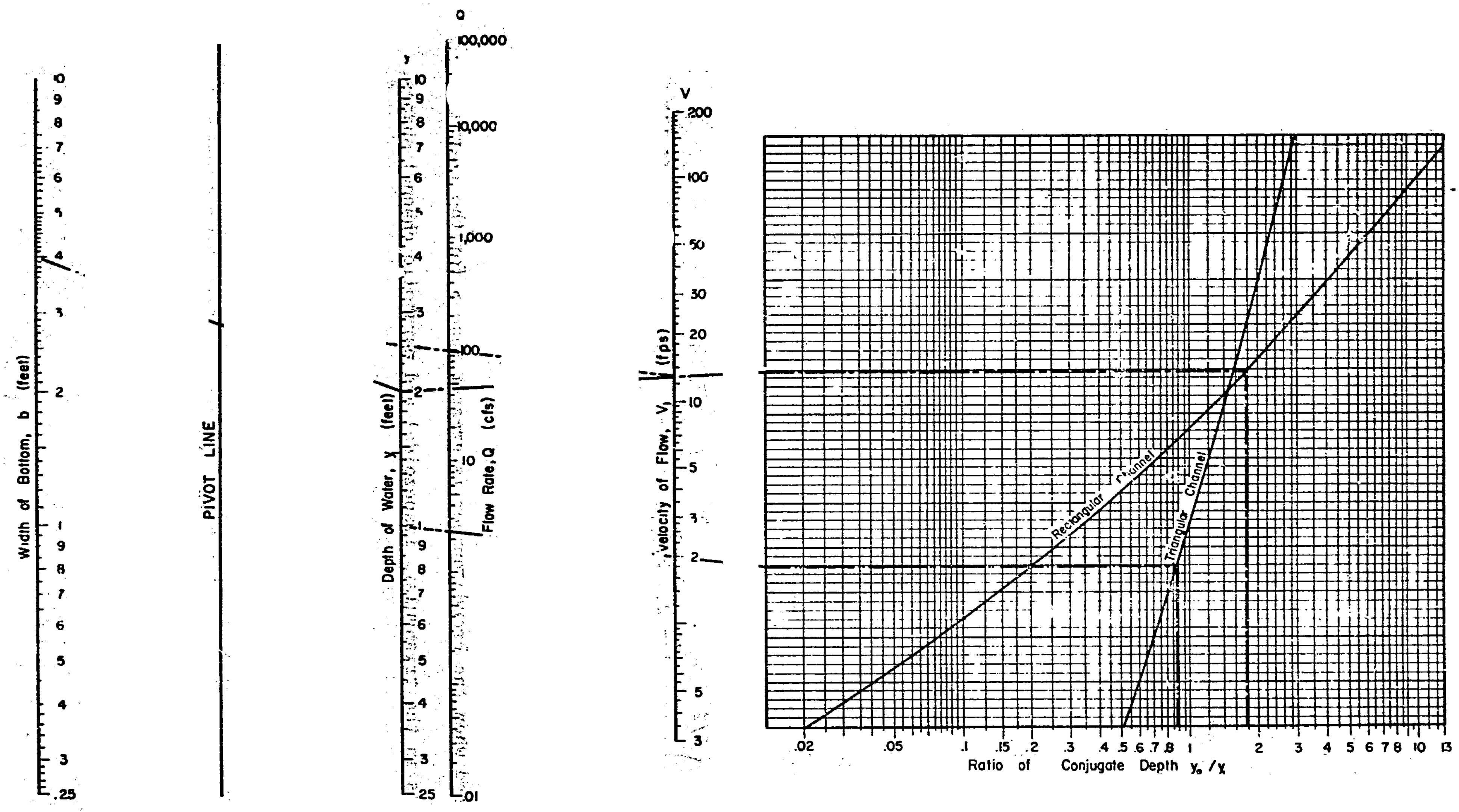


Figure 4-3. Nomogram for solution of hydrolic jump in trapezoidal channels.
(Taken from Jeppson, 1965)

Equation 4-5 is

$$K_2'\left(\frac{y_2}{y_1}\right)\frac{M_2}{M_1} - K_1' = F_1^{\,2}\left[1 - \frac{M_1}{M_2}\right] \tag{4-12}$$

in which M is the multiplying factor which gives the area of a segment of the circle ($A = MD^2$) and is either evaluated from tables or geometry. In the second case, Equation 4-5 becomes,

$$\frac{M_2}{M_1}\left(\frac{y_2}{y_1}\right) - \frac{1}{2}\left(\frac{M_2}{M_1}\right)\frac{D}{y_1} - K_1' = F_1^{\,2}\left[1 - \frac{M_1}{M_2}\right] \tag{4-13}$$

For both of these cases, it is better to compute y_2/y_1 than to use a figure. However, the second case is as yet relatively undefinable so care should be exercised when working with this circumstance.

Parabolic channels. The sequent depth ratio for any parabolic shape can be expressed as;

$$\left(\frac{y_2}{y_1}\right)^{5/2} - 1 = 2.5\ F_1^{\,2}\left[1 - \left(\frac{y_1}{y_2}\right)^{3/2}\right] \tag{4-14}$$

For convenience, Equation 4-12 has been plotted in Figure 4-4.

Equation limitations. Although fairly good agreement has been found experimentally with the preceeding equations, some caution should be exercised when the Froude number is greater than 5 for the flatter side slopes in the triangular, trapezoidal and parabolic channel shapes.

SAF Stilling Basin

The St. Anthony Falls (SAF) stilling basin is shown in Figure 4-5. This structure should give adequate performance for a range of Froude numbers F_1 at section 1 varying from 1.7 to 17. Normally, the length of basin is less than twice the sequent flow depth y_2 which is very short. Thus, this particular structure is quite economical. Blaisdell

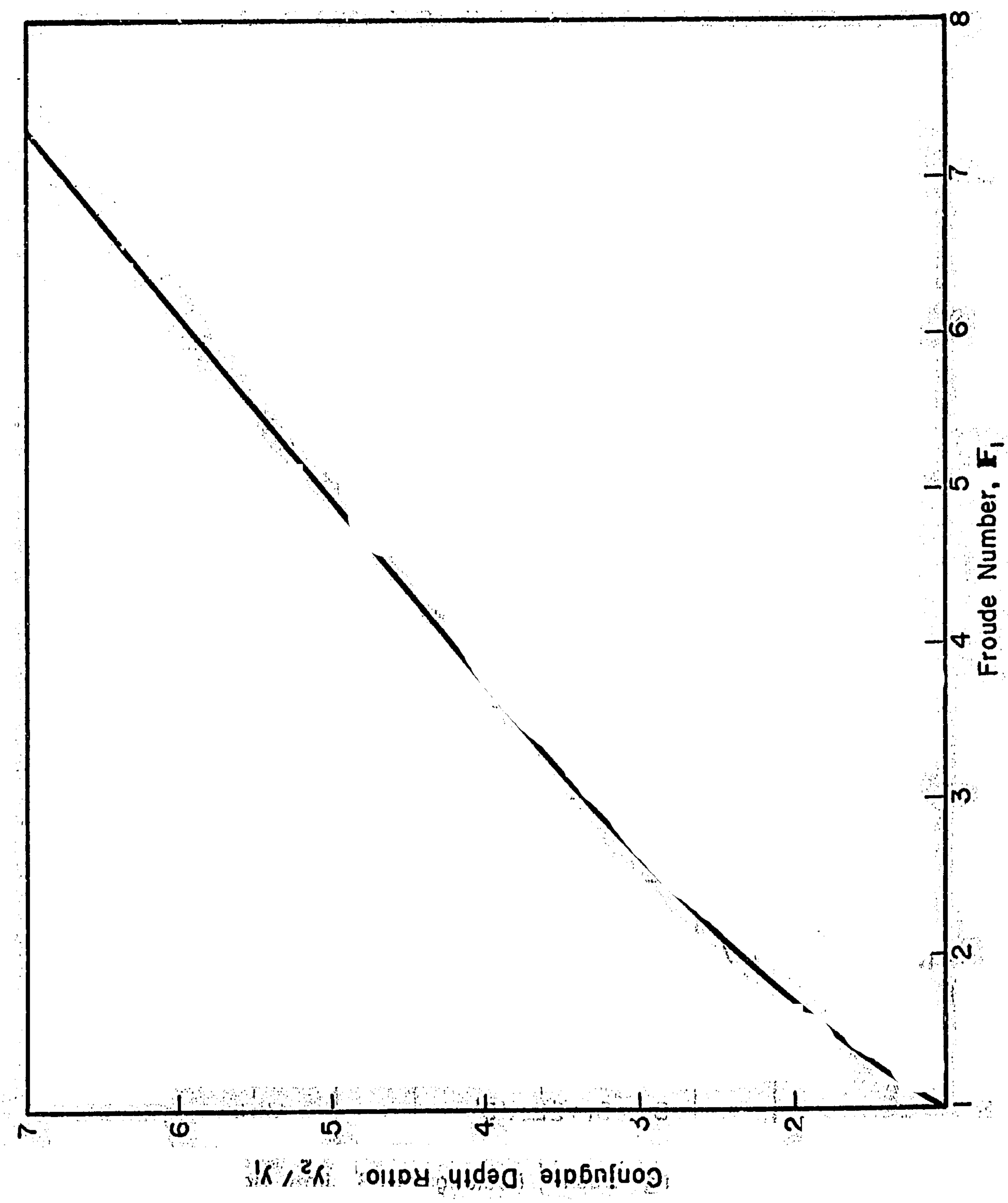


Figure 4-4 Relation between Froude number and sequent depth ratio for parabolic channels

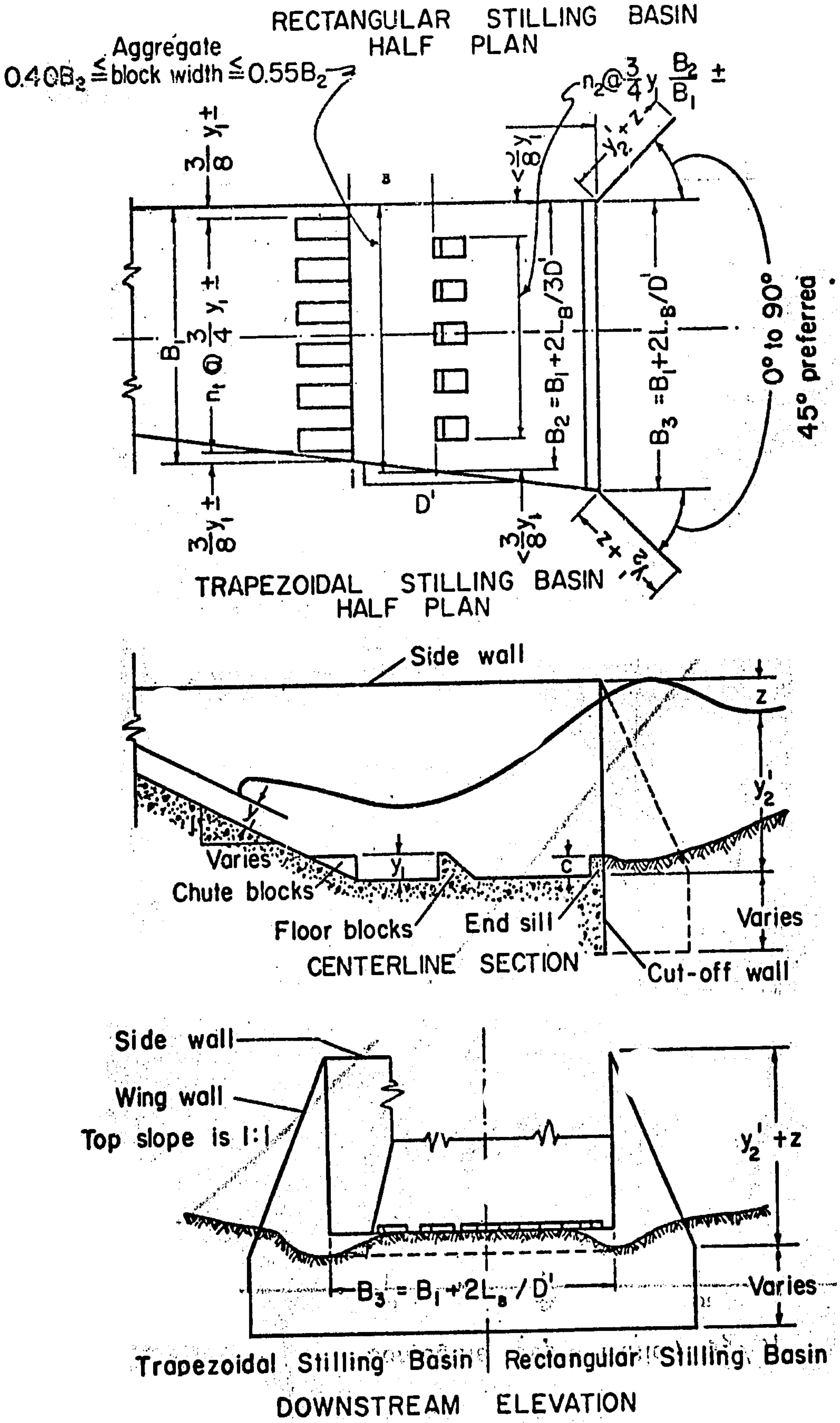


Figure 4-5 SAF Stilling Basin II

(1948) has listed the criteria for designing the SAF stilling basin:

1. The length L_B of the stilling basin for Froude numbers between $F_1 = 1.7$ and $F_1 = 17$ is determined by $L_B = 4.5y_2/F_1^{0.76}$.
2. The height of the chute blocks and floor blocks is y_1 , and the width and spacing are approximately $0.75y_1$.
3. The distance from the upstream end of the stilling basin to the floor blocks is $L_B/3$.
4. No floor blocks should be placed closer to the side wall than $3y_1/8$.
5. The floor blocks should be placed downstream from the openings between the chute blocks.
6. The floor blocks should occupy between 40 and 55% of the stilling basin width.
7. The widths and spacings of the floor blocks for diverging stilling basins should be increased in proportion to the increase in stilling-basin width at the floor-block location.
8. The height of end sill is given by $c = 0.07y_2$, in which y_2 is the theoretical sequent depth corresponding to y_1 .
9. The depth of tailwater above the stilling basin floor is given by $y_2' = (1.10 - F_1^2/120)\ y_2$, for $F_1 = 1.7$ to 5.5 ; by $y_2' = 0.85y_2$, for $F_1 = 5.5$ to 11; and by $y_2' = (1.00 - F_1^2/800)\ y_2$, for $F_1 = 11$ to 17.
10. The height of the side wall above the maximum tailwater depth to be expected during the life of the structure is given by $z = y_2/3$.
11. Wing walls should be equal in height to the stilling-basin side walls. The top of the wing wall should have a slope of 1 on 1.

12. The wing wall should be placed at an angle of 45° to the outlet center line.
13. The stilling-basin sidewalls may be parallel (as in a rectangular stilling basin) or they may diverge as an extension of the transition side walls (as in a trapezoidal stilling basin).
14. A cutoff wall of nominal depth should be used at the end of the stilling basin.
15. The effect of entrained air should be neglected in the design of the stilling basin.

USBR Stilling Basin II

The U.S. Bureau of Reclamation (Peterka, 1964) has developed design criteria for numerous stilling basins under varying hydraulic conditions. Stilling Basin II has been developed as an energy dissipator for spillways and large canal structures. A definition sketch for this stilling basin is shown in Figure 4-6. The design criteria are listed below:

1. Set apron elevation to utilize full sequent tailwater depth, plus an added factor of safety if needed. An additional factor of safety is advisable for both low and high values of the Froude number (see Figure 4-7). A minimum margin of safety of 5 percent of y_2 is recommended.
2. Basin II may be effective down to a Froude number of 4 but the lower values should not be taken for granted.
3. The length of basin can be obtained from the intermediate curve on Figure 4-8.
4. The height of chute blocks is equal to the depth of flow entering the basin, or y_1 (Figure 4-6). The width and spacing should be equal to approximately y_1 ; however, this may be varied to

eliminate fractional blocks. A space equal to $y_1/2$ is preferable along each wall to reduce spray and maintain desirable pressures.

5. The height of the dentated sill is equal to $0.2y_2$, and the maximum width and spacing recommended is approximately $0.15y_2$. On the sill, a dentate is recommended adjacent to each side wall (Figure 4-6). The slope of the continuous portion of the end sill is 2:1. For narrow basins, which contain only a few dentates according to the above rule, it is advisable to reduce the width and the spacing. However, widths and spaces should remain equal. Reducing the width and spacing actually improves the performance in narrow basins; thus, the minimum width and spacing of the dentates is governed only by structural considerations.

6. It is not necessary to stagger the chute blocks with respect to the sill dentates. In fact, this practice is usually inadvisable from a construction standpoint.

7. The verification tests on Basin II indicated no perceptible change in the stilling basin action with respect to the slope of the chute preceding the basin. The slope of chute varied from 0.6:1 to 2:1 in the verification tests. Actually, the slope of the chute does have an effect on the hydraulic jump when the chute is nearly horizontal. It is recommended that the sharp intersection between chute and basin apron, Figure 4-6, be replaced with a curve of reasonable radius $(R \geq 4y_1)$ when the slope of the chute is 1:1 or greater. Chute blocks can be incorporated on the curved face as readily as on the plane surfaces.

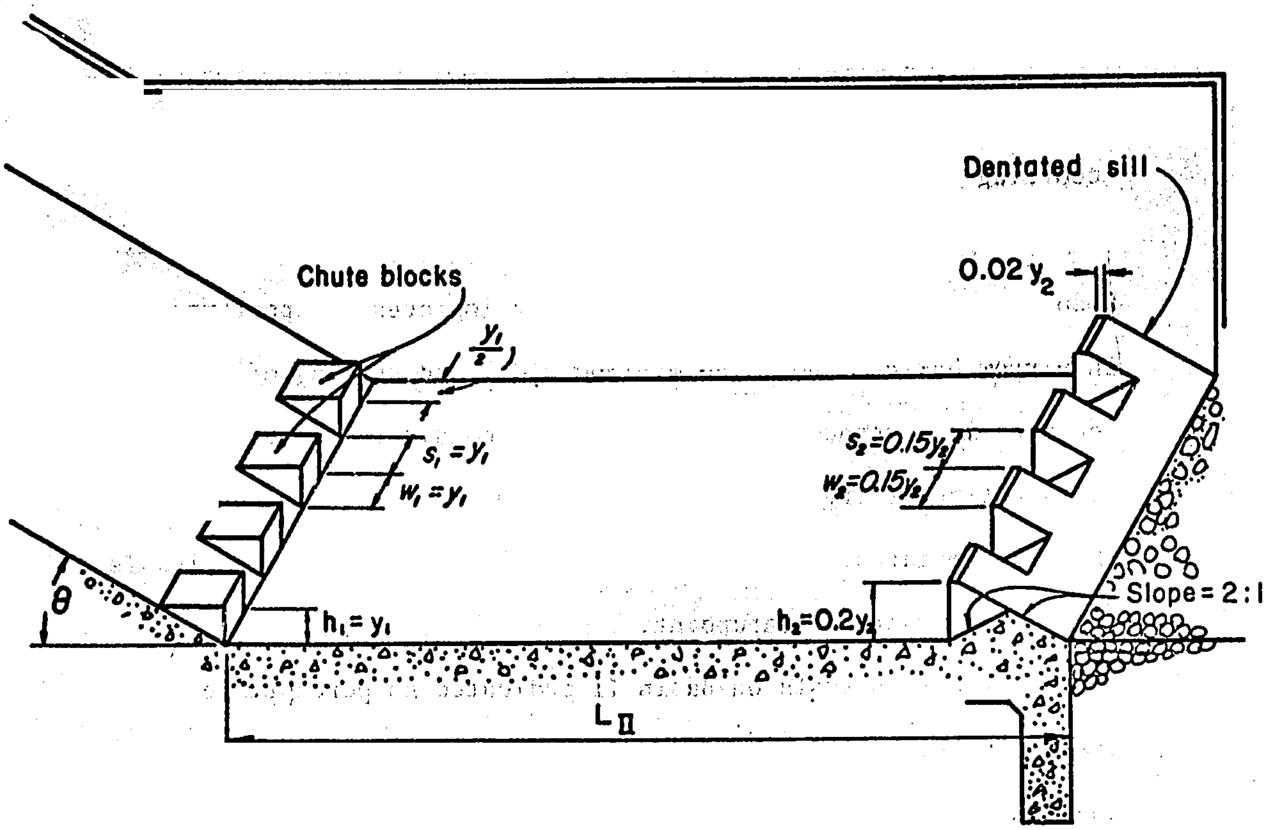


Figure 4-6 USBR Stilling Basin II

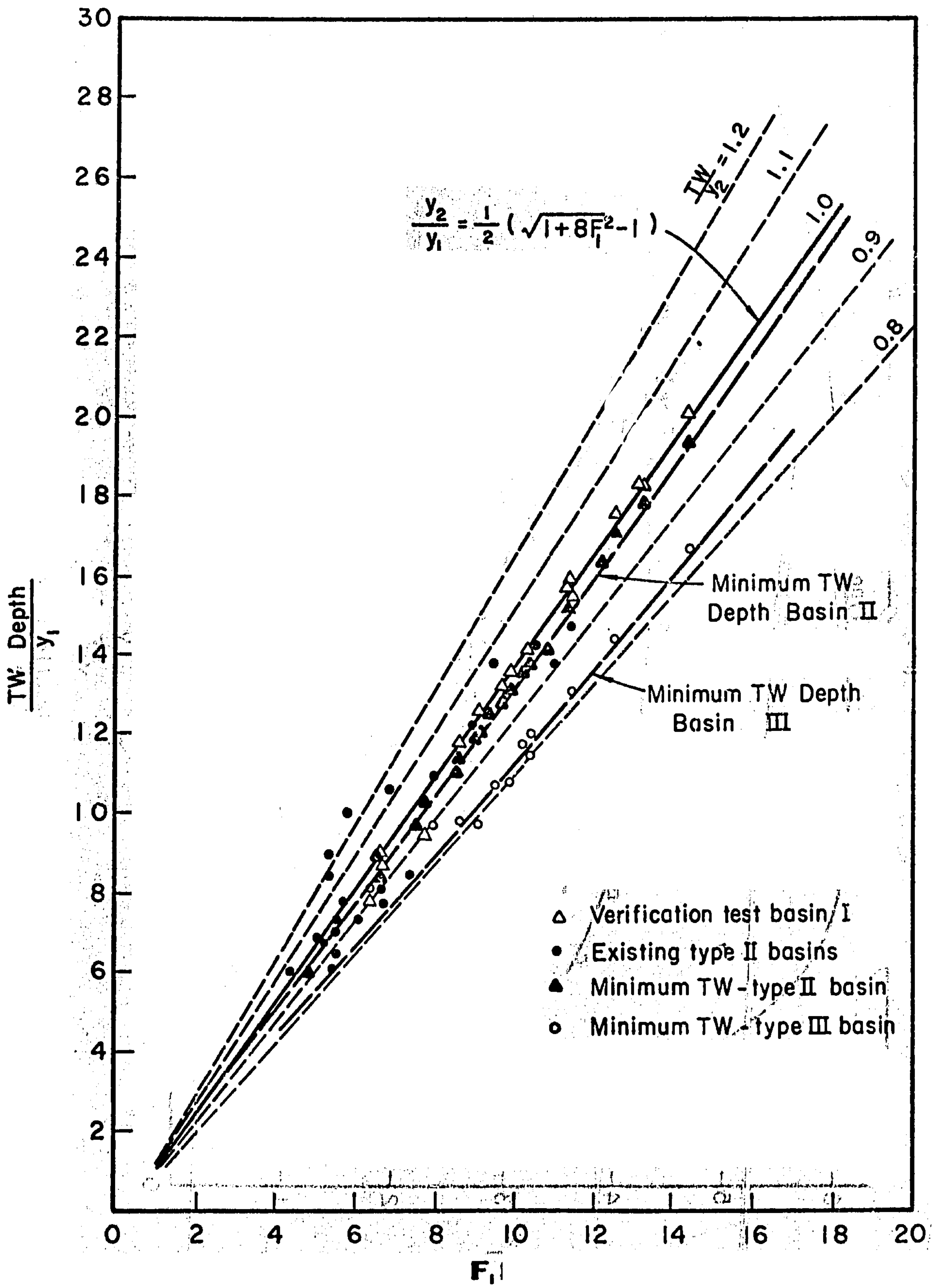


Figure 4-7 Minimum tailwater depths for USBR Stilling Basins I, II, III

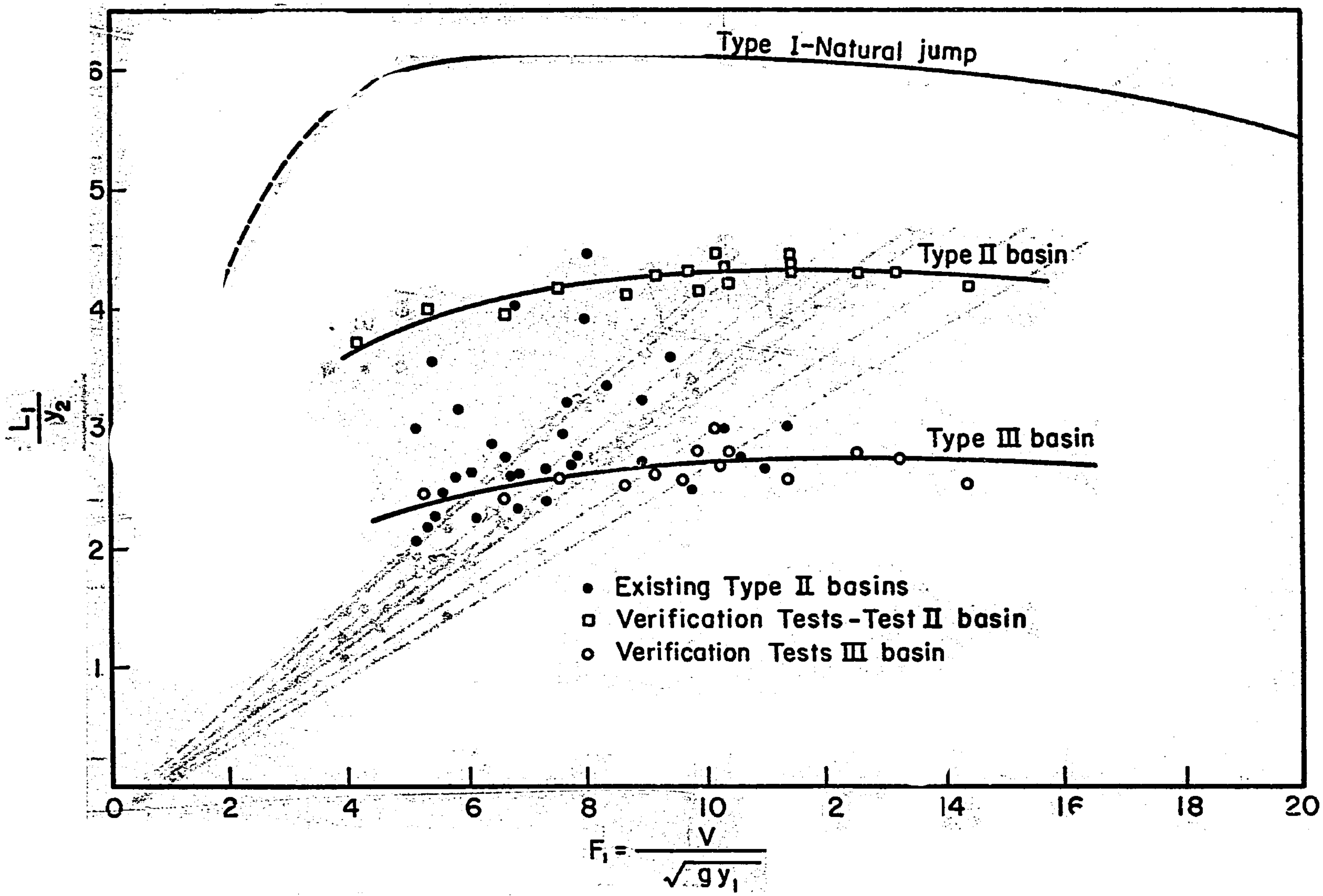


Figure 4-8 Length of hydraulic jump on a horizontal floor for USBR Stilling Basins I, II, III

Following the above rules will result in a safe, conservative stilling basin for spillways up to 200 feet high and for flows up to 500 cfs per foot of basin width, provided the jet entering the basin is reasonably uniform both as to velocity and depth. For greater falls, larger unit discharges, or possible asymmetry, a model study of the specific design is recommended.

USBR Stilling Basin III

The USBR Stilling Basin III, which is shown in Figure 4-9, gives good performance for entrance Froude numbers F_1 greater than 4.5. The length of this stilling basin is generally $2.75y_2$, which is considerably longer than the SAF stilling basin, but only half the length of the free hydraulic jump. The design criteria developed by the U.S. Bureau of Reclamation (Peterka, 1964) are listed below:

1. The stilling basin operates best at full sequent tailwater depth y_2. A reasonable factor of safety is inherent in the sequent depth for all values of the Froude number (Figure 4-7) and it is recommended that this margin of safety not be reduced.
2. The length of basin, which is less than one-half the length of the natural jump, can be obtained by consulting the curve for Basin III in Figure 4-8. As a reminder, an excess of tailwater depth does not substitute for pool length, or vice versa.
3. Stilling Basin III may be effective for values of the Froude number as low as 4.0, but this cannot be stated for certain.
4. The height, width, and spacing of chute blocks should equal the average depth of flow entering the basin, or y_1. The width of the blocks may be decreased, provided spacing is reduced a

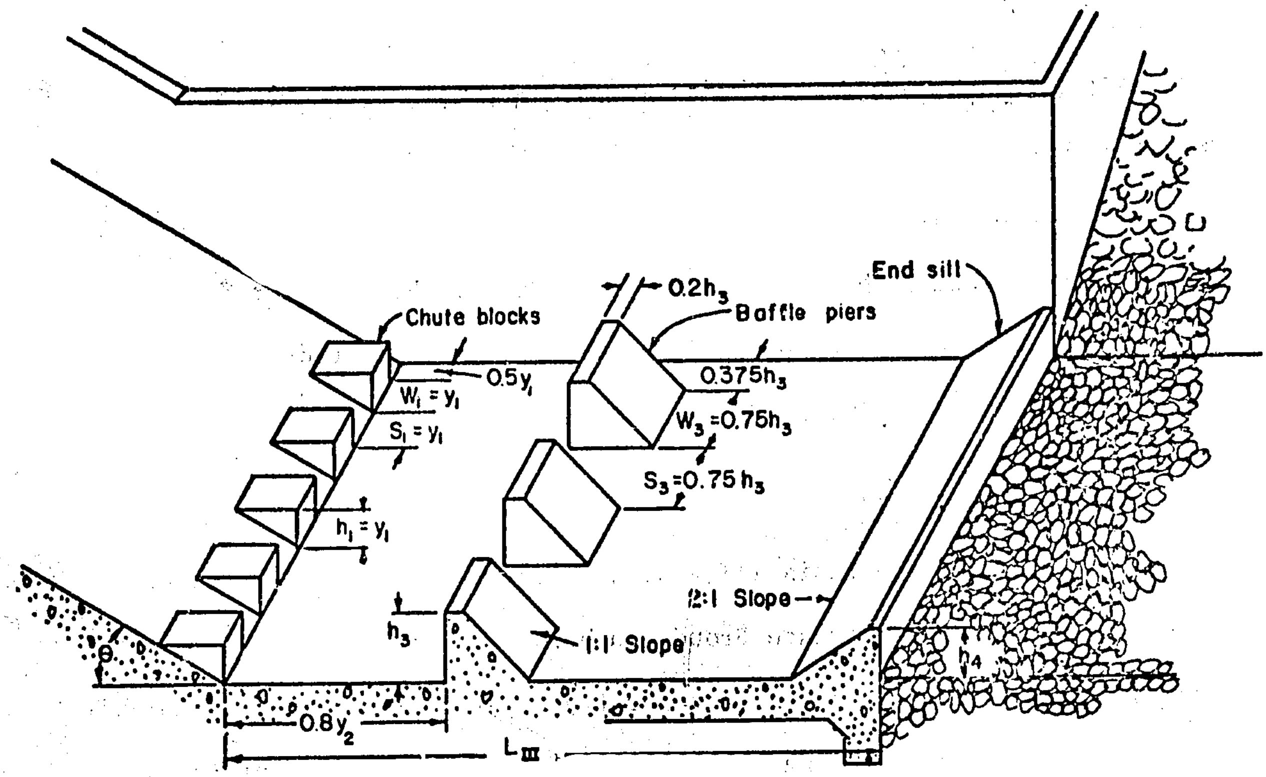


Figure 4-9 Recommended geometry for USBR Stilling Basin III

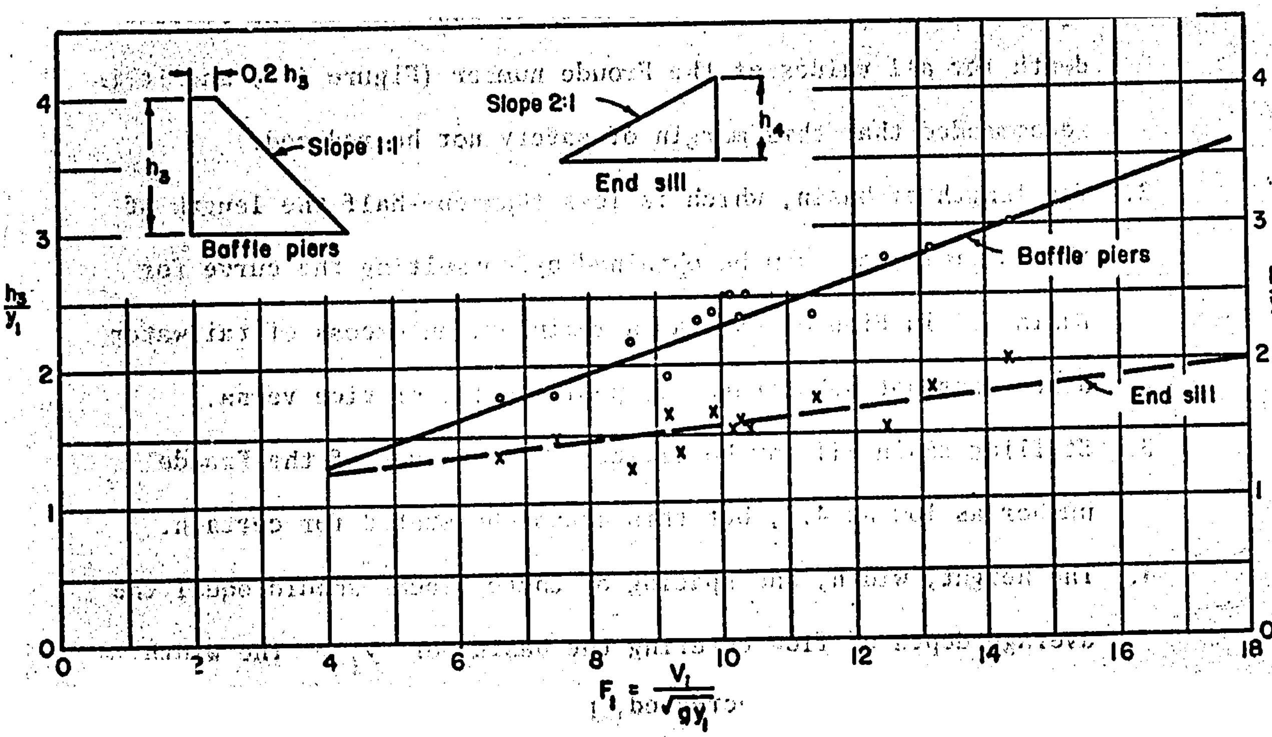


Figure 4-10 Height of baffle piers and end sill for USBR Stilling Basin III

like amount. Should y_1 prove to be less than 8 inches, the blocks should be made 8 inches high.

5. The height of the baffle piers varies with the Froude number and is given in Figure 4-10. The blocks may be cubes or they may be constructed as shown in Figure 4-9; the upstream face should be vertical and in one plane. The vertical face is important. The width and spacing of baffle piers are also shown in Figure 4-9. In narrow structures where the specified width and spacing of blocks do not appear practical, block width and spacing may be reduced, provided both are reduced a like amount. A half space is recommended adjacent to the walls.
6. The upstream face of the baffle piers should be set at a distance of $0.8y_2$ from the downstream face of the chute blocks (Figure 4-9). This dimension is also important
7. The height of the solid sill at the end of the basin is given in Figure 4-10. The slope is 2:1 upward in the direction of the flow.
8. It is undesirable to round or streamline the edges of the chute blocks, end sill, or baffle piers. Streamlining of baffle piers may result in loss of half of their effectiveness. Small chambers to prevent chipping of the edges may be used. Rounding may be necessary for high velocities (greater than 40 fps) which may cause cavitation.
9. It is recommended that a radius of reasonable length ($R \gtrapprox 4y_1$) be used at the intersection of the chute and basin apron for slopes of 45° or greater.

10. As a general rule, the slope of the chute has little effect on the jump unless long flat slopes are involved.

Since Basin III is a short and compact structure, the above rules should be followed closely for its proportioning. If the proportioning is to be varied from that recommended, or if the limits given below are exceeded, a model study is advisable. Arbitrary limits for the USBR Stilling Basin III are set at 200 cfs per foot of basin width and 50 to 60 feet per second entrance velocity until experience demonstrates otherwise (Peterka, 1964).

Baffled Apron

The baffled apron shown in Figure 4-11 is a very effective energy dissipator. This structure has been used successfully on many irrigation projects to dissipate the energy at a drop structure in a canal or waste-way. The allowable discharge is 60 cfs per foot of chute width, with the performance improving as the unit discharge is decreased. The approach velocity to the apron should be less than critical velocity to ensure adequate performance. The design criteria for the baffled apron have been developed by Peterka (1964):

1. The design should be based on the maximum expected discharge Q.
2. The design discharge per unit width q may be as high as 60 cfs/ft of chute width W. The lower this unit discharge becomes, the less severe are the flow conditions at the base of the chute.
3. The entrance velocity V_1 should be as low as possible and less than critical velocity. A desirable entrance velocity is $V_1 = (gq)^{1/3} - 5$, which is Curve D in Figure 4-12. Flow

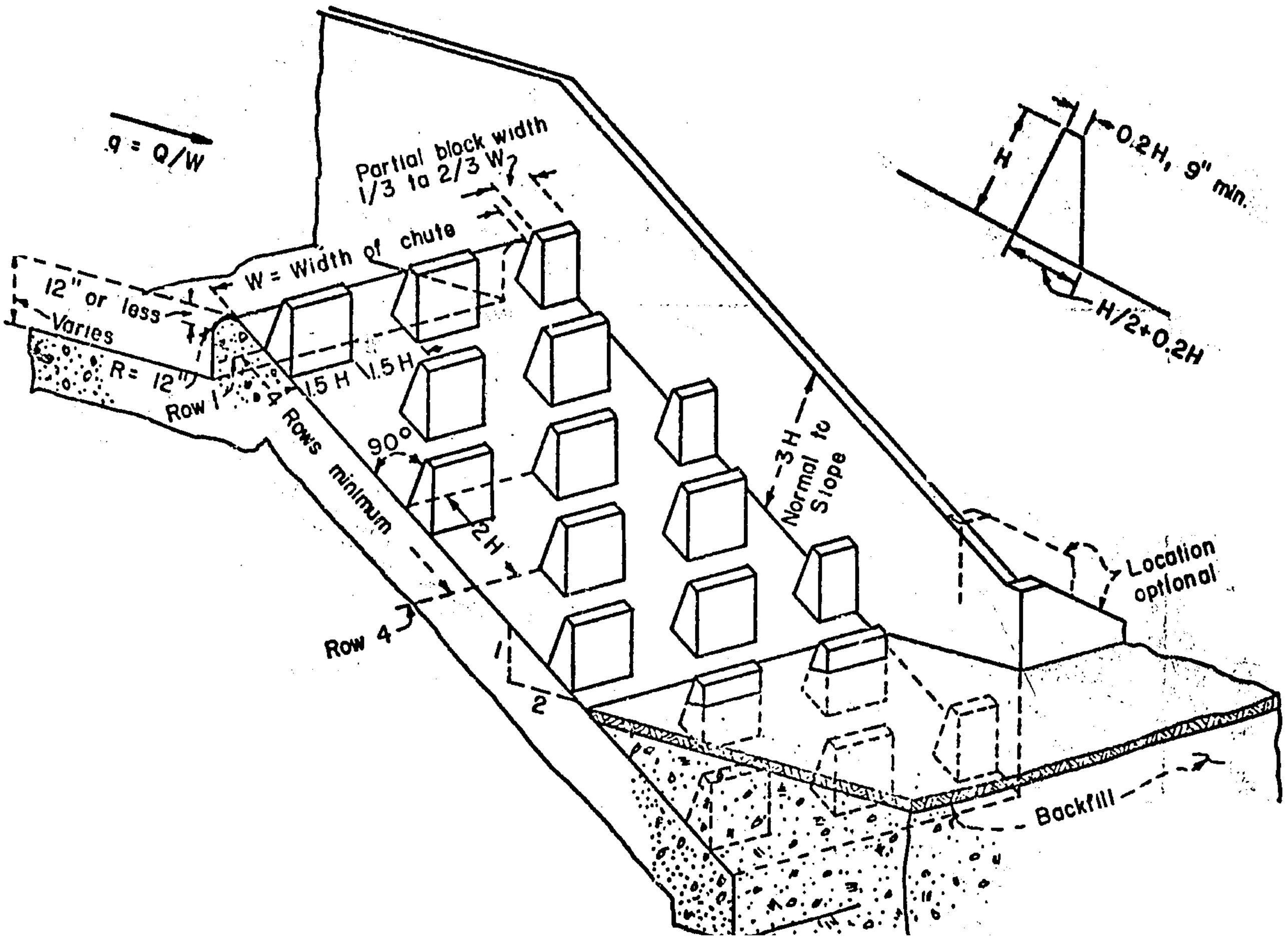


Figure 4-11 Baffled apron energy dissipator

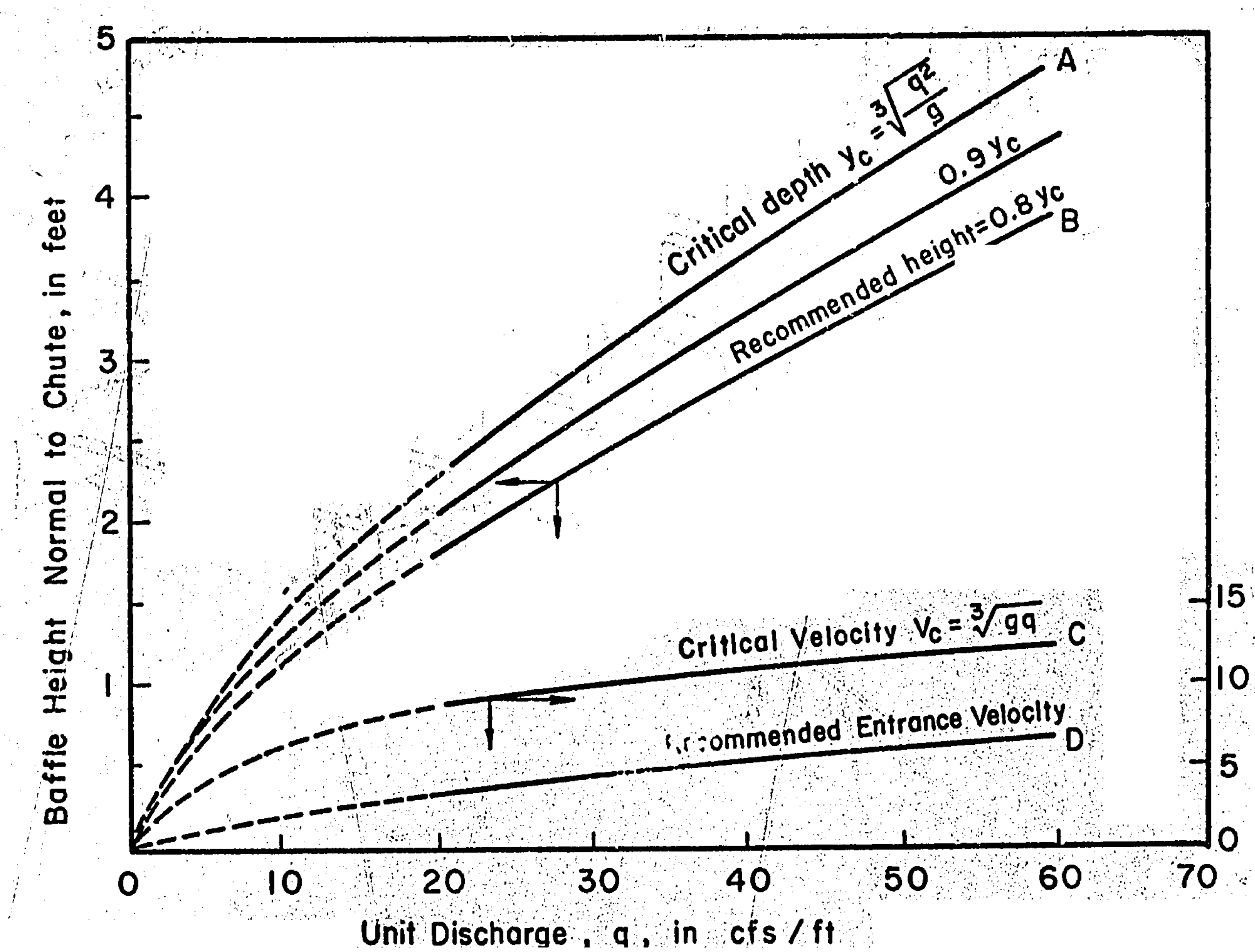


Figure 4-12 Basic proportions of a baffled apron

conditions are not acceptable when $V_1 = (gq)^{1/3}$, which is Curve C in Figure 4-12.

4. The vertical offset between the approach channel floor and the chute can be used for a stilling basin to create the desired approach velocity V_1. Use of a short radius curve to provide a crest on the sloping chute, and placing the first row of baffle piers close to the top of the chute, no more than 12 inches in elevation below the crest, are important considerations.
5. The baffle pier height H should be about $0.8y_c$, which is Curve B in Figure 4-12.
6. Baffle pier widths and spaces should be equal to about 1.5 H. but not less than H. Partial blocks, 1/3 H to 2/3 H, should be placed against the training walls in rows 1, 3, 5, 7, etc., alternating with spaces of the same width in rows 2, 4, 6, etc.
7. The maximum slope should not exceed 2:1 but may be flatter. The slope distance (along a 2:1 slope) between rows of baffle piers should be about 2H and not greater than 6 feet.
8. The baffle piers may have upstream faces normal to the apron floor or in a vertical plane.
9. Four rows are required to establish full control of the flow, although fewer will work. The chute should be extended below the normal downstream channel and at least 1 row of baffles should be buried in the back fill.
10. The chute training walls should be about 3H normal to the chute floor.
11. Six to twelve-inch riprap should be placed at the downstream end of the training walls to prevent erosion behind the chute.

Chapter 5

CLOSED CONDUIT TO OPEN CHANNEL STILLING BASINS

Closed conduits (pipelines) are frequently used in irrigation systems. Pipelines may be used to convey water down a hillside rather than using a drop structure or series of drop structures. Also, closed conduits are used for outlet works from dams, inverted syphons, culverts, and water distribution systems. In many cases, the flow velocity leaving the pipe outlet is excessive and would cause considerable scour in the open channel immediately downstream. In order to prevent excessive downstream scour, which might also result in scouring of the bed material underlying the closed conduit and consequent failure of the system, a number of energy dissipator structures have been developed for the outlets of closed conduits. Typical examples of generalized closed conduit to open channel stilling basins are the Contra Costa energy dissipator, the USBR impact stilling basin VI, the manifold stilling basin, and the USU (Utah State University) stilling basin.

Contra Costa Energy Dissipator

The Contra Costa energy dissipator (Keim, 1962) is a device for pipe outlets flowing partly full. A schematic diagram of the Contra Costa energy dissipator, along with its basic dimensions are shown in Figure 5-1. The design procedure for this structure, which is also shown in Figure 5-1, is as follows:

1. Determine the width W_c such that $D \leq W_c < 3D$ in which D is the inlet pipe diameter, or preferably, make W_c equal to the width of the downstream channel providing it is within the limits mentioned.

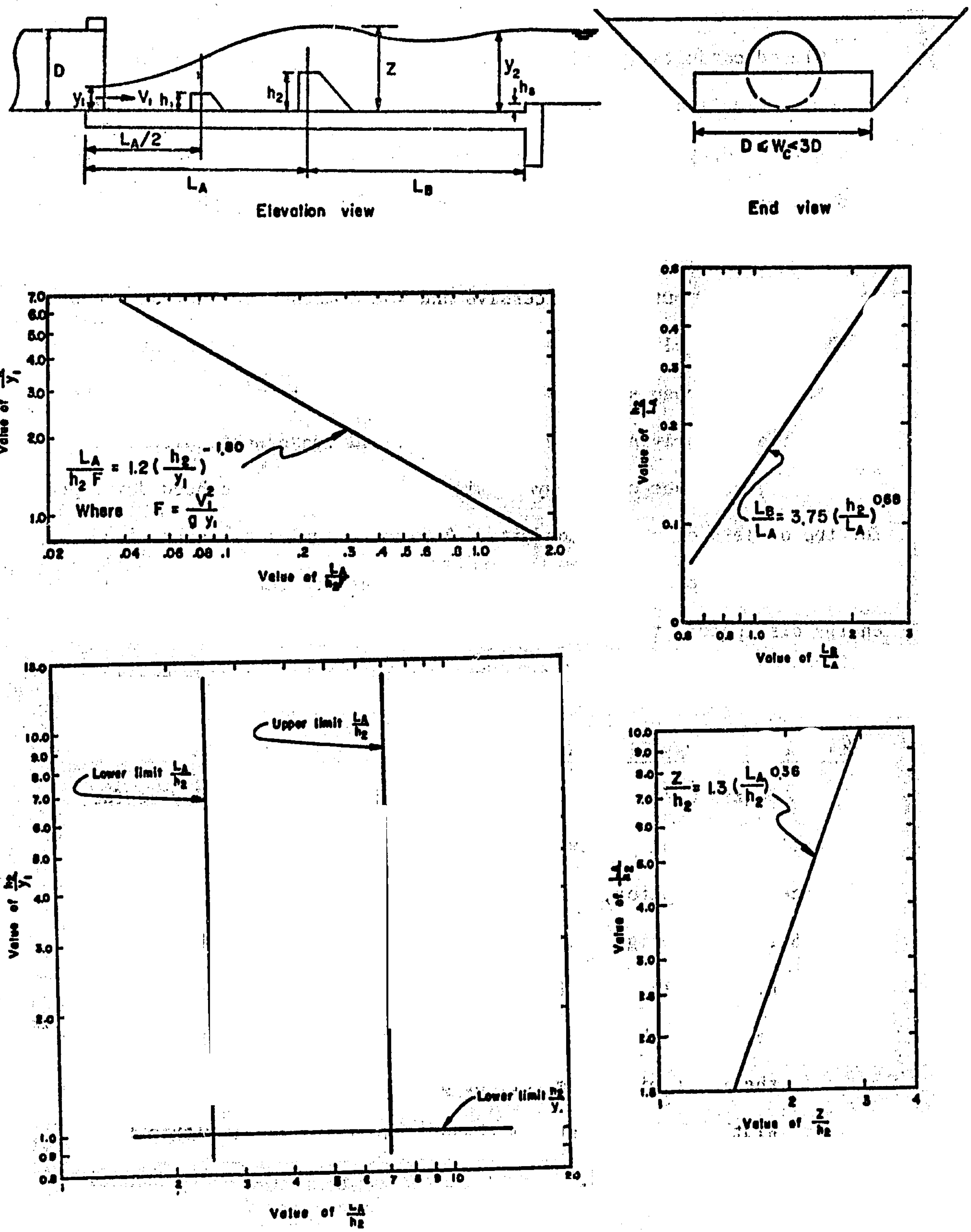


Figure 5-1 The Contra Costa energy dissipator

2. Letting L_A/h_2 be approximately 3.5, determine h_2/y_1, and solve for L_A.
3. Determine a value of L_B.
4. Determine a value of z.
5. Compute the value for the sill height, h_s, within the range $(0.6 - 0.9)y_2$.

Manifold Stilling Basin

The manifold stilling basin (Figure 5-2) is an effective energy dissipation device for closed conduit transitions to open channels using the vertical jet diffusion principle. The manifold structure itself has two primary functions:

1. To divide the flow into a number of jets having a length equal to the manifold width; and
2. To direct these jets vertically upward.

The dissipation of the kinetic energy takes place in the water above the manifold as the rising jets are steadily converted into turbulence, with the turbulence decaying through viscous shear (Fiala and Albertson, 1961).

The general design equation is

$$\frac{a}{V_1^2/2g} = \frac{C}{b/B_o} \tag{5-1}$$

in which

a is the boil height

V_1 is the initial velocity of the jet

g is the acceleration of gravity

C is a constant

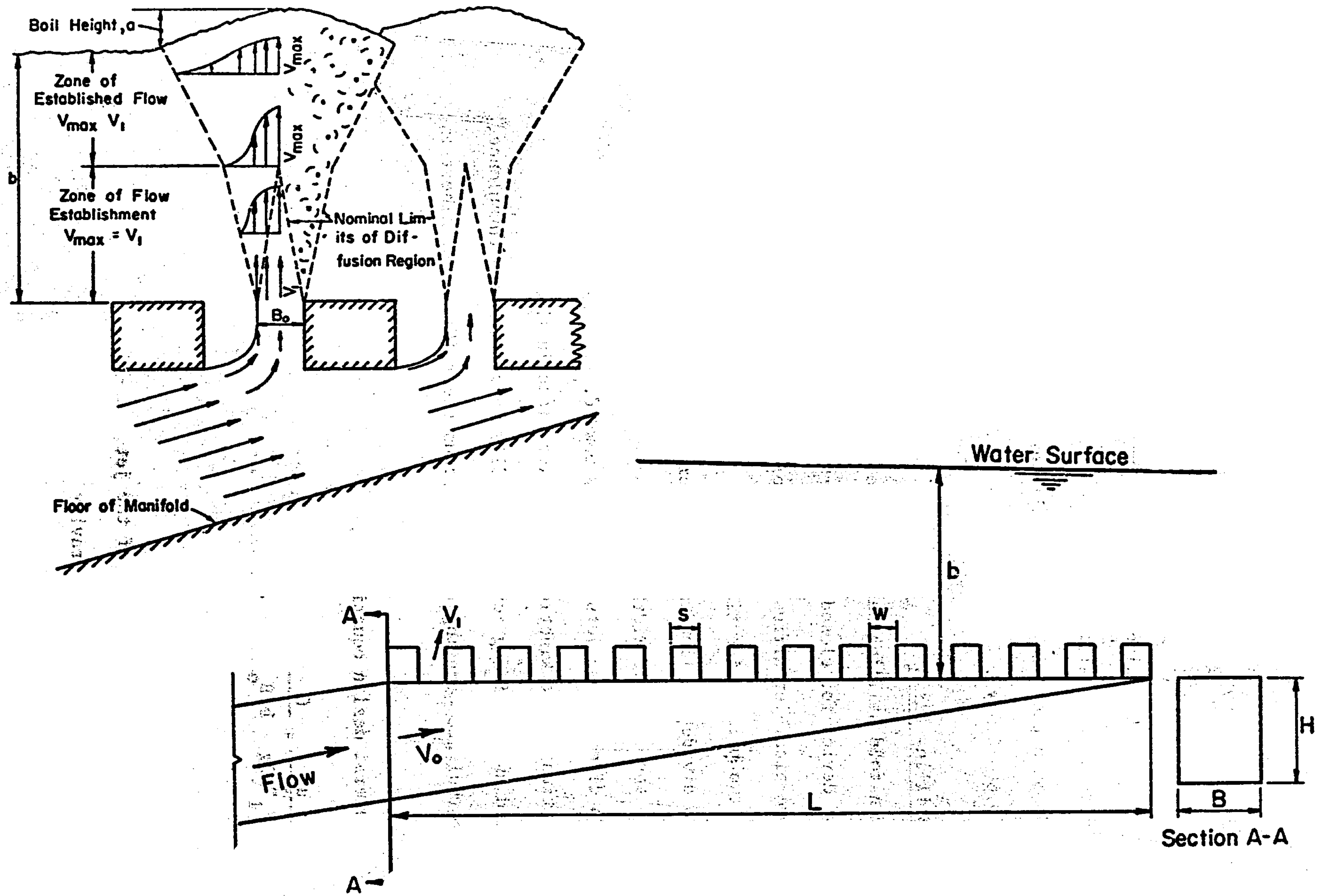


Figure 5-2 Schematic drawing of manifold stilling basin

h is the tailwater depth, and

B_o is the effective width of the jet.

Knowing the discharge Q tailwater level b and the inlet velocity V_o the manifold stilling basin design procedure is as follows:

1. Compute the inlet area A_o using the continuity equation.
2. Knowing the area, compute B and H,

   $$B = H = \sqrt{A} \qquad (5\text{-}2)$$

   in which B is the manifold width and H is the manifold height at the inlet.
3. Knowing B, compute the length L using a length to width ratio L/B of 8.
4. Assume a standard w/s ratio, 0.5, 1.0, or 2.0 in which s is the width of the manifold cross-bars.
5. Obtain a value of V_1/V_o and compute the value of V_1 based upon the relationships: (a) for $w/s = 0.5$, $V_1/V_o = 1.37$; (b) for $w/s = 1.0$, $V_1/V_o = 1.24$; and (c) for $w/s = 2.0$, $V_1/V_o = 1.13$.
6. Compute the open area of the jets B_oB using the continuity equation.
7. Compute the total open area of the manifold BW divide the value obtained in step 6 by the total open area of the manifold to get the percentage of area occupied by the jets, and finally, multiply this percentage by w to get a value for B_o.
8. Compute b/B_o, enter Figure 5-3 to obtain a value for $\frac{a}{V_1^2/2g}$ and knowing $V_1^2/2g$, solve for a boil height a.
9. Adjust the design if $\frac{a}{V_1^2/2g}$ is not reasonably close to 0.10.

The loser this value, the less erosive energy will be left in the jets.

10. Determine the value for the wave height h from Figure 5-4.

USBR Stilling Basin VI

The USBR stilling basin VI, shown in Figure 5-5, is an impact type energy dissipator which has been developed by the U.S. Bureau of Reclamation for use on pipe outlets, but it can be modified for open channel transitions. The basic dimensions for this structure along with the range of flows is given in Table 5-1.

USU Stilling Basin

The USU stilling basin is an impact type energy dissipator designed as a transition from pipe flow to open channel flow. The major mechanisms in dissipating the excess energy in the flow are shear drag, pressure drag, and vertical diffusion. The energy dissipation results from the submerged jet impinging upon a short-pipe energy dissipator and then being turned on itself along with part of the jet being turned downward through the slot in the bottom of the dissipator pipe. The energy of the flowing water is dissipated by diffusion of the jet from the inlet pipe into the stilling basin and the shearing between the jet itself and the surrounding water (Wei, 1968). The basic dimensions of the USU stilling basin, along with the design curves, are shown in Figure 5-6 (Flammer, Skogerboe, Wei, and Rasheed, 1970). The design procedure can be described as follows:

1. The dimensions of the short-pipe energy dissipator are a function of the inlet pipe diameter D_1 according to the following ratios:

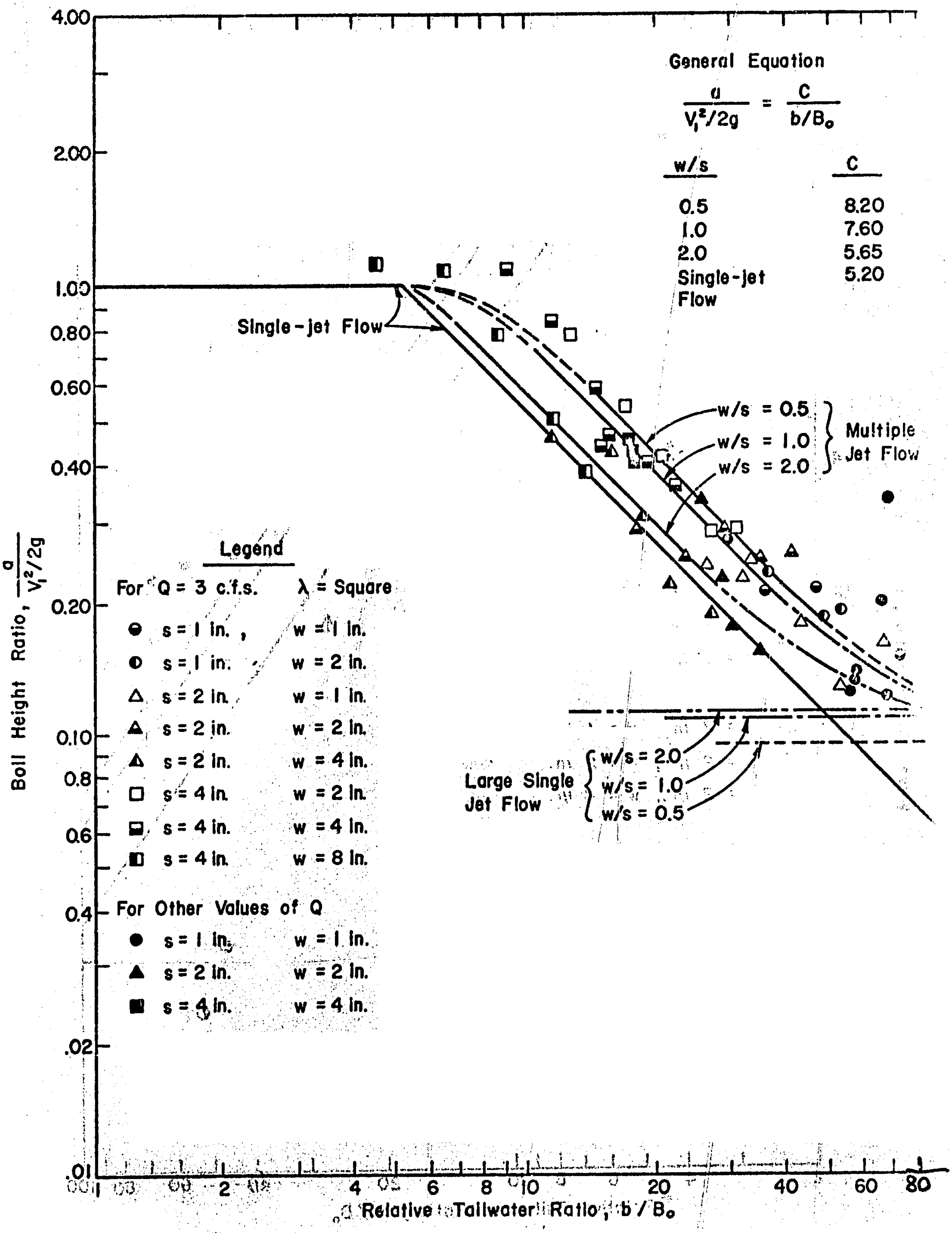


Figure 5-3 Variation of $a/(V_1^2/2g)$ with b/B_o and w/s for manifold stilling basin

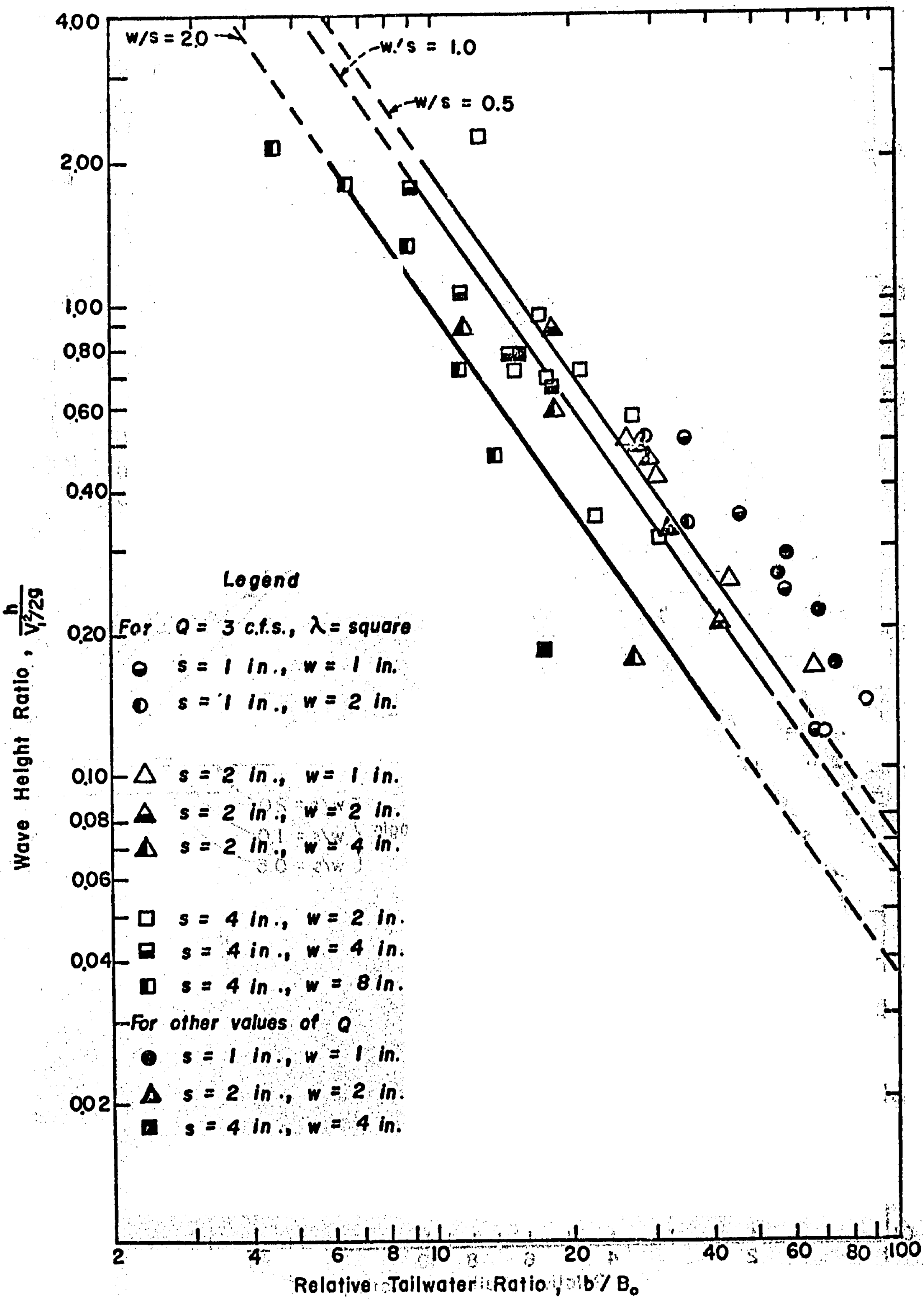


Figure 5-4 Variation of $h/(V_1^2/2g)$ with b/B_o and w/s for manifold stilling basin

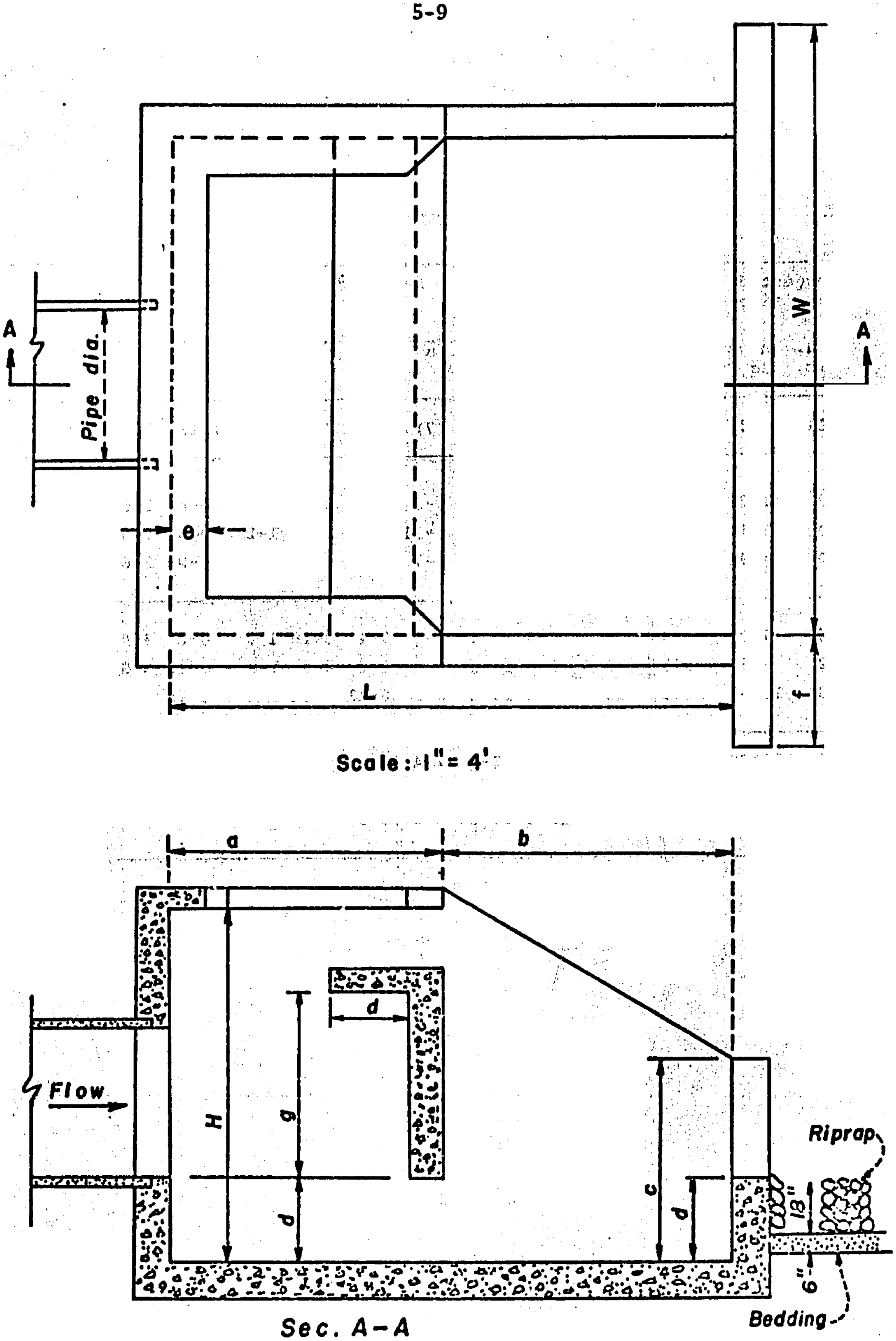


Figure 5-5 Sketch of USBR Stilling Basin VI

Table 5-1. Dimensions of USBR Stilling Basin VI

Suggested Pipe size		Max. dis-charge	Feet and Inches									
Dia. in.	Area (Sq Ft)	Q	W	H	L	a	b	c	d	e	f	g
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
18	1.77	21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	0-6	1-6	2-1
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	0-6	2-0	2-6
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	0-8	2-6	3-0
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	0-8	3-0	3-6
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	0-10	3-0	3-11
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	0-10	3-0	3-11
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	1-0	3-0	4-11
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	1-0	3-0	5-4
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-0	1-3	3-0	6-2

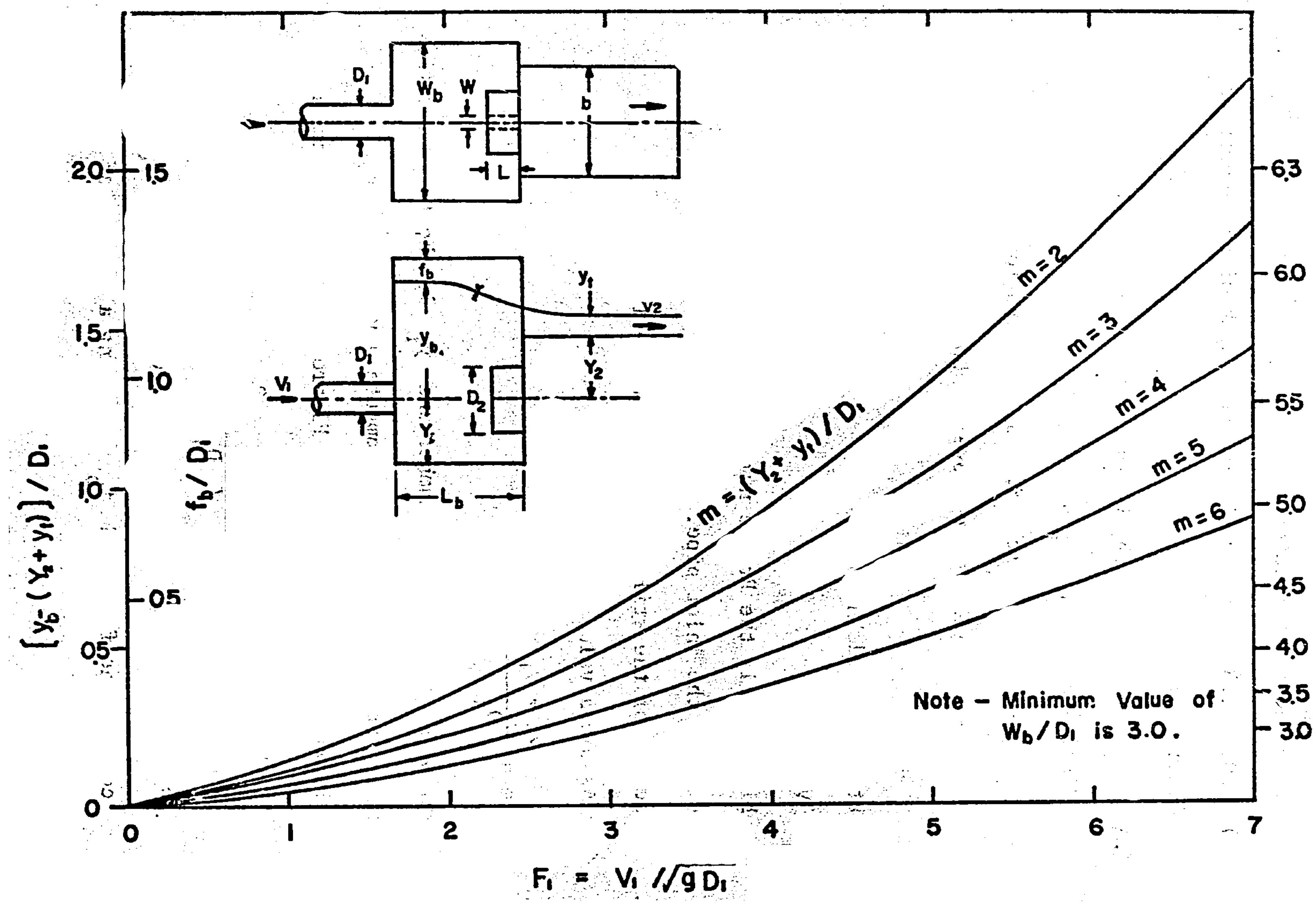


Figure 5-6 Design curves for USU Stilling Basin

(a) $D_2/D_1 = 2.0$, in which D_2 is the diameter of the short-pipe energy dissipator.

(b) $L/D_1 = 1.0$, in which L is the length of the shortpipe energy dissipator.

(c) $W/D_1 = 0.5$, in which W is the width of the slot in the dissipator pipe.

(d) $Y_1/D_1 = 1.5$, in which Y_1 is the elevation of the center line of the inlet pipe above the bottom of the stilling basin.

2. Compute the Froude number,

$$F_1 = V_1/\sqrt{gD_1} \tag{5-3}$$

in which F_1 is the Froude number and V_1 is the mean flow velocity in the inlet pipe.

3 The design of the stilling basin is as follows:

(a) Assume a value of m,

$$m = \frac{y_2 + y_t}{D_1} \tag{5-4}$$

in which m is an assumed number and y_t is the tailwater flow depth in the open channel, and compute a value for y_2.

(b) Knowing m and F_1, enter Figure 5-6 and determine a value for the expression,

$$\Delta = \left[y_b - (Y_2 + y_t)\right] \quad D_1 \tag{5-5}$$

(c) From Figure 5-6, determine the freeboard ratio f_b/D_1 compute the freeboard f_b but using a minimum of six inches

(d) Compute the length of the stilling basin L_b using the equation,

$$L_b/D_1 = 2.5\ (D_2/D_1 - 1) + 1.0 \qquad (5\text{-}6)$$

(e) From Figure 5-6, determine the stilling basin width W_b from the W_b/D_1 ratio.

4. The value of m is completely arbitrary and should be tried at various values in order to arrive at the most economical design. As the value of m is decreased, the flow in the stilling basin and the outlet channel will become less turbulent with a consequent decrease in the scour potential.

Chapter 6

VERTICAL DROPS

A definition sketch for a vertical drop structure is shown in Figure 6-1. In order to determine the length of basin L_d at which the jet strikes the basin floor, equations reported by Blaisdell (1954) describing the nappe of the freely falling jet are used. These equations are listed under the following section, "Straight Drop Spillways." White (1943) developed from application of momentum, the theoretical equation listed below which yields the flow depth y_1 .

$$\frac{y_1}{y_c} = \frac{\sqrt{2}}{1.06 + \sqrt{\frac{h}{y_c} + \frac{3}{2}}} \tag{6-1}$$

This equation agrees quite well with experimental work by Moore (1943) The specific energy at section 1 is obtained from the equation

$$\frac{E_1}{y_c} = \frac{y_1}{y_c} + \frac{y_c^2}{2y_1^2} \tag{6-2}$$

Curves representing the above equations are shown in Figure 6-2 along with the experimental work by Moore. The length of the hydraulic jump L_j is approximately six times the sequent flow depth y_2 but can be determined from the curve for the free hydraulic jump shown in Figure 4-8

When designing a drop structure, the discharge Q and the geometry of the approach section would be known. Consequently, the critical depth y_c can be computed, which allows the computation of

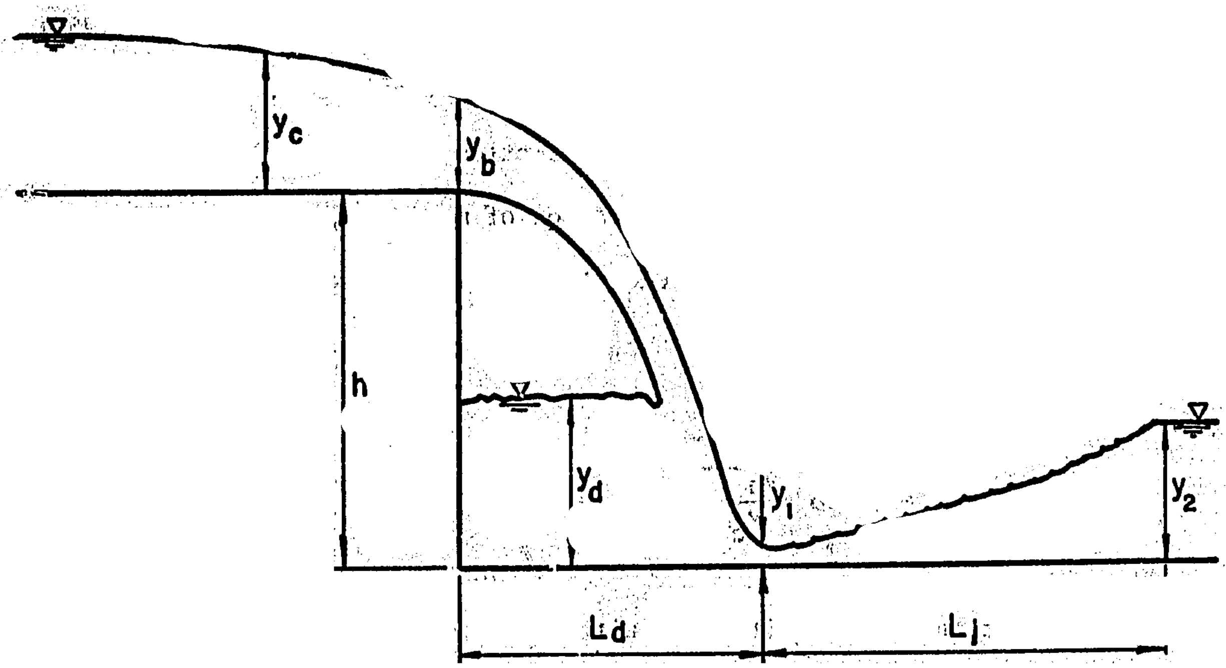


Figure 6-1 Basic type of vertical drop structure

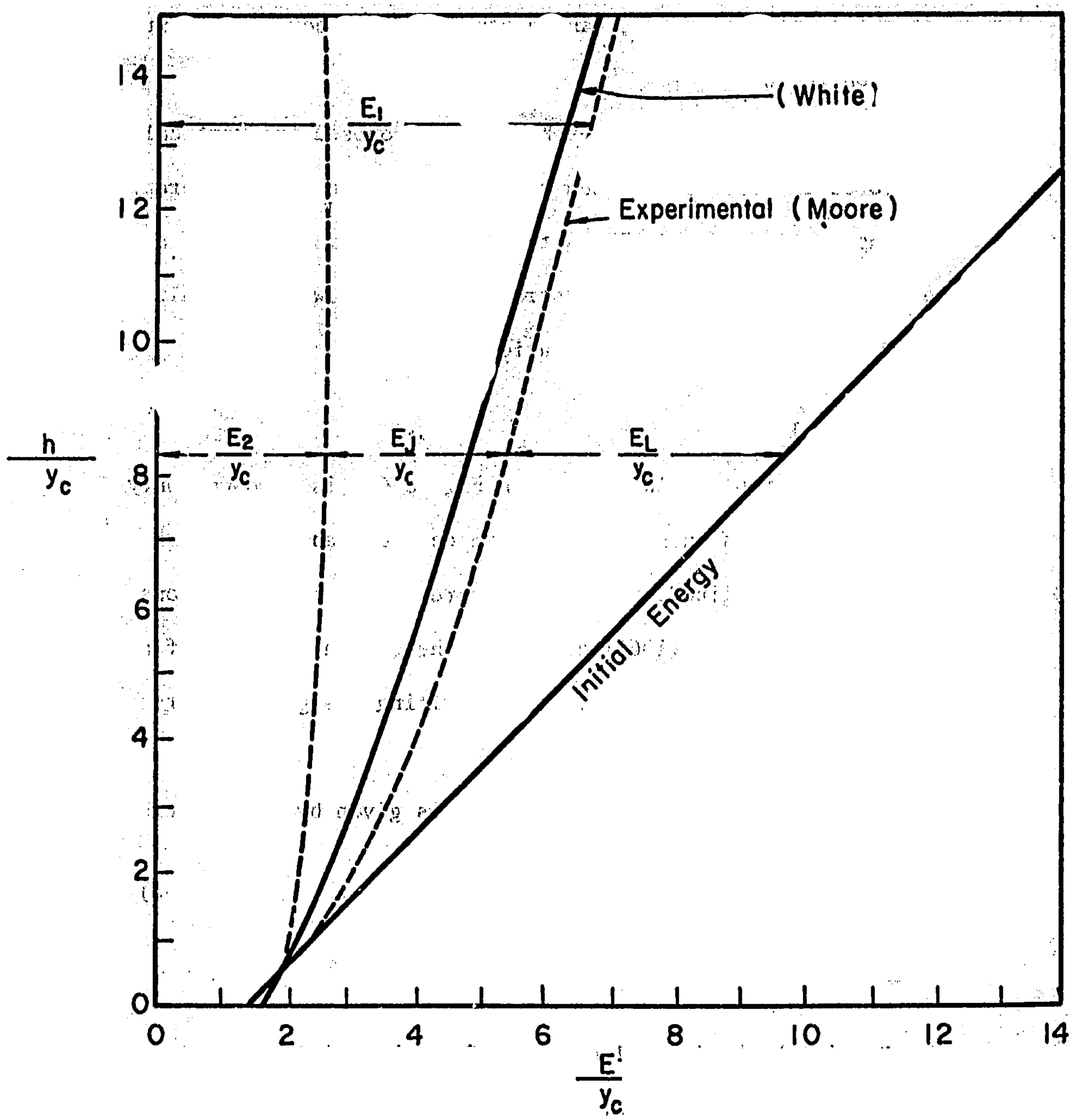


Figure 6-2. Energy dissipation at the base of the free overfall

y_1 from Equation 6-1. The sequent flow depth y_2 can be obtained from nomographs (Figs. 4-2 and 4-3). The length of the hydraulic jump L_j can be obtained from Figure 4-8 and L_d the horizontal length of the nappe trajectory, can be computed from equations listed in the section to follow.

Various types of appurtenances have been proposed for vertical drop structures in order to shorten the length of the basin, thereby reducing construction costs. Three types of vertical structures are described below, which include the straigh drop spillway, USBR alternative stilling basin IV, and dissipation bars.

Straight Drop Spillway

The straight drop spillway shown in Figure 6-3 is used commonly for low cost structures in small discharge channels and the design parameters are well defined. One of the more recent investigations by Donnelly and Blaisdell (1965) expresses the design in terms of an equation for the nappe trajectory. The resulting design procedure is listed below:

1. The total length of the basin L_b is given by the relation,

$$L_b = X_a + 2.55\ y_c \tag{6-3}$$

in which

X_a is the horizontal distance from the spillway crest to the point where the average of the upper and lower nappe strikes the floor, and

y_c is the critical depth of flow

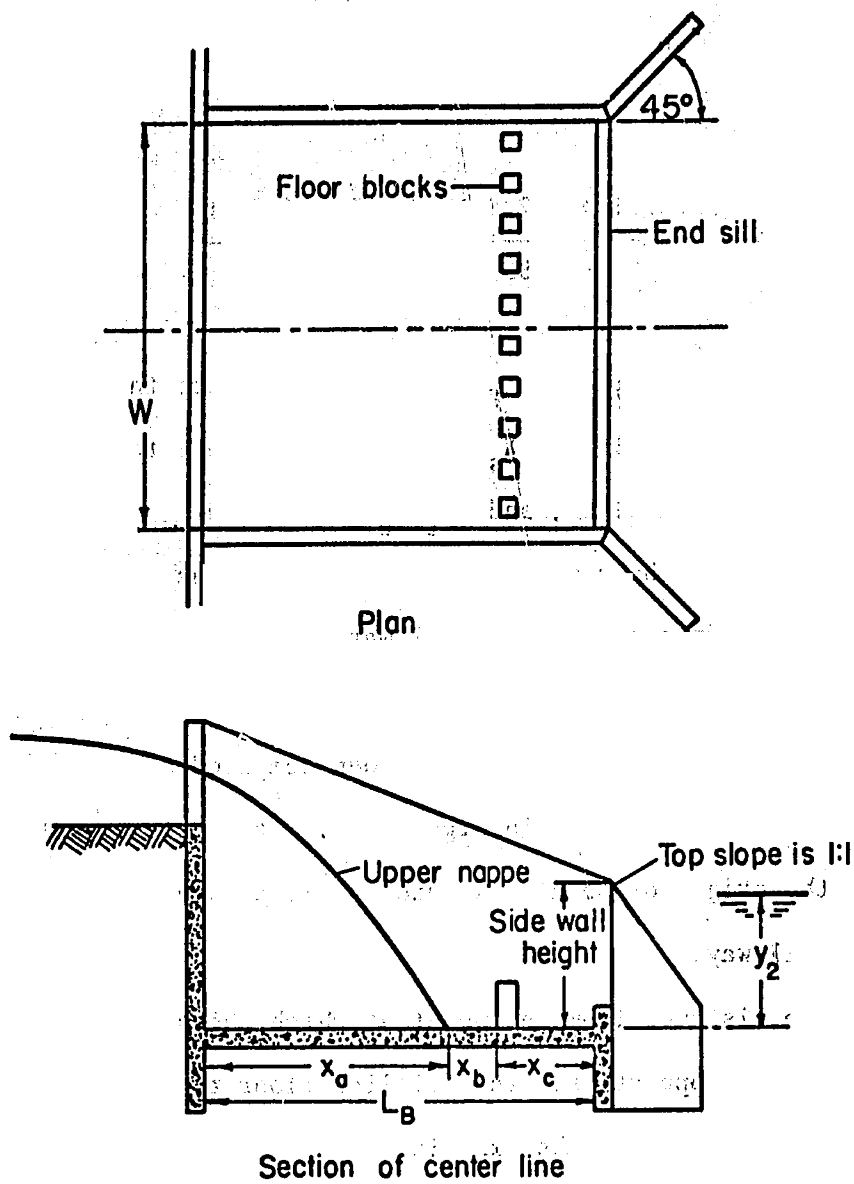


Figure 6-3 Straight drop spillway stilling basin

The value of X_a can be determined by the equation

$$X_a = \frac{X_f + X_s}{2} \tag{6-4}$$

in which

$$X_f/y_c = -0.406 + (3.195 - 4.386\ h/y_c)^{0.5} \tag{6-5}$$

and

$$X_s/y_c = \frac{0.691 + 0.228\ (X_t/y_c)^2 - (h/y_c)}{0.185 + 0.456\ X_t/y_c} \tag{6-6}$$

In these equations X_f is the point where the upper nappe strikes the floor, X_s is the point where the lower nappe strikes the floor, and X_t/y_c is the value obtained from Equation 6-5 by substituting X_t (for the point where the upper nappe strikes the tailwater) for X_f and y_t (the distance below the crest to the surface of the tailwater) for h . This equation has been computed and plotted for design convenience in Figure 6-4. It should be noted that the origin of points X and y is at the crest of the spillway.

2. The distance from the point at which the surface of the upper nappe strikes the stilling floor to the upstream of the floor blocks x_b is

$$x_b = 0.8\ y_c \tag{6-7}$$

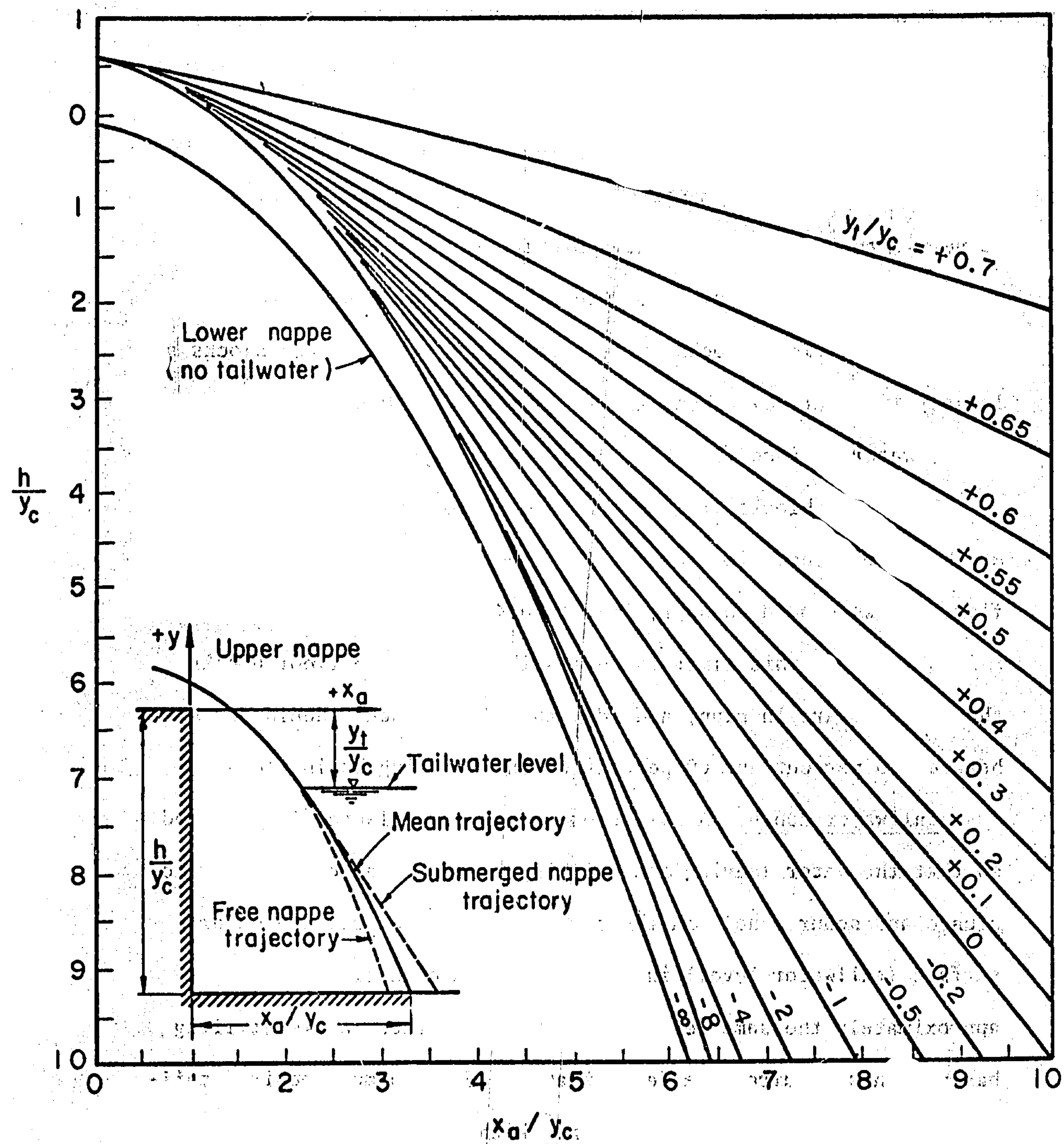


Figure 6-4. Graphical solution to nappe trajectory

3. The distance between the upstream face of the floor blocks and the end of the stilling basin x_c is:

$$x_c = 1.75\ y_c \tag{6-8}$$

Floor blocks. Sufficient distance is required between the point at which the upper nappe strikes the basin floor and the floor blocks to permit the stream to become approximately parallel to the floor before it reaches the blocks. When the distance between the upper nappe and the floor blocks x_b is less than 0.5 y_c, the blocks are largely ineffective. When x_b is increased to 0.8 y_c, the appearance of the water surface is satisfactory (Donnelly and Blaisdell, 1965).

The floor blocks are proportioned as follows: (a) the height of the floor blocks is 0.8 y_c; (b) the width and spacing of the floor blocks should be approximately 0.4 y_c, but a variation of ± 0.15 y_c from this limit is permissible; (c) the floor blocks should be square in plan; and (d) the floor blocks should occupy between 50 percent and 60 percent of the stilling basin width.

Tailwater depth. A sufficient depth of tailwater is required so that the water leaving the stilling basin has no opportunity to plunge and scour a hole in the stream bed; therefore, the water surface (tailwater level) in the downstream channel should be at approximately the same level as the water surface in the stilling basin. This requires the determination of a minimum required tailwater depth for the stilling basin. With the end sill at 0.4 y_c, the minimum desirable depth y_2 above the floor of the stilling basin is 2.15 y_c. Anything less than this depth can cause higher velocities over the sill which will in turn cause excessive scour.

Distance to end sill. If the distance between the floor blocks and the end sill is too short, neither the blocks nor the end sill is fully effective. The minimum distance between the floor blocks and the end sill x_c that prevents excessive scour is 1.75 y_c . Distances greater than 1.75 y_c have little beneficial effect on the scour pattern.

Sidewall height. An additional height of sidewall is required above the nominal tailwater level to prevent overtopping of the sidewalls as a result of natural water surface fluctuations, turbulence within the stilling basin caused by the floor blocks, and boils and standing waves resulting from the floor blocks and end sill. Donnelly and Blaisdell (1965) recommended that the top of the sidewalls be located a minimum distance of 0.85 y_c above the tailwater level. This will give a freeboard above the maximum boil height of 0.25 y_c .

Wingwalls. Wingwalls located at an angle of 45° with the stilling basin center line and having a top slope of 1 on 1 are recommended. This is the same design found suitable in hydraulic model tests of the box inlet drop spillway outlet and the St. Anthony Falls (SAF) stilling basin (Donnelly and Blaisdell, 1965).

Approach channel. The shape of the approach channel does affect the stilling basin performance. It is important, therefore, that certain minimum conditions be met with regard to the shape of the approach channel. The approach channel should have the following qualifications: (a) be level with the crest of the spillway; (b) have the toe of the dike or the toe of the side slope intersect the approach channel floor at the ends of the spillway notch, the approach channel at the headwall should have a bottom width equal to the spillway notch length w ; and (c) be protected by riprap or paving for a distance upstream from the headwall equal to two times the notch depth.

USBR Alternative Basin IV

The characteristics of a hydraulic jump for Froude numbers between 2.5 and 4.5 results in inefficient energy dissipation for most stilling basins. In this range of Froude numbers, the jump is not fully developed and undular waves occur. The USBR Alternative Stilling Basin IV (Peterka, 1964), shown in Figure 6-5, is an efficient method for dissipating energy and avoiding waves for many of the small drops encountered in irrigation canals. The design is based primarily on the equation,

$$L = \frac{Q}{CSN\,(2gy)^{1/2}} \tag{6-9}$$

in which

L is the length of the beams

Q is the total discharge in cfs

C is an experimental coefficient (approx. 0.245)

S is the width of a space in feet,

N is the number of spaces, and

y is the depth of flow in the upstream canal.

Dissipation Bars

Katsaitis (1966) has developed a vertical drop structure having two rows of dissipation bars are placed in the stilling basin as shown in Figure 6-6. The design information described below has been taken from the paper by Katsaitis.

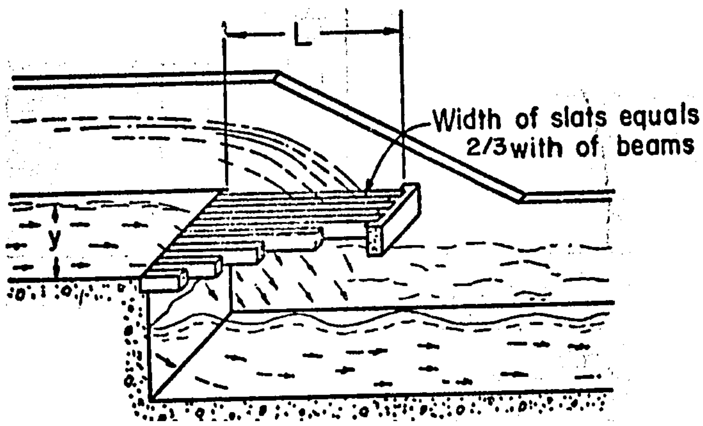


Figure 6-5 USBR Alternative Stilling Basin IV

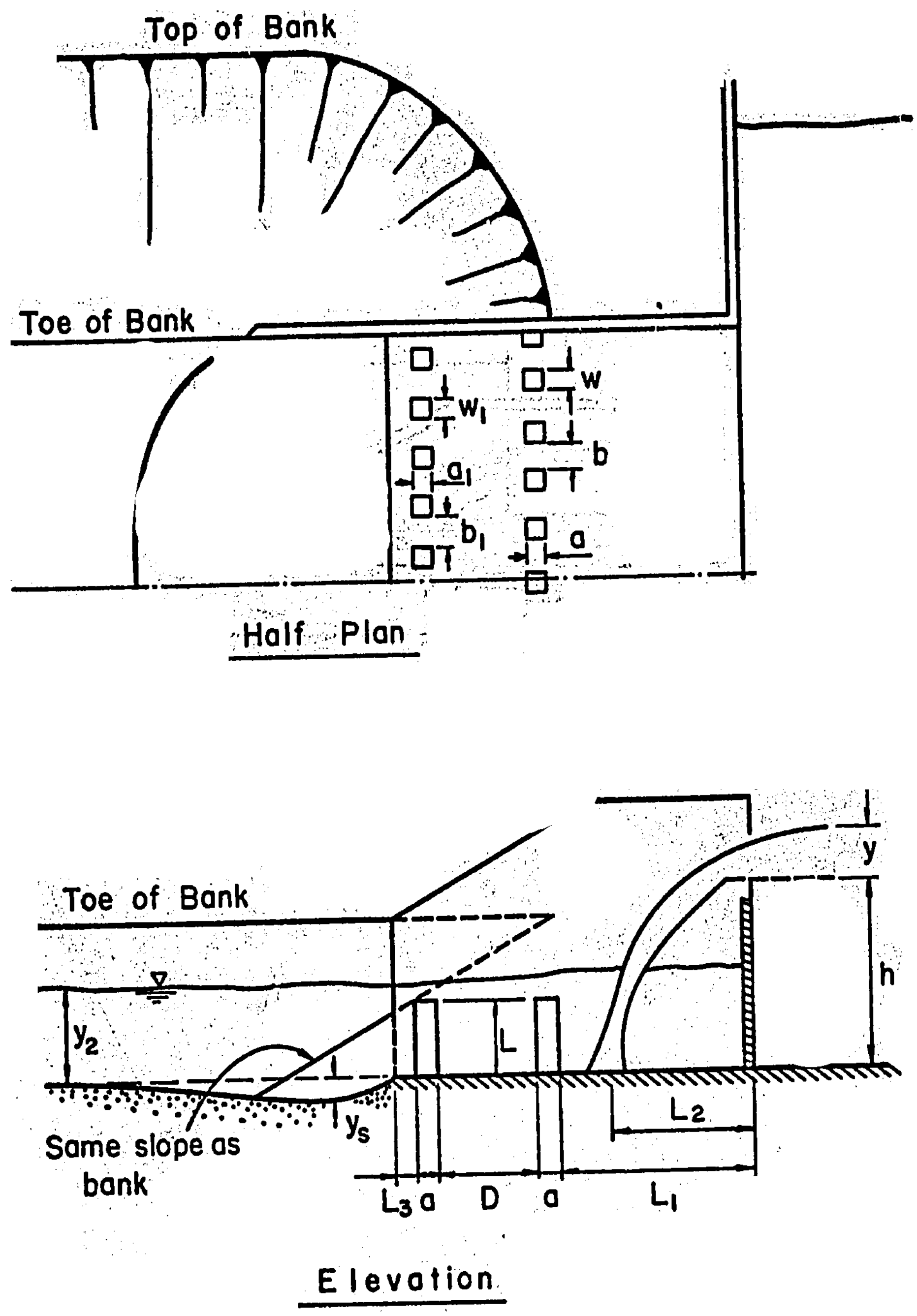


Figure 6-6 Drop structure with dissipation bars

Flow conditions. The definition sketch of flow over a weir as shown in Figure 6-7 can be used to describe the development of an equation for determining the length L_2 which is the horizontal distance from the weir (or drop structure) crest to the centerline of the nappe trajectory where it strikes the stilling basin floor. The reference point for the equations of rectilinear motion, which are listed below, is the centroid of the jet cross-section where the lower nappe has reached it's highest elevation (Figure 6-7). The horizontal distance between the reference point and the weir crest is H'/2, where H' is the total head (specific energy) above the highest point of the lower nappe.

The equations of motion can be written as

$$X = (Vo)_x t \qquad (6\text{-}10)$$

$$V_z = (Vo)_z + gt \qquad (6\text{-}11)$$

$$Z = (Vo)_z t + 1/2gt^2 \qquad (6\text{-}12)$$

$$V_z^2 = (Vo)_z^2 + 2gZ \qquad (6\text{-}13)$$

in which

X is the horizontal distance from the reference point

$(Vo)_x$ is the initial horizontal velocity at the reference point

$(Vo)_z$ is the initial vertical velocity

t is time

V_z is the final vertical velocity, and

Z is the total height of fall measured from the reference point (Figure 6-7

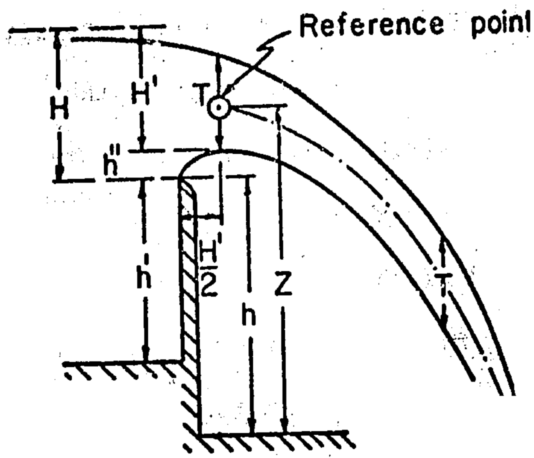


Figure 6-7 Definition sketch for free jet flow over a weir

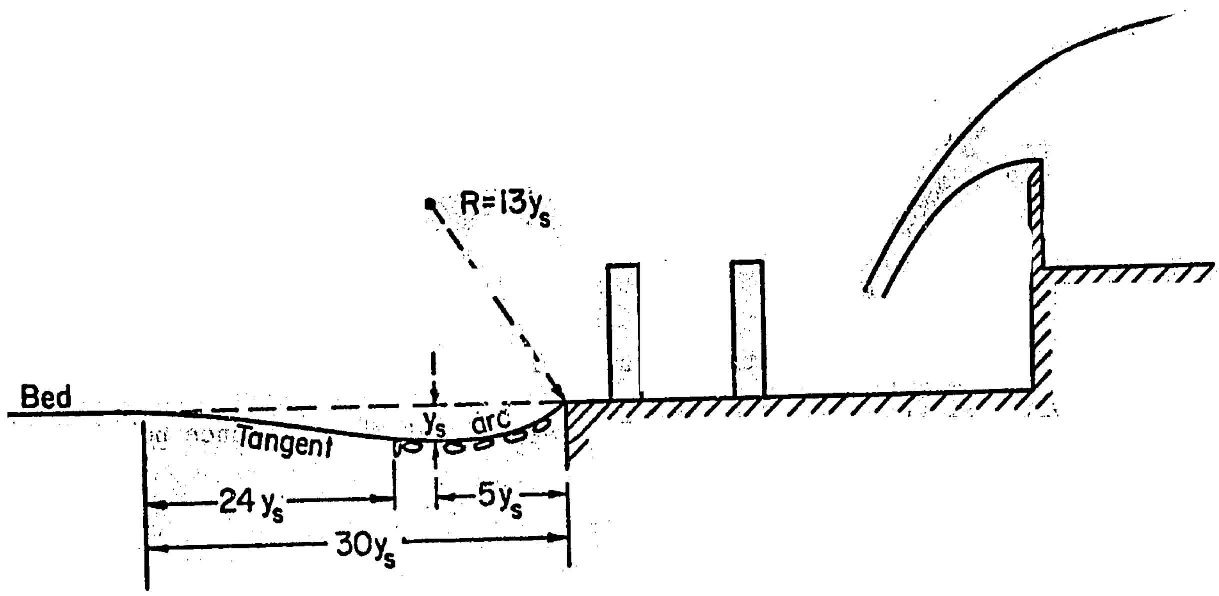


Figure 6-8 Design criteria for scour hole

Use of the above equations, along with knowing the horizontal location of the reference point, will yield a relationship for determining the vertical location of the reference point.

$$Z = h + 0.373H \tag{6-14}$$

Therefore, the available head (specific energy) above the height Z is 0.627H.

The axis of the nappe may be considered perpendicular to a vertical section at a distance 0.5H' from the crest and the horizontal velocity of the filament in the nappe axis may be computed from the relation $V^2/2g$. In this case

$$V_x = (0.627H\ (2g))^{1/2} \tag{6-15}$$

At the reference point, the water particles have no vertical velocity. Consequently, the theoretical vertical velocity of the jet as it strikes the stilling basin floor can be obtained from Equation 6-13 as

$$V_z = (2gZ)^{1/2} \tag{6-16}$$

Therefore, the resultant velocity of the jet as it strikes the floor will be

$$V^2 = V_x^{\,2} + V_z^{\,2} \tag{6-17}$$

A relationship for the time of travel can be obtained from Equation 6-12

$$t = (2Z/g)^{1/2} \tag{6-18}$$

The horizontal distance traveled in this time can be determined by combining Equation 6-18 with Equation 6-15.

$$\begin{aligned} L_2 &= X + 0.5H' \\ &= (V_x)t + 0.5H' \\ &= (0.627H(2g))^{1/2}(2Z/g)^{1/2} + 0.5H' \\ &= 2(0.627HZ)^{1/2} + 0.5H' \end{aligned} \qquad (6\text{-}19)$$

The above equation (Eq. 6-19) is based upon an aerated nappe. In practice, if the nappe is not fully aerated, the free jet will strike the stilling basin floor at a shorter distance from the overflow (weir) crest. Thus, the computation of L_2 using Equation 6-19 would result in a conservative design.

Based upon observations of scour immediately downstream from the drop structure, Katsaitis (1966) found that when L_1 was less than $1.4L_2$, a small increase in bed scour occurred. As a consequence, the following equation can be used to determine L_1.

$$L_1 = 1.5L_2 \qquad (6\text{-}20)$$

Also, the length D has been determined from model studies to give satisfactory performance when

$$D = \frac{0.17}{(g/V^2)} \qquad (6\text{-}21)$$

<u>Design of the structure.</u> The designs for numerous vertical drop structures with dissipation bars are listed in Tables 6-1, 6-2, and 6-3 for the usual range of discharges and drop heights encountered in irrigation systems. Interpolation may be used for intermediate drop conditions. Structures having values of h and H other than those

Table 6-1. Design of Vertical Drop Structures with Dissipation Bars for Drop Heights of 1, 2, and 3 feet

Total Head (H) in ft.	0-5	1-0	0-5	1-0	1-5	0-5	1-0	1-5	2-0
Height of Crest (h_1)	1 ft.		2 ft.			3 ft.			
h + Γ/2 = 0.373H	0.1865	0.3730	0.1865	0.3730	0.5595	0.1865	0.3730	0.5595	0.7460
$Z = h_1 + 0.373H$	1.1865	1.3730	2.1865	2.3730	2.5595	3.1865	3.3730	3.5595	3.7460
$V_x^2 = 0.627H(2g) = 40.379H$	20.190	40.379	20.190	40.379	60.569	20.190	40.379	60.569	80.758
$V_z^2 = 2gZ = 64.4Z$	76.411	88.421	140.811	152.821	164.832	205.211	217.221	229.232	241.242
$V^2 = V_x^2 + V_z^2$	96.601	128.800	161.001	193.200	225.401	225.401	257.600	289.801	322.000
g/V^2	0.3333	0.2500	0.2000	0.1667	0.1429	0.1429	0.1250	0.1111	0.1000
$(H'/2) = 0.435H$	0.2175	0.4350	0.2175	0.4350	0.6525	0.2175	0.4350	0.6525	0.8700
$t^2 = 2Z/g = .0622Z$	0.0738	0.0854	0.1360	0.1476	0.1592	0.1982	0.2098	0.2214	0.2330
t	0.2717	0.2922	0.3688	0.3842	0.3990	0.4452	0.4580	0.4705	0.4827
V_x	4.493	6.355	4.493	6.355	7.785	4.493	6.355	7.785	8.987
$x = V_{xt}$	1.221	1.857	1.657	2.442	3.106	2.000	2.911	3.663	4.338
$L_2 = (H'/2) + x$	1.439	2.292	1.875	2.877	3.759	2.218	3.346	4.316	5.208
$L_1 = 1.5L_2$	2.16	3.44	2.81	4.32	5.64	3.33	5.02	6.47	7.81
$L_4 = W = 0.04/(g/V^2)$	0.12	0.16	0.20	0.24	0.28	0.28	0.32	0.36	0.40
$D = 0.17/(g/V^2)$	0.50	0.68	0.85	1.02	1.19	1.19	1.36	1.53	1.70
$M = 2.728H\ (g/V^2)$	0.4546	0.6828	0.2728	0.4548	0.5847	0.1949	0.3410	0.4545	0.5456
$y_s/H = M - 0.218$	0.237	0.465	0.055	0.237	0.367	0	0.123	0.237	0.328
$y_s = H(y_s/H)$	0.1	0.5	0	0.2	0.6	0	0.1	0.4	0.7
Make D	0.50	0.68	0.85	1.02	1.19	0.85	1.36	1.53	1.70

Table 6-2. Design of Vertical Drop Structures with Dissipation Bars for Drop Heights of 4 feet and 5 feet

Total Head (H) in ft.	0-5	1-0	1-5	2-0	2-5	0-5	1-0	1-5	2-0	2-5	3-0
Height of Crest (h_f)	4 ft.					5 ft.					
$h + T/2 = 0.373H$	0.1865	0.3730	0.5595	0.7460	0.9325	0.1865	0.3730	0.5595	0.7460	0.9325	1.1190
$Z = h_1 + 0.373H$	4.1865	4.3730	4.5595	4.7460	4.9325	5.1865	5.3730	5.5595	5.7460	5.9325	6.1190
$V_x^2 = 0.627H(2g) = 40.379H$	20.190	40.379	60.569	80.758	100.948	20.190	40.379	60.569	80.758	100.948	121.137
$V_z^2 = 2gZ = 64.4Z$	269.611	281.621	293.632	305.642	317.653	334.011	346.021	358.032	370.042	382.053	394.064
$V^2 = V_x^2 + V_z^2$	289.801	322.000	354.201	386.400	418.601	354.201	386.400	418.601	450.800	483.001	515.201
g/V^2	0.1111	0.1000	0.0909	0.0833	0.0769	0.0909	0.0833	0.0769	0.0714	0.0666	0.0625
$(H'/2) = 0.435H$	0.2175	0.4350	0.6525	0.8700	1.0875	0.2175	0.4350	0.6525	0.8700	1.0875	1.3050
$t^2 = 2Z/g = .0622Z$	0.2604	0.2720	0.2836	0.2952	0.3068	0.3226	0.3342	0.3458	0.3574	0.3690	0.3806
t	0.5103	0.5215	0.5325	0.5433	0.5539	0.5680	0.5781	0.5880	0.5978	0.6075	0.6169
V_x	4.493	6.355	7.785	8.987	10.047	4.493	6.355	7.785	8.987	10.047	11.006
$x = V_{xt}$	2.293	3.314	4.146	4.883	5.565	2.552	3.674	4.578	5.372	6.104	6.790
$L_2 = (H'/2) + x$	2.511	3.749	4.799	5.753	6.652	2.770	4.109	5.321	6.242	7.192	8.095
$L_1 = 1.5L_2$	3.77	5.62	7.20	8.63	9.98	4.16	6.16	7.85	9.36	10.79	12.14
$L_3 = W = 0.04/(g/V^2)$	0.36	0.40	0.44	0.48	0.52	0.44	0.48	0.52	0.56	0.60	0.64
$D = 0.17/(g/V^2)$	1.53	1.70	1.87	2.04	2.21	1.87	2.04	2.21	2.38	2.55	2.72
$M = 2.728H\ (g/V^2)$	0.1515	0.2728	0.3720	0.4545	0.5245	0.1240	0.2272	0.3147	0.3896	0.4542	0.5115
$y_s/H = M - 0.218$	0	0.055	0.154	0.237	0.307	0	0.009	0.096	0.172	0.236	0.294
$y_s = H(y_s/H)$	0	0.1	0.2	0.5	0.8	0	0	0.1	0.3	0.6	0.9
Make D	1.19	1.70	1.87	2.04	2.21	1.53	1.70	2.21	2.38	2.55	2.72

Table 6-3. Design of Vertical Drop Structures with Dissipation Bars for Drop Heights of 6 feet and 8 feet

Total Head (H) in ft.	0-5	1-0	1-5	2-0	2-5	3-0	0-5	1-0	1-5	2-0	2-5	3-0
Height of Crest (h_1)	6 ft.						8 ft.					
[illegible] /2 0.373H	0.1865	0.3730	0.5595	0.7460	0.9325	1.1190	0.1865	0.3730	0.5595	0.7460	0.9325	1.1190
[illegible] h, + 0.373H	6.1865	6.3730	6.5595	6.7460	6.9325	7.1190	8.1865	8.3730	8.5595	8.7460	8.9325	9.1190
V_x = 0.627H(2g) = 40.379H	20.190	40.379	60.569	80.758	100.948	121.137	20.190	40.379	60.569	80.758	100.948	121.137
V_z^2 = 2gZ = 64.4Z	398.411	410.421	422.432	434.442	466.453	458.464	527.211	539.221	551.232	563.242	575.253	587.264
$V^2 = V_x^2 + V_z^2$	418.601	450.800	483.001	515.200	547.401	579.601	547.401	579.600	611.801	644.000	676.201	708.401
g/V^2	0.0769	0.0714	0.0666	0.0625	0.0588	0.0555	0.0588	0.0555	0.0526	0.0500	0.0476	0.0455
(H'/2) 0.435H	0.2175	0.4350	0.6525	0.8700	1.0875	1.3050	0.2175	0.4350	0.6525	0.8700	1.0875	1.3050
[illegible] 2Z/g - .0622Z	0.3848	0.3964	0.4080	0.4196	0.4312	0.4428	0.5092	0.5208	0.5324	0.5440	0.5556	0.5672
[illegible]	0.6203	0.6296	0.6387	0.6478	0.6567	0.6654	0.7136	0.7217	0.7297	0.7376	0.7454	0.7531
[illegible]	4.493	6.355	7.785	8.987	10.047	11.006	4.493	6.355	7.785	8.987	10.047	11.006
[illegible] V_{xt}	2.787	4.001	4.972	5.822	6.598	7.323	3.206	4.586	5.680	6.629	7.489	8.289
L (H'/2) + x	3.005	4.436	5.625	6.692	7.686	8.628	3.424	5.021	6.333	7.499	8.577	9.594
L_1 = 1.5L	4.50	6.65	8.44	10.04	11.53	12.94	5.14	7.53	9.50	11.25	12.87	14.39
L W = 0.04/(g/V^2)	0.52	0.56	0.60	0.64	0.68	0.72	0.68	0.72	0.76	0.80	0.84	0.88
D = 0.17/(g/V^2)	2.21	2.38	2.55	2.72	2.89	3.06	2.94	3.07	3.23	3.40	3.57	3.74
[illegible] 2.728H (g/V)	0.1049	0.1948	0.2725	0.3410	0.4010	0.4542	0.0802	0.1514	0.2152	0.2728	0.3246	0.3724
y_s/H = M - 0.218	0	0	0.055	0.123	0.183	0.236	0	0	0	0.055	0.107	0.155
$Y_s = H(y_s/H)$	0	0	0.1	0.3	0.5	0.7	0	0	0	0.1	0.3	0.5
Make D	1.87	2.04	2.55	2.72	2.89	3.06	2.55	2.72	2.89	3.40	3.57	3.74

in the tables are solved by substituting the desired values in the proper equations, which are also listed in the tables. The design procedure developed by Katsaitis (1966) is described below.

1. If the scour depth y_s is found to be zero, then the resulting value of D may be more than necessary. The size of D to be used in such cases is that given by the lower weir with zero scour at the same discharge. Therefore, a final row is provided in Tables 6-1, 6-2, and 6-3, which is entitled "Make D," showing how much D could be made.
2. The height of bars, if made equal to twice the critical depth of maximum flow, would be sufficient to protect both bed and banks.
3. The propagated waves in these structures are usually of small wave-length and dissipate themselves within a short distance. However, very strong surface waves are generated when the tailwater is near the crest level, thereby resulting in severe bank scour. This condition is not frequently encountered with drop structures and would normally be avoided. If such a condition exists, the waves could be dissipated by one row of inverted bars hanging from a beam as well as the two rows of floor bars. However, this application has not yet been investigated systematically (Katsaitis, 1966).
4. The height of bars should span the full depth of flow when smooth flows are necessary for a downstream flow measuring device.

5. The usual sill found at the downstream end of stilling basins must be excluded because it destroys the streamlined flow produced by the bars. The longitudinal shape of the scour bed downstream from the dissipation structure should be designed as shown in Figure 6-8. Having found y_s (see Step 9), the value of radius R used to form the depression in the channel bed is found by

$$R = 13y_s \tag{6-22}$$

6. The extension wall is formed by drawing a straight line on the side wall (Fig. 6-6), such that it touches the top of the second row of bars and has the slope of the banks. Comparison tests of this triangular extension wall with a rectangular extension wall of equal length have shown equal performance, which can be attributed to having high velocities occur in this structure near the bed where the triangular walls have their maximum length. The extension walls constitute a simple transition between the rectangular section of flow within the structure and the trapezoidal section of an earthen channel. They extend to a distance where the velocity becomes low and thus eliminate separation of flow which creates eddies and tends to cause scour around them. The dissipation bars greatly reduce the velocity of flow exiting from the structure, thereby shortening the required length of the extension walls.

7. The tailwater depth may vary to any degree without affecting the performance of the bars.

8. The higher the drop the more efficient this structure becomes for the same discharge. This can be seen in the worked examples listed in Tables 6-1, 6-2 and 6-3. Although the energy increases with drop height, the parabolic trajectory of the nappe results in the jet hitting the floor at an angle closer to 90°, which is the ideal angle for full impact. On the other hand, in low drops with large flows, the impact angle is much smaller than 90° and most of the energy dissipation occurs because of the dissipation bars, with the floor assisting very little in the energy dissipation process. Consequently, the designer should attempt to construct high drop structures.

9. The length L_1 is determined from Equation 6-20, while the value of L_2 is obtained from Equation 6-19 and D from Equation 6-21. Also, L_3 is made equal to W. The scour depth y_s may be computed from

$$y_s/h = 2.728(H(g/V^2)) - 0.2177 \tag{6-23}$$

The width of the bars W may be obtained from

$$W(g/V^2) = 0.04 \tag{6-24}$$

If the bar width resulting from Equation 6-24 is not satisfactory, it may be changed. In such a case, any percentage variation from the value 0.04 in Equation 6-24 will also change the value of y_s/H by the same percentage multiplied by 1.2. Hence, the scour depth y_s must be adjusted accordingly.

Performance. Dissipation bars proved very effective in a model with a St. Anthony Falls (SAF) type of energy dissipator (Katsaitis, 1966). Comparison tests were made by replacing the SAF structure with a dissipation bars structure of equal length. The result was that the dissipation bars structure passed 50 percent more than the design discharge without scour. The same discharge with the SAF energy dissipator produced severe scour.

Chapter 7

LOOSE BOUNDARY STILLING BASINS

Although reinforced concrete is usually used in the construction of stilling basins for energy dissipation structures, it frequently becomes desirable from an economic standpoint to place loose rock as a lining for the stilling basin. The use of rock, or riprap, can be particularly advantageous for small drop heights, h.

Flow Parameters

The problem of designing a riprapped stilling basin for vertical drop structures has been recently investigated by Smith and Strang (1967). A definition sketch for the flow situation studied by Smith and Strang is shown in Figure 7-1. The stabilized scour depth, Y_s was evaluated using as variables the head H, the drop height h the tail-water flow depth Y_2 and the mean stone diameter D_s. These variables can be combined into the following dimensionless ratios:

$$Y_s/h = f(H/h,\ Y_2/h,\ D_s/h) \qquad (7\text{-}1)$$

The experiments were conducted in a flume 1 foot wide and 37 feet long. The three stone sizes used in these studies had an average size of 0.021 feet, 0.041 feet, and 0.089 feet.

Data on scour hole depth has been plotted by Smith and Strang (1967) in the form of dimensionless curves as shown in Figure 33. The value of Y_s/h represents the stabilized scour depth below an initially horizontal bed. The data on scour depth was collected under steady flow operation at the specified values of H/h, Y_2/h, and h/D_s.

The volume of stone scoured from below the initial bed level must be deposited above the initial bed level. The actual depth of the deposit may be more or less than the depth of scour, depending upon the

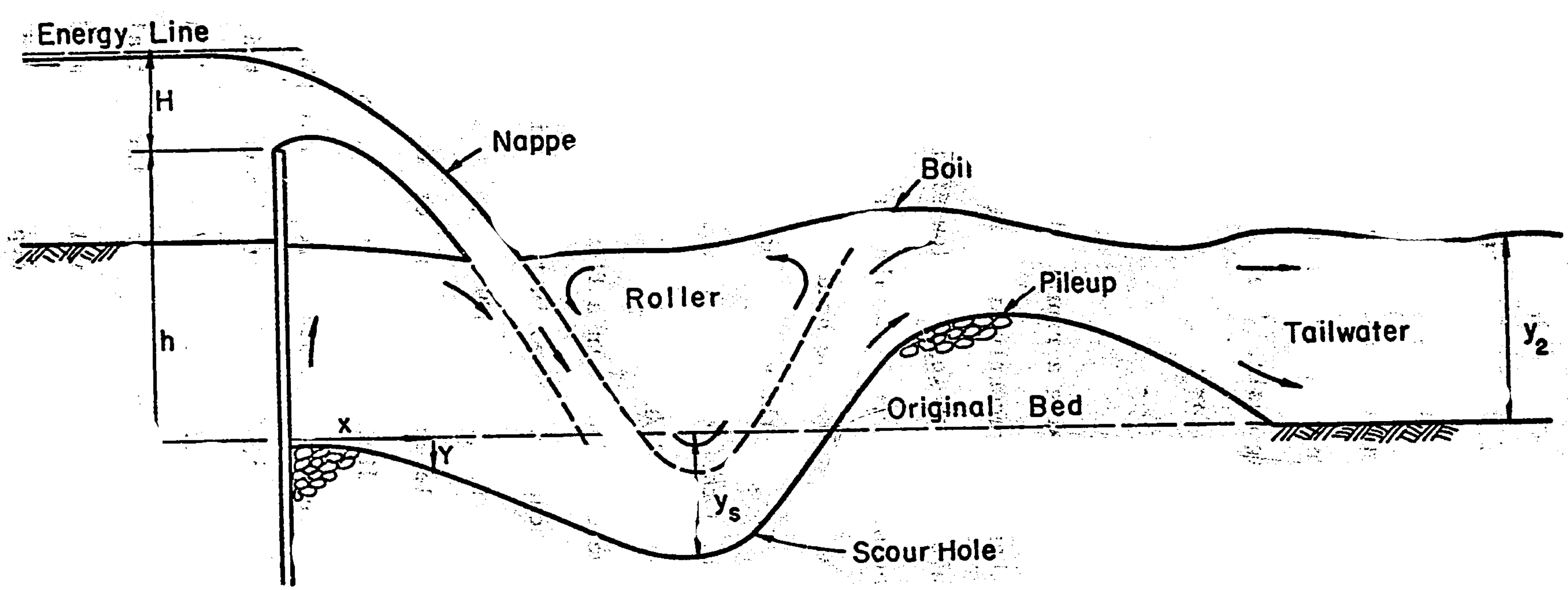


Figure 7-1 Definition sketch for loose boundary stilling basin

tailwater condition. The top of the deposit must always be below the tailwater level at design flow. When the tailwater is relatively deep, the depth of the deposit may exceed the scour depth. If the tailwater is shallow, the flow over the top of the pile may be supercritical, which in turn will limit the height to which the deposit can rise. Between these conditions, it is possible to have the scour depth equal to the deposit depth.

The maximum scour depth calculated from Figure 7-2 need not correspond to the maximum discharge. If the overflow crest is partially submerged at maximum discharge, the maximum calculated scour may occur for a smaller discharge. The condition corresponding to maximum scour can be determined by using Figure 7-2 in conjunction with a tailwater rating curve. For this case, hydrograph tests by Smith and Strang (1967) showed that the actual scour depth will exceed the calculated scour. When Y_2 exceeds h, a flat jet trajectory is produced which erodes the top of the previously stabilized pile and moves some of the stone further downstream, thereby resulting in a deepended scour hole on the subsequent hydrograph when the pile is reformed.

Design Procedure

The hydrograph tests raised considerable doubt about the long term stability of the riprapped stilling basin if based on a deep scour depth and a corresponding deep pileup on the downstream channel. The pileup is an integral part of the ultimate stable shape. Should any stone be removed from the pile for any reason, the size of the scour hole will increase accordingly. With the stone bed placed at the level of the downstream channel and $Y_c = 0.5$ h, performance was marginal. Although

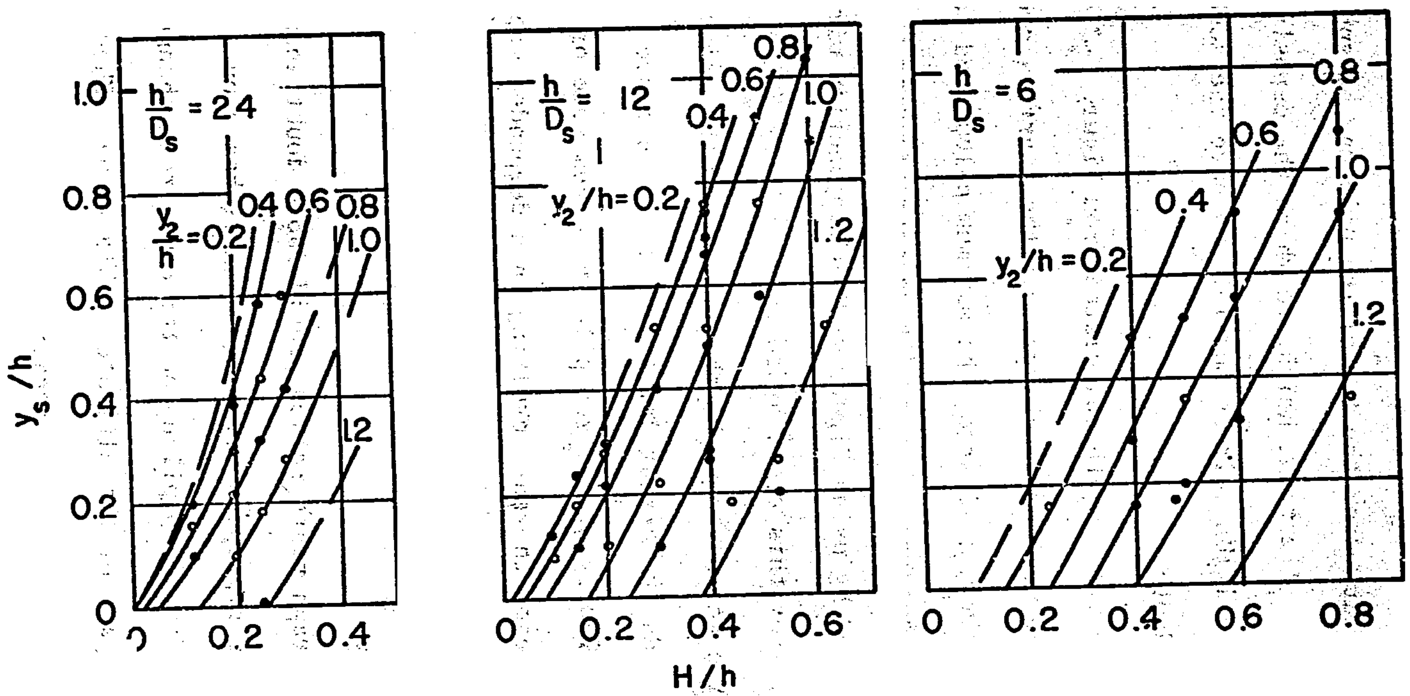


Figure 7-2 Scour depth relationships for vertical drop structures with loose boundary stilling basin.

not tested under hydrograph conditions, Smith and Strang (1967) believe that the scour depth Y_s should not exceed half the drop height (0.5h). A deep scour depth will result in a high downstream deposit with supercritical flow occurring over the top of the rock deposit. This flow condition would require a greater length of side protection, otherwise side eddies could further erode the rock deposit. The chances of stone removal of eddies, ice, debris, animals, or man are far greater when the deposit is relatively high above the channel bed.

Smith and Strang (1967) reasoned that if the stone bed was lowered relative to the downstream channel, and if the design scour depth was reduced, the performance under hydrograph conditions would be considerably improved. Their proposed design is shown in Figure 7-3. A depression of $2y_s/3$ was selected so that the top of the deposit would not form as high above the downstream channel bed. The data in Figure 7-2 can still be applied to this case, except that the drop height and tailwater depth must be taken relative to the original stone bed and not the channel bed. Hence, h' and Y_2' must be substituted for h and Y_2 in using Figure 7-2.

The method outlined above is intended to eliminate the occurrence of supercritical flow over the pile, giving a more stable design, and allowing the length of side protection to be reduced (Smith and Strang, 1967). Also, since $H/h' < H/h$ and $Y_2'/h' > Y_2/h$, the scour depth will be reduced and the volume of stone necessary to construct the basin will actually be less than if the stone were placed at the higher elevation.

In order to secure an adequate cutoff, the depth of the vertical wall must extend below the bottom of the excavation for the stone. Therefore, it is unlikely that it would be economical to exceed a design depth of 0.3 h' for Y_s. At this value, a total wall height of at least 2h would be required. In order to meet this limitation in the field, it may be necessary to increase D_s, decrease h (by using more drops), or decrease H (by using a wider structure).

The surface of the stone should be placed level, and the scour hole should be allowed to develop by natural means. This is simpler than attempting to preform the hole. In any case, the shape of the scour hold will vary slightly as the head changes. Smith and Strang (1967) recommend that the depth of stone placed in the stilling basin be 1-1/2 times the maximum scour depth obtained from Figure 7-2. This recommendation will provide ample margin of safety as long as the tail-water level at maximum discharge remains at, or below, the overflow crest. In case of a low weir on a natural watercourse, which may become submerged at maximum flow, an initial stone depth of at least $2Y_s$ should be provided. The tests by Smith and Strang showed that a base length of $4H + h'/4$ will safely accommodate any scour hole regardless of the value of Y_2', since Y_2' is the variable having the least effect on scour hole position.

The riprapped stilling basin design shown in Figure 7-3 was subjected to hydrograph tests using $h' = 0.5$ ft., $h'/D_s = 12$, and $Y_s/h' = 0.3$. The surface of the stone bed was depressed 0.2h' below the original downstream channel bed, so 0.2P° was added to each tailwater depth to determine the tailwater depth Y_2 above the stone bed.

The performance of the design was highly satisfactory. The depth of the pile became stabilized in one hydrograph, supercritical flow downstream from the scour hole was eliminated and wave action was reduced. This design is recommended for field use (Smith and Strang, 1967).

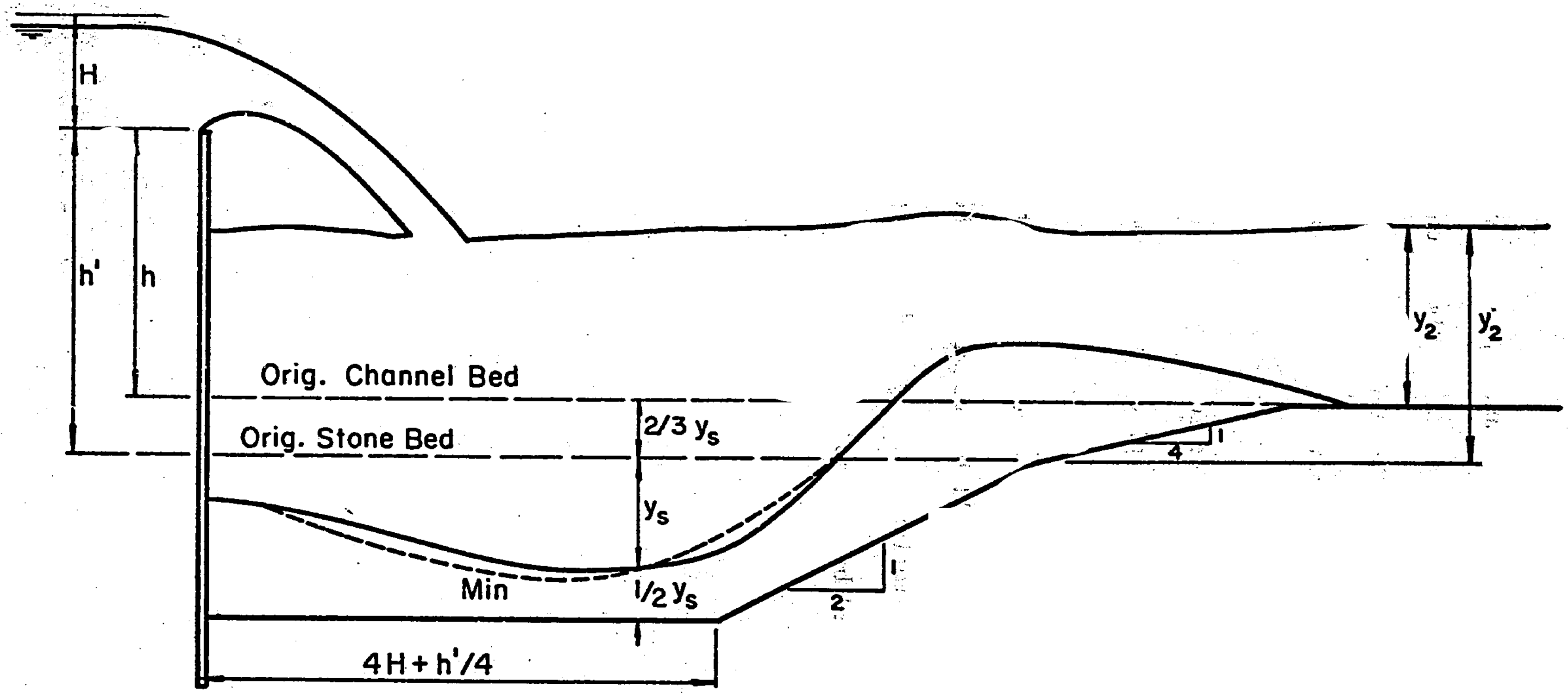


Figure 7-3 Recommended design of loose boundary stilling basins for vertical drop structures

Since the data reported by Smith and Strang is valid only for two-dimensional flow, they recommend that third dimension effects at the ends of the overflow be eliminated by constructing vertical abutment walls. Also, the sidewalls would have to extend downstream about twice the base length of the stone bed.

In order to prevent migration of foundation material under the stone bed, a filter layer should be placed at the boundary of the excavation line (Smith and Strang, 1967). This is particularly important if the underlying material contains fine sand or silt. The criterior for designing such a filter layer will be described in the following section.

The filter layer is similar to the graded rip-rap explained in the next chapter as determined by Hallmark.

Chapter 8

BED AND BANK PROTECTION

The use of natural open channels for the transportation of water presents the possibility of scour. Loose boundary channels are constantly shifting alignment due to this phenomenon while man-made channels through natural earth must be carefully designed so as to either keep local velocities below that causing scour (critical) or to maintain an equilibrium between deposition and entrainment at any one point. Where the discharge cannot be controlled so as to keep velocities below critical, some means of channel protection must be employed.

Rock has been used for lining floodway channels, river improvements, and below hydraulic structures as a means of protection against the scouring action of flowing water. It is usually believed that very large boulders would prevent scour, but in order to reduce costs, it is desirable to know the smallest size rock that can safely be used in a particular design situation. The problem then is to establish the laws and relationships that will allow prediction of the flow conditions under which a given size particle will be moved from its position.

Incipient Motion

Rubey (1938) discusses three theories regarding incipient motion. The first, reported by Leslie in 1829, is the "sixth-power law" which states that the weight of the largest particles that can be moved by a stream varies as the sixth power of the stream velocity. A second theory is that of a critical tractive force such that the diameter of the largest particle moved by a stream varies as the depth of flow times the slope of the energy gradient. The third theory reported by Rubey is that incipient motion is caused by lift forces due to velocity gradients acting on the particle.

Many investigators have reported velocities, often referred to as competent velocities, required to move various diameters of sand and gravel particles. Some of the earliest investigators were DuBuat in 1786, Bouniceau in 1845, Blackwell in 1857, Sainjon in 1871, Suchier in 1874, and Gilbert in 1914 as summarized in Figure 8-1 by Qazi (1958). Primarily, these studies related the average velocity of flow to the diameter of particle that could be expected to be moved by the flow.

Isbash (1936), investigated the stability of stones in place on a rock fill. The stability equation developed by Isbash is

$$U_{mc} = y_i 2g(S_s-1)\ (D_s)^{\frac{1}{2}}\ (\cos \phi)^{\frac{1}{2}} \tag{8-1}$$

in which

U_{mc} = mean velocity that will just move the stone, ft/sec

y_i = coefficient

g = gravitational acceleration, ft/sec^2

S_s = specific gravity of stone

D_e = diameter of stone, ft

ϕ = angle of inclination of plane that sone lies on

Grimm and Leupold (1939) utilized the stability equation developed by Isbash in analyzing experimental work of the Corps of Engineers on the incipient motion of stones up to 5.5-inches in diameter. They found a two-fold variation of the coefficient y_i (1.21 to 2.39).

An anonymous (1936) Russian article listed (Table 8-1) competent mean velocities for natural roughness elements from 0.005 to 200 millimeters. The most significant aspect of this publication was the inclusion of the depth of flow as a parameter describing the particle size that could be moved by the flow.

Results of investigations by Jakobson in 1945, Meyer-Peter and Muller in 1948, Sundborg in 1956, and Hedar in 1962, are reported by Andersson (1964) and compared in Figure 8-2. The equations developed by these investigators are similar in form to that developed by Isbash except that the depth of flow is taken into account. Both Sundborg and Hedar include the angle of repose of the material as an added variable.

Table 8-1. USSR data on permissible velocities for noncohesive particles.

Material	Particle diameter mm	Mean velocity ft/sec
Silt	0.005	0.49
Fine Sand	0.05	0.66
Medium Sand	0 .25	0.98
Coarse Sand	1.00	1.80
Fine Gravel	2.50	2.13
Medium Gravel	5.00	2.62
Coarse Gravel	10.00	3.28
Fine Pebbles	15.0	3.94
Medium Pebbles	25.0	4.59
Coarse Pebbles	40.0	5.91
Large Pebbles	75.0	7.87
Large Pebbles	100.00	8.86
Large Pebbles	150.0	10.83
	200.0	12.80

USSR corrections of permissible velocity for depth for noncohesive materials.

	Average Depth						
Meters	0.30	0.60	1.00	1.50	2.00	2.50	3.00
Feet	0.98	1.97	3.28	4.92	6.56	8.20	9.84
Correction Factor	0.8	0.9	1.00	1.1	1.15	1.20	1.25

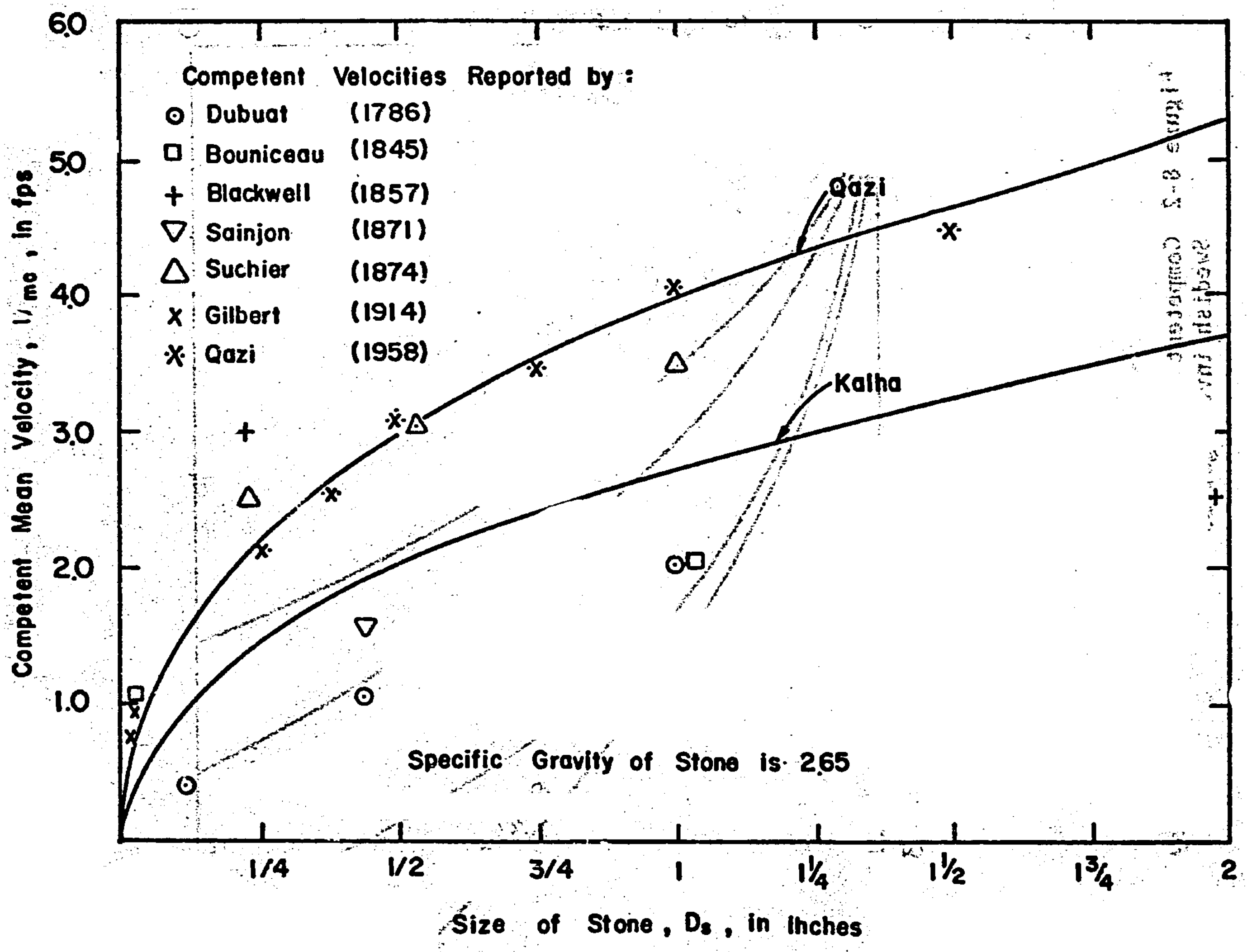


Figure 8-1 Competent velocity of stone

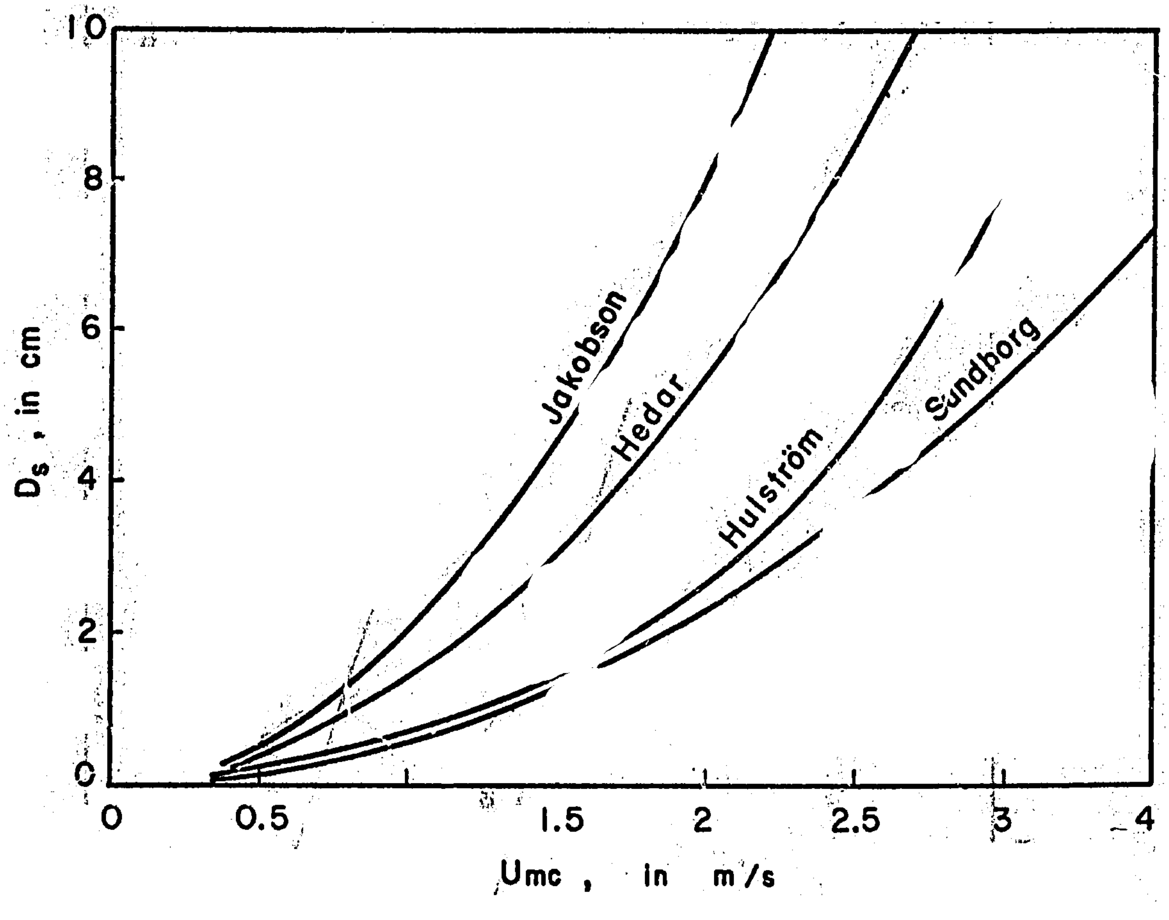


Figure 8-2 Competent velocity relationships reported by Swedish investigators

White (1940) has also studied the problem of incipient motion. Like Rubey, he understood the necessity for studying the flow conditions surrounding a particle rather than just the bulk flow donditions. The following excerpt illustrates White's thinking.

> Like any solid boundary, the loose granular bed exerts a drag upon the fluid, and the accompanying shear stress is transmitted from bed to fluid almost wholly by the more prominent grains in the uppermost layer. Each such grain transmits a small force, and the manner in which it does so again depends upon the type of motion, though it is now the local motion which is concerned, rather than that of the main stream.

Einstein and El Samni (1949) investigated the pressure fluctuations about a bed of hemispheres having a diameter of 0.225 feet. The significant results of their research was that, (a) the effective location of a rough boundary was found to be at 0.2 diameters below the top of the hemispheres as shown in Figure 8-3, and (b) the coefficient of lift for the hemispheres was constant when the flow velocity at a particular point above the bed was used in computing the lift forces.

Ippen and Verma (1953) studied the incipient motion of plastic and glass spheres varying in diameter from 2 to 4 millimeters and having specific gravities of 1.28 and 2.38, respectively. The use of edge velocity, U_{ec} (the velocity at the top edge of the particle), was introduced into their analysis of lift and drag forces, although edge velocity, lift, and drag were not measured directly. Also, the fall velocity w for spheres was used in the analysis.

Roberge and Peixotto (1956) extended the work of Ippen and Verma by studying natural roughness elements of stone and coal varying in diameter from 0.0125 to 0.0333 feet and having specific gravities of 2.65 and 1.30, respectively. The concept of edge velocity was used in describing the surface resistance, form drag, and lift, as shown

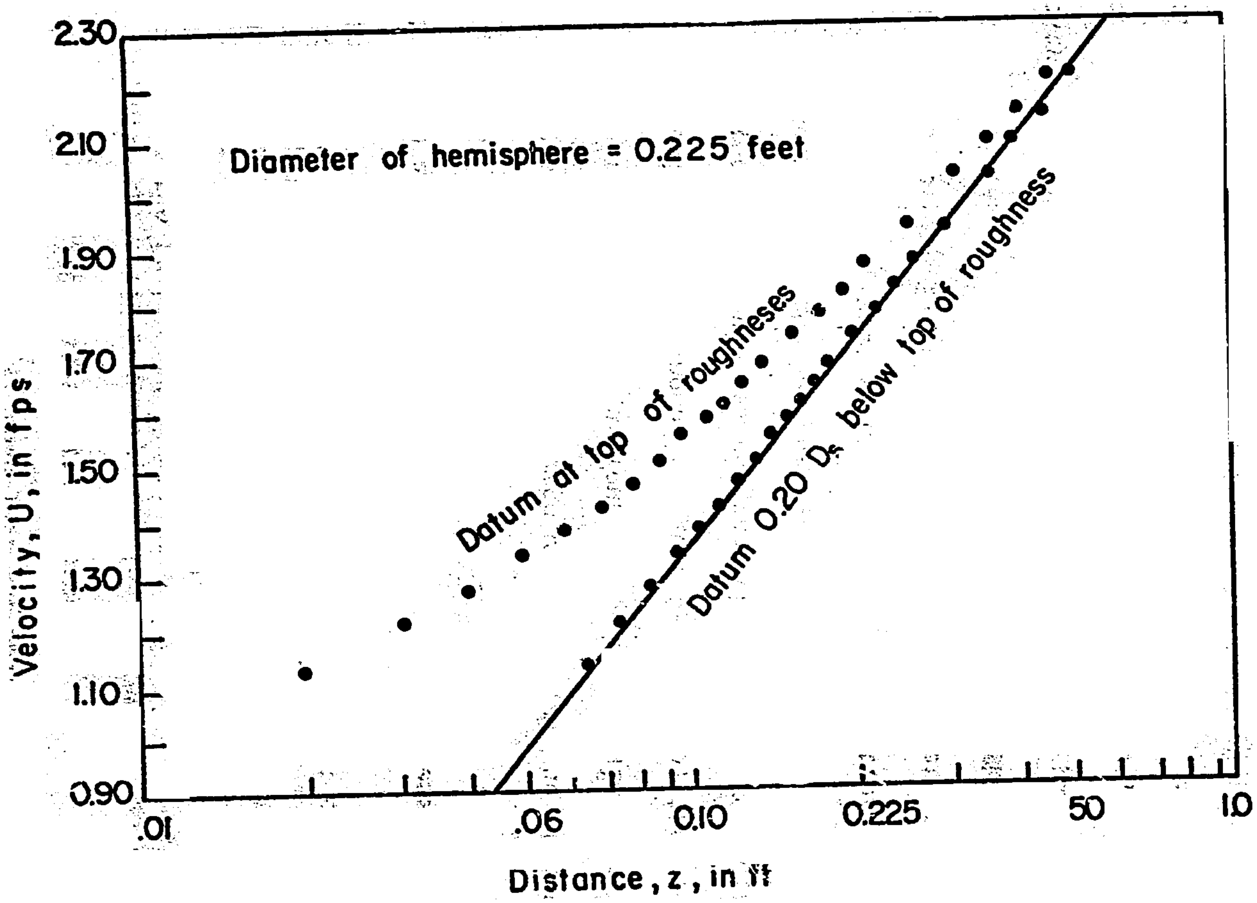


Figure 8-3 Vertical velocity distribution for hemispherical bed as reported by Einstein and El Samni (1949)

Surface resistance: $$F_s = C_s \rho A \frac{U_{ec}^2}{2} \tag{8-2}$$

Form drag: $$F_D = C_D \rho A \frac{U_{ec}^2}{2} \tag{8-3}$$

Lift: $$F_L = C_L \rho A \frac{U_{ec}^2}{2} \tag{8-4}$$

in which

F_s = surface resistance drag, lb.

F_D = form drag, lb

F_L = lift, lb.

C_s = coefficient of surface resistance

C_D = coefficient of form drag

C_L = coefficient of lift

ρ = density of water, slugs/ft.^{3}

A = area exposed to flow, ft.^{2}

U_{ec} = competent edge velocity, ft./sec

The edge velocity was taken as the velocity measured 0.07 inches above the particle. Actual drag and lift forces were not measured. For large Reynolds numbers, the equation developed by Roberge and Peixotto for incipient motion is

$$U_{ec} = \left[\frac{gD_s\ (S_s-1)}{0.96}\right]^{1/2} \tag{8-5}$$

A comparison of Equation 8-5 with results reported by other investigators for particles having a specific gravity of 2.65 is shown in Figure 8-4.

Yalin (1965) has applied dimensional analysis to the problem of sediment transport to arrive at four dimensionless parameters.

$$\frac{\rho U_{*c}^2}{\gamma_s' D_s} = \frac{\tau_c}{\gamma_s' D_s} = f\left[\frac{U_{*c} D_s}{\nu}, \frac{D_s}{d}, \frac{\rho_s}{\rho}\right] \tag{8-6}$$

in which

U_{*c} = competent shear velocity

γ_s' = submerged specific weight of roughness element

= $\rho g(S_s - 1)$

τ_c = competent bed shear stress

ν = kinematic viscosity of fluid

d = depth of flow

ρ_s = denisty of roughness element

The classic diagram developed by Shields (1936) plots $\tau_c/(\gamma_s' D_s)$ against $U_{*c}D_s/\nu$ as shown in Figure 8-5.

Neill (1967) has substituted U_{mc} for U_{*c} in Equation 8-6.

$$\frac{\rho U_{mc}^2}{\gamma_s' D_s} = f\left[\frac{U_{mc} D_s}{\nu}, \frac{D_s}{d}, \frac{\rho_s}{\rho}\right] \tag{8-7}$$

In the analysis reported by Neill, the Reynolds number was ignored since ...."the Reynolds number is irrelevant for normal-density $\left[S_s = 2.65\right]$ material if the grain-size exceeds approximately 3mm."

Also, Neill ignored the density ratio since the particles used in his experimental program were of comparable density. The results of Neills investigation is shown in Figure 8-6 which can be described by the equation

$$\frac{\rho U_{mc}^2}{\gamma_s' D_s} = 2.0\left[\frac{D_s}{d}\right]^{1/3} \tag{8-8}$$

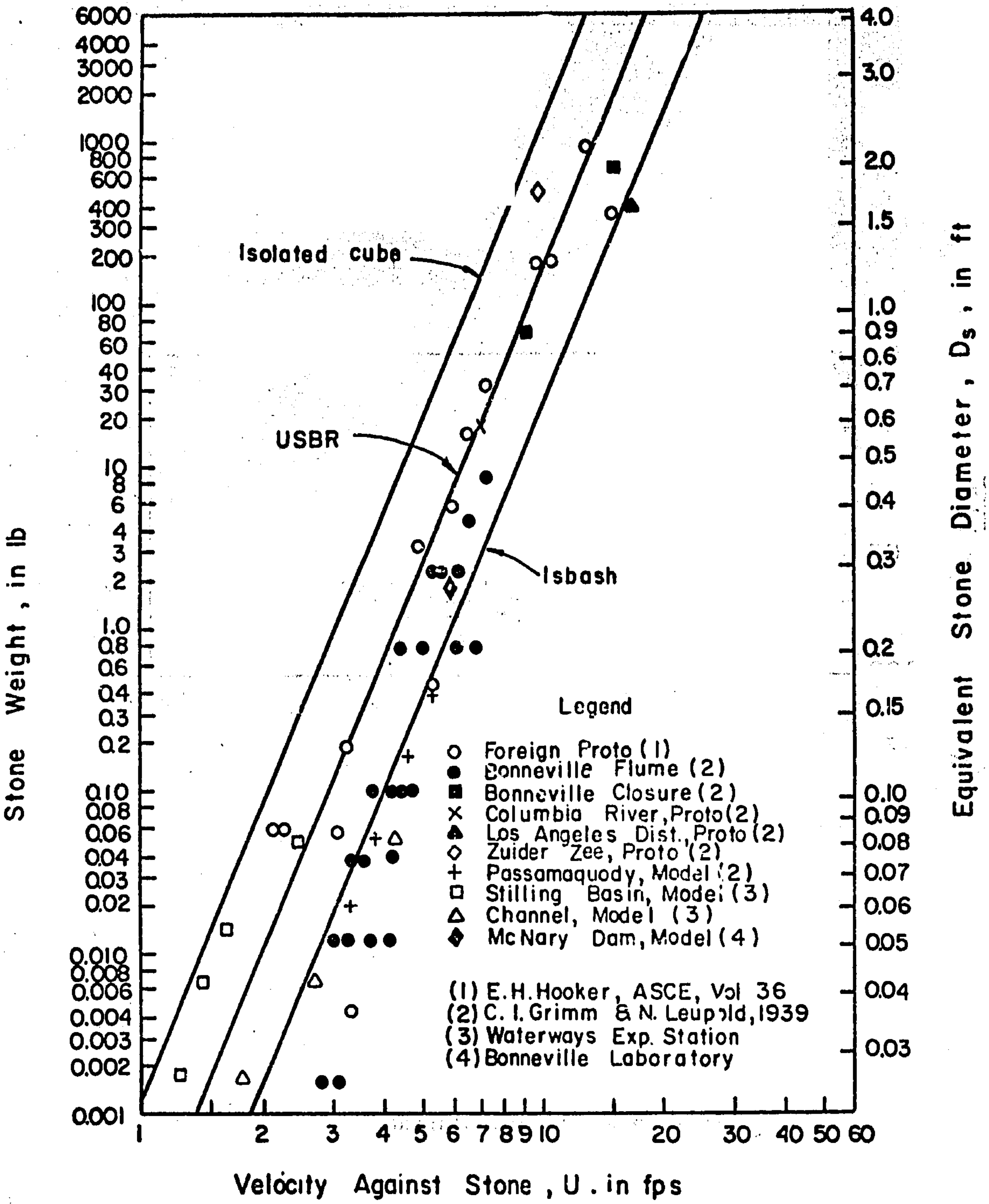


Figure 8-4 Comparisons of various velocity-particle size relationships

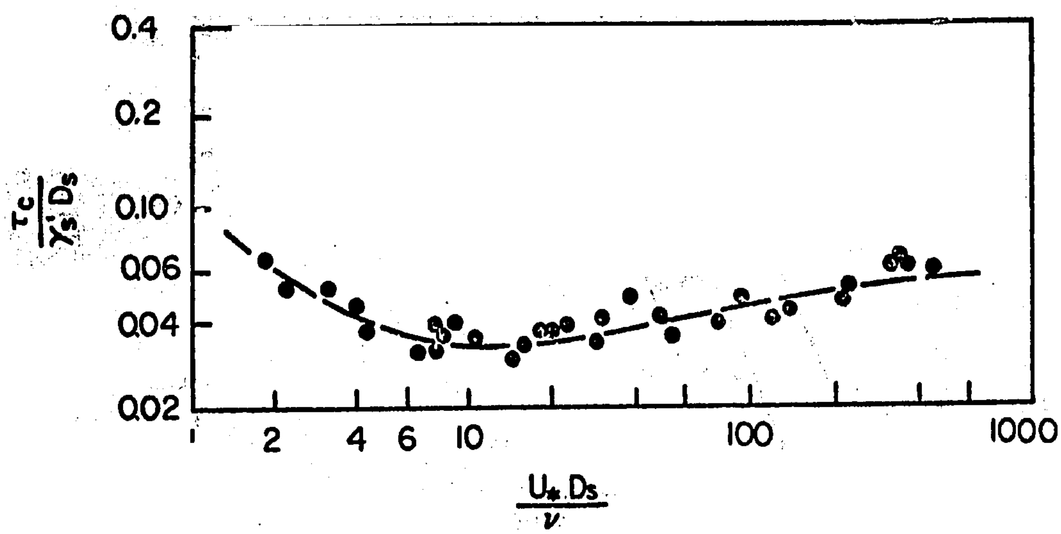


Figure 8-5 Shields diagram

Anderson, Paintal, and Davenport (1968) have developed a design procedure for riprap lined channels. Manning's equation can be written as

$$Q = \frac{1.49}{n} AR^{2/3} S_e^{1/2} \tag{8-9}$$

in which

- n is a flow resistance coefficient
- A is the cross-sectional area of flow
- R is the hydraulic radius (where $R = A/P$ and P is the wetted perimeter)
- S_e is the slope of the energy line

If uniform flow exists in a channel, S_e is equal to the slope of the channel bed S_b . Anderson, Paintal, and Davenport (1968) use the following equation for computing n

$$n = 0.0395 D_{50}^{1/6} \tag{8-10}$$

The effective size of the riprap mixture D_{50} is that size of which 50 percent of the material is finer by weight and is measured in feet.

The other equations utilized by Anderson, Paintal, and Davenport (1968) in developing a design procedure for riprap lining of open channels are

$$\tau_c = 4 D_{50} \tag{8-11}$$

and

$$\tau_{o(max)} = 1.5 \gamma R S_b \tag{8-12}$$

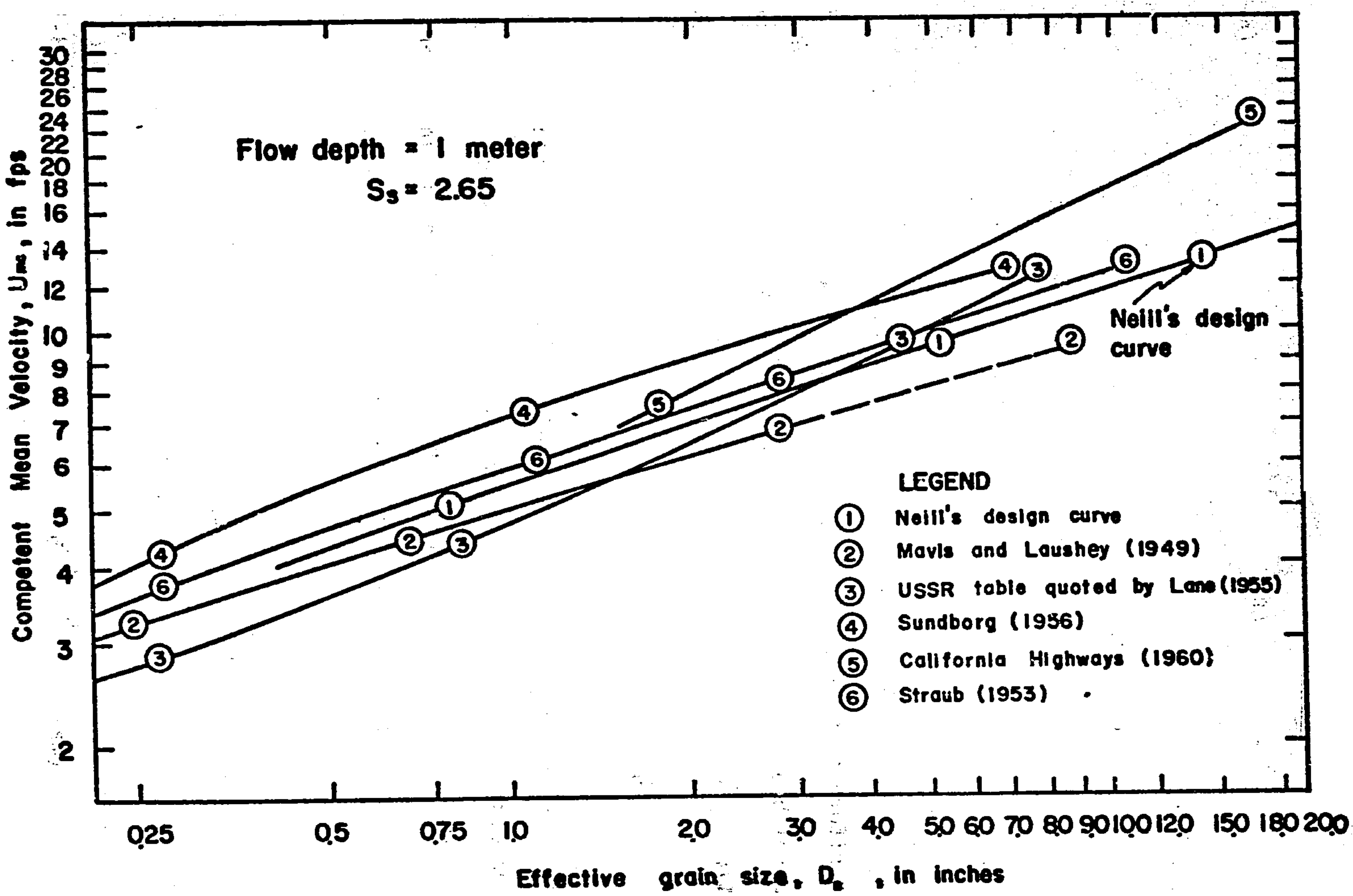


Figure 8-6 Plot of Neills design curve and comparable data by others

in which τ_c is the critical boundary shear for which the riprap will be stable and $\tau_{o(max)}$ is the maximum boundary shear occurring on periphery of a channel. The coefficient of 1.5 in Equation 8-12 is and estimated ratio of the maximum boundary shear to the mean boundary shear for a trapezoidal channel.

The value of the Shields function $\tau_c/(\gamma_s' D_s)$ commonly used in checking the stability of a channel bed is 0.06. An interesting comparison can be made between the criterion of Equation 8-11 used by Anderson, Paintal, and Davenport (1968) and the Shields function. If it is assumed that $D_{50} = D_s$, then Equation 8-11 can be substituted into the Shields function and if the specific gravity of the riprap is assumed to be 2.65, a numerical value of the Shields function is obtained, which is 0.039. Interestingly, Neill (1968) states that a value of 0.03 for the Shields function is necessary to insure that only a very small amount of bed material will be moved by the flowing water. Therefore, it would appear that the design procedure developed by Anderson, Paintal, and Davenport (1968) should be valid, since it is a compromise between the usual value of 0.06 and the results of Neill's work.

Design Procedure

The following design procedure and design charts have been developed by Anderson, Paintal, and Davenport (1968). Experimental data have been used in conjunction with flow characteristics to describe the characteristics of the riprap lining and the channel dimensions necessary to convey a given discharge on a given slope. Based upon these relationships, Ancerson, Paintal, and Davenport (1968) have assumed that, for purposes of design, the following conditions are applicable.

1. The channel to be designed will be essentially straight and of trapezoidal cross-section. The effect of bends in the alignment of the channel on the resulting design will be treated as a corrective factor that will be applied to the design.
2. The flow will be essentially uniform and can be described by the Manning flow formula:

$$V = \frac{1.49}{n} R^{2/3} S_b^{1/2} \qquad (8\text{-}13)$$

in which the symbols are as previously defined. Certain precautions must be taken at the entrance and outlet of the channel and consideration given to these regions of possible nonuniform flow.
3. Since it is presumed that the riprap used in the channel will be stable and that the channel alignment is essentially straight, Mannings n , the roughness coefficient, will depend only upon the effective size of the rock fragments that make up the riprap and can be expressed as

$$n = 0.0395\, D_{50}^{1/6} \qquad (8\text{-}10)$$

The effective size of the riprap mixture D_{50} is that size of which 50 per cent of the material is finer by weight and is measured in feet.
4. The critical boundary shear τ_c which represents the maximum shear for which the riprap will be stable is directly proportional to the effective size D_{50} or

$$\tau_c = 4\, D_{50} \qquad (8\text{-}11)$$

5. The boundary shear stress is not uniformly distributed around the wetted perimeter of the channel. The magnitude and location of the maximum shear on the boundary depends upon the shape of the cross section. For the wider trapezoidal channels, the maximum shear occurs at the center of the bed with lesser maxima on the side slopes. For narrow trapezoidal channels, the maximum shear occurs on the side slope. The excess of the maximum boundary shear over the mean shear varies somewhat with the width-depth ratio. In order to simplify the computation and design charts the ratio of the maximum boundary shear stress is taken to be 1.5 times the mean for all trapezoidal channels (Anderson, Paintal, and Davenport, 1968).

$$\tau_{o(max)} = 1.5\gamma R S_b \qquad (8\text{-}12)$$

6. Because of the component of the force of gravity acting on the riprap in the direction of the side slope, the critical boundary shear stress for the riprap on the sloping side is less than that for riprap on the bed. The ratio of the critical boundary shear on the sloping side to the critical boundary shear acting on a similar particle on the bed is

$$K = \frac{\tau_{cs}}{\tau_{cb}} = \sqrt{1 - \frac{\sin^2 \phi}{\sin^2 \theta}} \qquad (8\text{-}14)$$

7. The discharge to be conveyed in the channel and the topographic slope upon which it is to be constructed are prescribed by external conditions; that is, they are independent variable. Under certain circumstances the size of the riprap that may

also be considered an independent variable and the channels designed to take this into account.

The ratio of width to depth for trapezoidal channels must be limited to practical values. This may be done arbitrarily within certain limits. The ratio P/R , where P is the wetted perimeter and R is the hydraulic radius, is a minimum for a triangular channel. For channels with side slopes of 2.5:1 P/R (min) equals 11.6; for 3:1 P/R (min) equals 13.3; and for side slopes of 4:1 P/R (min) equals 17. Since trapezoidal channels with side slopes of 4:1 or less are relatively rare, the minimum P/R ratio is taken as 13.3. The upper limit of P/R is set at 30, since wider channels become uneconomical. In practice, the ultimate design will probably be between these limits.

The development of design charts for trapezoidal channels by Anderson, Paintal, and Davenport (1968) is founded upon the basic equations described above. The combination of these equations with the continuity equation results in a relationship between the principal variables of discharge, slope, shape of channel, and size of riprap material.

Starting with the basic Manning formula (Equation 8-13), the roughness coefficient defined by Equation 8-10 may be introduced with that result that

$$V = \frac{37.7}{D_{50}^{1/6}} R^{2/3} S_b^{1/2} \qquad (8\text{-}15)$$

Further, since the stability of the riprap depends upon the boundary shear not exceeding the critical boundary shear for the riprap, the condition for stability is that

$$ \tag{8-16}$$

The combination of Equations 8-11 and 8-12 results in an expression for R in the form

$$R = \frac{D_{50}}{1.5\gamma S_b} = 0.0428 \frac{D_{50}}{S_b} \tag{8-17}$$

When Equation 8-17 for the hydraulic radius is substituted into Equation 8-15 the mean velocity can be expressed, in terms of the size of the riprap and the longitudinal slope of the channel, as

$$V = 4.60 \frac{D_{50}^{1/2}}{S_b^{1/6}} \tag{8-18}$$

Introducing the continuity equation defined as

$$Q = VA = VPR = VR^2 \cdot \frac{P}{R} \tag{8-19}$$

into Equation 8-18 and using Equation 8-17, results in an equation relating the discharge, the longitudinal slope of the channel, the size of the riprap, and the shape of the channel as defined by P/R in the form

$$Q = \frac{1}{118} \frac{D_{50}^{5/2}}{S_b^{13/6}} \frac{P}{R} \tag{8-20}$$

This equation shows that, for a given discharge and slope, the size of riprap that is needed to protect the channel depends also upon the channel shape. For given values of P/R, Equation 8-20 can be plotted with the discharge Q as a function of the slope S_b with D_{50} as a parameter. The values of P/R to be used can be taken as the upper and lower limits of practical channels. These limits have been taken as

P/R = 13.3 and P/R = 30 for the reasons given previously. Figures 8-7 and 8-8 represent Equation 8-20 for P/R = 13.3 and P/R = 30, respectively. For these values of P/R, the charts give the size of riprap required to line a channel having the given discharge Q and slope S_b such that the riprap is stable. The determination of D_{50} from each chart provides two limits within which the actual size must fall. Once the size of riprap between these two limits has been chosen, the velocity and the hydraulic radius can be determined from Figures 8-9 and 8-10, which are graphical representation of Equations 8-17 and 8-18 and give V and R in terms of the known values od D_{50} and S_b. Obtaining the mean velocity from Figure 8-9, the required cross sectional area is obtained from Figure 8-10, which is a graphical representation of the continuity equation.

The required side slopes are obtained from Figures 8-12 and 8-13, which are obtained by combining Equation 8-14 with the following equation

$$\frac{\tau_{s(max)}}{\tau_{b(max)}} = 0.8 \qquad (8\text{-}21)$$

Equation 8-21 states that the riprap on the side slopes should have a resistance to movement which is 0.8 times the movement resistance of the riprap placed on the channel bed. Thus, the shearing stress on the sloping sides is appreciably less than that on the bottom. At the same time, the critical shear stress for riprap on the side of the trapezoidal channel is also less than that for the same size on the bed. Now, if the side slopes are adjusted so that the ratio of critical shear on the side to that on the bed is equal to the ratio of the boundary shears on the side to that on the bottom, then the riprap will be equally stable on the side and on the bottom. The result is given in

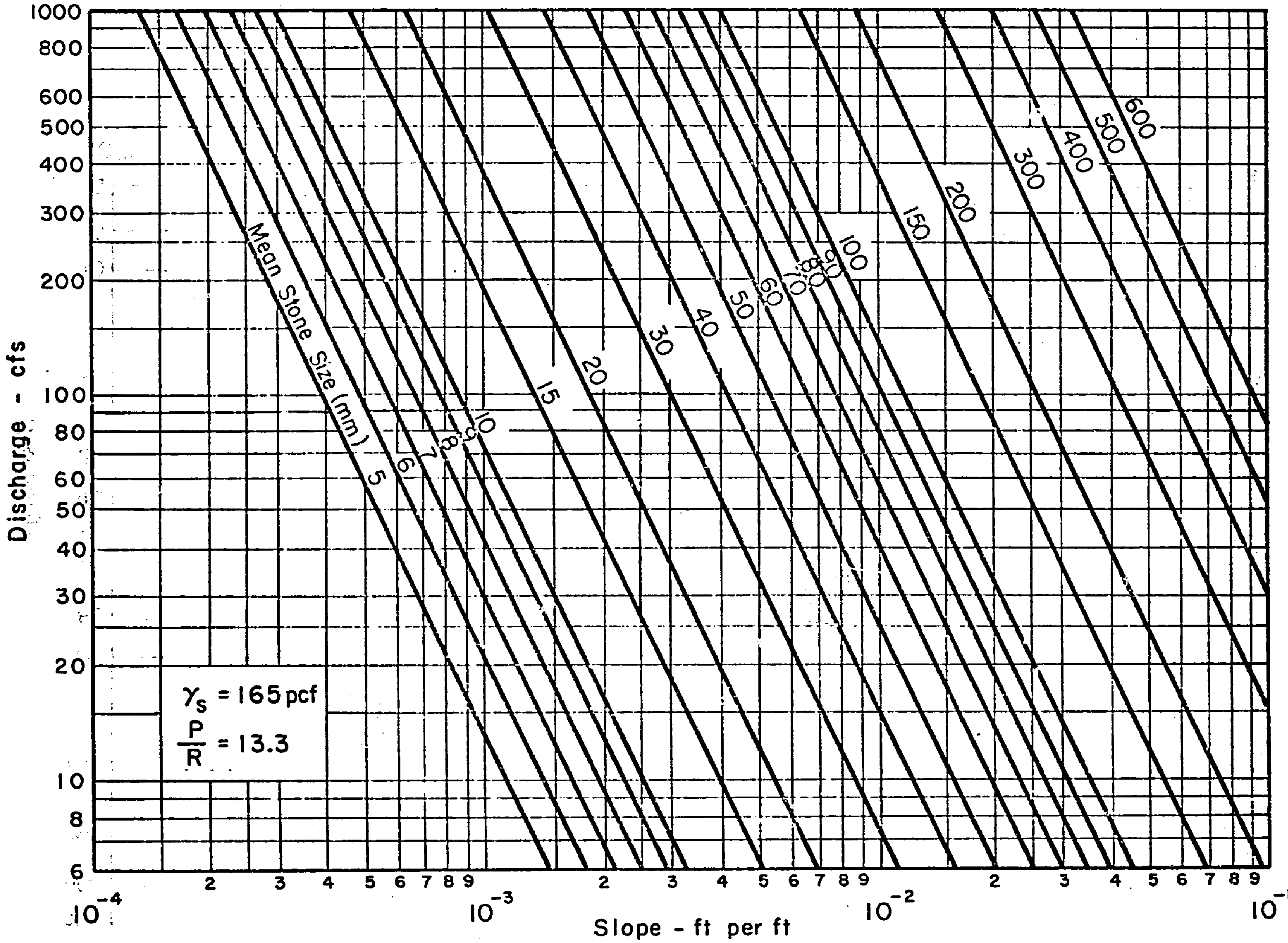


Figure 8-7 Minimum size (mean) of stone riprap that will be stable in trapezoidal channels with P/R = 13.3 for various combinations of discharge and slope

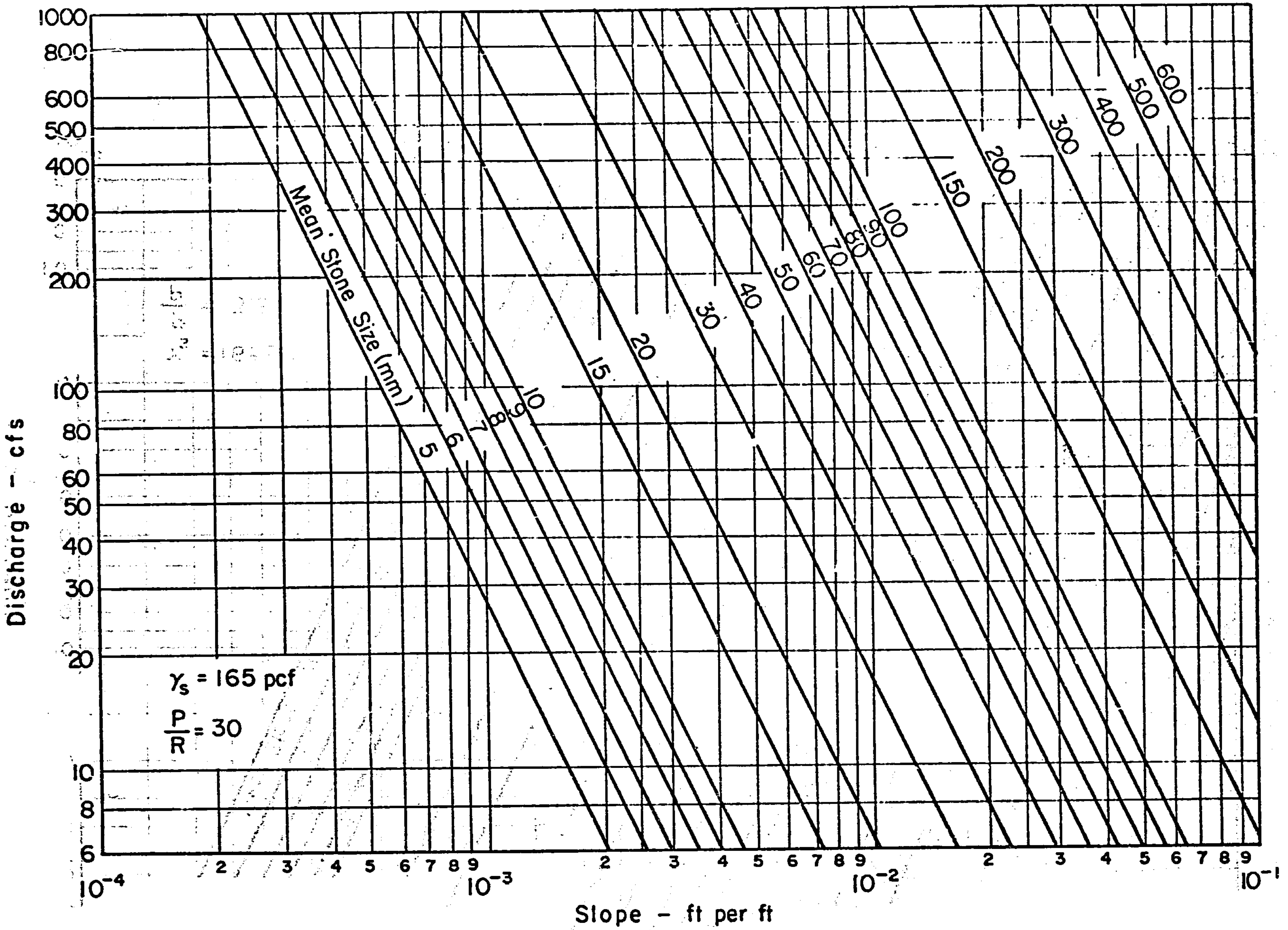


Figure 8-8 Minimum size (mean) of stone riprap that will be stable in trapezoidal channels with P/R = 30 for various combinations of discharge and slope

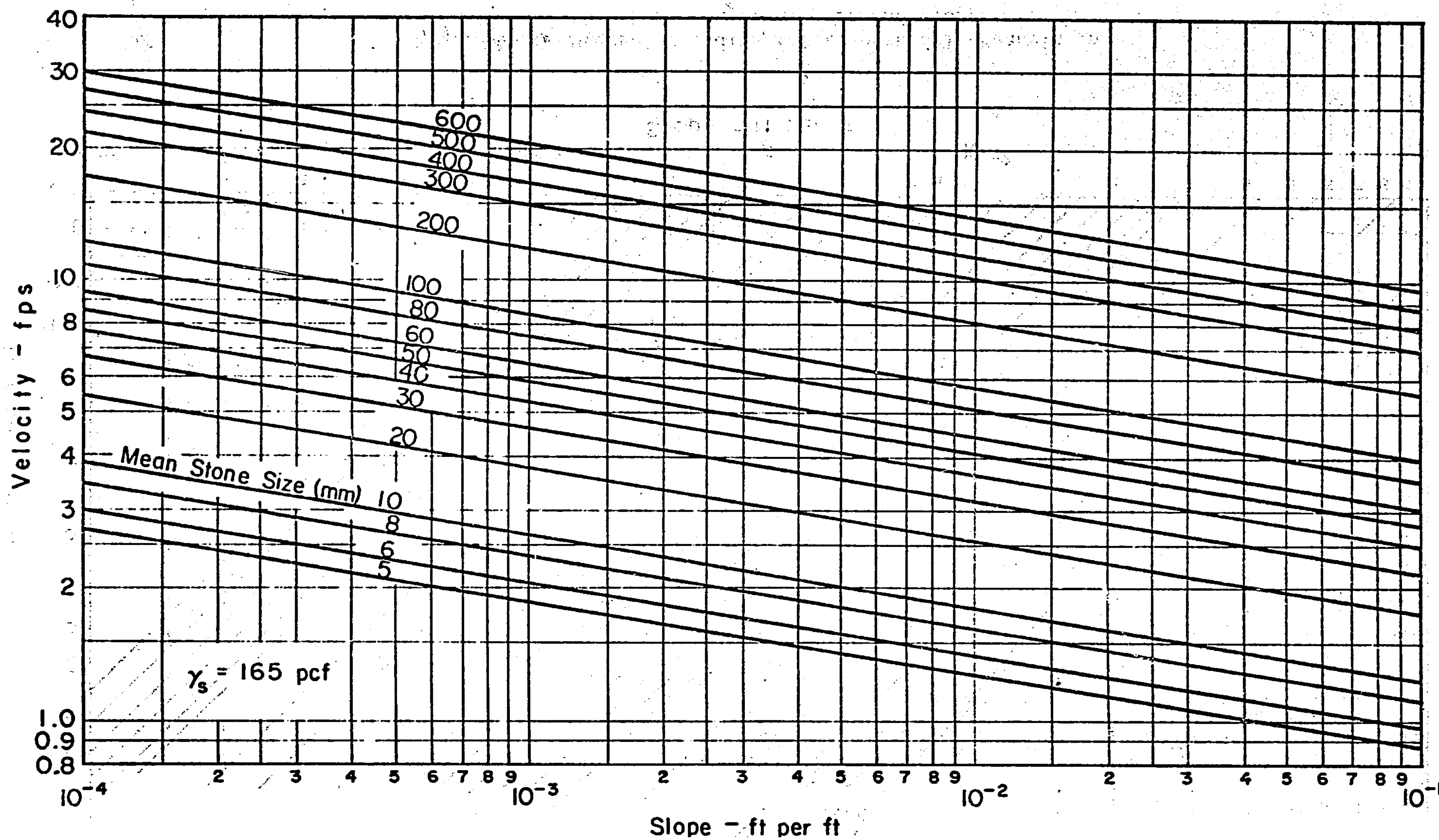


Figure 8-9 Maximum mean velocity for stable riprap in trapezoidal channels for various mean stone sizes and slopes

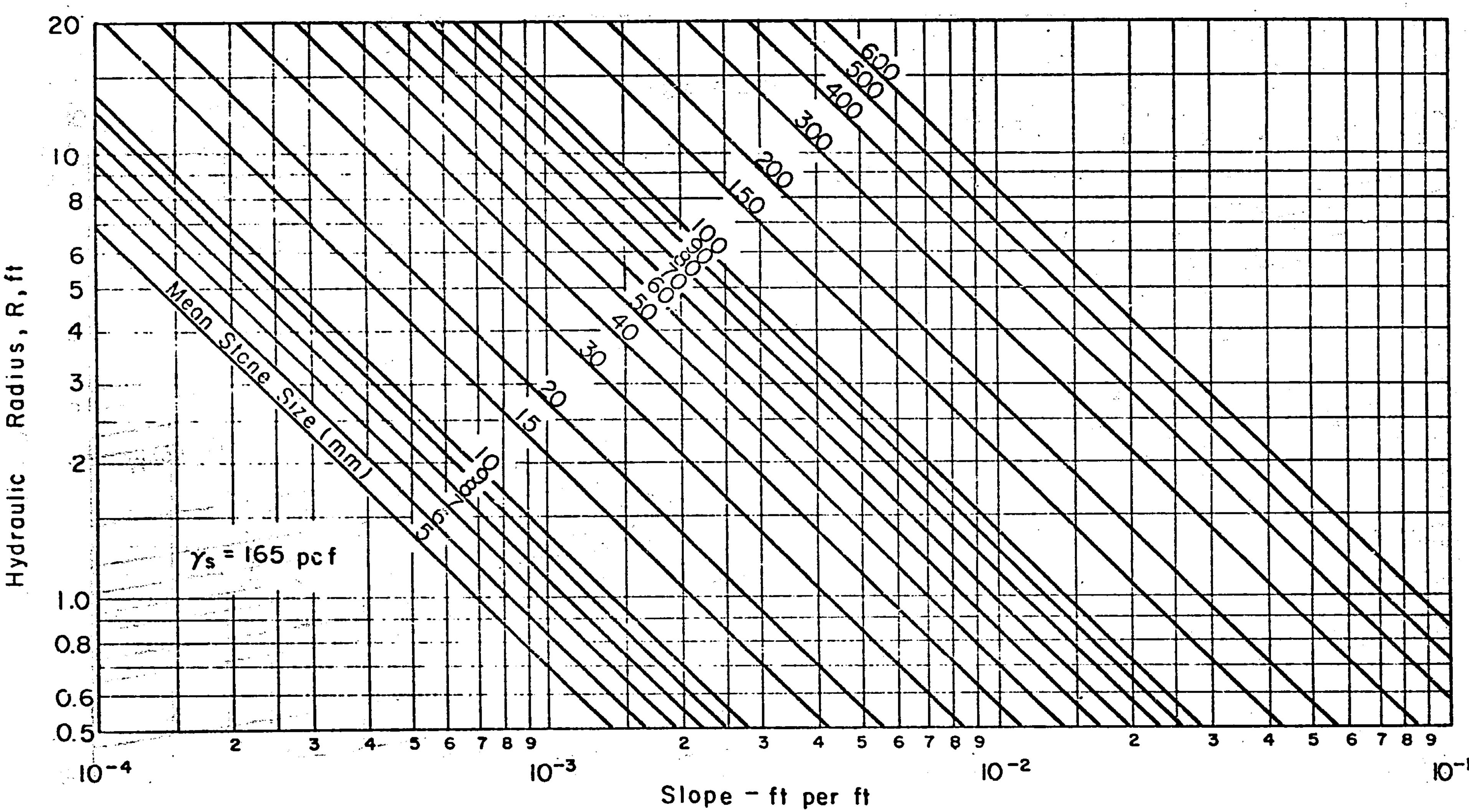


Figure 8-10 Hydraulic radius for trapezoidal channels in terms of mean stone size and slope

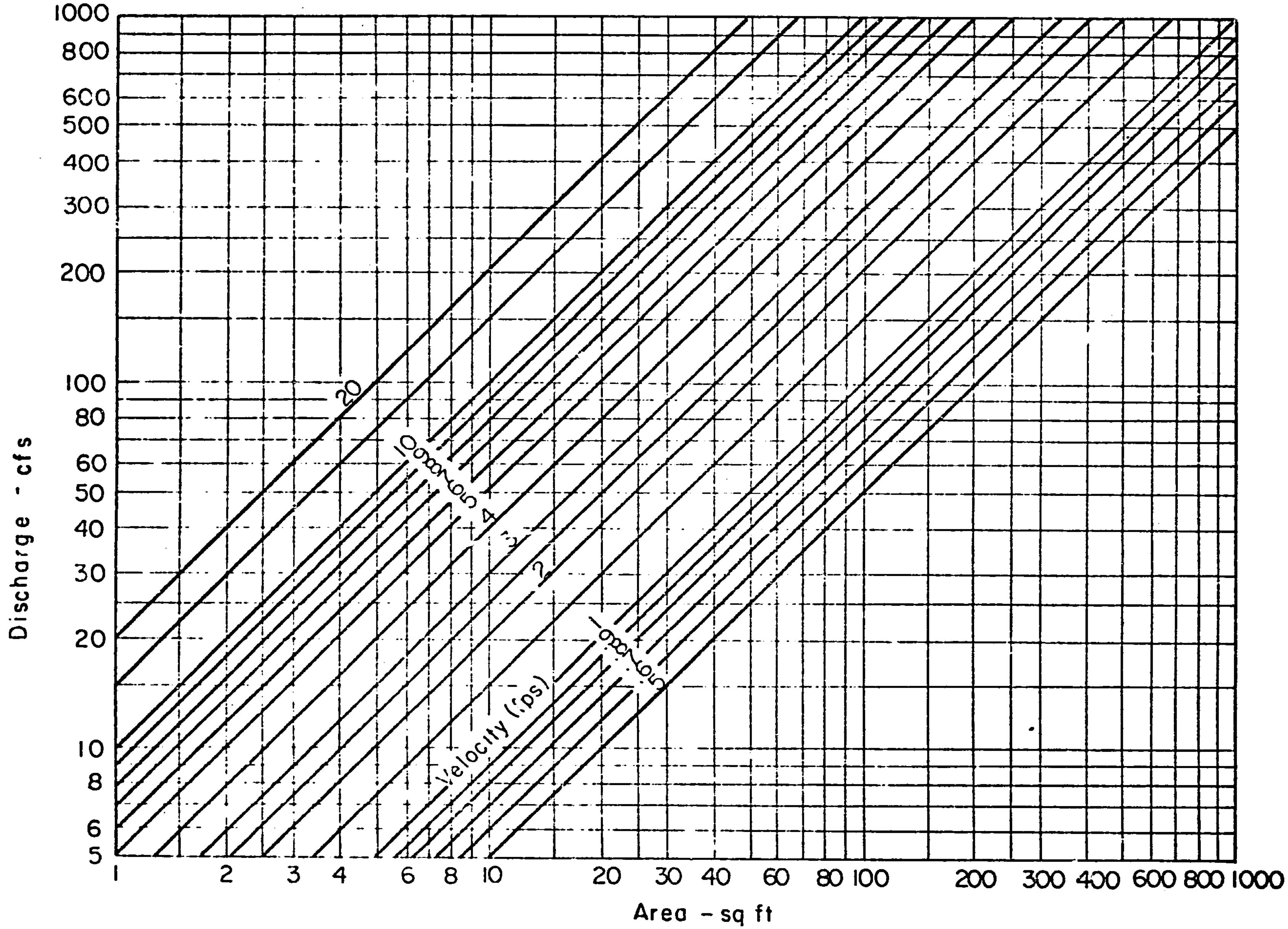


Figure 8-11 Area of a trapezoidal channel in terms of discharge and maximum mean velocity

Figures 8-12 and 8-13. Given the size of riprap D_{50} and its angularity, the angle of repose is taken from Figure 8-12. Using this angle of repose, the side slope is determined from Figure 8-13. The curve in Figure 8-13 represents the actual variation between angle of repose and side slope, but for practical reasons the range of side slopes was divided into three groups and each assigned a value (Anderso , Paintal, and Davenport, 1968)

Having chosen the side slope and having determined the cross sectional area and the hydraulic radius, the channel geometry is obtained directly from the appropriate chart of Figures 8-14 ahrough 8-18. In these design charts, the side slope is established so that the riprap on the side is as stable as that on the bottom and the size of the riprap is a minimum consistent with the superimposed discharge and slope. If, for any reason, it is desirable to make the side slopes steeper than what is given by the design charts, the size of the riprap can be increased to accommodate the increased side slope at the cost of some loss of efficiency on the channel bottom. Assuming that the angle of repose is a constant, the value of K in Figure 8-19 can be determined for both the design slope and the desired steeper slope. Assuming further that the riprap on the bottom remains unchanged, K becomes proportional to the critical boundary shear stress of the side riprap, and therefore through Equation 8-21. K is also proportional to D_{50}. Therefore,

$$D'_{50} = D_{50} \frac{K}{K'} \qquad (8\text{-}22)$$

in which D'_{50} is the required size of riprap on the steeper side slope.

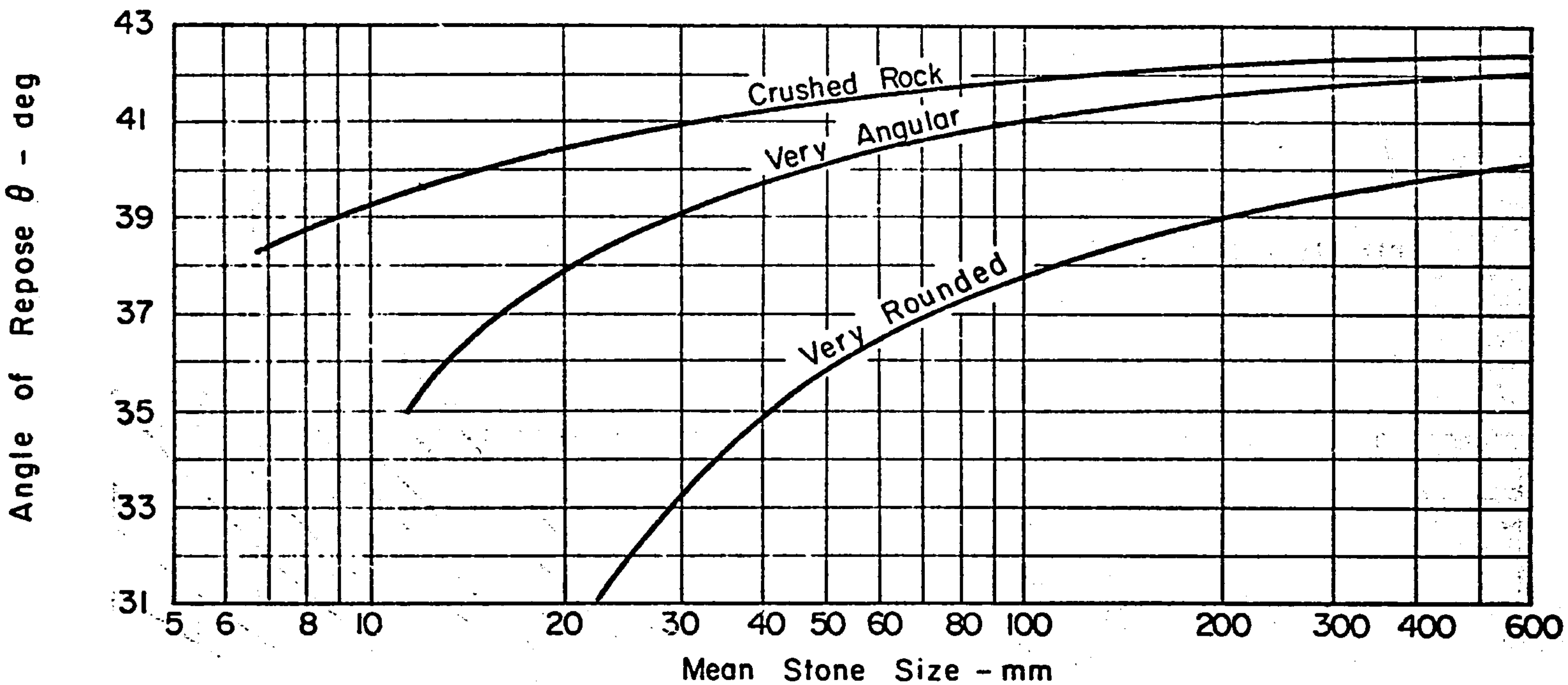


Figure 8-12 Angle of repose of riprap in terms of mean size and shape of stone

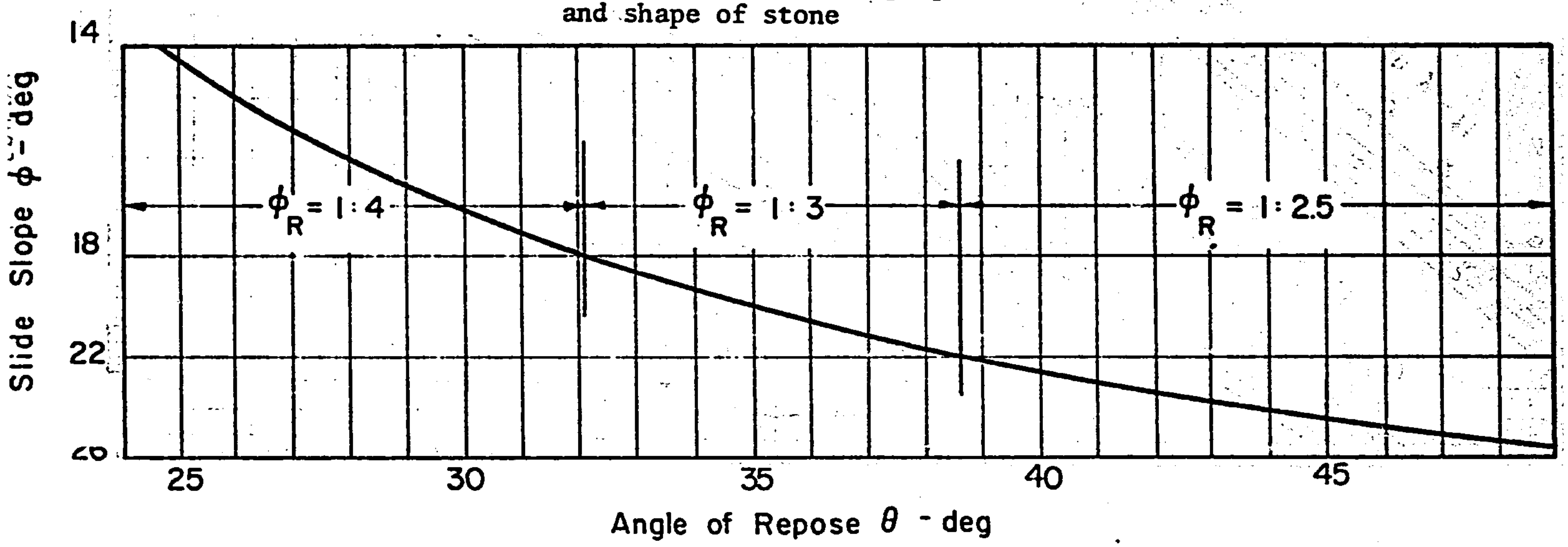


Figure 8-13 Recommended side slopes of trapezoidal channels in terms of riprap angle of repose

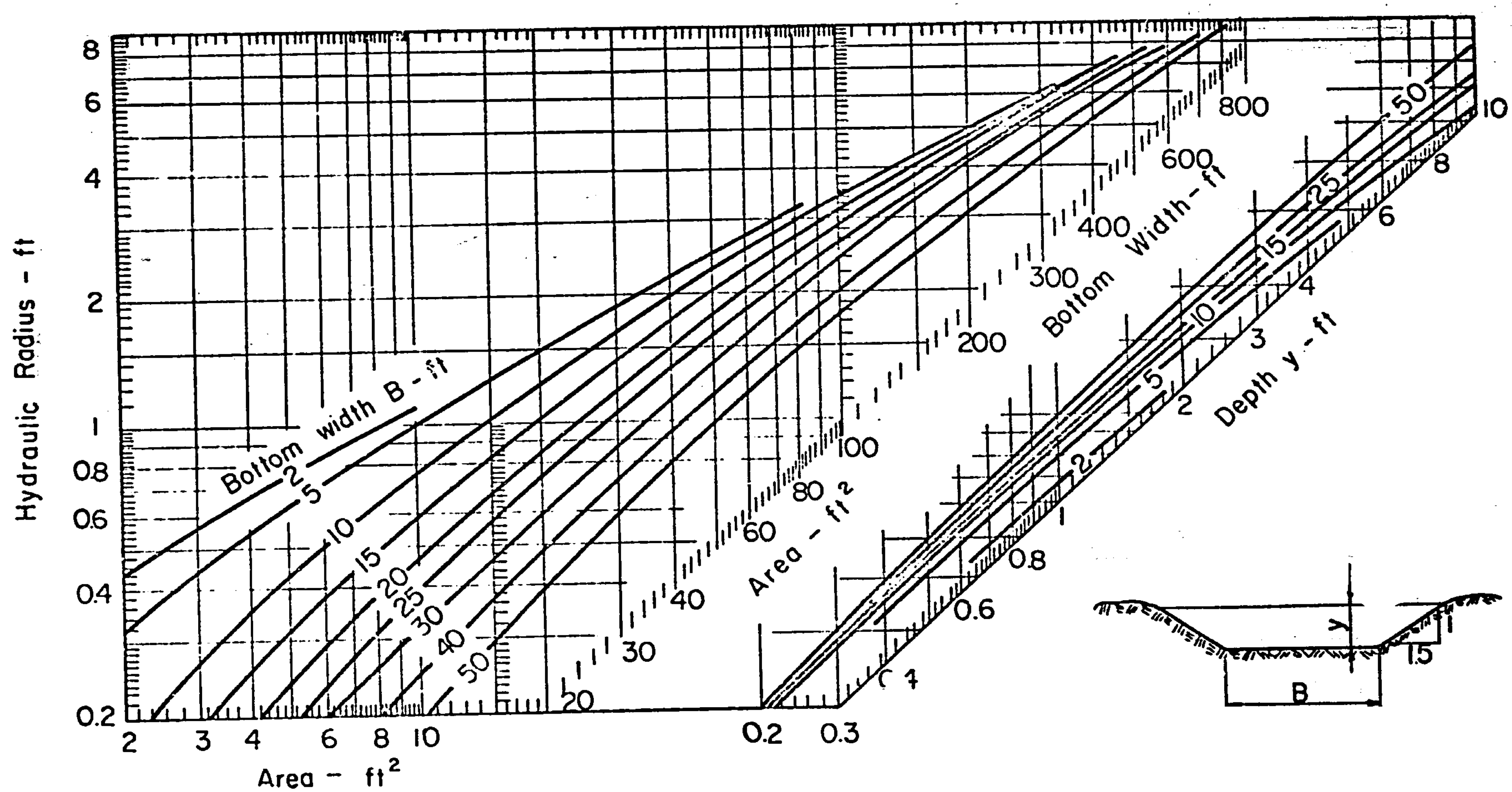


Figure 8-14 Geometry of trapezoidal channels with 1.5:1 side slopes

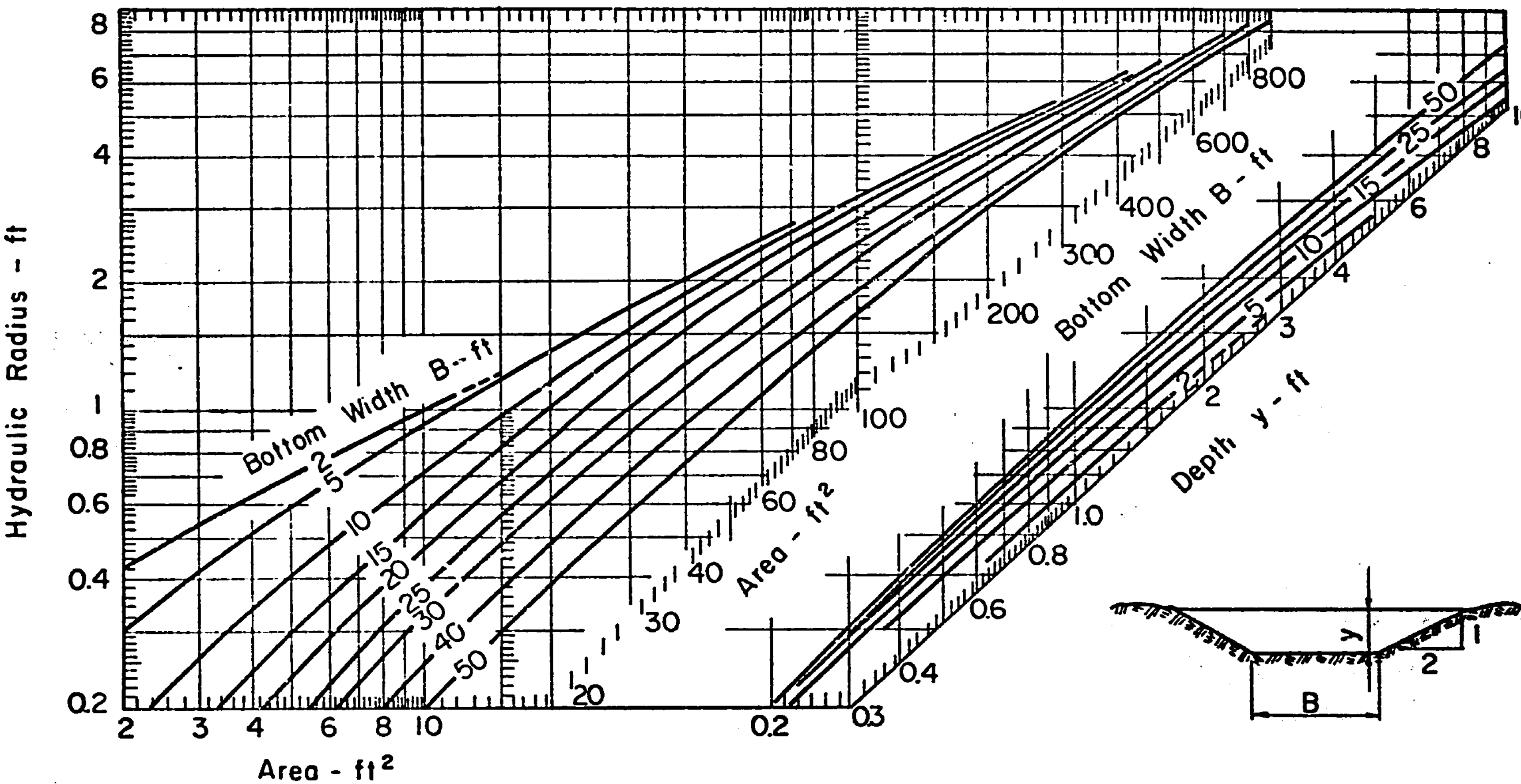


Figure 8-15 Geometry of trapezoidal channels with 2:1 side slopes

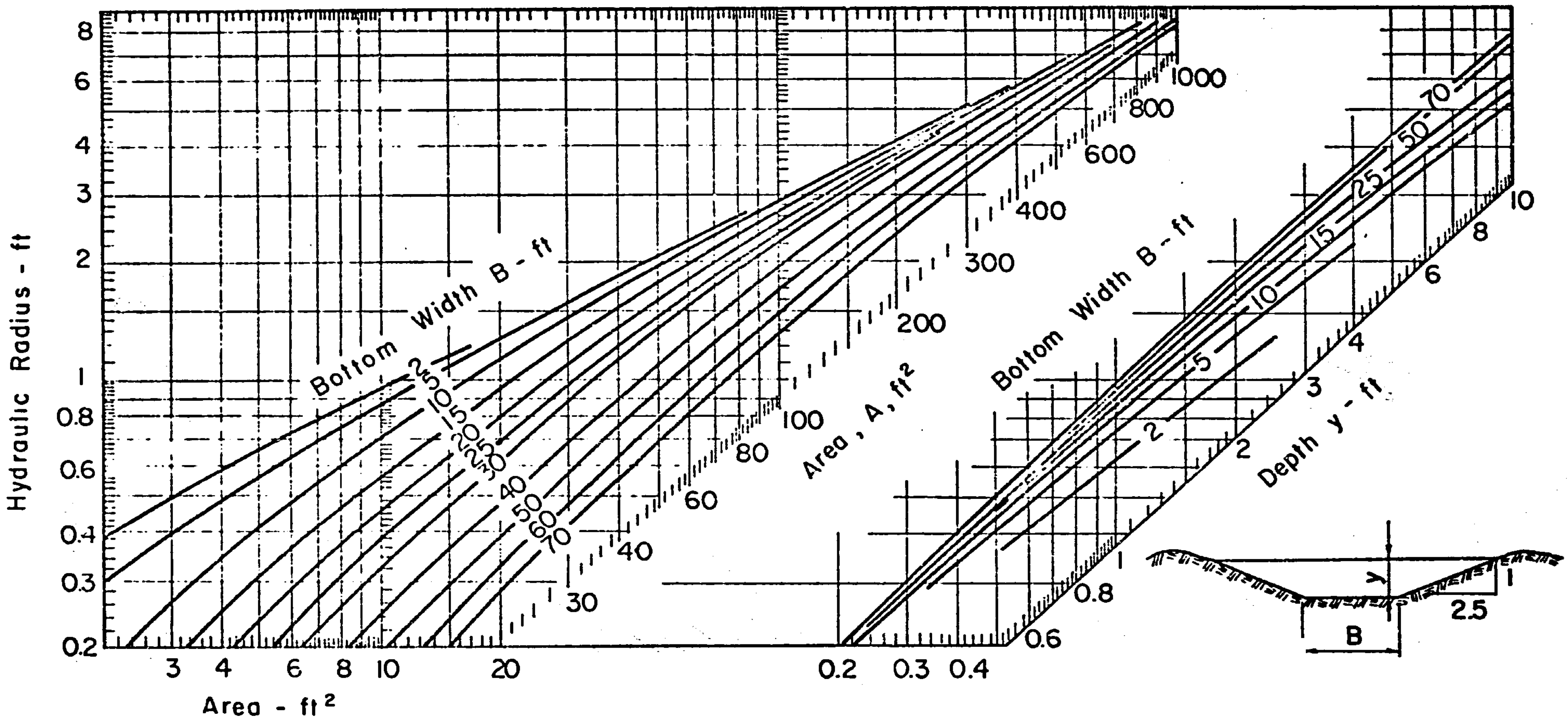


Figure 8-16 Geometry of trapezoidal channels with 2.5:1 side slopes

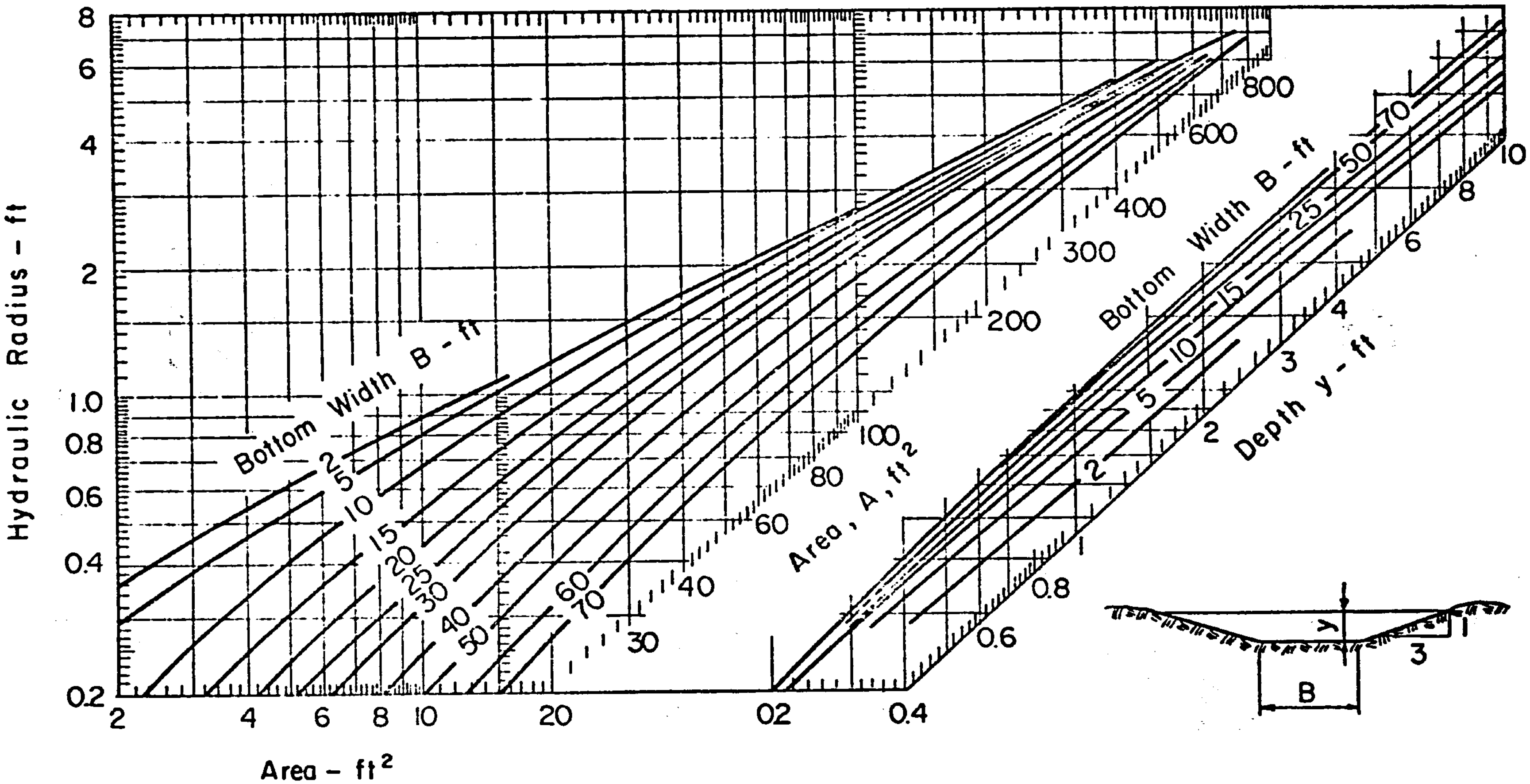


Figure 8-17 Geometry of trapezoidal channels with 3:1 side slopes

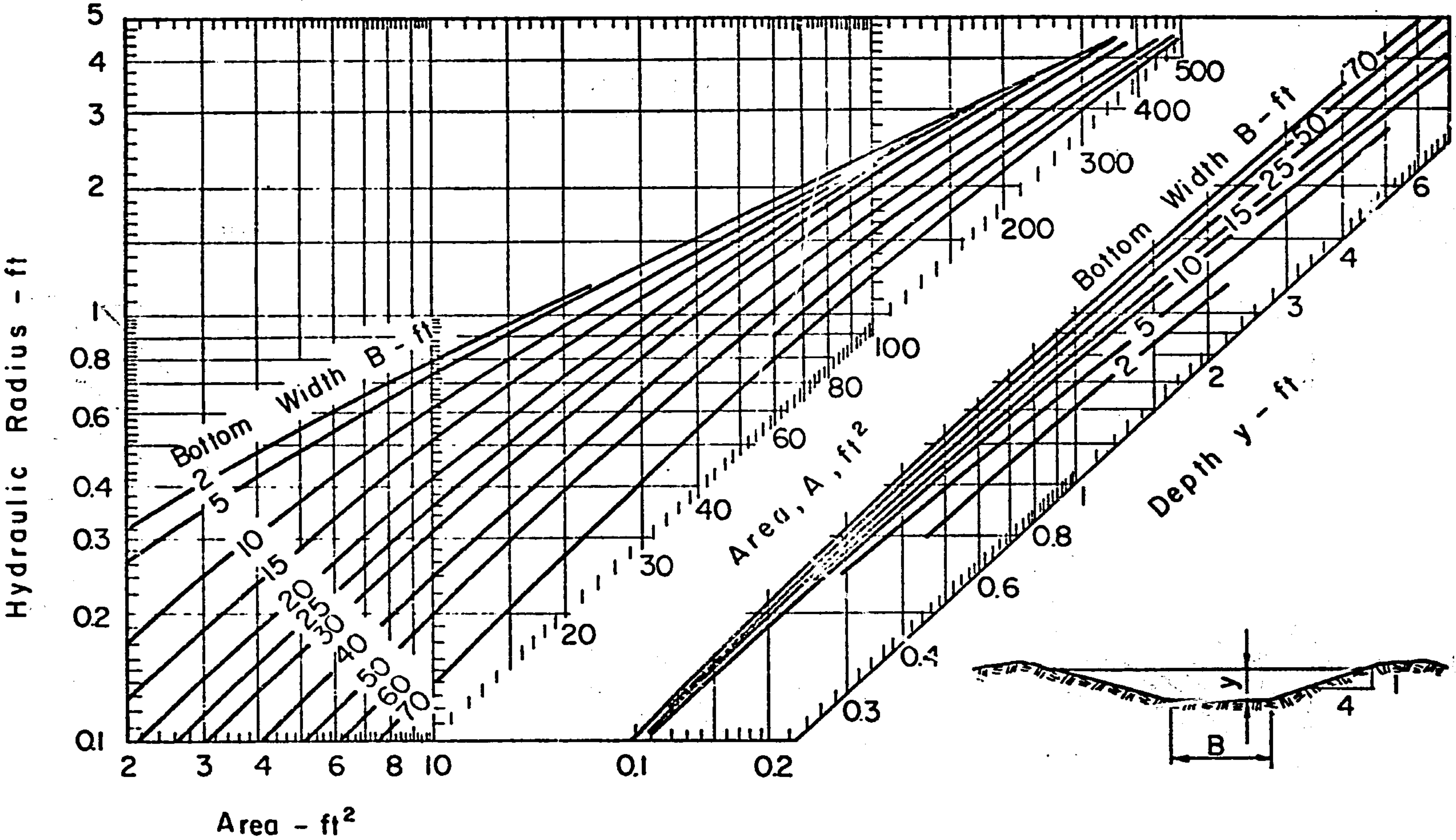


Figure 8-18 Geometry of trapezoidal channels with 4:1 side slopes

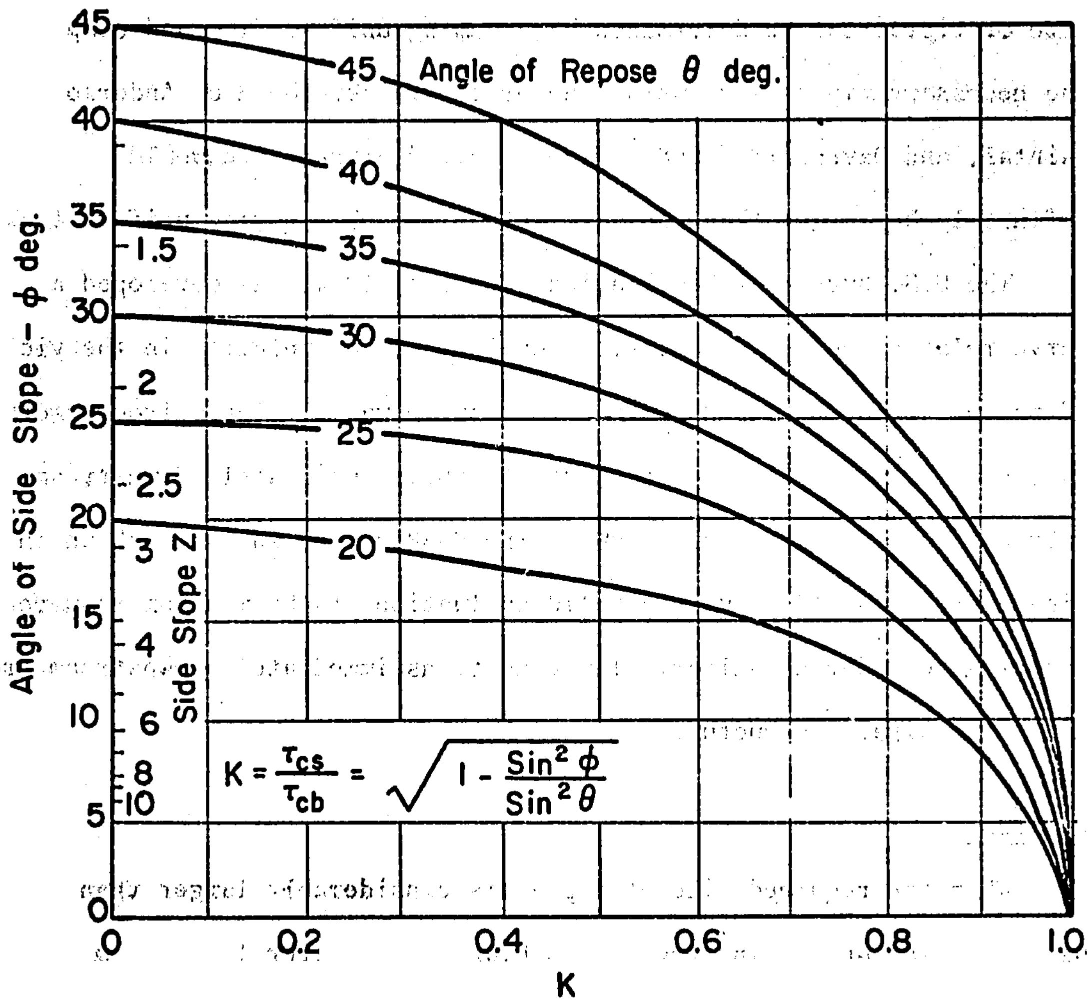


Figure 8-19 Ratio of critical shear on side slopes to critical shear on channel bed for noncohesive sediment

Nonuniform Flows

In designing an energy dissipator structure, one of the problems is to determine the riprap requirements immediately downstream from the structure when the outlet channel has a loose boundary. Usually, considerable judgement is necessary in making this evaluation. A minimum size of riprap can be determined by assuming uniform flow and computing the necessary riprap size using the procedure developed by Anderson, Paintal, and Davenport (1968). The required riprap size would definitely be larger than the size determined by assuming uniform flow.

The U.S. Bureau of Reclamation (Peterka, 1964) has developed a curve relating bottom velocity, which is the flow velocity in the vicinity of the particle, to riprap size. This curve was developed from numerous case studies of success and failure of riprap immediately downstream from energy dissipator structures. The USBR curve, which is shown in Figure 8-14, should provide a valid evaluation of the minimum required riprap size under nonuniform flow conditions immediately downstream from energy dissipator structures.

Filters

When the required size of riprap is considerably larger than the base material underlying the riprap blanket, a filter layer of material may be required between the base material and the riprap in order to prevent leaching. Leaching is the process by which the finer material underlying the riprap is picked up and carried away by turbulent eddies, waves, jets, and surges that penetrate the riprap blanket through the interstices of the rock particles. Leaching can be minimized if the riprap blanket is thick enough, the interstices are closed or reduced in size, a protective layer of intermediate sized material is interposed between the base material and the riprap, or the base material is sufficiently cohesive to prevent unraveling and erosion of the individual particles.

The criteria which are frequently used in determining whether or not a filter layer is required can be defined as

$$\frac{D_{15}\text{ Riprap}}{D_{85}\text{ Base}} < 5 \tag{8-23}$$

$$5 < \frac{D_{15}\text{ Riprap}}{D_{15}\text{ Base}} < 40 \tag{8-24}$$

$$\frac{D_{50}\text{ Riprap}}{D_{50}\text{ Base}} < 40 \tag{8-25}$$

in which D_{15}, D_{50}, and D_{85} are the sizes of riprap and base material of which 15, 50, and 85 percent are finer. If these criteria (Equations 8-23, 8-24, and 8-25) are not met, then a filter layer is necessary. In fact, the possibility exists that more than one filter layer may be required, the above criteria must be met by each successive filter layer.

If, for example, a filter layer is required, then the criteria given by Equations 8-23, 8-24, and 8-25 must be modified according to the following equations:

$$\frac{D_{15}\text{ Riprap}}{D_{85}\text{ Filter}} < 5 \tag{8-26}$$

$$< \frac{D_{15}\text{ Riprap}}{D_{15}\text{ Filter}} < 40 \tag{8-27}$$

$$\frac{D_{50}\text{ Riprap}}{D_{50}\text{ Filter}} < 40 \tag{8-28}$$

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Base}} < 5 \tag{8-29}$$

$$5 < \frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Base}} < 40 \tag{8-30}$$

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Base}} < 40 \tag{8-31}$$

If only one filter layer is required, then the use of Equations 8-26 through 8-31 will yield a range of particle size distributions that will be satisfactory for the filter material.

Recommendations regarding the thickness of the riprap blanket or filter layer vary (Anderson, Paintal, and Davenport, 1968). The range is about 1.3 to 2 times the D_{50} size of the material. Other reccommendations state that the thickness of the material should be equal to the maximum particle size or 1.5 times the maximum particle size.

A natural, built-in series of filter layers is graded riprap which varies in size from the maximum required by the foregoing criteria down to the maximum size in significant quantity in the base material. All intermediate sizes also need to be present so that the riprap is well graded and maximum density. Such material is usually available at the lowest price as pit-run sand, gravel, cobbles, and boulders pit, or as crusher-run material from a rock crusher plant.

The importance of graded riprap is shown in Figure 8-20 from Hallmark and Smith (1965) in which the depth of scour base material (maximum size 1/4 inch) is reduced to some extent by 1-inch to 2-inch riprap, reduced even more by 1/4-inch to 1/2-inch riprap, and reduced the most by the graded riprap ranging in size from 1/4-inch to 2- inches. This also shows it is more important to have the sizes of riprap (1/4-1/2)

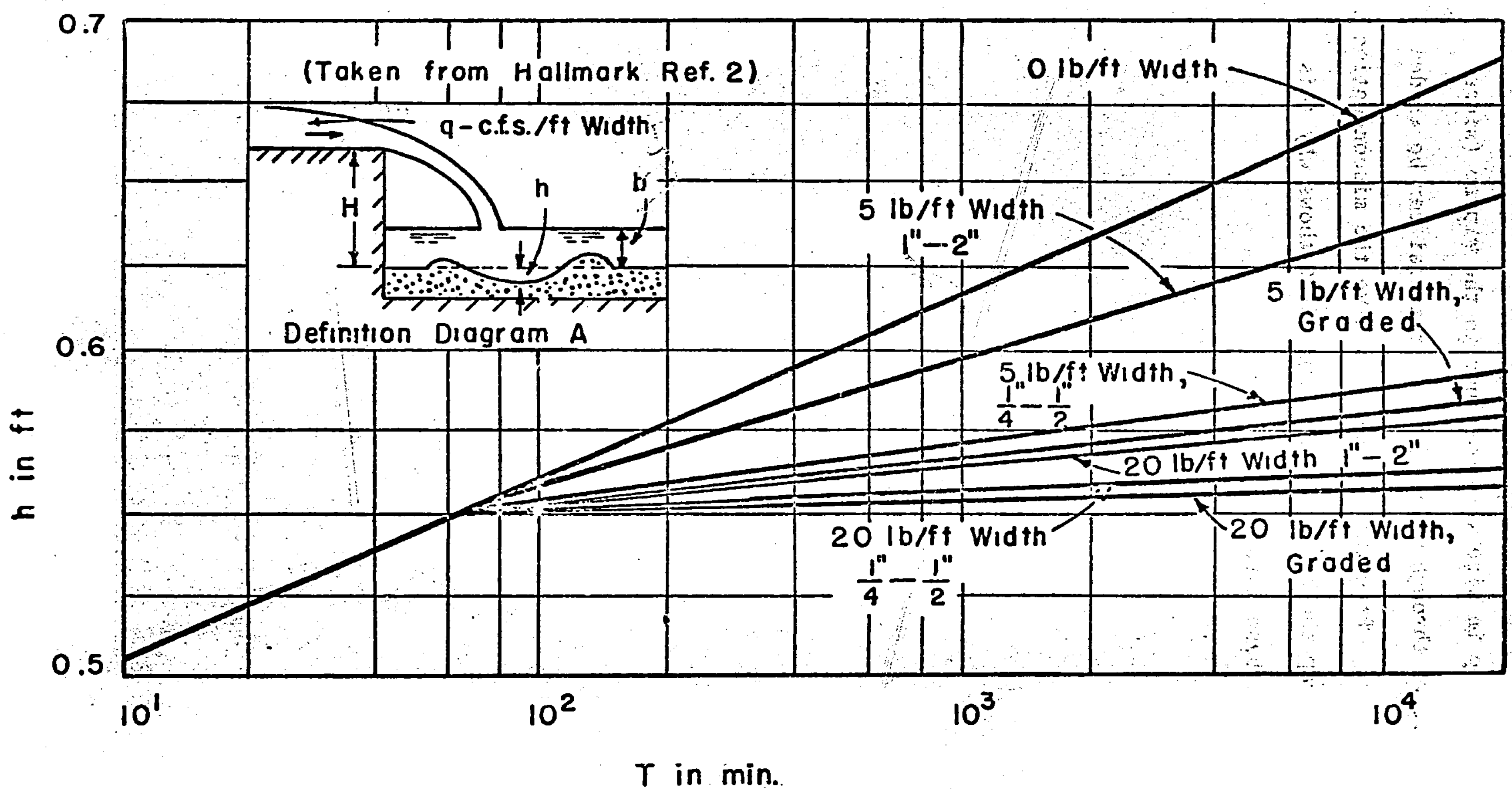


Figure 8-20. Variation in Rate of Scour With Size, Gradation, and Quanity of Armorplate for Free Overfall Flow Conditions

immediately larger than the base material than the much larger (1-2) material which leaves a gap in size with openings in the riprap (since it is not maximum density) through which the base material can be eroded by jets, waves, and surges. Frequently, designers have the misconceptior that "the larger the riprap the better," but Figure 8-20 shows why this is not true with alluvial base material.

Chapter 9

WAVE SUPPRESSORS

In some cases, a problem may develop in the outlet channel from an energy dissipator structure due to wave action. The effect of the wave action may be detrimental because of bank scour or periodic overtopping of the banks or because a measuring device may be downstream. Usually, this problem is not anticipated. Thus, corrective measures are required after construction is completed which might consist of placing riprap along the banks, raising the height of the banks, or constructing a structure for suppressing wave action. A wave suppressor structure may also be desirable in order to improve the approach flow conditions if a flow measurement station or flow measuring device is located a short distance downstream -- thereby improving the accuracy of the flow measurement.

A raft-type wave suppressor is shown in Figure 9-1. This structure consists of two or more perforated slabs held rigidly in place. In the design of a raft-type wave suppressor, the following considerations must be taken into account:

1. The rafts should be perforated in a regular pattern.
2. The rafts must be thick enough to prevent the wave troughs from breaking free from the underside.
3. At least two rafts should be used, separated by at least 3 times the raft width.
4. The rafts should be rigid and held stationary. Where it is desirable to suppress waves at less than maximum discharge, some means of adjustment should be made.

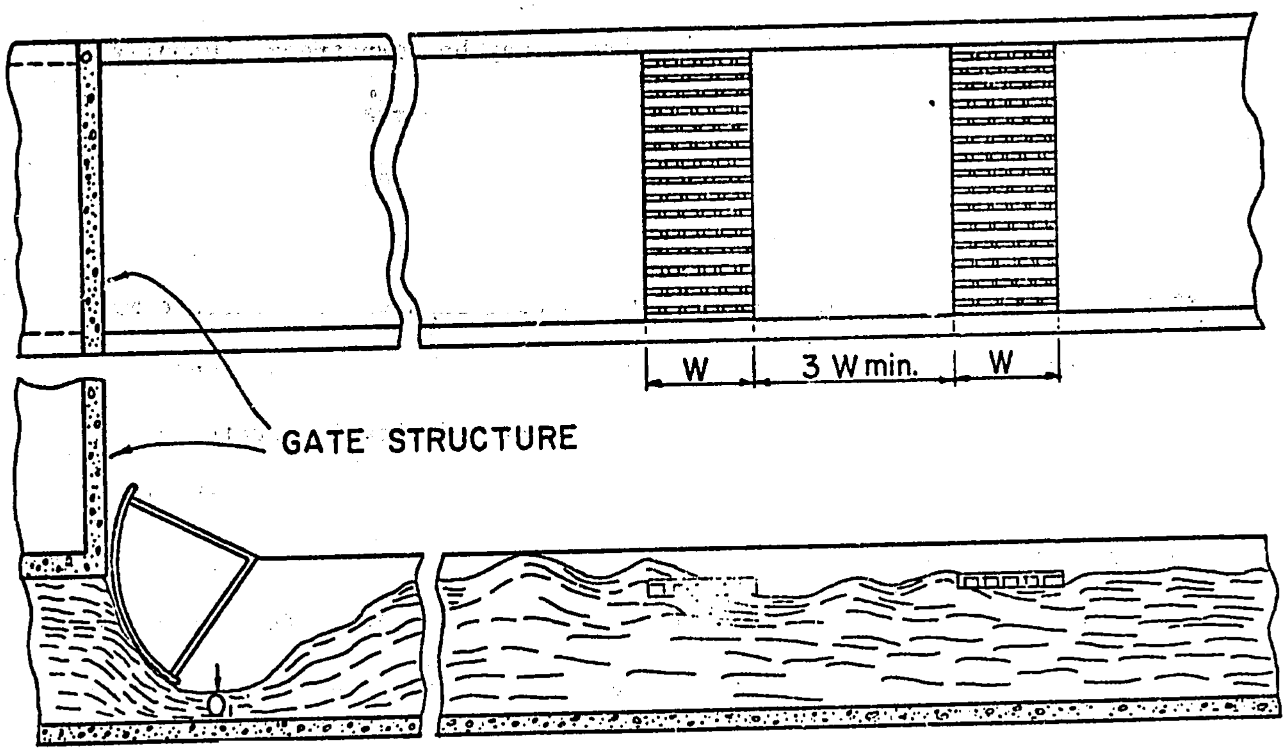


Figure 9-1 Schematic drawing of a raft-type wave suppressor

5. The ratio of the hole area to the total area of the raft may be from 1:6 to 1:8.

6. The top surface of the raft is usually placed at the mean water surface elevation.

The underpass-type wave suppressor shown in Figure 9-2 is effective for wave suppression. Essentially, it consists of a horizontal roof placed in the channel with a head wall high enough to cause all flow to pass beneath the roof. General design criteria include:

1. The roof will produce the greatest wave reduction when the underside is submerged (h''/y_2) to 33 percent of the maximum flow depth. The greatest wave reduction is for waves of short wave length.

2. The length of the underpass should be $1.0\ y_2$ to $1.5\ y_2$ for 60 to 75 percent wave height reduction, $2\ y_2$ to $2.5\ y_2$ for values up to 88 percent, and for values up to 93 percent reduction in wave height the length should be from $3.5\ y_2$ to $4\ y_2$, where y_2 is the downstream water elevation. Included in the length for the $3.5\ y_2$ to $4\ y_2$ suppressors is a 4:1 sloping roof extending from the underpass roof elevation to the tailwater surface which acts as a draft tube.

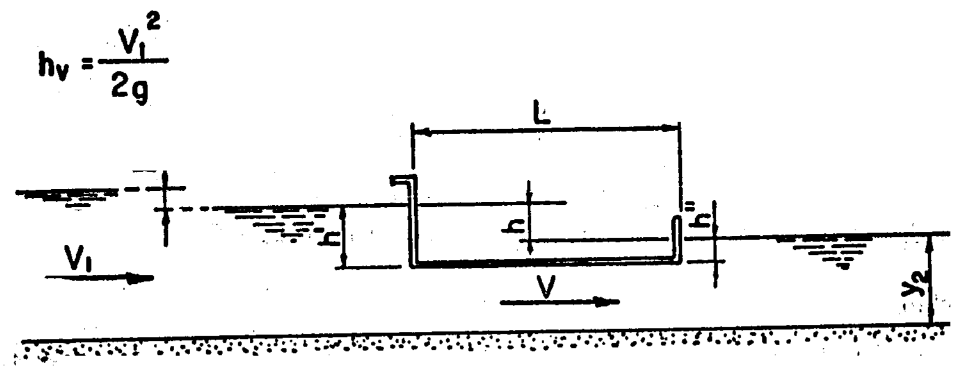
3. The backwater effect of the underpass is shown in Figure 9-2 for the range of lengths. The basic flow equation for the structure is,

$$Q = CA\sqrt{2g\ (h + h_v)} \qquad (9\text{-}1)$$

in which

Q is the discharge,

C is the constant determined from Figure 9-2,



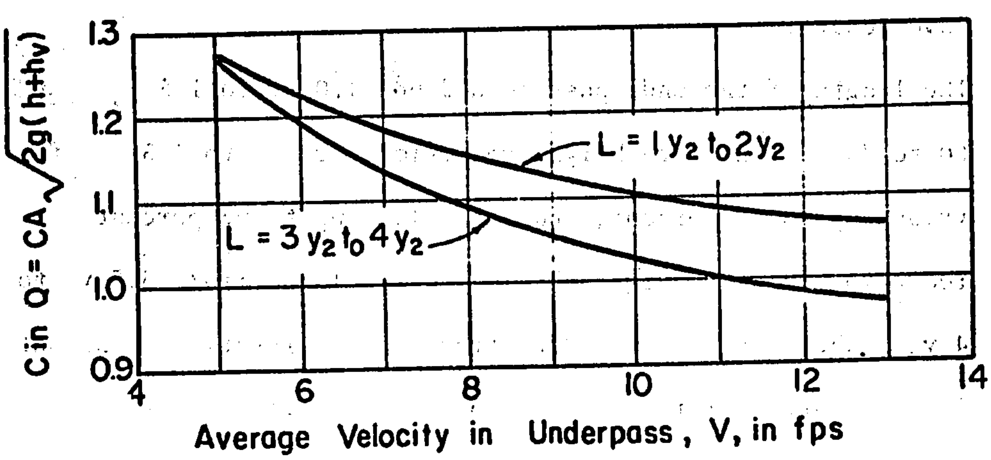


Figure 9-2 Schematic drawing of an underpass-type wave suppressor

A is the cross-sectional flow area in the underpass,

g is the acceleration of gravity,

h is the head through the constriction, and

h_v is the upstream velocity head.

Chapter 10

SUMMARY

The three structural components of combination check-drop-energy dissipator structures are: (1) the inlet or appraoch section; (2) the stilling basin; and (3) the outlet channel. The hydraulic design of each component has been discussed in this report.

At the inlet or approach section, two different structural conditions can occur, with consequent effects on flow conditions. For the situation in which the geometry of the approach section is constant with the flow dropping off the end of the approach section, or entering a chute having a slope greater than critical slope, the flow conditions at the end of the approach section are known.

If the flow entering the stilling basin passes over a check structure (weir or orifice), then the flow characteristics of the free jet are known if the overflow crest is sharp-crested (thin-plate weir) or has a curvilinear shape (ogee crest) conforming to the lower nappe of a free jet passing over a sharp-crested weir or the control gate has a sharp edge and large upstream section. The overflow crest of most check structures are neither sharp-crested (edged) nor do they have an ogee crest. Thus, there is a problem in predicting the flow characteristics of the free jet passing over or through most check structures.

In many cases, check structures are also used for flow regulation in that a portion of the discharge is allowed to flow over the check flashboards. Consequently, it becomes desirable to determine the relationship between depth and discharge for the check flashboards to allow accurate stream regulation, as well as evaluating the free jet characteristics of the overflow in order to properly design the stilling basin located immediately downstream. Also, the corners of flashboards become rounded with time, which affects the flow characteristics of the overflow. A laboratory experimental design to collect the necessary information would have to include such variables as thickness of flashboards, rounding of flashboard edges, height of overflow crest above the approach channel floor, geometry of check structure, and geometry of approach channel.

Numerous investigators have reported specific design information for particular geometries of energy dissipator structures. The variety in types of energy dissipator structures does allow the designer considerable latitude in meeting most field situations. Of particular importance is the more general work of Moore (1943) and White (1943), which provides sufficient information for designing small rigid-boundary stilling basins.

A considerable amount of research has been accomplished regarding appurtenances in stilling basins. For vertical drop structures, a systematic study would appear to be in order to evaluate the effectiveness of end sills, floor blocks, and dissipation bars. Such an evaluation would have to cover a wide range of unit discharge and height of drop to be of any value in systematizing the hydraulic design of stilling basins, including appurtenances, for vertical drop structures.

The design of loose-boundary, or riprap, stilling basins below vertical drop structures has been reported by Smith and Strang (1967). Their studies should yield good results for small drop heights. Because the process of scour and sediment transportation is so complicated, there is a real need to duplicate the general experimental design of

Smith and Strang, but using much larger drop heights, discharge, and particle sizes.

Determining the required size of riprap to be placed in a loose-boundary outlet channel immediately downstream from a stilling basin is very difficult. First of all, our present criterion for sizing riprap is a function of mean velocity or shearing stress, neither of which take into account the effects of turbulence and eddy size upon the movement of particles. The flow immediately downstream from a stilling basin may be highly turbulent. Thus, using the mean velocity in the outlet channel to arrive at a required size of riprap would yield a diameter much too small. At the present time, the USBR curve is probably the best criterion for this situation, since it is based on numerous case studies of riprap failures and successes below energy dissipator structures. Better design information is needed for riprap sizing near stilling basins. The simplest type of information would be relations between mean velocity and bottom velocity (velocity near the top of the particle) for movement of various sizes of riprap placed downstream from various types of energy dissipator structures. Another unknown which should be incorporated in this experimental design is the variation of bottom velocity with distance downstream in order to determine the channel length for which riprap is required.

APPENDIX A

ANNOTATED BIBLIOGRAPHY

CHECK-DROP-ENERGY DISSIPATOR STRUCTURES IN IRRIGATION SYSTEMS

ANNOTATED BIBLIOGRAPHY

Ackermann, N. L. and Undan, R. 1970. Forces from submerged jets. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 96, No. HY11, Paper No. 7675, Nov., pp. 2231-2240.

The force on rigid surface produced by an impinging jet is a function of the physical characteristics of the fluid in which the jet is submerged. In the present experimental investigation, flow conditions are considered where a submerged circular jet impinges upon a disk whose diameter and distance from the origin of the jet are variable. For such conditions, the force produced by the impinging jet is found to depend upon the nozzle and disk diameters, and the momentum flux of the disk in the incident flow field. A stagnation pressure model was developed relating these variables when the disk was smaller than the local diameter of the spreading jet.

Advani, R. M. 1967. Energy dissipation through hydraulic jump. 12th Congress, IAHR, Fort Collins, Colorado, Vol. 3, C29, pp. 249-251.

The subject of energy dissipation below spillways and other hydraulic structures is of vital importance to irrigation and hydraulic engineers particularly in view of the increased tempo of construction of such structures these days all over the world. There are several devices through which the desired energy dissipation may be accomplished, but the most effective one seems to be through the formation of hydraulic jump below such structures. The usual practice is to have a channel of rectangular shape for the hydraulic jump device, perhaps for ease of computation. By expressing the basic equations in terms of suitable nondimensional parameters as outlined in this paper, the computation work is greatly simplified.

Albertson, M. L., Dai, Y. B., Jensen, R. A. and Rouse, H. 1950. Diffusion of submerged jets. Transactions of the ASCE, Vol. 115, Paper No. 2409, pp. 1571 - 1594.

As the direct result of turbulence generated at the borders of a submerged jet, the fluid within the jet will undergo both lateral diffusion and deceleration, and at the same time fluid from the surrounding region will be brought into motion. The approximate characteristics of the corresponding mean flow pattern are derived analytically, with the exception of a single experimental constant, through assumptions that : (1) the pressure is hydrostatically distributed throughout the flow; (2) the diffusion process is dynamically similar under all conditions; and (3) the longitudinal component of velocity within the diffusion region varies according to the normal probability function at each cross section. Experimental data are presented which justify the analysis and provide the necessary coefficients for flow from both slots and orifices. All results are reduced to a form immediately useful for design purposes.

Anderson, A. G., Paintal, A. S. and Davenport, J. T. 1968. Tentative design procedure for riprap lined channels. St. Anthony Falls Hydraulic Laboratory, Project Report No. 96, Project No. HR 15-2, Minneapolis, Minnesota, June, pp. 1-67.

This report describes the interrelationships and develops design criteria by which a riprap lined drainage channel can be proportioned and the riprap lining specified for a given discharge and longitudinal slope. These relationships so developed have been reduced to design charts, the use of which permits rapid and simple establishment of channel shape and size as well as the properties of the riprap lining. Limited experimental data are presented which serve to verify the design procedure, to test the efficacy of channels designed according to this procedure, and to examine somewhat more closely the phenomenon of leaching of base material through the riprap interstices. These experiments, while preliminary in character, indicate that the design procedures are suitable and incorporate sufficiently large factors of safety to provide stable channels.

Andersson, S. 1964. Stability of armour layer of uniform stones in running water. Swedish Geotechnical Institute, Reprints and Preliminary Reports No. 6, pp. 21-25; Stockholm.

Equations describing the beginning of movement for coarse granular particles are summarized. These equations have been

developed by numerous investigators and are based on hypotheses of shearing stress or velocity.

Anonymous. 1936. The maximum permissible mean velocity in open channels. Gedrotekhnecheskoe Stroitelstov.

For particle diameters up to 200 mm, the maximum mean velocity allowable in an open channel without moving the particles is given. Adjustment factors for the allowable mean velocity are given which take into account the effects of flow depth.

Argyropoulos, P. A., Advani, R. M., Leutheusser, H. J., Campbell, F. B., and Smith P. M. 1964. End depth for circular channels. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 90, No. HY3, Paper No. 4050, Sept. pp. 261-283.

The recent supplementary theoretical attempts and experimental tests examined by the authors concerning the solution of the end depth problem in open channels are reported and the practical significance of brink depth is explained.

Austin, L. H., Skogerboe, G. V., and Bennett, R. S. 1970. Subcritical flow at open channel structures: open channel expansions. OWRR Project No. B-018-Utah, Utah Center for Water Resources Research, Utah State University, Logan, Utah - 84321. Aug., pp. 1-71.

The intent of the writers is to develop a basis for the design of open channel expansions utilizing the techniques previously developed for flow measuring flumes and weirs. This method for analysis would employ the use of specific energy rather than flow depth, but this substitution has been found valid through previous analysis of data from other research.

Bakmeteff, B. A., and Matzke, A. E. 1938. The hydraulic jump in sloped channels. Transactions of the ASME, Feb. pp. 111-118.

The discussion of a previous paper on the hydraulic jump in sloped channels made it evident that the existing literature carried no clear exposition of the subject, and moreover that no

systematic experimental evidence was available which would permit an engineer to dare practical conclusions. This paper is an attempt to clarify this situation.

Bilashevsky, N. N. 1965. The mechanism of the local scour behind the spillway structures with aprons and the influence of the macroturbulence upon the scour. 11th Congress of the International Association of Hydraulic Research, Leningrad. pp. C30.1-C 30.6.

The authors wide range experimental study on the velocity pattern and on the bottom scour holes behind spillway structures shows that the scour ability of a flow depends on the magnitude of the maximum actual near-bottom velocities.

Blaisdell, F. W. 1948. Development and hydraulic design, St. Anthony Falls stilling basin. Transactions of the ASCE, Vol. 113, Paper No. 2342, pp. 483-561.

Tests made to develop rules for the design of a stilling basin to dissipate hydrodynamic energy are described in this paper. The energy dissipator has been named the SAF stilling basin, SAF (denoting St. Anthony Falls) being a coined word used to differentiate this design from other stilling basin designs. The objective in presenting this paper is to present data in sufficient detail to permit an independent analysis by the reader so that he can arrive at his own conclusions. The results are summarized for convenience of reference so that, after evaluating the design, the reader can obtain all the equations and information essential to the hydraulic design of the SAF stilling basin without "thumbing" through the entire paper.

Blaisdell, F. W., Donelly, C. A., and Yalamanchili, K. 1969. Abrupt transition from a circular pipe to a rectangular open channel. St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Technical Paper No. 53, Series B, July.

The development of criteria and a generalized procedure for the design of an abrupt transition from a circular pipe to a rectangular open channel are presented. The equations developed describe the locations of the water surface elements to within an average of 0.11 pipe diameters of their correct locations.

The maximum anticipated location error is ± 1.4 pipe diameters. The equations for the envelope curves covering the crests of the sidewall waves, which determine the channel sidewall height, provide an average freeboard of 0.08 pipe diameters and a maximum freeboard of 0.31 pipe diameters. When the envelope equations are used only 2% of the wall waves will overtop the sidewalls, the maximum overtopping being 0.04 pipe diameters.

Blaisdell, F. W. 1954. Equation of the free falling nappe. Journal of the Hydraulics Division, ASCE, Separate No. 482, Vol. 80, Aug., pp. 1-46.

A general equation for the form of the nappe is developed in this paper. The equation is not valid close to the crest, but applies to that portion of the nappe that is free-falling, where pressures within the nappe are atmospheric. The constants in the equation have been evaluated for the vertical sharp-crested weir having approach channel-depths ranging from deep to zero (the free overfall); that is, over the entire range of subcritical approach velocities. The evaluation is made using nappe coordinates obtained by others. The equation is checked by comparing its predictions with a number of published profiles. The comparison is shown to be excellent for the lower approach velocities and good at the higher approach velocities.

Butcher, A. D., and Atkinson, J. D. 1932. The cause and prevention of bed erosion with special reference to the protection of structures controlling rivers and canals. Minutes of the Proceedings of the Institution of Civil Engineers, London, Vol. 235, pp. 175-193.

The experiments described in this paper were started in connection with certain problems of erosion which had arisen at the Sennar dam controlling the Blue Nile, about 200 miles South of Khartoum. The continuance of the erosion rendered necessary the provision of some form of protection and a start was made by designing horizontal aprons for the first twenty sluices which were screened by an island from the main river channel.

Chang, F. M. and Karim, M. 1970. Erosion protection for the outlet of small and medium culverts. South Dakota University, Brookings, South Dakota, Report PB190565, Feb., pp. 1-52.

This is a pilot study to investigate and evaluate the feasibility of an erosion control work for the outlet of small and medium culverts. The proposed control work consists of a recessed stilling basin armored with gravel and a tranverse impact wall. The primary objectives of this investigation were to find the dimensions of the stilling basin and a proper location of the impact wall for the design flow discharge for two tailwater condition simulating a discharge into a receiving channel. Two things were considered important in regard to the satisfactory functioning of this control work: (1) stabilization of the stilling basin, and (2) minimization of the scour below the impact wall.

Chitale, S. S. 1959. Energy dissipation in hydraulic jump below weirs and falls. Irrigation and Power, Vol. 16, No. 4, Oct. pp. 465-477.

Both the Indian as well as American practices of design of hydraulic jump type horizontal cisterns for energy dissipation below weirs and barrages are examined. The length of the cistern according to Indian practice is based on the height of jump while American designs correlate it with downstream depth of flow. Using experimental data, a relation between length and height of jump is first established and on this basis the length of cistern according to the Indian practice in terms of downstream depth is obtained.

Curtis, D. D., Martinez, J. E., and Vasquez, V. 1956. Coefficient of contraction for a submerged jet. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 81, No. HY4, Paper No. 1038, August, pp. 17-19.

In computing the rate of discharge for a submerged orifice, it is customary to assume that the coefficient of jet contraction is the same as it would be if the jet were free. This experiment is undertaken to compare both the geometric and the kinematic characteristics of a free and a submerged jet issuing from the same orifice under the same differential head.

Diskin, M. H. 1961. Hydraulic jump in trapezoidal channels. Water Power, January., pp. 12-17.

Reports of experiments on the hydraulic jump in non-rectangular channels indicate that the conjugate depths of the jump, as measured, agree quite closely with theoretical values derived by the momentum equation. The computation of the theoretical conjugate depths, however, is a tedious process which involves a graphical solution or successive approximations. The method presented in this article is based on an equation similar to that proposed by Elevatorski but derived in a different way.

Diskin, M. H. 1961. End depth at a drop in trapezoidal channels. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 86, No. HY4, Paper 2851, July, pp. 11-32.

The momentum equation is used to derive a general equation for the end depth in prismatic channels in mild slopes at an abrupt drop. The equation is solved directly for exponential channels and in a tabular form for trapezoidal channels. Experiments on two trapezoidal channels are reported and the results are compared with the theoretical values of end depth derived by use of the momentum equation.

Doddiah, D. D. 1949. Comparison of scour caused by hollow and solid jets of water. M.S. Thesis, Colorado Agricultural and Mechanical College, Fort Collins, Colorado, Dec.

The problem for which this thesis seeks to furnish an answer may be stated as follows: What is the scouring capacity of a hollow jet of water and how does it compare with that of a solid jet? An attempt to further analyze this problem in a search for its solution will be much facilitated by a review of the work that other investigators have already done in this field.

Doddiah, D. D., Albertson, M. L. and Thomas, R. A. 1953. Scour from jets. Proceedings, Minnesota, International Hydraulics Convention, Minneapolis, Minnesota, September, pp. 161-169.

Scour from jets of water which might be found under natural conditions was given rather detailed treatment by Schoklitsh in 1935. In this work, Schoklitsch describes a variety of conditions under which scour might occur and gives data for design of certain structures involving scour. Doddiah, in 1949, made a study of scour resulting from circular jets issuing vertically downward onto a bed of alluvial material covered by a pool of water having various depths. More recently, Thomas has studied the scour resulting from a two-dimensional jet or sheet of water issuing from a free overfall and impinging on an alluvial bed also covered by a pool of water having various depths. This paper reports the studies made by Doddiah and Thomas and compares the study of Thomas with the equation of Schoklitsch.

Donnelly, C. A., and Blaisdell, F. W. 1965. Straight drop spillway stilling basin. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 91, No. HY3, Paper No. 4328, May, pp. 101-131.

The development of generalized design rules for a straight drop spillway stilling basin is described in this paper. This stilling basin was developed because experience had shown that there was no satisfactory stilling basin for the straight drop spillway. The stilling basin can be used for a wide range of discharge, head on the crest, crest length, height of drop, and downstream tailwater level. A method of computing the stilling basin length for all tailwater levels is presented. The design rules developed as a result of the laboratory tests were carefully checked and verified. An example shows how these rules can be applied to the design of a field structure.

Einstein, H. A., and E. A. El Samni, 1949. Hydrodynamic forces on a rough wall. Review of Modern Physics, Vol. 21, pp. 520-524.

The dynamic forces which a turbulent flow exerts on the individual protrusions of a rough wall have been measured. it was found that even in the case of an extremely high relative roughness, the drag force on the protrusions may be determined from the logarithmic friction laws. The lift force, which was measured, was divided into a constant average value and a random fluctuation superimposed over the average. Statistical analysis showed that the frequency of lift forces followed the normal error law. This fact seems to indicate that in the description of turbulence near a rough wall, the pressures must be regarded as the primary influence, not the velocities.

Elevatorski, E. A. 1959. Hydraulic energy dissipators. Engineering Societies Monographs, McGraw-Hill Book Company, Inc., New York.

This book draws upon many sources of material in presenting experimental as well as design data to serve both research and design engineers. A partial list of references to such literature is given at the end of each chapter.

Fiala, G. R., and Albertson, M. L. 1961. Manifold stilling basin. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 86, No. HY4, Paper 2863, July, pp. 55-81.

This paper presents the manifold stilling basin as a device for dissipating excess kinetic energy. This device has certain important advantages for some conditions. Two successful field installations of the manifold stilling basin have already been made. Both installations were made at the outlet ends of the pipe drops in canals in which the vertical drop of the water surface was several tens of feet.

Flammer, G. H., Skogerboe, G. V., Wei, C., and Rasheed, H. 1970. Closed conduit to open channel stilling basin. Proceedings of the American Society of Civil Engineers, Journal of Irrigation and Drainage, Vol. 96, No. IR1, Paper No. 7124, March, pp. 1-11.

Criterion have been developed for designing a stilling basin to serve as a transition from closed conduit flow to open channel flow for a fully submerged pipe outlet. The unique feature of the stilling basin is the short-pipe energy dissipator for the basin configuration. The expanding characteristics of a submerged jet were used in establishing the length of the stilling basin. The unsteadiness of the water surface and the relative boil height in the model basin were used as the criteria for evaluating the effectiveness of the structure for energy dissipation. Relations between the tailwater depth, the outlet flume floor elevation, the height of boil in the stilling basin, the width of the stilling basin, and the amount of freeboard have been studied. The interrelationship among these variables have been shown graphically.

Forster, J. W. and Skrinde, R. A. 1950. Control of the hydraulic jump by sills. Transactions, ASCE, Vol. 115, pp. 973-1022.

The object of the study described herein was to investigate the performance of such sills in level rectangular channels, and to attempt to formulate a general method of design. The experimental results are shown to compare favorably with a theoretical analysis and data of general application are presented dimensionlessly. Two types of sills were investigated. First, a vertical nonaerated weir with a sharp crest and second, an abrupt rise in the channel bottom. Charts have been prepared which may be used to determine sill dimensions for preliminary design purposes and as a guide in analysing or predicting the effect of such sills under given conditions.

Foss, W. L. 1959. Experiences with canal drops of various designs. Proceedings of the 8th General Meeting of IAHR, Vol., 21, August, pp. 1-24.

The development of the design of special drop structures for canals on the St. Mary Irrigation Project in Southern Alberta is described. This included large scale model experiments carried out in the open, beside a fall on an irrigation canal. Equations for the hydraulic design of structures of any size and capacity are proposed, the design being in terms of canal dimensions. Criteria are developed by which the performance of any structure can be compared with that of the model. The main principle involved in the design is the spreading of the stream within the structure back to the average canal width before and during the hydraulic jump.

Grimm, C. I., and Leupold, N. 1939. Hydraulic data pertaining to the design of rock revetment. U.S. Army, Corps of Engineers, North Pacific Division.

Discusses current revetment practices. Tabulates and plots available laboratory data of movement of solids by flowing water. Evaluates Bonneville, Passamaquoddy, Isbash, WES, Groat, and Hooker data. Includes observed prototype data on Columbia River, Zuider Zee, and in the Los Angeles District.

Gyorke, O. 1961. Energy dissipation in protected beds downstream of river barrages in the case of shallow stilling pools. 9th Congress IAHR, Dubrovnik, Yugoslavia, pp. 184-196.

Principles for the design of an effective energy dissipating system are summarized in four points. It can be seen from the typical flow-patterns, that the stilling pools of shallow depths and with high terminal sills are impracticable. Theoretical conclusions are supported by the results of experiments with tailwater apron designs according to an apron test of a river barrage to be built into a river-bed consisting of fine soil.

Hallmark, D. E. 1955. Influence of particle size gradation on scour at base of free overfall. M. S. Thesis, Colorado Agricultural and Mechanical College, Colorado, Aug.

The objective of this study is to obtain information that will ultimately lead to the use of gravel for amorplating stilling basins. The subject for immediate investigation is the effect of size gradation of bed material on scour at the base of a free overfall. This investigation has been limited to a maximum steady flow discharge of 0.5 cfs per linear foot of crest width. The maximum height of drop has been four feet. The sizes of the bed material have been limited to ¼ in. mean diameter and 1/32-in. mean diameter. Armorplating gravels have been tested on the 1/32-in. bed material only. A constant channel width has been used to maintain two-dimensional flow.

Hallmark, D. E. and Albertson, M. L. 1956. Recent developments in the design of a simple overfall structure. Proceedings of the Four States Irrigation Council, Fifth Annual Meeting, Denver, Colorado, January.

This paper describes the work carried out at Colorado A & M Coll. in an attempt to develop design criteria for a simple, economical and effective drop structure for canals and conveyance systems.

Hallmark, D. E., and Smith, G. L. 1965. Stability of channels by armorplating. Proceedings, ASCE, Journal of the Waterways and Harbors Division, Vol. 91, No. WW3, Paper 4452, Aug., pp. 117-135.

The results of experimental and theoretical investigations are presented for certain sediment characteristics found pertinent to the control of localized scour in alluvial channels. The relationship between fall velocity of the sediment particle, velocity at the beginning of sediment motion, tractive force, and bed shear velocity is developed in terms of the nominal particle diameter.

Hickox, G. H. 1944. Aeration of spillways. Transactions of the American Society of Civil Engineers, Vol. 109, Paper 2215, pp. 537-566.

A method of computing the size of air vent needed for aeration of spillways, based on all the available data, is presented in the paper. The effect of insufficient aeration on the reduction of pressure beneath the nappe and on the discharge is discussed. Formulas and diagrams for calculation of these effects are included.

Ingram, L. F., Ottman, R. E. and Tracy, H. J. 1956. Surface profiles at a submerged overfall. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 81, No. HY4, Paper No. 1038, August, pp. 12-16.

Depending upon the interrelationship of a number of variables, the free surface at a submerged overfall will assume one of several basically different forms. In addition to the coordinate position, the pertinent variables include the velocity and depth of the oncoming flow; the depth of the tailwater in the forebay; the channel slope, roughness, and cross-sectional dimension; and the dimensions of the forebay. For the purpose of this exploratory study, the channel was made smooth and horizontal and the drop as great as possible, changes in boundary alignment were limited to the vertical, and the surface configuration was observed in the crest vacinity only.

Ippen, A. T., and R. P. Verma. 1953. The motion of discrete particles along the bed of a turbulent stream. Proceedings, Minnesota International Hydraulics Convention, pp. 7-20.

The movement of glass and plastic spheres having diameters between 2mm and 4mm were observed on a flume bed roughened with sand particles. The bed roughness was described by an effective roughness computed from the Karman logarithmic laws. The entrainment function of Shields was checked and found untenable where the bed load and bed consist of different particle sizes. A new entrainment function was derived that was more generally valid for the range of experimental results.

Isbash, S. V. 1936. Construction of dams by depositing rock in running water. Transactions, Second Congress on Large Dams.

Summarizes experiments for construction of dams by depositing rock in running water. Author briefly reviews theory resulting from small-scale experiments and gives equations applicable to various stages of construction. Riprap composed of 15- to 500-lb rock. Large rock not moved. No basic data given in report. A tentatively recommended curve for riprap design is given.

Ishihara, T., Iwasa, Y., and Ihda, K. 1960. Basic studies on hydraulic performances of overflow spillways and diversion weirs. Disaster Prevention Research Institute, Kyoto University, Bulletin No. 33, March, pp. 1-29.

This paper describes the theoretical characteristics of round crested weirs which are one of the controlling devices for released discharge from a reservoir or a main stream and the verification of the theory by experimental research. The head-discharge relationship of a round crested weir is theoretically estimated through the hydraulic characteristics of control section mathematically obtained by the geometric properties of the basic dynamic equation. The theory described in this paper can also be applied to the control structures of overflow spillways. Some contributions to the design procedures for control structures are also presented.

Jeppson, R. W. 1965. Graphical solutions to frequently encountered fluid flow problems. Utah Water Research Laboratory, Utah State University, Logan, Utah, June, pp. 1-27.

This publication presents nomograms and charts which solve the hydraulic equations most frequently used for both open channel and pipe flow problems. This method of solution should be of considerable practical value over "cut and try" approaches usually applied to these kinds of problems. The use of the nomograms and charts is described briefly and illustrated by example problems.

Kandaswamy, P. K. 1957. Characteristics of flow over terminal weirs and sills. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 82, No. HY4, Paper No. 1345, Aug., pp. 1-11.

Generalized experimental results are presented to show the variations in both the discharge coefficient and the nappe profile for two-dimensional flow over a vertical sharp-crested weir as the ratio of head to depth of flow changes continuously from zero to unity.

Katsaitis, G. D. 1966. The use of dissipation bars in channel drop structures. The Journal of the Institution of Engineers, Australia, Paper No. 2005, Jan-Feb., pp. 9-18.

A systematic investigation of the use of dissipation bars in stilling basins for vertical drop structures is presented. The dissipation bars were found to be effective energy dissipators. Design equations have been derived which express the efficiency of these bars in terms of scour in the outlet channel. Example designs are listed.

Keim, R. S. 1962. The Contra Costa energy dissipator. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 87, No. HY2, Paper 3077, March, pp. 109-122.

The laboratory development of the Contra Costa energy dissipator is described. The dissipator is used in the re-establishment of natural channel flow conditions at culvert outfalls where uncontrolled, excessively high effluent velocities at depths less than half the culvert diameter otherwise would cause undesirable damage.

Kindsvater, C. E., and Carter, R. W. 1957. Discharge characteristics of rectangular thin-plate weirs. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 82, No. HY6, Paper No. 1453, Dec., pp. 1-35.

The flow pattern for rectangular, thin plate weirs is not subject to complete mathematical analysis and experiment is presented which provides a simple, direct solution for the discharge and a convenient method of compensating for the influence of viscosity and surface tension. The effects of viscosity and surface tension are related to an increase in the effective head and a decrease in the effective notch width. Thus, the combined effects of the fluid properties are accounted for with adjustment coefficients which are applied to measured values of the head and width. Consequently, the coefficient of discharge is defined as a function of the width contraction ratio and the head weir-height ratio, only.

Kindsvater, C. E. 1942. The hydraulic jump in sloping channels. Transactions of the ASCE, Vol. 67, Paper No. 2228, Nov. pp. 1107-1154.

Common forms of the hydraulic jump in sloping channels have been classified into three general cases, and an analysis is presented which leads to a practical method of computing the dimensions of the jump. Satisfactory agreement between analysis and experiment was obtained from laboratory tests on a channel with a 1 on 6 sloping floor. Conclusions drawn from this investigation indicate that experiments on other slopes might eventually yield a satisfactory treatment for hydraulic jumps on any slope within the practical range.

Lane, E. W. 1942. Spillways and stream-bed protection works. Handbook of applied hydraulics. Edited by Davis. McGraw-Hill Book Co., New York, Section 9, pp. 333-357.

The design procedures for various types of spillways is given, along with requirements for scour protection in natural river channels below stilling basins for spillway structures.

Luthra, S. D. 1950. Dissipation of energy below overfall dams. The Central Board of Irrigation Journal, November. pp. 660-665.

The paper presents a method of dissipation of energy of water flowing over a spillway dam which has been developed as a result of a large series of experiments. The existing methods such as hydraulic jump or an upturned bucket were tested for securing protection against scour below an overfall spillway.

Massey, B. S. 1961. Hydraulic jump in trapezoidal channels, an improved method. Water Power, June, pp. 232-233.

The author presents a simple and exact solution of the equations, in preference to approximate methods, in solving the problem relating to the hydraulic jump occurring in trapezoidal channels.

Moore, W. L., and Morgan, C. W. 1957. The hydraulic jump at an abrupt drop. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 82, No. HY6, Paper 1449, Dec., pp. 1-21.

The hydraulic jump may form at various locations relative to a low abrupt drop in a rectangular channel. The role of the drop in determining the form of the jump and in stabilizing its position is clarified by analysis and experiment. An example illustrates the application of the results to the analysis of stilling basin design.

Moore, W. L. 1943. Energy loss at the base of a free overfall. Transactions of the ASCE, Vol. 108, Paper No. 2204, pp. 1697-1714.

Experimental studies were made of a free overfall with a view to obtaining information that would be of value to designers of hydraulic structures. Detailed laboratory measurements showed that the energy losses at the base of a fall were of appreciable magnitude. These measured energy losses were applied in the development of a rational formula for calculating the height of the hydraulic jump below a fall. Limited information was also obtained on the length characteristics of the jump and on the effect of the submergence of the jump on energy dissipation. The presence of standing water behind the fall is explained and its height is calculated by application of the momentum equation.

Morris, B. T., and Johnson, D. C. 1943. Hydraulic design of drop structures for gully control. Transactions of the ASCE, Vol. 108, Paper No. 2198, pp. 887-940.

In the stabilization of gullies, small overflow dams are used to retain silt and to control the stream grade. These dams are simple drop structures similar to those used in irrigation canals. In this paper, the development of rules for the proportioning of such dams is described in terms of the hydraulic requirements for structure performance. The formulas included in the design rules are presented graphically for convenience in application. These rules are based on the accumulated experience of engineers in irrigation and soil conservation work and on the results of a series of laboratory test programs.

Murley, K. A. 1970. Irrigation channel structures, Victoria, Australia. Proceedings of the American Society of Civil Engineers, Journal of the Irrigation and Drainage Division, Vol. 96, No. IR2, Paper No. 7371, June, pp. 131-150.

The basic types of structures used in irrigation channels at Victoria are overflow drops (check regulators, measuring weirs and vertical drops), gate operated regulators undershot flow, access crossings and pipe structures used for culverts, siphons, and chutes, scouring tendencies and scour control arrangements of the structures used as observed in the field and as studies in model tests are considered in this paper measuring structures have been covered separately but some comments are given on hydraulic design for pipe structures and orifice measurement with submerged flow.

Nakagawa, H. 1969. Flow behavior near the brink of free overfall. Disaster Prevention Research Institute, Kyoto University, Vol. 18, Part 4, Bull. No. 149, March, pp. 65-76.

As for the free overfall, the boundary conditions can be determinative only at the terminal section, so that the flow behaviors in this case have been confirmed only by the experimental procedure, or the theoretical solution of the flow to be obtained would be restricted by the terminal section. The author tries to clarify the controlling mechanism of free overfall by one-dimensional analysis with the aid of experimental research and to develop a more exact method of analysis of the flow characteristics of free overfall, based on universal laws obtained by experimental investigation.

Neill, C. R. 1967. Mean-velocity criterion for scour of coarse uniform bed-material. Proceedings, XII, Congress, IAHR, Vol. 3, Paper C6, pp. 46-54.

New experimental data is presented on incipient motion of uniform bed materials ranging from 6mm to 30mm in diameter. The data have been correlated with comparable data by previous investigators to develop a dimensionless expression for scour of coarse uniform material. A design nomogram is presented relating competent mean velocity to grain size, specific gravity, and depth of flow. Predictions of the formula are compared with those of earlier design curves and formulas.

Neill, C. R. 1968. A re-examination of the beginning of movement for coarse granular bed materials. Report Int. 68, Hydraulics Research Station, Wallingford, Berkshire, England. June.

Flume experiments on low movement rates of uniform coal grains, glass spheres, and coal mixtures are described and analyzed. Reasonably consistent results were obtained using a numerical criterion for beginning of movement. A 3-parameter relationship is proposed expressing a dimensionless grain displacement rate as a function of the original Shields parameters. Empirically, the beginning of movement for uni-model mixtures of moderate dispersion can be estimated by inserting the D_{50} size by weight into the expression for uniform materials.

Opie, T. R. 1968. Scour at culvert outlets. M. S. Thesis, Colorado State University, Fort Collins, Colorado, March.

The procedures used in, and the results of, experiments to determine the size and geometry of scour holes in flat, loose rock beds at culvert outlets are given. A review of four recent approaches to the problem is also included. From a dimensional analysis, the depth of scour at such an outlet is related to the discharge and bed characteristics. The depth of scour has been related to the length, width, and volume of scour. The relations are severely restricted in their application to the range of outlet conditions. Practical examples are given. Results are presented in graphic form.

Peterka, A. J. 1964. Hydraulic design of stilling basins and energy dissipators. Engineering Monogram No. 25, U.S. Dept. of the Interior, Bureau of Reclamation, Denver, Colorado, Sept. pp. 1-222.

Generalized designs are given for stilling basins and energy dissipators of several kinds, as well as associated appurtenances. General design rules are presented so that the necessary dimensions for a particular structure may be easily and quickly determined, and the selected values checked by others without the need for exceptional judgment or extensive previous experience. Proper use of the material in this monograph will eliminate the need for hydraulic model tests on many individual structures, particularly the smaller ones. Designs of structures obtained by following the recommendations will be conservative in that they will provide a desirable factor of safety.

Pillai, N. N., and Unny, T. E. 1964. Shapes for appurtenences in stilling basins. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 92, No. HY3, Paper No. 3888, May, pp. 1-21.

New shapes for stilling basin appurtenances in the form of wedge blocks with an apex angle of 120° are proposed based on detailed analysis of the existing literature on the subject and by further experimentation. The new type recommended causes greater energy dissipation in a shorter length than the commonly adopted rectangular blocks. Also, the modified shape is expected to be free from cavitation danger at the large velocities encountered in high dams. The overfall arrangement of the stilling basin shows an appreciable reduction in the size of the basin.

Prasad, R. 1970. Numerical method of computing flow profiles. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 96., No. HY1, Paper No. 7005, Jan., pp. 75-86.

A general method of computing flow profiles based on numerical integration is presented. The differential equation of gradually varied flow may be numerically integrated using Manning's (or any other) formula for energy slope by the method described in this paper.

Qazi, N. A. 1958. Stability of canal linings. M.S. Thesis, University of Toronto, Ontario.

Presents historical data reported by previous investigators on incipient motion. In addition, laboratory experiments were conducted using particles ranging in size from 0.25 to 1.5 inches.

Rajaratnam, N. 1963. Method of hydraulic equivalents for critical-flow computations. Water Power, Feb. pp. 79-80.

A modified method of hydraulic equivalents for critical-flow computations in open channels is presented.

Rajaratnam, N. 1965. Submerged hydraulic jump. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 91, No. HY4, Paper No. 4403, July, pp. 71-94.

The writer has attempted to solve the submerged hydraulic jump as the case of a plane turbulents wall jet under an adverse pressure gradient over which a backward flow has been placed. Using the writer's results, an analysis is presented of the forward flow in the submerged jump as a plane wall jet.

Rajaratnam, N. 1967. Diffusion of submerged sluice gate flow over a drop. 12th Congress IAHR, Fort Collins, Colorado, Vol. 4, Paper No. 17, pp. 156-163.

The paper presents an experimental study of the diffusion of a supercritical stream emerging under a submerged sluice gate situated over an abrupt drop. The length of the eddying region, the characteristics of the velocity profile, and bed shear stress after reattachment are studied. It has been found that after a certain distance downstream of the reattachment line, the supercritical stream could be treated as a reattached plane turbulent wall jet which behaves very much like the corresponding classical wall jet. The variation of the bed shear stress is expressed in the form of a dimensionless similarity plot.

Rajaratnam, N., and Muralidhar, D. 1964. End depth for exponential channels. Proceedings of the ASCE, Journal of the Irrigation and Drainage Division, Vol. 90., No. IR1, Paper No. 3819, March, pp. 17-39.

An extensive investigation of the end depth problem for exponential channels is presented. General theoretical equations have been developed for all cases. From a large number of carefully conducted experiments, the end depth ratio for horizontal free overfalls has been found to 0.795 for the triangular shape and 0.772 for the parabolic shape. For sloping channels, the end depth ratio has been found to be a function of the relative slope. For the rectangular free overfall, it was found that the already available results for the confined case could be used with little error for the unconfined case.

Rajaratnam, N., and Subramanya, K. 1968. Hydraulic jumps below abrupt symmetrical expansions. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 94, No. HY2, Paper No. 5860, March, pp. 481-503.

An experimental study has been made of the R-jump and S-jump occurring below abrupt symmetrical expansions in rectangular channels. A new characteristic length has been developed to correlate the mean flow characteristics of the S-jump. The S-jump has been treated as a three-dimensional turbulent wall jet.

Rand, W. 1955. Flow geometry at straight drop spillways. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 81, Paper No. 791, Sept., 13 pp.

The flow pattern at a straight drop spillway can be described by a number of characteristic length terms: the drop length, that is the distance from the vertical drop wall to the toe of the non-submerged nappe, the length of the hydraulic jump if it begins at the toe of the nappe, the depth of flow downstream from this jump, and the depth of the under-nappe pool between the drop wall and the nappe. All these values are represented as functions of the discharge and of the height of the drop. The results are given by a collective plot of dimensionless terms. Two geometrical properties of the flow pattern are established, consisting in practically constant relationships between some of the terms. The determination of flow geometry is important for the design of straight drop stilling basins.

Rand, W. 1970. Sill-controlled flow transitions and extent of erosion. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 96, No. HY4, Paper No. 7212, April, pp. 927-939.

Sill-controlled flow transitions in open channels, dependent on the geometry of a rigid (fixed-bed) boundary, have been described earlier. This investigation deals with the same flow transitions where the channel downstream of the sill is erodible. A similarity concept for erosion, to be confirmed by experimental evidence, will be established as an extension to the similarity criteria valid for a rigid boundary. As a result, prediction of the extent of erosion is expected to become possible for a wide variety of sill-controlled flow transitions, including the natural hydraulic jump, and flow transitions present in the hydraulic jump stilling basins.

Rao, G. N. S., Seetharamaiah, K., and Swamy Chandrasekhara, N. V. 1960. Dissipation of energy of a circular jet submerged in water. La Houille Blanche, No. 6, Nov. pp. 704-713.

This paper deals with the study of the dissipation of energy of a high velocity flow into a standing mass of water. At the instant when the jet enters the standing pool of water, the jet possesses the maximum energy, which is gradually dissipated along its passage in the surrounding medium. Some studies were undertaken with a view of studying the mechanism by which the energy of the jet is reduced to a minimum. Studies are also reported regarding various conditions of flow.

Rao, G. N. S. and Rajaratnam, N. 1963. The submerged hydraulic jump. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 89, No. HY1, Paper No. 3404, January, pp. 139-410.

An investigation on the submerged hydraulic jump is presented. The submerged jump has been defined with the introduction of a definition sketch and a submergence factor. Theoretical and experimental equations have been developed for the main flow parameters. A general equation has been derived for the energy loss in the submerged jump. All the theoretical developments have been experimentally verified for the range of supercritical Froude numbers from 2.94 to 10.0 and for the submergence factor to values of approximately 4. It has also been established that, in a submerged jump, high velocities continue along the bottom for considerable distances, thereby causing scour.

Roberge, R. A., and Peixotto, E. D. 1956. Similitude of incipient motion. Unpublished M.S. Thesis, Massachusetts Institute of Technology, Cambridge, Mass.

This is a laboratory investigation of conditions controlling incipient motion of graded rock particles composing the bed of a turbulent stream. A resistance coefficient was obtained for various sized material and plotted against Reynolds number.

Rouse, H. 1936. Discharge characteristics of the free overfall. Civil Engineering, Vol. 6, No. 4, April, pp. 257-260.

The free overfall can be used as a flow meter without calibration. Although the flow at the overfall is not parallel, the crest section is that of true minimum energy and hence is the actual control section. The crest depth is a constant percentage of the computed critical depth for parallel flow.

Rouse, H. 1958. Turbulence characteristics of the hydraulic jump. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 83, No. HY1, Paper No. 1528, Feb., pp. 1-30.

Hot wire measurements of the turbulence in an air-flow model of the hydraulic jump are described for Froude numbers 2, 4, and 6. Results are analyzed and interpreted in the light of the momentum and energy. In an initial analytical section, the modeling method is justified and the differential and integral forms of the momentum and energy equations pertinent to the investigation are explained.

Rube, W. W. 1938. The force required to move particles on a stream bed. U. S. Geological Survey, Professional paper 189-E, pp. 121-141.

Three theories regarding the movement of particles from a stream bed are discussed: (1) the impact theory, which states that the weight of the largest particle moved by a stream varies as the sixth power of the velocity; (2) critical tractive force theory; and (3) hydraulic lift theory, which is concerned with the difference in pressure above and below the particle. The laboratory data of Gilbert has been used to test these theories.

Shields, A. 1936. Application of similarity principles and turbulence research to bed-load movement. Translated from German by W. P. Ott and J. C. van Vchelen for California Institute of Technology, Pasadena, California.

This is a translation of the original German report which deals with general problems of bed load movement with particular reference to the influence of weight and shape. By using particles of barite, granite, coal, and amber, a range of particle weights could be studied. Investigation of grain shape covered rounded, angular, and sharp-edged grains. The development of the Shields function is explained, along with its physical importance.

Shih, C. C. and Parson, D. F. 1967. Some hydraulic characteristics of trapezoidal drop structures. Proceedings of the 12th Congress, IAHR, Fort Collins, Colorado, Vol. 3, pp. 249-260.

This study is concerned with some of the hydraulic characteristics of flow over drop structures connecting two trapezoidal channels at different elevations. The drop structures are formed by an abrupt drop in the horizontal channel bottom, and are equipped with or without a weir. The dimensional analysis, which was based on theoretical considerations of the flow problem, resulted in a set of dimensionless parameters, namely the relative depth, the drop number, and geometric parameters of the drop structure. Experimental results are presented in graphical form through dimensionless parameters for various trapezoidal drop structures.

Shihmen Reservoir Project. 1963. Hydraulic model studies on the overflow section and stilling basin of Shihmen Afterbay Weir. Engineering Department, Shihmen Development Commission in Cooperation with Taipei Hydraulic Laboratory, National Taiwan University, Technical Monograph No. 3, Taipei, Taiwan, Republic of China, August., pp. 1-28.

The results of tests involving the hydraulic performances of the ogee crest, the sloping apron, the stilling basin, and the downstream river bed including pressures, water surfaces and scour measurements are reported. The tests also included a study of the hydraulic performance under conditions of tailwater levels progressively lower than those assumed in the basic design.

Shukry, Ahmed, M. 1957. The efficacy of floor sills under drowned hydraulic jumps. Proceedings of the ASCE, Journal of the Hydraulic Research, Vol. 82, No. HY3, Paper 1260, June, pp. 1-9.

The results of hydraulic tests on the performance of various types of floor sills are presented. The study is mainly concerned with low-head river barrages which are generally operated under conditions of drowned jumps. The distribution of velocity for various types and location of sills was recorded by a Pitot-static tube. The efficiency of any sill against bed scour is indicated by the rate of adjustments of the flow to the normal distribution in the downstream channel.

Silvester, R. 1964. Hydraulic jump in all shapes of horizontal channels. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 92, No. HY1, Paper No. 3754, Jan., pp. 23-55.

Exact solutions are provided for the conjugate depths and energy loss for hydraulic jumps in rectangular, triangular, parabolic, circular, and trapezoidal channels in terms of the upstream Froude number. These agree with previous solutions derived in terms of other dimensionless parameters and are verified by published data and local tests. The equation for the length of the jump for the various shapes of channel is obtained in terms of the upstream depth and Froude number. However, the constant in the relationship must be determined experimentally because it is dependent on the general proportions of any given channel shape.

Simons, D. B., Stevens, M. A., and Watts, F. J. 1970. Flood protection at culvert outlets. Colorado State University, Civil Engineering Dept., Report CER 69-70 DBS-MAS-FJ W4, 211 pp.

In this study, several classes of information concerning flood protection at culvert outlets are presented. The information is related to the flow conditions at culvert outfalls and to the hydraulics of rigid basins and outlet basins stabilized with rock riprap. In addition, the characteristics of high tailwater and non-scouring, low tailwater basins are covered.

Skogerboe, G. V., and Hyatt, L. M. 1967. Analysis of submergence in flow measurement flumes. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 92, No. HY4, Paper No. 5348, July, pp. 183-200.

The calibration curves which describe submergence in flow measuring flumes are developed by a combination of dimensional analysis and empiricism. The parameters developed in this manner are further verified by the theoretical submerged flow equation developed from momentum relationships. A flat-bottomed rectangular measuring flume was used to generate data necessary for establishing the parameters describing submerged flow. The resulting form of the discharge equation has been verified for a trapezoidal flat-bottomed flume and a Parshall flume. For any particular flume geometry, both the free flow and submerged flow equations can be placed on a single graph.

Skogerboe, G. V., Walker, W., Hacking, B. B. and Austin, L. H. 1970. Research topics for small irrigation structures. Proceedings of the ASCE, Journal of the Irrigation and Drainage Division, Vol. 96, No. IR3, Paper No. 7542, Sept., pp. 309-318.

The primary purpose of this effort has been to delineate specific research topics which could be accomplished as a thesis by graduate student at the master of science level. In most cases, pursuit of the suggested topics requires the collection of laboratory data regarding discharge, flow depths, and consequent energy losses.

Smith, C. D. 1959. The effect of sidewall height on the hydraulic jump on a continuously sloping chute. 8th Congress of the International Association For Hydraulic Research, Montreal, August, pp. 65.

In three-dimensional tests of the hydraulic jump on a sloping chute, it was shown that the height of the sidewall had an important effect on the performance. The results of these studies and the consequent design criteria are presented.

Smith, C. D. 1962. Brink depth for a circular channel. Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 87, No. HY6, Paper No. 3327, November, pp. 125-134.

The relationship between the discharge and the depth at the brink of a freely discharging circular section is investigated. Through application of the momentum equation between a section upstream from the brink and a section downstream from the brink, the limits for the brink depth are established from theory. Experimental test points are found to lie between the limits so delineated. A dimensionless plot is provided, from which the discharge versus brink depth can be calculated for design purposes for any size of pipe.

Smith, C. D., and Strang, D. K. 1967. Scour in stone beds. Proceedings of the 12th Congress, IAHR, Fort Collins, Colorado, Vol. 3, pp. 65-73.

The problem of scour due to nappe impingement in a stone bed downstream from a vertical drop structure is reported. The stone size and areal extent necessary for a dependable design was determined. The data is presented in dimensionless charts.

Smith, G. L. 1957. Scour and energy dissipation below culvert outlets. Colorado Agricultural and Mechanical College, Fort Collins, Colorado, Report CER 57 GLS 16, April.

The first section of this material introduces the concept of kinetic energy dissipation as a function of the effects of viscosity fluid motion. The second section, which considers the concentration of excessive kinetic energy as learned from engineering experience, consists of the general classifications: (a) eroding velocities in conveyance systems; (b) control structures installed in conveyance systems; and (c) outlet structures such as culverts and tunnels used to by-pass water between conveyance systems. The third and final section reviews the various types of energy dissipation structures which have been developed on the basis of knowledge gained from experience, experiment, and theoretical analysis.

Smith, G. L. 1961. Scour and scour control below cantilevered culvert outlets. Colorado State University, Civil Engineering Dept., Report CER 61 GLS 14, 109 pp.

An investigation of the phenomenon of scour and scour control below a cantilevered culvert outlet under steady, uniform flow conditions was investigated. Primary consideration was given to the kinematics of the jet causing scour, the outlet boundary geometry, and the characteristics of both the bed material of the alluvial channel and the graded gravel (armor-plate) used for scour control.

Stevens, M. A. 1969. Scour in riprap at culvert outlets. Ph. D. Dissertation, Colorado State University, Fort Collins, Colorado, Jan.

The object of this study was to develop design criteria for riprapped stilling basins at circular culvert outlets. The data for the design aids were collected from models of culverts and rock basins; model pipe diameters ranged from 6 inches to 36 inches and rock riprap from 0.5 inches to 7 inches. For the type of flow encountered at culvert outlets, the Froude model law was tested in various sized models and found to be sufficient for scaling results to prototype installations. The important variables in the hydraulic design were the discharge, pipe diameter, brink depth, tailwater level, and representative rock diameter. Since such large riprap was required to prevent scour at culvert outlets, emphasis was placed on rock basins in which scour was allowed. Examples of the design riprapped basins are solved in detail.

Task Force on Energy Dissipators for Spillways and Outlet Works. 1964. Energy dissipators for spillways and outlet works. Proceedings, ASCE, Journal of the Hydraulics Division, Vol. 92, No. HY1, Jan., pp. 121-147.

The history and development of hydraulic jump and roller bucket stilling basins are given, as well as the present (1963) state of knowledge for design of these basin types. Criteria for the selection of basin type and the need for basin appurtenance are given in the light of some often overlooked considerations. The primary conclusions of the report are summarized as follows: Many factors other than established empirical or model relationships must be considered in the design of such basins, such as the absolute size of the structure, frequency of operation, and durability of river bed downstream. However, extensive use of hydraulic models over the past 25 years has done more to establish correct design criteria for such basins than any other single procedure. Future progress in improving the performance of such stilling basins will be assured with further laboratory tests and testing and reporting the findings of tests made on the resulting full size structures. An extensive bibliography on all types of energy dissipators for spillways and outlet works, believed to be essentially complete through 1962, is included.

Thomas, R. A. 1953. Scour in a gravel bed at the base of a free overfall. M.S. Thesis, Colorado Agricultural and Mechanical College, Fort Collins, Colorado, May, 117 pp.

Extensive erosion or sedimentation in localized areas has often resulted from the conveyance of water in irrigation systems in the Western United States. The most widespread occurrence of excessive erosion is in irrigation canals and open drains which were constructed too steep and whose grades have not been corrected. The experiments reported in this thesis were conducted in a glass-walled flume in the Hydraulics laboratory at Colorado A & M College. Emperical formulae for the depth of scour were developed for each size of gravel.

Watts, F. J. 1968. Hydraulics of rigid boundary basins. Ph. D. Dissertation, CSU, Fort Collins, Colorado, Aug. 237 pp.

The object of the study was to develop design criteria for three classes (A, B & C) of rigid boundary energy dissipating structures. Design aids developed during this study include: dimensionless coefficients for the energy and momentum equations

which correct for nonhydrostatic pressure distribution and nonuniform velocity distribution at the outfall sections of circular and rectangular conduits; dimensionless water surface contours and velocity vectors for freely expanding jets supported on the bottom, downstream of circular and rectangular abrupt expansions; drag coefficients for roughness elements of known size and spacing; and other minor criteria. Design procedures based on continuity of flow and the balance of impulse and momentum from station to station are presented for the three classes of basins.

White, C. M. 1940. The equilibrium of grains on the bed of a stream. Proceedings, Royal Society of London, Series A, Vol. 174, pp. 322-338.

The discussion focuses upon the fluid flow in the vicinity of the particles rather than just the bulk flow parameters which might bring about movement of a particle. The forces required to move an individual particle, the shearing stress at the stream bed, and the effects of turbulence are discussed.

Whittington, R. B. 1969. Convergent stilling basins. Institution of Civil Engineers, Proceedings, Vol. 43, pp. 157-173.

The paper deals with the operation of convergent stilling basins in the "drowned jump" condition. Particular attention is paid to the occurrence of "sweep-out". A momentum equation is given for the calculation of the "backing-up" depth at the entrance to the basin; this yields good agreement with experimental results. The paper attempts to relate the observed phenomena to the fundamental work of Ippen on oblique shock waves.

Yalin, M. S. 1965. Similarity in sediment transport by currents. Hydraulics Research Paper No. 6, Hydraulics Research Station, Wallingford.

Dimensional analysis has been applied to the problem of incipient motion of bed particles. The criterion developed by this technique are verified by hydraulic experiments.

Zimmerman, F., and Maniak, U. 1967. Scour behind stilling basins with end sills of baffle-piers. 12th Congress IAHR, Fort Collins, Colorado, Paper C14, Vol. 3, Sept. pp. 117-124.

The nature of scour in the movable bed of a river downstream from a stilling basin of a weir permits a judgment to be made on the rate of the dissipation of energy in the stilling basin. Scour is an important criterion in determining adequate dimensions for stilling basins. In this paper, two rows of baffle piers are used in the stilling basin and design formulas are developed with regard to minimizing scour. The results of the model tests are confirmed by field observations.

APPENDIX B

EXPLANATION AND LISTING OF COMPUTER PROGRAM TO EVALUATE FLOW CONDITION AT THE BASE OF SMALL IRRIGATION DROP STRUCTURES

The following computer program can be used to evaluate the hydraulic flow conditions at the base of small irrigation drop structures. Basically, the program handles the following cases as described in an earlier section:

(1) Rectangular shaped drop structures which include straight drop spillways of varying height and discharge, and inclined drops of varying horizontal length, height, and discharge.

(2) Trapezoidal shaped drops such as the straight drop spillway and inclined drops. In addition to the parameters varied for the rectangular case, side slope and bottom width can be varied.

The scope of the program is to initialize with the flow conditions of the inlet and compute the conditions at the base of the drop. These conditions include the depth, Froude number and conjugate depth for the flow. Before using the program, several limitations should be observed:

(1) Inclined drops in small irrigation systems are often steep, so that the application of the gradually varied flow equation to compute the flow profile is not entirely justified.

(2) The depth at the inlet has not been experimentally evaluated, consequently an assumption must be made between .7 times the critical depth and the critical depth.

(3) For the case of the straight drop spillway, the approach channel has been assumed to be horizontal. If such is not the case, adjustment can be made.

Provision has been made in the program to vary the channel roughness if desired, as well as add the velocity distribution coefficient.

To better facilitate the possible use of the program, a list of variables and their definitions have been included below. Also the functions of the program subroutine have been tabulated.

Program Definitions

A	Velocity head coefficient
AN	Mannings roughness coefficient
AT	Froude number
B	Bottom width of channel, always equal to 1 for rectangular channel
DL	Incremental division in incline length for surface profile computations
EL	Length of incline
H	Height of drop structure
S	Side slope of the channel sides
SO	Slope of the incline
UQ	Unit discharge for rectangular channels and total discharge for the trapezoidal channel
X	Horizontal length of the drop structure
Y1	Depth at the base of the drop
Y2	Conjugate depth at base or the drop
Yc	Critical depth

Subroutine Descriptions

Subroutine YDOTZ	This subroutine is the numerical solution to the differential equation describing gradually varied flow. Used exclusively in the incline evaluation.
Subroutine TRAPC	Solution to the critical depth in trapezoidal channel.
Subroutine TRAPZ	Solution to the conjugate depth in trapezoidal channels.
Subroutine PUTOUT	Output of computations.

```
      PROGRAM THOM(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      COMMON H(20),DV(20),UQ(20),YC(20),Y1(20,20),AL(20,20),Y2(20,20),BD
     1(20,20),AT(20,20),AW(20,20),X(20),B(12),S(10)
      READ(5,2) NQ,NH,NX,NS,NB,NOPER,NCD
    2 FORMAT(24I3)

      NQ=NUMBER OF DISCHARGES, NH=NUMBER OF DROP HEIGHTS, NX= NUMBER OF
      DROP CHANNEL LENGHTS, NS= NUMBER OF CHANNEL SIDE SLOPES,AND NB= NU
      MBER OF CHANNEL BOTTOM WIDTHS
      IF NCD IS 1 THE PROGRAM WORKS THE RECTANGULAR PROBLEM
      IF NOPER IS 0 THE PROGRAM WORKS THE STRAIGHT DROP SPILLWAY
      IF NOPER IS 1 THE PROGRAM WORKS THE INCLINED DROP
      IF NCD IS 0 THE PROGRAM WORKS THE TRAPEZOIDAL PROBLEM

      READ(5,1)(UQ(I),I=1,NQ)
      READ(5,1)(H(I),I=1,NH)
      READ(5,1)(X(I),I=1,NX)
      READ(5,1) (S(I),I=1,NS)
      READ(5,1) (B(I),I=1,NB)
    1 FORMAT(12F6.1)
      DH =0.1
      A=1.05
      AN=0.018
      DO 12 I=1,NH
      DO 12 J=1,NQ
      Y1(I,J)=0.0
      Y2(I,J)=0.0
   12 AT(I,J)=0.0
      DO 11 K=1,NX
      DO 11 II=1,NB
      DO 11 IJ=1,NS
      DO100J=1,NQ
      IF(NCD.EQ.1) GO TO 600
      CALL TRAPC(J,IJ,II,Y)
      GO TO 603
  600 CONTINUE
      YC(J)=(UQ(J)*UQ(J)/32.176)**0.333
  603 CONTINUE
      DO100I=1,NH
      Y=0.8*YC(J)
      IF(NOPER.EQ.0)GO TO 555
      I2=0
      Q1=UQ(J)
      B2=S(IJ)
      FL=SQRT(H(I)**2+X(K)**2)
      B1=B(II)
      SO=H(I)/X(K)
      I3=0
  101 CONTINUE
      IF(I3.EQ.1) GO TO 115
      IF(I2.GE.10) GO TO 115
      CALL YDOTZ(Y,I3,B1,B2,Q1,AN,A ,SO,DL,EL)
      I2=I2+1
```

```
THOM      FORTRAN EXTENDED VERSION 2.0

      GO TO 101
  115 Y1(I,J)=Y
      GO TO 556
  555 CONTINUE
      Y1(I,J)=0.54*H(I)*(YC(J)/H(I))**1.275
  556 CONTINUE
      IF(NCD.EQ.1) GO TO 604
      IF(Y1(I,J)) 700,700,701
  700 Y2(I,J)=0.0
      AT(I,J)=0.0
      GO TO 605
  701 CONTINUE
      CALL T2AR2 (J,IJ,II,Y3,T)
      Y2(I,J)=Y3
      ZKT=H(II)/(S(IJ)*Y1(I,J))
      AT(I,J)=Q(J)/(S(IJ)*5.67*Y1(I,J)**2.5*(ZKT+1.))
      GO TO 605
  604 CONTINUE
      IF(Y1(I,J)) 702,702,703
  702 Y2(I,J)=0.0
      AT(I,J)=0.0
      AL(I,J)=0.0
      GO TO 605
  703 CONTINUE
      AL(I,J)=UQ(J)**2   /32.176/Y1(I,J)**3
      Y2(I,J)=Y1(I,J)*(SQRT(1.+8.*AL(I,J))-1.)/2.
      AT(I,J)=SQRT(AL(I,J))
  605 CONTINUE
  100 CONTINUE
      CALL PUTOUT(NH,NQ,K,NS,NB,II,IJ,NCD,NOPER)
   11 CONTINUE
      STOP
      END
```

YDOTZ   FORTRAN EXTENDED VERSION 2.0

```
      SUBROUTINE YDOTZ(Y,L,B,S,Q,AN,AL,SO,DL,EL)
      FXTZ(B,S,AN,SO,AL,Q,Y)=(SO-((AN*Q)**2*(B+2.*Y*SQRT(1.+S*S))** 1.33
     13 )/(2.21*((B+S*Y)*Y)**  3.333 ))/(1.-((AL*Q*Q*(B+2.*S*Y))/(32.176
     2*((B+S*Y)*Y)**3)))
      YDOT1=FXTZ(B,S,AN,SO,AL,Q,Y )
      YDOT2=YDOT1
      I=0
  101 TEMP=YDOT2
      I=I+1
      Y2=Y +(YDOT1+YDOT2)/2.*DL*EL
      IF(I.GT.50) L=1
      IF(I.GT.50) Y=0.0
      IF(I.GT.50)  GO TO 111
      IF(Y2.LT.0.0)  Y2=-Y2
      YDOT2=FXTZ(B,S,AN,SO,AL,Q,Y2)
      IF(ABS(TEMP-YDOT2).GT.0.0001) GO TO 101
      Y=Y2
  111 CONTINUE
      RETURN
      END
```

PUTOUT FORTRAN EXTENDED VERSION 2.0

```
      SUBROUTINE PUTOUT(NH,NQ,K,NS,NB,II,IJ,NCD,NOPER)
      COMMON H(20),DV(20),UQ(20),YC(20),Y1(20,20),AL(20,20),Y2(20,20),BD
     1(20,20),AT(20,20),AW(20,20),X(20),B(12),S(10)
      IF(NCD.EQ.1) GO TO 200
      WRITE(6,201)
  201 FORMAT(1H1,/////,47X,*DESIGN OF TRAPEZOIDAL DROP STRUCTURES*/)
      GO TO 202
  200 CONTINUE
      WRITE(6,1)
    1 FORMAT(1H1,/////,47X,*DESIGN OF RECTANGULAR DROP STRUCTURES*///)
  202 CONTINUE
      IF(NOPER.EQ.0) GO TO 203
      WRITE(6,76) X(K)
   76 FORMAT(1H0,*LENGTH OF DROP SRTUCTURE*,F6.2)
  203 CONTINUE
      IF(NCD.EQ.1) GO TO 204
      WRITE(6,205) B(II),S(IJ)
  205 FORMAT(1H0,*BOTTOM WIDTH*,F6.2,10X,*SIDE SLOPE*,F6.2//)
  204 CONTINUE
      WRITE(6,3)
    3 FORMAT(1H ,*HEIGHT*,50X,*UNIT DISCHARGE/CRITICAL DEPTH*)
      LK=1
      WRITE(6,57) (UQ(J),J=1,NQ)
   57 FORMAT(1H ,//,10X,20F6.1)
      WRITE(6,44)(YC(J),J=LK,NQ)
   44 FORMAT(1H ,10X,20F6.3/)
      WRITE(6,109)
  109 FORMAT(1H0,65X,*F1 / Y1 / Y2*)
      DO 108 I=1,NH
      WRITE(6,5)H(I),(AT(I,J),J=LK,NQ)
    5 FORMAT(1H0,F5.1,5X,20F6.2)
      WRITE(6,56)(Y1(I,J),J=LK,NQ)
      WRITE(6,56) (Y2(I,J),J=LK,NQ)
   56 FORMAT(1H ,10X,20F6.3)
  108 CONTINUE
  106 CONTINUE
      RETURN
      END
```

TRAPC   FORTRAN EXTENDED VERSION 2.0

```
      SUBROUTINE TRAPC(J,IJ,II,Y)
      COMMON H(20),DV(20),UQ(20),YC(20),Y1(20,20),AL(20,20),Y2(20,20),BD
     1(20,20),AT(20,20),AW(20,20),X(20),B(12),S(10)
      DELX=1.0
      YD=0.0
101   CONTINUE
      A=B(II)+2.0*S(IJ)*YD
      A1=32.176*(YD*(B(II)+YD*S(IJ)))**3
      A2=UQ(J)**2  *A
      IF(A1-A2)102,103,104
102   IF(DELX.EQ.2.0)GO TO 105
      IF(DELX.EQ.3.0) GO TO 105
      YD=YD+0.1
      GO TO 101
104   IF(DELX.EQ.3.0)GO TO 103
      DELX=2.0
      YD=YD-0.01
      GO TO 101
105   DELX=3.0
      YD=YD+0.001
      GO TO 101
103   CONTINUE
      YC(J)=YD
      XC=S(IJ)*YC(J)/B(II)
      AEC=6.0*(1.0+2.0*XC+XC*XC)/(9.0+20.0*XC+10.0*XC*XC)
      ACB2=YC(J)/B(II)+S(IJ)*(YC(J)/B(II))**2
      ABB2=ACB2*AEC*B(II)*B(II)
      Y=-B(II)/2./S(IJ)+SQRT((B(II)/S(IJ))**2+4.*ABB2/S(IJ))/2.
      RETURN
      END
```

TRAP2 FORTRAN EXTENDED VERSION 2.0

```
      SUBROUTINE TRAP2(J,IJ,II,Y3,I)
      COMMON H(20),DV(20),UQ(20),YC(20),Y1(20,20),AL(20,20),Y2(20,20),BD
     1(20,20),AT(20,20),AW(20,20),X(20),B(12),S(10)
      D1=(B(II)+S(IJ)*Y1(I,J))*Y1(I,J)/(B(II)+2.0*S(IJ)*Y1(I,J))
      T=B(II)/(S(IJ)*Y1(I,J))
      AT(I,J)=UQ(J)/((B(II)+S(IJ)*Y1(I,J))*Y1(I,J))/SQRT(D1*32.176)
      ZIP=0.0
      R=1.0
  101 CONTINUE
      Z=R**4  +(2.5*T+1.0)*R**3  +(1.5*T+1.0)*(T+1.0)*R**2  +(0.5*T**2
     1+(T-3.*AT(I,J)**2 )*(T+1.))*R-3.*AT(I,J)**2 *(T+1.)**2
      IF(Z)102,103,104
  102 IF(ZIP.EQ.2.0)GO TO 105
      IF(ZIP.EQ.3.0) GO TO 105
      R=R+0.1
      GO TO 101
  104 IF(ZIP.EQ.3.0) GO TO 103
      ZIP=2.0
      R=R-0.01
      GO TO 101
  105 ZIP=3.0
      R=R+0.001
      GO TO 101
  103 Y3=R*Y1(I,J)
      RETURN
      END
```

APPENDIX C

DESIGN EXAMPLES

DESIGN C-1
INCLINED DROP
WITH USBR STILLING BASIN II

References

(a) Chapter 4, Pages 3-5.

(b) Chapter 4, Pages 14-19.

(c) Peterka, 1964.

(d) Prasad, 1970.

Problem Description

A 10 foot drop (h = 10 ft, Figure A-1) between two 10 foot wide rectangular channels carrying 100 cfs is to be accomplished by an inclined drop with an USBR Stilling Basin II to dissipate excess energy and thus protect the downstream channel. The tailwater depth is 3 feet (Y_2) and the horizontal length of the drop is to be 30 feet (x = 30 ft, Figure 4-1).

Design Assumptions

(a) Based upon the study results cited in Ref. (c), the following design criteria are recommended for the USBR Stilling Basin II. Refer to Figure 4-1 and Figure 4-6.

1. The stilling basin should include a 5% safety factor for tailwater. Thus, the elevation of the apron should be 0.05 Y_2 below the floor elevation determined from the tailwater depth and flow depth after the hydraulic jump.
2. The Froude munber of the flow entering the basin should exceed 4.
3. The height of chute blocks and the width and spacing of these blocks equals the depth entering the basin, Y_1. A

space equal to $Y_1/2$ is to be allowed adjacent to the wells.

4. The height of dentating sill equals 0.2 times the tailwater depth, Y_2. The maximum width and spacing of dentates is 0.15 Y_2. Slope of continuous part of sill is 2:1.

(b) In order to determine the depth, Y_1, it is necessary to compute the water surface profile down the incline. The method chosen was that described in Ref. (d). The assumption is made that the brink depth is 0.8 times the critical depth, Y_c. A Manning roughness factor n of 0.018 will be used.

Design

(a) Computation of water surface profile. For a unit discharge of 10 cfs/ft, Y_c = 1.459 feet, and thus the brink depth is 1.17 feet. Using the computer program listed in Appendix B, an end depth Y_1 of 0.535 feet is derived.

(b) Basic stilling basin dimensions. The inlet Froude number can be computed as 4.50 and thus from Figure 4-8,

$$L_1/Y_2 = 3.8 \qquad \text{or} \qquad L_1 = 11.4 \text{ feet.}$$

Chute block height, width, and spacing is 0.535 feet.

The other basic dimensions and the number of chute blocks and dentates is the end sill follow from the earlier assumptions.

DESIGN C-2
INCLINED DROP
WITH USBR STILLING BASIN III

References

(a) Chapter 4, pages 3-5

(b) Chapter 4, pages 19-22

(c) Peterka, 1964

(d) Prasad, 1970

Problem Description

A 10 foot drop between two 10 feet wide rectangular channels carrying 200 cfs is to be accomplished by an inclined drop with an USBR Stilling Basin III to dissipate excess energy for downstream protection. The expected tailwater depth is 5 feet, the horizontal length of the incline is 30 feet, and the channel roughness coefficient (Manning's N) is 0.018.

Design Assumptions

(a) Based on study results cited in Ref. (c), the following design criteria are suggested for the USBR Stilling Basin III. Refer to Figures 4-1, 4-9 and 4-10.

1. The apron flow should be set 0.05 Y_2 feet below the floor elevation determined by subtracting the flow depth after the hydraulic jump from the tailwater elevation. This will allow a 5% safety factor for tailwater depth Y_2.

2. The Froude number entering the basin should exceed 4.

3. The height, width and spacing of chute blocks should equal the flow depth entering the basin Y_1. Chute blocks should be at least 8 inches high, but width and spacing may be

<u>Design Assumptions</u> (cont.)

reduced proportionately to accomodate smaller values of Y_1.

4. The upstream face of the baffle piers should be $0.8\ Y_2$ feet downstream from the chute block faces.
5. Slope of end sill is 2:1.
6. Stilling Basin appurtenances should not be streamlined.
7. The transition between chute and stilling basin should be rounded for chute slopes greater than 45° ($R > 4\ Y_1$).

(b) The computation of the water surface profile down the chute necessary to determine Y_1 was preformed using the method outlined in Ref. (d). As such, the brink depth is assumed to be $0.8\ Y_c$, when Y_c is the critical depth of flow.

<u>Design</u>

(a) Computation of water surface profile. For a unit discharge of 20 cfs/ft $Y_c = 2.315$ feet and the corresponding brink depth is 1.85 feet. Using the computer program used in Appendix B, and end depth Y_1 was found to be 0.8555 feet. This depth would correspond to a Froude number of 4.46.

(b) Stilling basin dimensions.

From Figure 4-8, the total length of the basin can be found as 11 feet.

From Figure 4-10, the height of the baffle piers h_3 is found to be 1.37 feet and the height of the end sill h_4 is also equal to 1.37 feet.

Other basic dimensions, including the distribution of chute blocks and baffle piers, follow from the design assumptions and information shown in Figure 4-9.

DESIGN C-3

CONTRA COSTA ENERGY DISSIPATOR

References

(a) Chapter 5 Pages 1-3

(b) Keim, 1962

Problem Description

A transition is to be made from a three foot wide rectangular conduct carrying 100 cfs to an open channel with an expected tailwater depth of 3.0 feet. Design a Contra Costa Energy Dissipator that will provide sufficient downstream protection.

Design Assumptions

(a) the inlet pipe is flowing partly full with an assumed flow depth at the culvert outlet of $Y_1 = 2.0$ feet.

(b) $D \leq W_c < 3D$

(c) $L_a/h_2 = 3.5$

(d) $0.6\ Y_2 \leq 0.9\ Y_2$

Design

(a) Compute L_a

$$F = \frac{V_1^2}{gY_1} = \frac{[100/(2.0)(3.0)]^2}{(32.176)(2.0)} = 4.32$$

from Figure 5-1, $h_2/Y_1 = 1.3$. Thus, $h_2 = 2.6$ feet and $L_a = 9.1$ feet.

(b) Compute L_b

$h_2/L_a = 1/3.5 = 0.2855.$

From Figure 5-1, $L_b/L_a = 1.6$. Thus, $L_b = 14.55$ feet.

(c) Determine Z.

From Figure 5-1 at $L_a/h_2 = 3.5$ $Z/h_2 = 2.C4$.

So, Z = 5.31 feet.

(d) Compute h_s.

$h_s = 0.6\ Y_2 = 1.8$ feet.

DESIGN C-4
MANIFOLD STILLING BASIN

References

(a) Chapter 5, pages 3-6.

(b) Fiala and Albertson, 1961

Problem Description

A transition is to be made from a two foot diameter pipe carrying 60 cfs to an open channel with an expected tailwater depth of 6.0 feet. Design the Manifold Stilling Basin which will provide sufficient energy dissipation to insure adequate downstream protection.

Design Assumptions

The basic geometry of the basin is illustrated in Figure 5-2, page 5-4. From Ref. (d), the suggested dimensional ratios include:

$L/B = 8$

$W/s = 0.5, 1.0$ or 2.0

$\frac{a}{V_1^2 2g}$ must be close to 0.10

for $W/s = 0.5 \quad V_1/V_o = 1.37$

$W/s = 1.0 \quad V_1/V_o = 1.24$

$W/s = 2.0 \quad V_1/V_o = 1.13$

Design

(a) Basic manifold dimensions.

inlet area, $A_1 = B^2 = H^2 = BH = \frac{\pi_1 d_1^2}{4} = 3.14 \text{ Ft}^2$

inlet velocity, $V_o = Q/D_o = 19.1$ Ft/Sec

manifold width and height, B and H = 1.37 Ft. $\therefore L = 10.95$ Ft.

The number of cross-bars is assumed to be 11 and a W/s ratio of 2.0 is initially assumed. Then, W = 0.333 Ft and S = 0.167 Ft.

(b) Basic jet geometry.

When W/s = 2.0, $V_1/V_o = 1.13$, so $V_1 = 2.16$ Ft/Sec.

From continuity, $A_1 = \frac{A_o V_o}{V_1} = 2.78 \text{ Ft}^2$

The total open area of the mainfold is (L)(B)(0.5) - (L)(B) (0.167) = 5.83 Ft^2

The percentage of area actually occupied by the jets is thus 47.3%. Now, B_o can be computed,

$$B_o = .473\ (0.333) = 0.1575 \text{ Ft}^2$$

and finally, $b/B_o = 38.1$

(c) Boil height.

$V_1{}^2/2g = 7.25$ and from Figure 5-3, when $b/B_o = 38.1$, $\frac{a}{V_1{}^2/2g} \approx 0.15$. Thus, a = 1.09 feet.

<u>Comments</u>

Best results are expected when $\frac{a}{V^{\,2}/2g}$ is reasonably close to 0.10. As a result, if the first trial differs somewhat from this value, another iteration should be made.

In actual construction, it is difficult to construct a device such as this in terms of the dimension arrived of above. Consequently, the designer should use standard dimensions commonly encountered with construction materials.

DESIGN C-5
USBR STILLING BASIN VI

References

(a) Chapter 5, pages 6, 9, and 10.

Problem Description

A transition is to be made from a four foot diameter pipe carrying 151 cfs to an open channel. Design a USBR Stilling Basin VI to accomodate this condition and protect downstream channels and appurtenances.

Design

From Table 5-1, all pertinent dimensions can be found.

W = 11 Ft. 9 in.	c = 4 Ft. 11 in.
H = 9 Ft. 0 in.	d = 2 Ft. 0 in.
L = 15 Ft. 8 in.	e = 0 Ft. 10 in.
a = 6 Ft. 9 in.	f = 3 Ft. 0 in.
b = 8 Ft. 11 in.	g = 3 Ft. 11 in.

DESIGN C-6
USU STILLING BASIN

References

(a) Chapter 5, pages 6, 11-13.

(b) Wei, 1968.

(c) Flammer, Skogerboe, Wei, and Rusheed, 1970.

Problem Description

A transition is to be made from a one foot diameter pipe carrying 20 cfs to an open channel with an expected tailwater depth of 1.5 feet. Design the USU Stilling Basin which will provide sufficient energy dissipation to insure adequate downstream protection.

Design Assumptions

The basic geometry of the basin is shown in Figure 5-6, page 5-11. From Ref. (c), optimal dimensional ratios were noted during testing of the basin, these include:

(a) $D_2/D_1 = 2.0$ (c) $W/D_1 = 0.5$

(b) $L/D_1 = 1.0$ (d) $Y_1/D_1 = 1.5$

In addition, the freeboard, f_b, should not be less than 0.5 feet.

Design

(a) Incorporating the basic design assumptions suggested in Ref. (c):

$D_2 = 2.0$ feet $W = 0.5$ feet

$L = 1.0$ feet $Y_1 = 1.5$ feet

(b) Froude Number F, may be computed as follows

$$F_1 = V_1/\sqrt{D_1 g} \qquad [5\text{-}3]$$

$$= \left(\frac{20}{\pi/4}\right)\Big/\sqrt{32.176} = 4.49$$

(c) Basic basin dimensions not already known may be determined as follows:

1. assume M = 3 where

$$m = \frac{Y_2 + Y_t}{D_1} \qquad [5\text{-}4]$$

then Y_2 = 2.5 feet

2. Knowing m and F_1, enter Figure 5-6 and get a value of Δ where,

$$\Delta = Y_b - \left[(Y_2 + Y_t)\right] / D_1 = 0.75 \qquad [5\text{-}5]$$

Therefore, Y_b = 4.75 feet

and f_b/D_1 = 0.6

Therefore, f_b = 0.6 feet

3. Compute length of stilling basin, Lb,

$$L_b = 2.5\ (D_2/D_1 - 1) + 1.0 \qquad [5\text{-}6]$$

Therefore, L_b = 3.5 feet

4. Determine stilling basin width W_b from Figure 5-6,

W_b/D_1 = 4.6

Therefore, W_b = 4.6 feet

<u>Comments</u>

Repetition of the preceding steps should be made with various values of m to arrive at the most economical design. As the value of m is decreased, the flow will become less turbulent and thus a resulting decrease in scour potential will exist in the downstream channel.

DESIGN C-7
STRAIGHT DROP SPILLWAY

References

(a) Chapter 6, pages 4-10.

(b) Donnelly and Blaisdell, 1965.

Problem Description

A drop of ten feet is to be designed between two rectangular channels carrying 20 cfs/Ft. Design a Straight Drop Sillway to facilitate this drop in a manner safe to existing downstream conditions.

Design Assumptions

(a) $L_b = X_a + 2.55\ Y_c$

(b) $X_b = 0.8\ Y_c$

(c) $X_c = 1.75\ Y_c$

(d) Floor Blocks are proportioned as follows:

height = $0.8\ Y_c$

width and spacing = $0.4\ Y_c$

square in plan should occupy 50-60% of basin width

(e) $Y_2 = 2.15\ Y_c$

(f) sidewall height = $0.85\ Y_c$ above tailwater

(g) low slope spillway crest

Design

(a) Critical depth, Y_c

$Y_c = (q^2/g)^{1/3} = 2.315$

$h/Y_c = \dfrac{10.0}{2.315} = 4.32$

(b) Computation of X_a.

From Figure 6-4, $X_a/Y_c = 4.6$ when $h/Y_c = 4.32$ and $Y_2 = 5$ feet.

Thus, X_a = 4.6 (2.315) = 10.75 feet

and L_b = 10.75 + 5.95 = 16.70 feet.

(e) Other basic dimensions

X_b = 1.87 feet

X_c = 4.1 feet

floor blocks: height = 1.87 feet

width spacing 0.9 feet

and sill = 0.93 feet

side wall height from floor = 7 feet.

Comments

Both the conditions upstream and downstream from the drop have considerable influence on the adequacy of the design. Any uncertainties, particularly with respect to the tailwater depth, must be provided for by lowering the stilling basin floor to insure that the hydraulic jump occurs in the stilling basin.

DESIGN C-8
VERTICAL DROP STRUCTURE
WITH DISSIPATION BARS

References

(a) Chapter 6, pages 10, 12-22.

(b) Katsaitis, 1966.

Problem Description

A drop in elevation of six feet is to be designed for the transition between two rectangular channels carrying 10 cfs/Ft. Design a vertical drop structure with dissipation bars to accomplish this drop in a manner which will protect all downstream facilities. Note that the crest is not in weir form. (See Figure 6-6).

Design Assumptions

(a) From elementary principles of dynamics, the following relatio ships can be developed. (Refer to Figure 6-7 and Equations 6-10 through 6-24.)

Design

(a) Total specific energy above crest is

$H = Y_c + V_c^2/2g = 2.19$ feet

h_1 = drop height = 6 feet

(b) Basic flow geometry

$Z = h_1 + 0.373H = 6.0 + .815 = 6.815$ feet 6-14

$V_x^2 = 0.672H(2g) = 40.379H = 88.4\ V_x = 7.64$ fps 6-15

$V_z^2 = 2gZ = 64.42 = 439,\ V_z = 20.95$ fps 6-16

$V^2 = V_x^2 + V_z^2 = 479.379,\ V = 21.9$ fps 6-17

$g/V^2 = .0671$

$(H'/2) = 0.435\ H = 0.95$

$t^2 = (2Z/g) = .0622Z = .424$, $t = .651$ sec 6-18

$x = V_x t = 4.96$ feet

$L_2 = (H'/2) + X = 5.91$ feet 6-19

$L_1 = 1.5L_2 = 8.85$ feet 6-20

$L_3 = W = 0.04/(g/V^2) = 0.596$ feet 6-24

$D = 0.17/(g/V^2) = 2.53$ feet 6-21

$M = 2.738H\ (g/V^2) = .409$ 6-23

$Y_s/H = M - 0.2177 = .1913$ 6-23

$Y_s' = .419$ feet 6-23

Comment

The designer should note that the preceding design is based on a step-by-step procedure outlined in Tables 6-1, and 6-3.

DESIGN C-9
RIPRAP FILTER MATERIAL

References

(a) Chapter 8, pages 8-33 to 8-36.

Problem Description

The range of allowable size distributions for a filter material is to be determined. A sample of a local source of riprap was run through a series of sieves and the weight of sampie retained on each sieve is listed below. A respresentative sample of the soil through which the channel is to be excavated yielded the size distribution listed below.

Channel Excavation		Riprap	
Sieve Opening in inches	Weight Retained in Grams	Sieve Opening in inches	Weight Retained in Pounds
0.50	68	18	77
0.25	141	15	108
0.125	182	12	124
0.08	217	9	67
0.03	161	6	34
0.02	133	3	21
0.01	90	2	10
Total Weight	992	1	4
		Total Weight	445

Design Assumptions

(a) For a given open channel, it was determined that the required riprap size at the design discharge have a minimum value of C_{50} of 11.5 inches. First of all, a determination must be made if a filter layer is required. Then, if a filter is required, the gradation requirements for the filter layer must be established to insure that the base material will not be eroded.

(b) The criteria which are frequently used in determining whether a filter layer is required can be defined as

$$\frac{D_{15}\ \text{Riprap}}{D_{85}\ \text{Base}} < 5 \qquad \text{Equation 8-23}$$

$$5 < \frac{D_{15}\ \text{Riprap}}{D_{15}\ \text{Base}} < 40 \qquad \text{Equation 8-24}$$

$$\frac{D_{50}\ \text{Riprap}}{D_{50}\ \text{Base}} < 40 \qquad \text{Equation 8-25}$$

Where D_{15}, D_{50}, and D_{85} are the sizes of riprap and base material of which 15, 50, and 85 percent are finer. If these criteria (Equations 8-23, 8-24, and 8-25) are not met, then a filter layer is required.

(c) If a filter layer is reauired. the following equations can be used to determine the size distribution of the particles for the filter material.

$$\frac{D_{15}\ \text{Riprap}}{D_{85}\ \text{Filter}} < 5 \qquad \text{Equation 8-26}$$

$$5 < \frac{D_{15}\ \text{Riprap}}{D_{15}\ \text{Filter}} < 40 \qquad \text{Equation 8-27}$$

$$\frac{D_{50}\ \text{Riprap}}{D_{50}\ \text{Filter}} < 40 \qquad \text{Equation 8-28}$$

$$\frac{D_{15}\text{Filter}}{D_{85}\ \text{Base}} < 5 \qquad \text{Equation 8-29}$$

$$5 < \frac{D_{15}\text{Filter}}{D_{15}\ \text{Base}} < 40 \qquad \text{Equation 8-30}$$

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Base}} < 40 \qquad \text{Equation 8-31}$$

Design

From the data obtained by the sieve analysis, the values of D_{15}, D_{50}, and D_{85} were calculated.

Channel Excavation	Riprap
D_{15} = 0.0245 inches	D_{15} = 8.8 inches
D_{50} = 0.11 inches	D_{50} = 14.0 inches
D_{85} = 0.335 inches	D_{85} = 18.3 inches

These values are then used in equations 8-23, 8-24, and 8-25 to determine if a filter is needed.

Eq. 8-23: $\frac{D_{15}\ \text{Riprap}}{D_{85}\ \text{Base}} < 5 \quad \frac{8.8\ \text{inches}}{0.335\ \text{inches}}$ = 26.3 which is not less than 5

Eq. 8-24 $5 < \frac{D_{15}\ \text{Riprap}}{D_{15}\ \text{Base}} < 40 \quad \frac{8.8\ \text{inches}}{0.0245\ \text{inches}}$ = 359 which is not less than 40

Eq. 8-25 $\frac{D_{50}\ \text{Riprap}}{D_{50}\ \text{Base}} < 40 \quad \frac{14\ \text{inches}}{0.11\ \text{inches}}$ = 1.27 which is not less than 40

Therefore, it can be concluded that filter is required.

Knowing that a filter is required, the next step is to determine the values of D_{15}, D_{50}, and D_{85} for a suitable filter material. By using equations 8-26 through 8-31, these gradation values can be calculated.

Eq. 8-26: $\dfrac{D_{15}\text{ Riprap}}{D_{85}\text{ Filter}} < 5$

Solving for D_{85} filter, this becomes

$$D_{85}\text{ filter} > \frac{D_{15}\text{ Riprap}}{5}\text{ . Therefore,}$$

$$D_{85}\text{ filter} > \frac{8.8\text{ inches}}{5} > 1.76\text{ inches}$$

So, D_{85} filter must be greater than 1.76 inches.

Eq. 8-27: $5 < \dfrac{D_{15}\text{ Riprap}}{D_{15}\text{ filter}} < 40.$

Solving for D_{15} filter this becomes:

$$D_{15}\text{ filter} < \frac{D_{15}\text{ Riprap}}{5}\quad\text{and}$$

$$D_{15}\text{ filter} > \frac{D_{15}\text{ Riprap}}{40}\text{ . Therefore,}$$

$$D_{15}\text{ filter} < \frac{8.8\text{ inches}}{5} = 1.76\text{ inches and}$$

$$D_{15}\text{ filter} > \frac{8.8\text{ inches}}{40} = 0.22\text{ inches.}$$

Eq. 8-28 $\dfrac{D_{50}\text{ Riprap}}{D_{50}\text{ Filter}} < 40.$

Solving for D_{50} filter,

$$D_{50}\text{ Filter} > \frac{D_{50}\text{ Riprap}}{40}$$

$$D_{50} \text{ filter} > \frac{14.0 \text{ inches}}{70} = 0.35 \text{ inches}$$

Therefore, the filter that will work with the riprap is described as follows:

0.22 inches < D_{15} filter < 1.76 inches, D_{50} filter > 0.35 inches, and D_{85} filter > 1.76 inches.

Eq. 8-29: $\dfrac{D_{15} \text{ filter}}{D_{85} \text{ Base}} < 5.$

Solving for D_{15} filter,

$$D_{15} \text{ filter} < 5\ (D_{85} \text{ Base}).$$

Therefore,

$$D_{15} \text{ filter} < 5\ (0.335 \text{ inches}) = 1.68 \text{ inches}$$

Eq. 8-30: $5 < \dfrac{D_{15} \text{ filter}}{D_{15} \text{ Base}} < 40.$

Solving for D_{15} filter this becomes:

$$D_{15} \text{ filter} > 5\ (D_{15} \text{ Base})$$

and

$$D_{15} \text{ filter} < 40\ (D_{15} \text{ Base})$$

Therefore,

$$D_{15} \text{ filter} > 5\ (0.0245) = 0.123 \text{ inches}$$

$$D_{15} \text{ filter} < 40\ (0.0245) = 0.98 \text{ inches}$$

Eq. 8-31: $\dfrac{D_{50} \text{ filter}}{D_{50} \text{ Base}} < 40$

Solving for D_{50} filter

$$D_{50} \text{ filter} < 40\ (D_{50} \text{ Base})$$

so,

$$D_{50} \text{ filter} < 40\ (0.11) = 4.4 \text{ inches}$$

Therefore, the filter that must be used with the base is described as follows:

0.123 inches < D_{15} filter < 0.98 inches and D_{50} filter < 4.4 inches

In order to obtain a better feeling for how all of the numbers are related to each other, they are plotted as shown in Figure C-1. Here, the upper and lower limits of D_{15}, D_{50}, and D_{85} that were calculated are plotted and smooth curves are drawn through the points. The area between the highest minimum size curve and the lowest maximum size curve (cross hatched area) is the range through which the values for the filter material may vary.

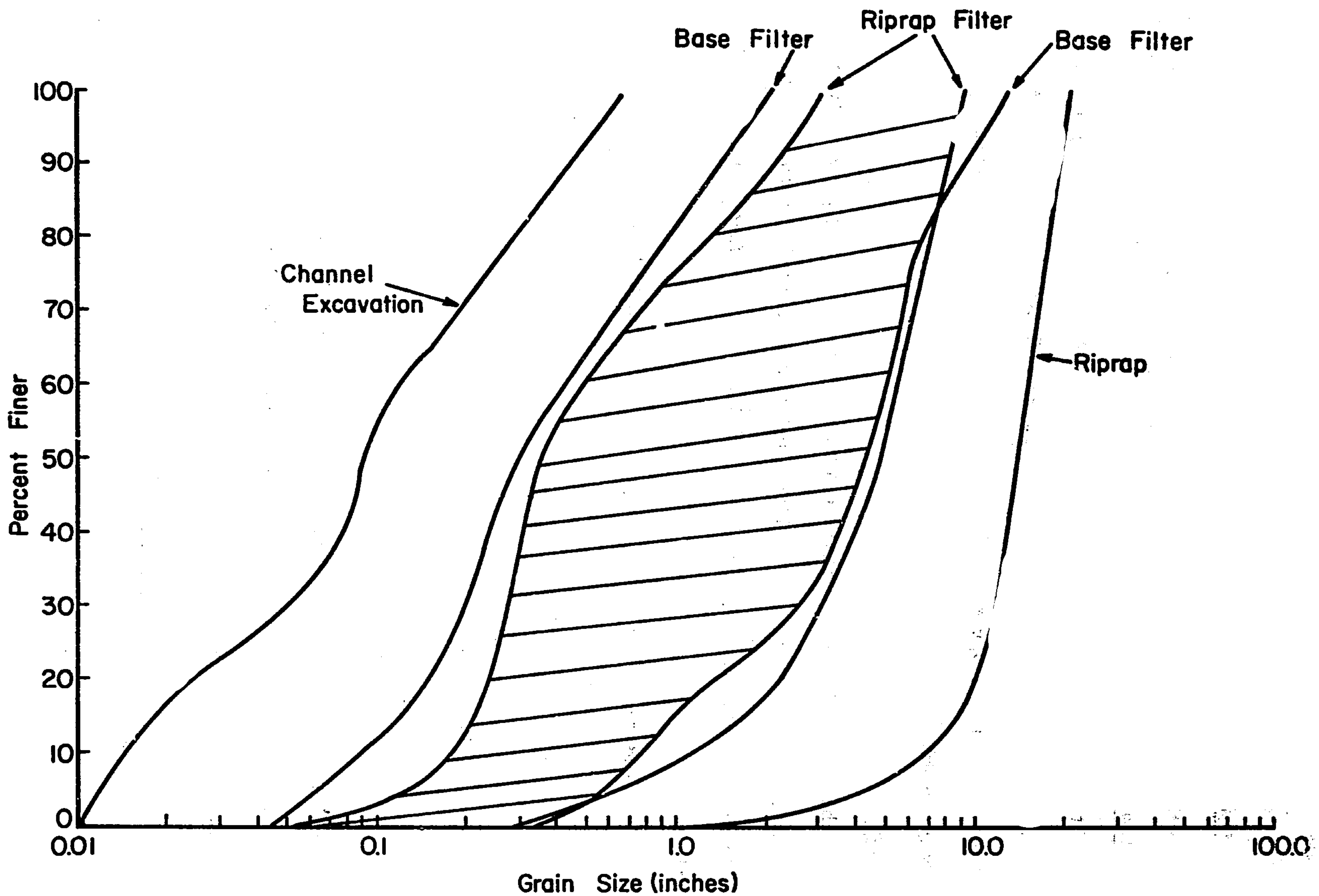


Figure C-1. Example of allowable filter gradation.

DESIGN C-10
RIPRAP REQUIREMENTS

Reference

(a) Chapter 8, pages 17-19.

Problem Description

It is necessary to determine the size of riprap required for the following trapezoidal channel. Flow rate = 50 cfs; Bottom width = 5 feet, and side slope = 2.5:1. Twobed slopes will be investigated; namely, 0.001 and 0.01.

Design

The design has to be divided into two parts; namely, the size of riprap required for the channel bed and that required for the sloping side walls. For the bed slope, S_b, of 0.001, the design proceeds as follows:

To find the value of D_{50} required, Equation 8-20 is used

$$Q = \frac{1}{118} \frac{D_{50}^{5/2}}{S_b^{13/6}} \frac{P}{R}$$

Solving for D_{50} yields

$$D_{50} = \left[\frac{Q\ 118\ S_b^{13/6}}{P/R}\right]^{2/5}$$

The value of P/R has been found to vary from a value of 13.3 to 30. Using these two values as upper and lower limits, the corresponding values of D_{50} are obtained.

$$D_{50} = \left[\frac{50\ \text{ft}^3/\text{sec}\ (118)(1 \times 10^{-3})^{13/6}}{13.3}\right]^{2/5} = 0.0282 \text{ ft, or}$$

8.6 mm.

$$D_{50} = \left[\frac{50\ ft^3/sec\ (118)(1 \times 10^{-3})^{13/6}}{30.0}\right]^{2/5} = 0.0022\ ft, \text{ or}$$

6.7 mm.

Since both values of D_{50} are reasonably close together, the upper value would be selected. The reason for this is to provide a margin of safety. Therefore, the value of D_{50} for use on the bed of the channel would be selected as 9 mm, which corresponds to a value of approximately 0.03 feet.

Since the riprap on the side of the channel also has a force due to gravity acting in such a way as to cause it to move down to slope, the stones must be larger in order to have an equal resistance to movement as those on the channel bed. Equation 8-14 is used to calculated a proportionality constant K which relates the value of D_{50} for the bed to D_{50} of the side, where:

$$K = \frac{T_{cs}}{T_{cb}} = \sqrt{1 - \frac{5\ in^2\ \phi}{5\ in^2\ \theta}} \quad \text{.................(8-14)}$$

The values of θ and ϕ are obtained from Figs. 8-12 and 8-13, respectively.

Therefore, assuming that crushed rock is to be used, the value of D_{50} for the sides is calculated as follows:

$$\theta = 39^\circ$$

$$\phi = 22^\circ$$

Solving for K,

$$K = \sqrt{1 - \frac{5\ in^2\ (22^\circ)}{5\ in^2\ (39^\circ)}} = 0.799, \text{ say } 0.8$$

Then,

$$D_{50s} = \frac{D_{50b}}{K}$$

So,

$$D_{50s} \quad \frac{0.03 \text{ ft}}{0.8} \quad 0.0375 \text{ ft, or } 11.4 \text{ mm.}$$

Therefore, since this is the minimum value for D_{50s}, the riprap for use on the sides would be selected as having a D_{50} of at least 12 mm.

For a value of bed slope equal to 0.01, the proceedure would be the same and proceed as follows:

$$D_{50} = \left[\frac{50 \text{ ft}^3/\text{sec } (118)(0.01)^{13/6}}{13.3}\right]^{2/5} = 0.21 \text{ ft.} = 64 \text{ mm.}$$

$$D_{50} = \left[\frac{50 \text{ ft}^3/\text{sec } (118)(0.01)^{13/6}}{30.0}\right]^{2/5} = 0.15 \text{ ft.} = 46 \text{ mm.}$$

Therefore, to be conservative, a D_{50} of 64 mm. could be used, although a riprap size between 46 mm and 64 mm could be used, depending on the value of P/R actually used.

From Tables 8-12 and 8-13, we find that $\theta = 41.5^{\circ}$ and $\phi = 23.5^{\circ}$ for crushed rock.

Therefore,

$$K = \sqrt{1 - \frac{5 \text{ in}^2 (23.5)}{5 \text{ in}^2 (41.6)}} = 0.799, \text{ say } 0.8$$

Then,

$$D_{50s} = \frac{0.21 \text{ ft}}{0.8} = 0.262 \text{ ft, or } 81 \text{ mm.}$$

DESIGN C-11
DESIGN OF RIPRAP LINED CHANNEL

Reference

(a) Chapter 8, pages 14-32.

Problem Description

A riprap lined trapezoidal channel is required to carry 50 cfs with a bed slope S_b of 0.001.

Design

The limiting values of P/R for trapezoidal channedls are 13.3 and 30.0. The riprap requirement for each is computed using Equation 8-20.

$$Q = \frac{1}{118} \frac{(D_{50})^{5/2}}{(S_b)^{13/6}} \quad P/R$$

Solving for D_{50},

$$D_{50} = \left[\frac{Q(118)(S_b)^{13/6}}{P/R}\right]^{5/2}$$

The flow velocity is then calculated using Equation 8-18, where

$$V = 4.60 \frac{D_{50}^{\frac{1}{2}}}{S_b^{1/6}}$$

Knowing the flow rate Q and the flow velocity F, the cross-sectional area of flow is calculated from the continuity equation.

$$A = Q/V$$

For a P/R value of 13.3,

$$D_{50} = \left[\frac{50(118)(1 \times 10^{-3})^{13/6}}{13.3}\right]^{2/5}$$

$$= 2.84 - 10^{-2} \text{ ft, or } 8.66 \text{ mm.}$$

This same value can be obtained using Fig. 8-7.

Therefore, a value for D_{50} of about 2.5×10^{-2} ft, or 8 mm, would be selected.

The allowable velocity in the channel would be,

$$V = 4.6 \frac{(2.5 \times 10^{-2})^{1/2}}{(1 \times 10^{-3})^{1/6}}$$

$$= 2.3 \text{ feet per second}$$

The value of the hydraulic radius R can now be determined using Fig. 8-10.

R = 1.1

Knowing the flow rate Q and the flow velocity V, the cross-sectional area can be calculated.

$$A = \frac{50 \text{ ft}^3/\text{sec}}{2.3 \text{ ft/sec}} = 21.7 \text{ ft}^2$$

For use with the figures, this can be rounded to 22 ft^2.

The geometry of the channel can now be determined using Figs. 8-14, 8-15, or 8-17. Since these graphs each represent a different value of side slope Z, three different channel geometries will be obtained. It should also be noted that these graphs are prepared in such a way as to produce geometries such that the material on the sides is as stable as that on the bottom.

Channel 1: Z = 1.5, B = 17 ft, Y = 1.3 ft (Fig. 8-14)

Channel 2: Z = 2.0, B = 16 ft, Y = 1.35 ft (Fig. 8-15)

Channel 3: Z = 3.0, B = 11 ft, Y = 1.5 ft (Fig. 8-17)

The value of Y obtained for each channel is the actual water depth and does not contain any allowance for channel freeboard. To obtain the channel depth required, the freeboard would have to be added. For this problem, the freeboard should be at least 0.5 ft.