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Hydraulic Design of Energy Dissipators for Culverts and Channels

Hydraulic Engineering Circular No. 14

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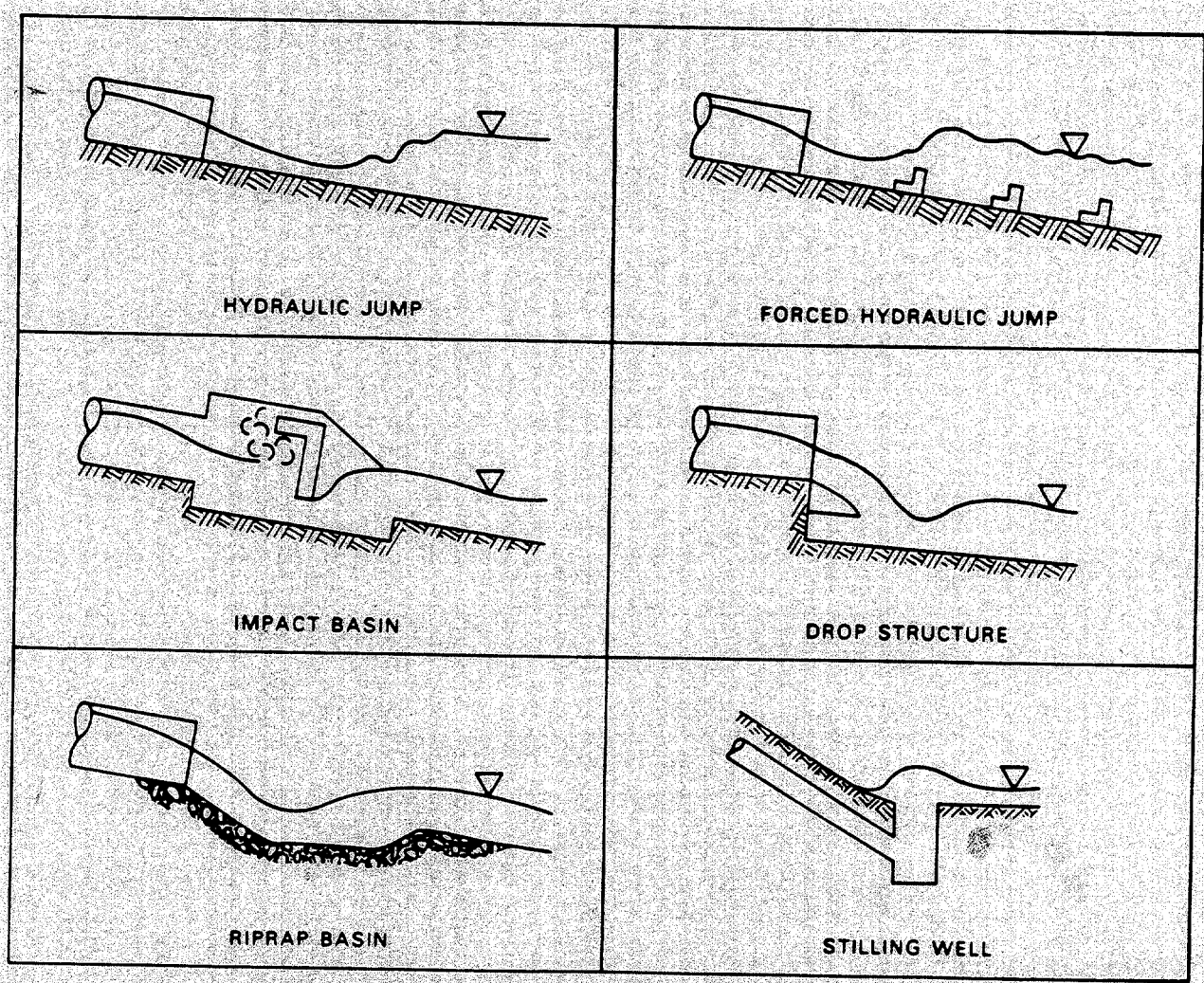

PREFACE

The purpose of this circular is to provide design information for analyzing energy dissipation problems at culvert outlets and in open channels. The first five chapters of the circular provide general information to support the remaining design chapters. Design chapters VI-XI cover the general types of dissipators: hydraulic jump, forced hydraulic jump, impact, drop structure, stilling well, and riprap. The design concept presented in chapter I is illustrated in chapter 12, Design Selection. In this chapter, the different dissipator types are compared using design problems.

Much of the information presented has been taken from the literature and adapted, where necessary, to fit highway needs. Recent research results have been incorporated, wherever possible, and a field survey was conducted to determine the States' present practice and experience.

ACKNOWLEDGMENTS

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The authors wish to express their gratitude to the many individuals and organizations whose research and designs are incorporated into this circular.

TABLE OF CONTENTS

THE HYDRAULIC DESIGN OF ENERGY DISSIPATORS

Chapter Subject	Page
Preface and Acknowledgments	i
Table of Contents	ii
List of Symbols	v
Metrication Conversion	vii
I. Design Concept	I
II. Erosion Hazards	II-1
A. Erosion Hazards At Culvert Inlets	II-1
Depressed Inlets	II-1
Headwalls and Wingwall	II-2
Inlet Failures	II-3
B. Erosion Hazards At Culvert Outlets	II-3
Types of Scour	II-4
Standard Culvert Outlet Treatments	II-5
C. Riprap Protection	II-5
III. Culvert Outlet Velocity and Velocity Modification	III-1
On Mild Slopes	III-1
On Steep Slopes	III-3
IV. Flow Transitions	IV-1
A. Culverts in Outlet Control	IV-1
Abrupt Expansion	IV-3

Design of Subcritical Flow Transitions IV-6

B. Culverts in Inlet Control IV-18

Supercritical Flow Contraction IV-18

Supercritical Flow Expansion IV-25

V. Estimating Erosion of Culvert Outlets V-1

VI. Hydraulic Jump VI-1

A. Nature of the Hydraulic Jump VI-1

B. Hydraulic Jump Expressions VI-4

Horizontal Channels VI-4

Sloping Channels VI-12

C. Jump Efficiency VI-16

VII. Forced Hydraulic Jump and Increased Resistance

A. Colorado State University (CSU) Rigid Boundary Basin VII-A-1

B. Tumbling Flow in Culverts VII-B-1

C. Increased Culvert Resistance VII-C-1

D. U.S. Bureau of Reclamation (USBR) Type II . . VII-D-1

E. USBR Type III VII-E-1

F. USBR Type IV VII-F-1

G. St. Anthony Falls (SAF) Basin VII-G-1

VIII. Impact Basins

A. Contra Costa VIII-A-1

B. Hook VIII-B-1

C. USBR Type VI VIII-C-1

D. Forest Service (Metal) VIII-D-1

IX. Drop Structures IX-1

X. Stilling Wells

A. Manifold Type X-A-1

B. Corps of Engineers X-B-1

XI. Riprap XI-1

XII. Design Selection XII-1

Appendix A (Computation Forms) A-1

LIST OF SYMBOLS

Note: For specific definitions refer to each chapter.

A	Cross-sectional area of flow
a	Acceleration
B	Width of rectangular culvert barrel
c	Proportionality constant; subscript for critical conditions
D	Diameter or height of culvert barrel
E	Energy
Fr	Froude number
f	Darcy-Weisbach resistance coefficient
F	Force
g	Acceleration of gravity
H	Energy head
h	Vertical dimension
H_L	Head loss (total)
H_f	Friction head loss
L	Distance, length, longitudinal dimension
M	Momentum
m	Mass
N	Number
n	Manning roughness coefficient; coordinate normal to flow direction
o	Subscript for culvert outlet parameters
P	Wetted perimeter
p	Pressure
Q	Discharge

q	Discharge per unit width
R	Hydraulic radius
Re	Reynolds number
r	Radius; cylindrical coordinate
S	Slope
S_f	Slope of energy grade line
S_o	Slope of the bed
S_w	Slope of the water surface
T	Top width of water surface
TW	Tailwater Depth
t	Time variable; thickness dimension
V	Mean velocity
$\forall$	Volume
v	Velocity at a point
W	Transverse dimension; width
w	Weight
y	Depth of flow
y_e	Equivalent depth = $(A/2)^{1/2}$
y_m	Hydraulic depth: area/top width (A/T)
y_n	Normal depth of flow
y_c	Critical depth of flow
z	Side slope; stream bed elevation
Z	Water surface elevation

α Kinetic energy coefficient; inclination angle

β Velocity (momentum) coefficient; wave front angle

γ Specific weight

Δ Small increment

θ Angle: inclination, contraction, central

μ Dynamic viscosity

ν Kinetic viscosity

ρ Mass density of fluid (1.94 slugs/cu. ft. for water)

Σ Summation symbol

τ Shear stress

Conversion Factors for British to Metric Units

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
feet per second	0.3048	meters per second
cubic feet per second	0.028317	cubic meters per second
pounds	0.453592	kilograms

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

THE HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS

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I. DESIGN CONCEPT

The failure of many highway culverts can be traced to unchecked erosion. Erosive forces which are at work in the natural drainage network are often increased by the construction of a highway. Interception and concentration of overland flow and constriction of natural waterways inevitably results in an increased erosion potential. To protect the highway and adjacent areas, it is sometimes necessary to employ an energy dissipating device. These devices cover a wide range in complexity and cost and the particular type selected will depend on the assessment of the erosion hazard. This assessment includes determining the ability of the natural channel to withstand erosive forces and the scour potential represented by the superimposed flow conditions. The purpose of this circular is to aid in selecting and designing an energy dissipator which will meet the requirements indicated by an erosion hazard assessment.

Energy dissipators should be considered part of a larger design system which includes the culvert, channel protection requirements (both upstream and down), and may include a debris control structure. When viewed from this standpoint, much of the input data will be available to the energy dissipator design phase from previous design steps. For example, the culvert design should provide: the design discharge; outlet flow conditions--velocity and depth; culvert type--size, shape, and roughness; culvert slope; operating characteristics--performance curve; and the standard culvert outlet design utilized--projecting, wingwalls, headwall, aprons, etc.

Much of the location data will also serve more than one design segment in the overall process. Vicinity and contour maps are essential to culvert, dissipator and channel designs. A debris assessment is a necessary input to selecting both a debris control structure and energy dissipator. The allowable scour estimate, which is related to location, is a design as well as a selection parameter.

Information generated as input and part of the energy dissipator design output will also be useful in subsequent design phases. The channel characteristics--slope, cross section, normal depth and velocity, bed and bank materials along with the flow characteristics at the dissipator exit, velocity and depth--are all essential to the design of channel protection.

These common data input and output requirements, although very important, are only one reason for considering the culvert, debris control, energy dissipator, and channel protection designs as an integrated system. The interrelationship of the various parts or individual designs within the system must be considered. For example, energy dissipators can change culvert performance and channel protection requirements; some debris-control structures represent losses not normally considered in the culvert design procedure; energy dissipator requirements might be substantially reduced, increased, or possibly eliminated by changes in the culvert design; and downstream channel conditions--velocity, depth and channel stability are important considerations in energy dissipator selection and design.

The designer might also consider energy dissipator design as a a minisystem involving numerous energy dissipation schemes with overlapping selection criteria. A combination of dissipator and channel protection might be used to solve specific problems. Figure I-1, "Conceptual Model--Energy Dissipator Design," indicates the input, output, and the various steps in the energy dissipator selection and design process. As indicated on the flow chart, the process begins by considering the standard design terminal structures normally employed. The initial step is to determine the flow conditions at the exit of the standard transition outlet and using these conditions estimate the scour which might be expected if the downstream channel were composed of unconsolidated sand.

This estimate represents an extreme condition; but, by comparing it with the subjective judgment of the erodibility of the actual material present in the channel, the designer is provided with a qualitative measure of the magnitude of the local erosion problem. This input, considered along with data on the long-term stability of the downstream channel which is discussed in the chapter on Erosion Hazards, enables the designer to reach a preliminary decision on energy dissipator needs.

The decision may be that no protection is required; that minimal protection and monitoring after each runoff event is needed; or that an energy dissipator or combination energy dissipator and channel protection is necessary.

Throughout the selection and design process, the designer should keep in mind that his primary objective is to protect the highway structure and adjacent area from excessive damage due to erosion. One way to help accomplish this objective is to return the flow to the downstream channel in a condition which approximates the natural flow regime. This also implies guarding against overdesign or employing dissipation devices which reduce flow conditions substantially below the natural or normal channel conditions.

If scour computation indicates the need for an energy dissipator, a logical next step is to investigate the possible ways of reducing or eliminating this need by modifying the outlet velocity or erosion potential. This involves analyzing the effects of various alterations of the culvert characteristics--changing slope, roughness, etc. These are discussed in the chapter on Outlet Velocity and design chapter VII. The cost of the culvert alteration and its effects on culvert performance compared with the cost of providing an energy dissipator are all important considerations in this investigation.

Preliminary energy dissipator selection is made by comparing the input constraints or design criteria--flow regime, debris problems, location, channel characteristics, allowable scour, etc.--to the attributes of the various energy dissipators.

This process may result in the selection of several energy dissipator designs or combination of designs which substantially satisfy the design criteria. Each situation is unique, however, and compromise between the various elements of the system and the exercise of engineering judgment will always be necessary.

Flow transition design, the next step in the process, is an essential part of many dissipator designs. The Flow Transition chapter provides guidance for the selection and design of this important appurtenance. Most situations encountered involve supercritical flow, indicating transitions must be carefully designed in order to minimize wave and flow separation problems and to provide uniform flow conditions at the dissipator entrance.

The individual dissipator designs have been qualified as to their area of application. The attributes delineated include: Froude number range for best performance; discharge velocity or other limitations; possible maintenance, operational, or location problems; maximum size; limiting characteristics such as culvert slope or shape.

The design output includes the detailed design information and sufficient data to make the final design selection or to indicate that a different design or designs should be considered. Design selection is discussed and detailed design problems and procedures are provided in the last chapter of the manual.

The circular contains sections which discuss erosion hazards and provide guidance on velocity reduction, flow transition designs, as well as a procedure for estimating scour in a sand bed channel. The design of free hydraulic jumps for various channel shapes and slopes is included along with energy dissipator designs which utilize forced hydraulic jumps. The design of several types of impact basins, drop structures, and stilling well or vertical flow devices are included. The last design chapter deals with the riprap basin.

Although it was not always possible, every effort has been made to treat energy dissipator design as illustrated by the conceptual model. The weakest areas are the initial scour determination and the economic data necessary for the selection process.

Throughout the circular, an attempt has been made to relate the designs to actual situations through example problems. Examples of the application of each type of energy dissipator are presented in chapter 12. Each of the design chapters includes the best available design information to date. The entire manual should be considered as a dynamic framework within which material will be added and deleted as new information becomes available.

CONCEPTUAL MODEL - ENERGY DISSIPATOR DESIGN

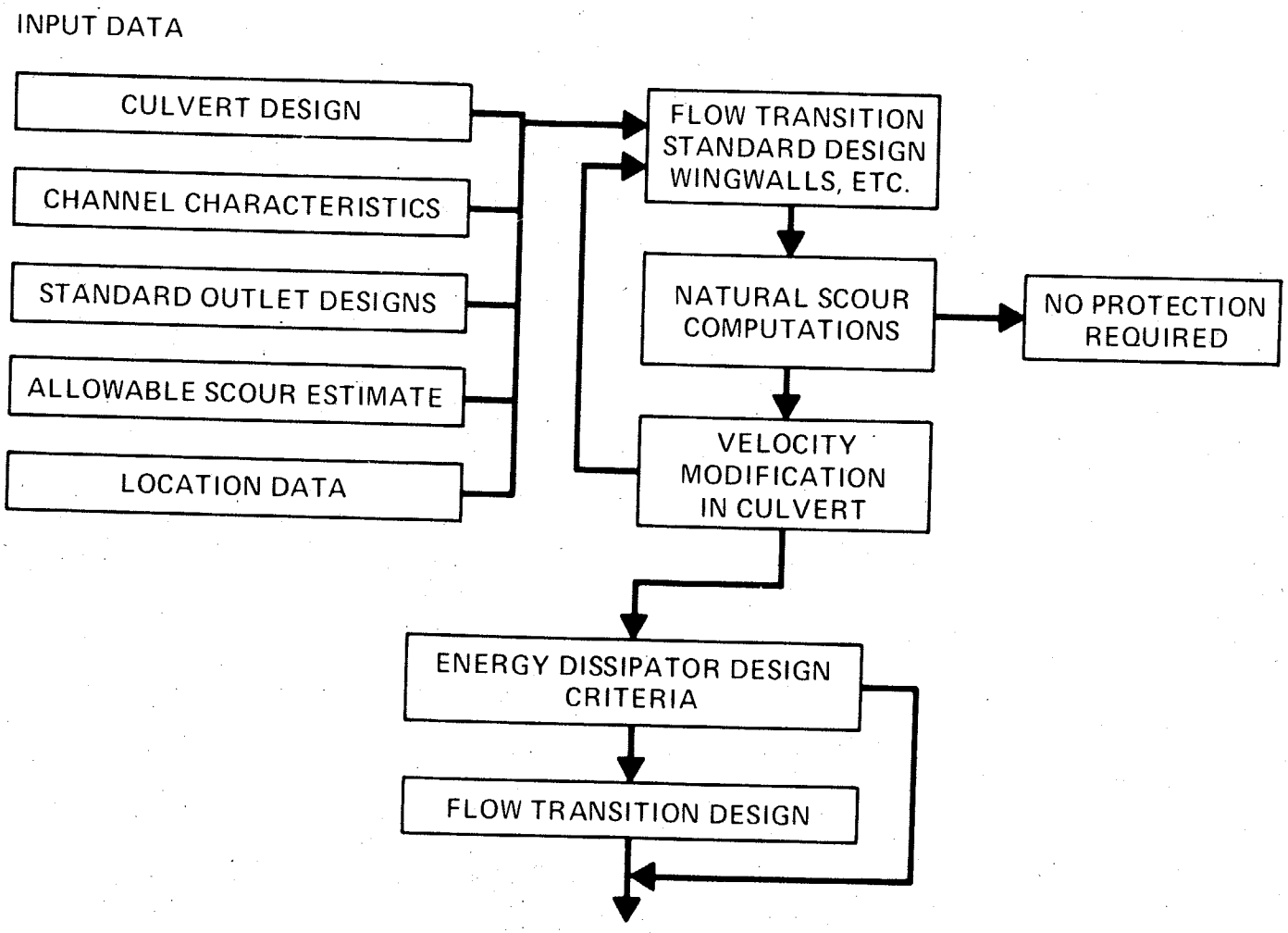


FIGURE I-1 CONCEPTUAL MODEL

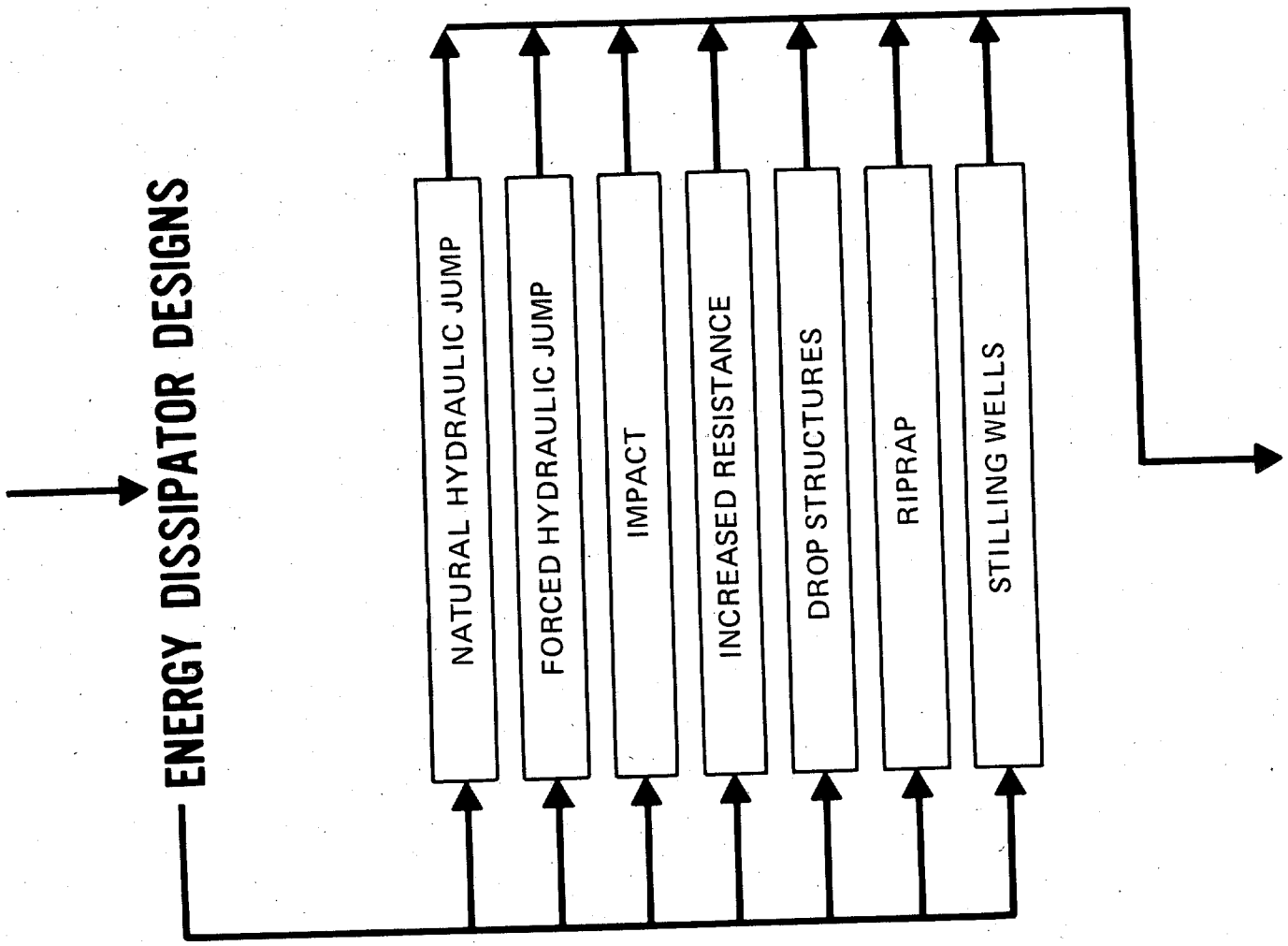


FIGURE I-1 CONCEPTUAL MODEL

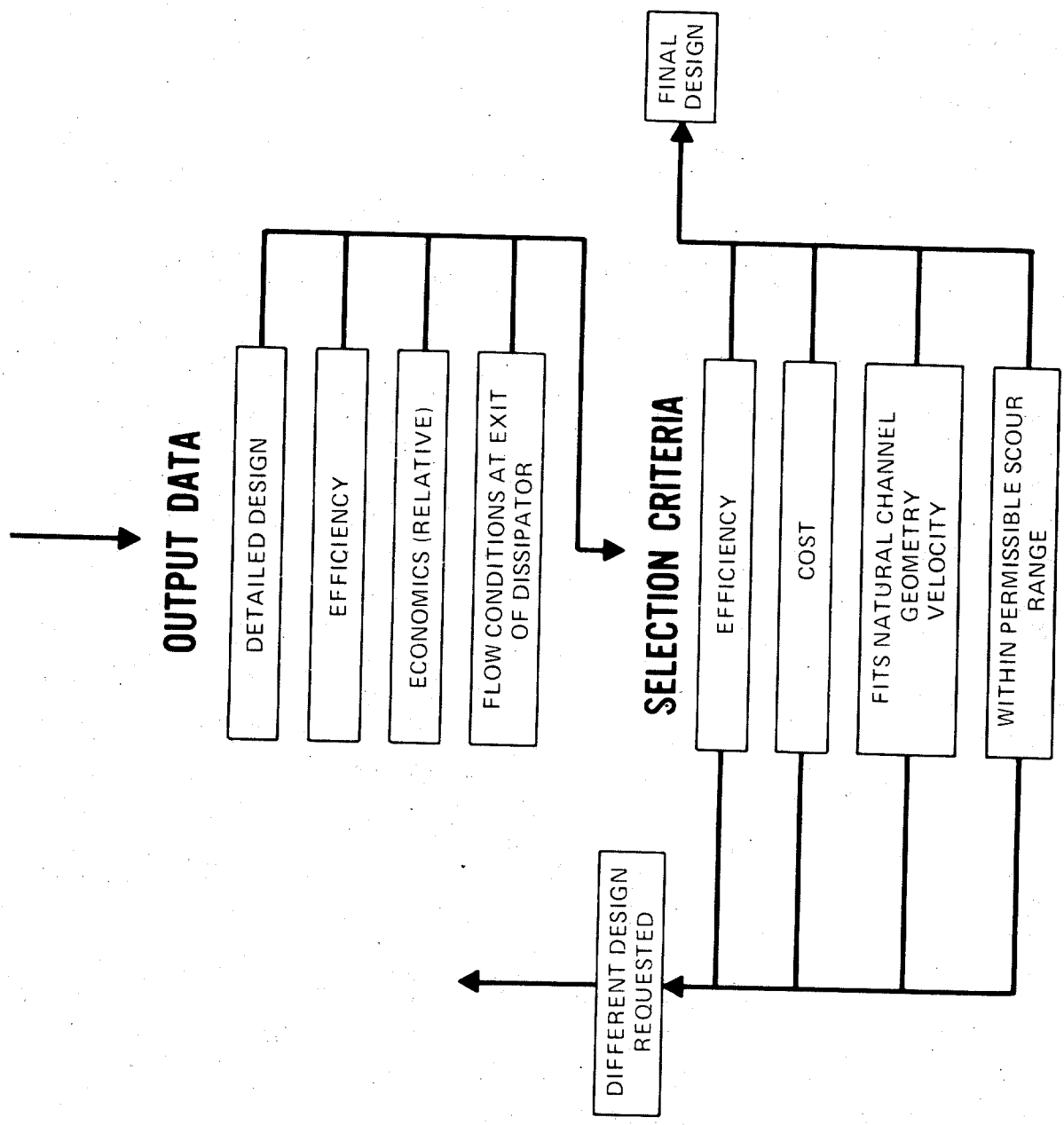


FIGURE I-1 CONCEPTUAL MODEL

II. EROSION HAZARDS

II-A - EROSION HAZARDS AT CULVERT INLETS

Erosion from vortexes, flow over wingwalls, and fill sloughing at culvert inlets is generally not a major problem. There are some exceptions, however. For example, where a confined approach channel is not aligned with the culvert axis, some degree of protection may be required. The area of greatest potential damage is on the outside of a sharp bend where the flow must turn to enter the culvert.

At design discharge, water will normally pond at the culvert inlet and flow from this pool will accelerate over a relatively short distance. Significant increases in velocity only extend upstream from the culvert inlet a distance about equal to the height of the culvert. Velocity near the inlet may be approximated by dividing the flow rate by the area of the culvert opening. The risk of channel erosion should be judged on the basis of this approach velocity.

It is essential that any protection provided also be adequate for flow rates less than the maximum design rate, since depth of ponding at the inlet is less and greater velocities may occur. This is especially true in channels with steep slopes where high velocity flow prevails.

DEPRESSED INLETS

Culvert inverts are sometimes placed below existing channel grades to increase culvert capacity or to meet minimum cover requirements. The depression may result in progressive degradation of the upstream channel unless resistant natural material or channel protection is provided. Hydraulic Engineering Circular No. 13 (II-A-3) discusses the advantages of providing a depression or fall at the culvert entrance to increase culvert capacity.

Culvert invert depressions of 1 or 2 feet are usually adequate to obtain minimum cover, and may be readily provided by modification of the concrete apron. The drop may be provided in two ways. A vertical wall may be constructed at the upstream edge of the apron, from wingwall to wingwall,

or where a drop may be considered undesirable, the apron slab may be constructed on a slope to reduce or eliminate the vertical face.

Caution must be exercised in attempting to gain the advantages of a lowered inlet where placement of the outlet flowline below the channel would also be required. Locating the entire culvert flowline below channel grade may result in deposition problems.

HEADWALLS AND WINGWALLS

Recessing the culvert into the fill slope and retaining the fill by either a headwall parallel to the roadway or by a short headwall and wingwalls does not produce significant erosion problems. This type of design decreases the culvert length and enhances the appearance of the highway by providing culvert ends that approximately conform to the embankment slopes. A vertical headwall parallel to the embankment shoulder line should have sufficient length so that the embankment spill cones remain clear of the culvert opening. Normally riprap protection of these spill cones is not necessary if slopes are sufficiently flat to remain stable when wet.

Wingwalls flared with respect to the culvert axis are commonly used and are more efficient than parallel wingwalls. The effects of various wingwall placements upon culvert capacity are discussed in HEC Nos. 5, 10, and 13 (II-A-1,2,3). Use of a minimum practical wingwall flare has the advantage of reducing the area requiring protection against erosion. The flare angle for the given type of culvert should be consistent with the recommendations of HEC No. 13.

If flow velocity near the inlet indicates a possibility of scour threatening the stability of wingwall footings, erosion protection should be provided. A concrete apron between wingwalls is the most satisfactory means for providing this protection. The slab has the further advantage that it may be reinforced and used to support the wingwalls as cantilevers.

It is not necessary to extend an inlet headwall (with or without wingwalls) to the maximum design headwater elevation. With the inlet and the slope above the headwall submerged, velocity of flow along the slope is low. Even with easily erodible soils, a vegetative cover is usually adequate protection in this area.

INLET FAILURES

Most of the inlet failures reported have occurred on large flexible-type pipe culverts with projected or mitered entrances without headwalls or other entrance protection. The mitered or skewed ends of corrugated metal pipes, cut to conform with the embankment slopes, offer little resistance to bending or buckling. When soils adjacent to the inlet are eroded or become saturated, pipe inlets can be subjected to buoyant forces. Lodged drift and constricted flow conditions at culvert entrances cause pressures which, while difficult to predict, have significant effect on the stability of culvert entrances.

To aid in preventing inlet failures of this type, protective features generally should include full or partial concrete headwalls and/or slope paving (Figures II-A-1 and II-A-2). Riprap can serve as protection in some instances, but concrete inlet structures anchored to the pipe are safer. Preformed concrete or metal end sections may be used in lieu of the inlet structures shown. Metal end sections for culvert pipes larger than 54 inches in height must be anchored to increase their resistance to failure. The figures also show inlet designs which should be used if such protection is considered necessary for pipes smaller than 4 feet in height.

Failures of inlets are of primary concern, but other types of failures have occurred. Seepage of water along the culvert barrel has caused piping or the washing out of supporting material. Hydrostatic pressure from seepage water or from flow under the culvert barrel has buckled the bottoms of large corrugated metal pipe arches. Good compaction of backfill material is essential to reduce the possibility of these types of failures. Also, where soils are quite erosive, special impervious bedding and backfill material should be placed for a short distance at the entrance, and further protection may be provided by cutoff collars placed at intervals along the culvert barrel or by a special subdrainage system.

II-B - EROSION HAZARDS AT CULVERT OUTLETS

Erosion at culvert outlets is a common condition. Determination of the flow condition, scour potential, and channel erodibility, should be standard procedure in the design of all highway culverts.

The only safe procedure is to design on the basis that erosion at a culvert outlet and downstream channel is to be expected. A reasonable procedure is to provide at least minimum protection, and then inspect the outlet channel after major storms to determine if the protection must be increased or extended. Under this procedure, the initial protection against channel erosion should be sufficient to provide some assurance that extensive damage could not result from one runoff event.

TYPES OF SCOUR

Two types of scour can occur in the vicinity of culvert outlets--local scour and general channel degradation. Culverts are generally constructed at crossings of small streams, and the majority of these streams are eroding to reduce their slopes. Channel degradation may proceed in a fairly uniform manner over a long length, or may be evident in one or more abrupt drops progressing upstream with every runoff event. The latter type, referred to as headcutting, can be detected by location surveys or by periodic maintenance inspections following construction. Information regarding the degree of instability of the outlet channel is an essential part of the culvert site investigation. If any substantial doubt exists as to long-term stability of the channel, measures for protection should be included in the initial construction.

Long term lowering of the stream channel through natural processes and local erosion at the culvert outlet may occur simultaneously. Local scour is the result of high-velocity flow at the culvert outlet, but its effect extends only a limited distance downstream. Natural channel velocities are almost universally less than culvert outlet velocities, because the channel cross section, including its flood plain, is generally larger than the culvert flow area. Thus, the flow rapidly adjusts to a pattern controlled by the channel characteristics.

The highest velocities will be produced by long, smooth-barrel culverts on steep slopes. These cases will no doubt require protection of the outlet channel at most sites. However, protection is also often required for culverts on mild slopes. For these culverts flowing full, the outlet velocity will be critical velocity with low tailwater and the full barrel velocity for high tailwater. Where the discharge leaves the barrel at critical depth, the velocity will usually be in the range of 10 to 20 feet per second.

STANDARD CULVERT OUTLET TREATMENT

Standard practice is to use the same treatment at the culvert entrance and exit. It is important to recognize that the inlet is designed to improve culvert capacity or reduce headloss while the outlet structure should provide a smooth flow transition back to the natural channel or into an energy dissipator. Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence. It may not be possible to satisfy both inlet and outlet requirements with the same end treatment or design. As will be illustrated in chapter IV, properly designed outlet structures are essential for efficient energy dissipator design; and in some cases, may substantially reduce or eliminate the need for other end treatments.

II-C - RIPRAP PROTECTION

Some energy dissipators provide exit conditions, velocity and depth, near critical. This flow condition rapidly adjusts to the downstream or natural channel regime; however, critical velocity may be sufficient to cause erosion problems requiring protection adjacent to the basin exit.

Figure II-C-1 provides the riprap size recommended for use downstream of energy dissipators. The length of protection can be judged based on the magnitude of the exit velocity compared with the natural channel velocity. The greater this difference, the longer will be the length required for the exit flow to adjust to the natural channel condition. A filter blanket should also be provided, see reference II-C-1.

II-A-1. Herr, Lester A. and Bossy, Herbert G., HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS, Federal Highway Administration, U.S. Government Printing Office, Washington D.C., 1965, 54 pp. (Hydraulic Engineering Circular No. 5)

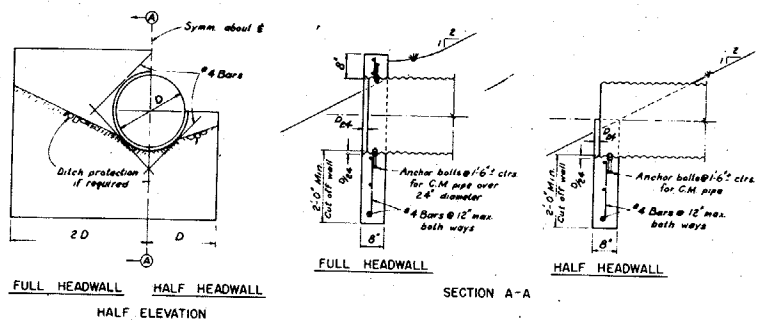
II-A-2. Herr, Lester A. and Bossy, Herbert G., CAPACITY CHARTS FOR THE HYDRAULIC DESIGN OF HIGHWAY CULVERTS, Federal Highway Administration, U.S. Government Printing Office, Washington D.C., 1965, 90 p. (Hydraulic Engineering Circular No. 10)

II-A-3. Harrison, L. J., Morris, J. L., Normann, J. M., and Johnson, F. L., HYDRAULIC DESIGN OF IMPROVED INLETS FOR CULVERTS, Federal Highway Administration, U.S. Government Printing Office, Washington, D.C., August 1972, 150 pp. (Hydraulic Engineer Circular No. 13)

II-B-1. Normann, J. M., DESIGN OF STABLE CHANNELS WITH FLEXIBLE LININGS, Federal Highway Administration, October 1975

II-B-2. AASHTO, Highway Drainage Guidelines, GUIDELINES FOR THE HYDRAULIC DESIGN OF CULVERTS, Vol. IV, 1975, 45 pp.

II-C-1. Searcy, James K., USE OF RIPRAP FOR BANK PROTECTION, Federal Highway Administration, Washington, D.C., 1967, pp. 43.



INLET STRUCTURES FOR CULVERT SIZES 18 IN. TO 36 IN. DIAMETER

SUMMARY OF QUANTITIES				
Culvert Dia.	Full Headwall		Half Headwall	
inches	Concrete Cu. Yd.	Reinforcing Steel Lbs.	Concrete Cu. Yd.	Reinforcing Steel Lbs.
18	0.57	45	0.18	15
24	0.85	65	0.26	20
30	1.15	85	0.34	25
36	1.50	115	0.43	35

Note: Quantities shown are for C.M. pipe culvert.

GENERAL NOTES

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1973.

Concrete: All concrete shall be Class A(AE) using Type II (low alkali) Portland cement and having a minimum 28-day compressive strength f'_c=3000 psi. The air entraining agent shall be approved by the engineer prior to acceptance for use. All exposed edges shall be chamfered ¾ inch unless otherwise noted.

Reinforcing Steel: Reinforcing steel shall conform to ASTM A-615, A-616 or A-617.

Anchor Bolts: Bolt and nut material shall conform to ASTM A-307. Bolts and nuts shall be galvanized after fabrication in accordance with ASTM A-153. Anchor bolts are not required for concrete pipe.

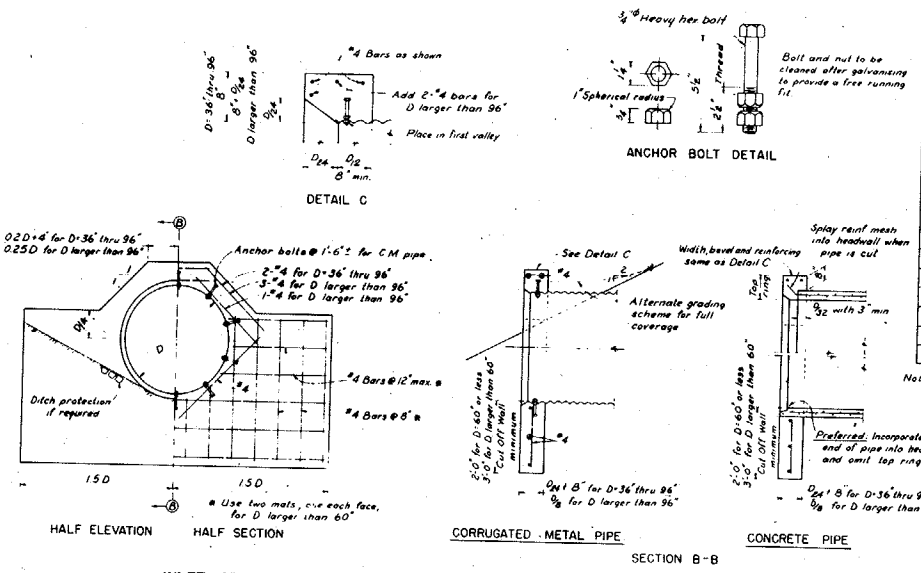
Cutoff Wall: The depth of wall shown on the plan may be reduced if rock is encountered at a higher elevation.

Multiple Pipe Installations: To permit careful placing and tamping of back fill material, clear spacing for multiple pipe installations shall be not less than one-half the diameter of the larger pipe between sides of adjacent pipes, but not required to exceed three feet, and in no case less than one foot.

Piping: When using pervious bedding and backfill, it is desirable to prevent seepage and piping by placing impervious material at the inlet. Cut off collars may be used in lieu of impervious material.

Skew: When culvert is skewed to embankment, the embankment may be contoured as shown on Sheet No. 4.

Preformed End Sections: Preformed end sections may be used in lieu of headwalls shown, if approved.



INLET STRUCTURE FOR CULVERT SIZES 36 IN. TO 180 IN. DIAMETER (NOT TO SCALE)

SUMMARY OF QUANTITIES		
Culvert Dia. inches	Concrete Cu. Yd.	Reinforcing Steel Lbs.
36	1.0	75
42	1.3	90
48	1.5	105
54	2.0	125
60	2.4	145
72	4.1	435
84	5.6	535
96	6.8	640
108	9.8	795
120	12.4	955
144	20.1	1255
180	37.6	1820

Note: Quantities shown are for C.M. pipe culvert.

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WASHINGTON, D. C.

CIRCULAR CULVERT END TREATMENT

INLET STRUCTURES FOR CONCRETE AND CORRUGATED METAL CULVERTS SIZES 18 IN TO 180 IN. DIAMETER

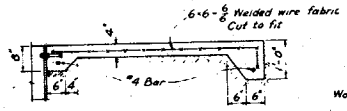
FIGURE II-A-1	NOV. 1973	SHEET NO. G-1-73

FIGURE II-A-1

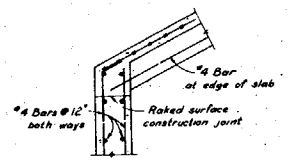
TYPICAL DIMENSIONS AND QUANTITIES

Culvert Dia. inches	S ft-in.	Square Headwall A ft-in.	Square Headwall L ft-in.	Square Headwall Concrete Cu.Yd.	Square Headwall Reinf. Lbs.	15° Skew A ft-in.	15° Skew L ft-in.	15° Skew Concrete Cu.Yd.	15° Skew Reinf. Lbs.	30° Skew A ft-in.	30° Skew L ft-in.	30° Skew Concrete Cu.Yd.	30° Skew Reinf. Lbs.	45° Skew A ft-in.	45° Skew L ft-in.	45° Skew Concrete Cu.Yd.	45° Skew Reinf. Lbs.	S ft-in.	Culvert Dia. inches
48	13-5	4-0	8-0	2.8	180	4-3	8-6	2.8	185	5-4	10-8	3.2	220	8-0	16-0	4.5	320	13-5	48
60	15-1 1/8	5-0	10-0	3.7	225	5-4	10-8	3.7	235	6-8	13-4	4.4	290	10-0	20-0	6.3	420	15-1 1/8	60
72	16-9 3/4	6-0	12-0	4.7	275	6-5	12-10	4.8	295	8-0	16-0	5.7	360	12-0	24-0	8.3	525	16-9 3/4	72
108	21-9 5/8	9-0	18-0	8.9	480	9-8	19-4	9.3	510	12-0	24-0	11.3	630	18-0	36-0	16.8	930	21-9 5/8	108
144	26-10	12-0	24-0	15.2	695	12-10	25-8	16.1	745	16-0	32-0	20.1	935	24-0	48-0	30.3	1395	26-10	144
180	31-10 5/8	15-0	30-0	23.7	940	16-1	32-2	25.5	1020	20-0	40-0	32.1	1285	30-0	60-0	48.7	1945	31-10 5/8	180

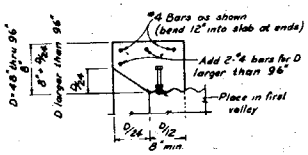
Note: To determine the dimension "A" for culvert sizes and skew angles not shown, multiply the pipe diameter by the square of the secant of the skew angle.



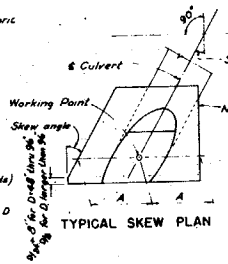
SECTION B-B



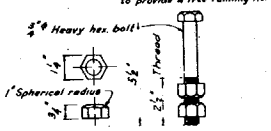
SECTION D-D



DETAIL E

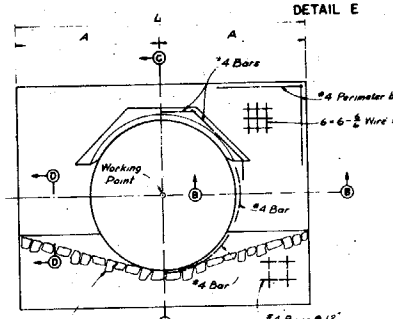


TYPICAL SKEW PLAN

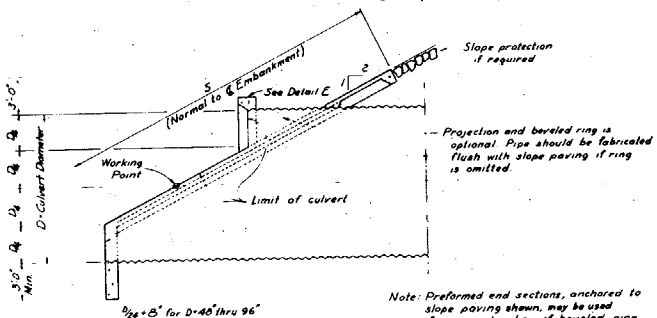


ANCHOR BOLT DETAIL

Anchor bolts to be set at 1'-6" ± centers around entire perimeter at end of pipe.



ELEVATION



SECTION C-C
(Normal to ℄ Embankment)

Note: Preformed end sections, anchored to slope paving shown, may be used if approved in lieu of beveled pipe.

GENERAL NOTES

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1973

Concrete: All concrete shall be Class A(AE) using Type II (low alkali) Portland Cement and having a minimum 28 day compressive strength f'c = 3,000 psi. The air entraining agent shall be approved by the engineer prior to acceptance for use. All exposed edges shall be chamfered 3/4 inch unless otherwise noted. Slope paving surface variations shall not exceed 1/2 inch in 10 feet.

Reinforcing Steel: Reinforcing steel shall conform to ASTM A-615, A-616 or A-617.

Welded steel wire fabric shall conform to ASTM A-185.

Anchor Bolts: Bolt and nut material shall conform to ASTM A-307. Bolts and nuts shall be galvanized after fabrication in accordance with ASTM A-153.

Cutoff Wall: The depth of wall shown on the plan may be reduced if rock is encountered at a higher elevation.

Multiple Pipe Installations: To permit careful placing and tamping of backfill material, clear spacing for multiple pipe installations shall be not less than one-half the diameter of the larger pipe between sides of adjacent pipes, but not required to exceed three feet.

Piping: When using pervious bedding and backfill, it is desirable to prevent seepage and piping by placing impervious material at the inlet. Cutoff collars may be used in lieu of impervious material.

Skew: When culvert is skewed to embankment slope, a square headwall may be used as an alternate and the embankment contoured as shown on Sheet No. 4.

(NOT TO SCALE)

U.S. DEPARTMENT OF TRANSPORTATION FEDERAL HIGHWAY ADMINISTRATION WASHINGTON, D.C.		
CIRCULAR CULVERT END TREATMENT		
INLET STRUCTURES FOR CORRUGATED METAL CULVERTS SIZES 48 IN. TO 180 IN. DIAMETER		
FIGURE II-A-2	NOV. 1973	SHEET NO. G-2-73

FIGURE II-A-2

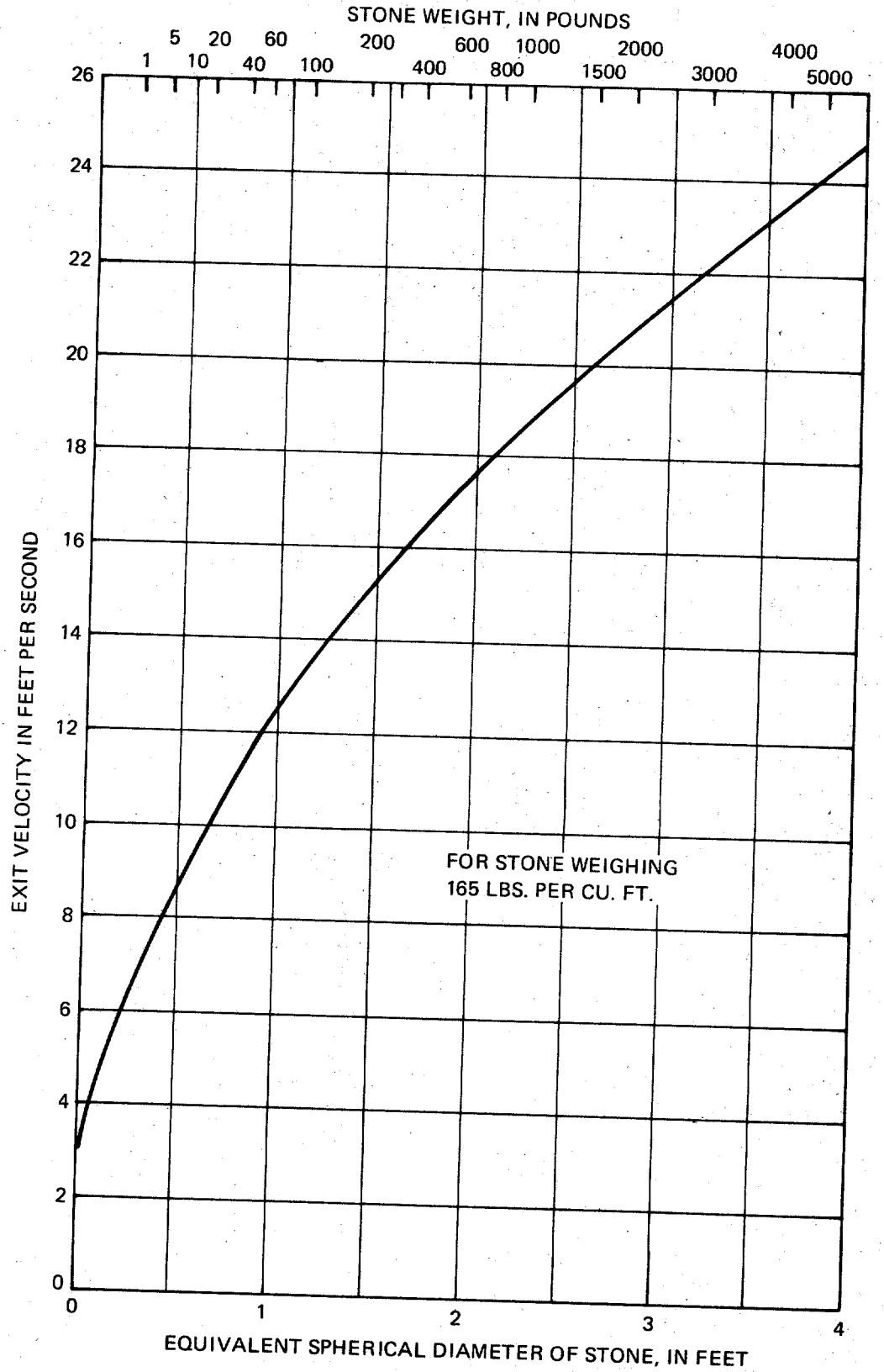


FIGURE II-C-1. RIPRAP SIZE FOR USE DOWNSTREAM OF ENERGY DISSIPATORS FROM REFERENCE II-C-1.

III. CULVERT OUTLET VELOCITY AND VELOCITY MODIFICATION

Culvert outlet velocity is one of the primary indicators of erosion potential. Outlet velocities are seldom less than 10 fps and range up to 30 fps for culverts on small or mild slopes and can exceed this for culverts on steep slopes. Under these conditions, it is reasonable to investigate measures to modify or reduce velocity within the culvert before considering an energy dissipator. Several possibilities exist, but the degree of velocity reduction is, in most cases, limited and must always be weighed against the increased costs which are generally involved.

The continuity equation, $Q = AV$, can be utilized in all situations to compute culvert velocities, either within the barrel or at the outlet. Since discharge will generally be known from culvert design, determining the flow area will define the velocity.

CULVERTS ON MILD SLOPES

Figure III-1, taken from HEC No. 5 (III-1), indicates the four types of flow for culverts on mild slopes, i.e., culverts flowing with outlet control.

Figure III-1A indicates a condition where high tailwater controls the culvert outlet velocity. In this case, outlet velocity is determined using the full barrel area. With this flow condition, it is possible to reduce the velocity by increasing the culvert size. The degree of reduction is proportional to the reciprocal of the culvert area. Selecting several culvert diameters provides a specific example:

CULVERT DIAMETER (ft.)	3	4	5	6
Percentage reduction of Outlet Velocity ($V = Q/A$)	44%	36%	31%	

For high tailwater conditions, erosion may not be a serious problem. It may be more important to determine if tailwater will always control, or if the conditions shown in figures III-1B, C, or D might occur under some circumstances.

When discharge is high enough to produce critical depth equal to the crown of the culvert barrel, the full flow condition shown in figure III-1B will occur. The outlet velocity reduction is again illustrated in the previous example. In this case, however, it is necessary to determine if the increased culvert dimensions result in brink depth below the culvert crown. When this occurs, the flow area used in the continuity equation is that associated with brink depth, which for this illustration is assumed to be critical depth. Figures III-3 through III-8 are included for convenience in determining critical depth for various shapes of culverts.

Example: A 3.0-foot CMP discharging 100 cfs, flowing full with tailwater of 2.0 feet.

From Figure III-4 critical depth (d_c) exceeds 3.0 feet.

Therefore, the barrel is flowing full to the end. From table III-2 with $d/D=1$, $A/D^2=0.785$ and $V=100/.785(9)=14.15$ fps.

Changing to a 4.0-foot CMP, changes d_c to 3.1 feet which is less than D so d_c controls outlet velocity.

$$V_{FULL} = \frac{100}{.785(16)} = 7.98 \text{ and } d_c/D = \frac{3.1}{4.0} = .78$$

$$V/V_{FULL} = 1.16 \text{ from figure VII-C-3 and } V = 1.16(7.98) = 9.25$$

This is a reduction of about 35 percent instead of the approximate 44 percent indicated in the previous example.

When culverts discharge as in figure III-1C and D with critical depth near the outlet, changing the barrel slope will have no effect on the outlet velocity as long as the slope is less than critical slope. Changing the resistance factor will change the depth at the outlet an insignificant degree and will, therefore, not modify the outlet velocity.

The initial step is to compute normal depth (tailwater) in the outlet channel, tables III-1 and 2 at the end of this section facilitates normal depth calculations with this, Figures III-9 and 10 may be used directly to determine outlet brink depths for rectangular and circular sections, respectively. These figures are dimensionless rating curves

which indicate the effect on brink depth of tailwater for culverts on mild or horizontal slopes. Values of $Q/BD^{3/2}$ and $Q/D^{5/2}$ for use with figures III-9 and 10 are included as table III-3.

When the tailwater depth is low, culverts on mild or horizontal slopes will flow with critical depth near the outlet. This is indicated on the ordinate of figures III-9 and 10. As the tailwater increases, the depth at the brink increase, at a variable rate, along the $Q/BD^{3/2}$ or $Q/D^{5/2}$ curve, until a point where the tailwater and brink depth vary linearly, the 45° line on figures III-9 and 10. Using these figures, the effects of changing culvert size may be determined. For example,

Q = 60 cfs (constant)
TW = 24 inches (constant)

D	$D^{5/2}$	$Q/D^{5/2}$	TW/D	Y_o/D
42	22.9	2.6	.57	.63
48	32.0	1.9	.50	.54
54	43.0	1.4	.44	.46
60	55.9	1.1	.40	.41

Y_o/D	Y_o	Y_c	A/D^2	A	V = Q/A	D
.63	2.2	2.4	0.52	6.4	9.4	42
.54	2.16	2.3	0.43	6.9	8.7	48
.46	2.1	2.3	0.35	7.1	8.5	54
.41	2.05	2.2	0.30	7.5	8.0	60

Changing culvert diameter from 3.5 to 5.0 feet, a 43 percent increase, results in a decrease of only 15 percent in the outlet velocity.

For culvert shapes other than rectangular and circular, the brink depth for low tailwater can be approximated from the critical depth curves figures III-4 through 8. Since critical depth is larger than brink depth, determining brink depth in this manner is not conservative, but is acceptable.

CULVERTS ON STEEP SLOPES

For the situation shown in A and B of figure III-2, it is convenient to determine normal flow conditions by the use of Manning's equation. The charts and tables of reference III-1 provide rapid solutions under these circumstances.

Increasing the barrel size for a given discharge and slope has little effect on velocity if the flow reaches normal depth, as it will within most culverts on steep slopes. For example, using a 5-foot diameter concrete pipe with

constant slopes as a base, the velocity in a 3-foot pipe will be 1.03 larger and the velocity in an 8-foot pipe will be 0.97 smaller. Less velocity change would be obtained for corrugated metal pipes.

Some reduction in outlet velocity can be obtained by increasing the number of barrels carrying the total discharge. Reducing the flow rate per barrel reduces velocity at normal depth, if the flowline slopes are the same. Substituting two smaller pipes with the same depth to diameter ratio for a large one reduces Q per barrel to one-half the original rate and the outlet velocity to approximately 87 percent of that in the single-barrel design. However, this 13 percent reduction must be considered in light of the increased cost of the culverts. In addition, the percentage reduction decreases as the number of barrels is increased. For example, using four pipes instead of three results in only an additional 5 percent reduction in outlet velocity. Furthermore, where high velocities are produced, a design using more barrels may still result in velocities requiring protection, with a large increase in the area to be protected.

Outlet velocities can also be modified by substituting a rough barrel for a smooth barrel. For a 60-inch concrete pipe, n = 0.012, on a 1-percent slope (S_o= 0.01), discharging at 100 cfs, V_n = 13.8 fps and S_c = 0.00325; using a c.m. pipe (n = 0.024) results in a critical slope of 0.015. Since S_c for the c.m. pipe is greater than the actual slope, the flow is subcritical and the outlet velocity will be critical velocity or 8.5 fps. Manning's equation V = $(1.49\ R^{2/3}S^{1/2})/n$ shows that V varies as $S^{1/2}/n$. For the critical slope situation (R is a constant), doubling the roughness results in a four-fold increase in critical slope. When using this method of velocity reduction, it should be remembered that changing the flow from supercritical to subcritical may result in a marked change in the headwater.

Substituting a "broken-slope" flow line for a steep, continuous slope is not recommended for controlling outlet velocity. Such a design is based on the assumption that the reduced slope of the lower barrel will control depth and velocity, as indicated by the Manning formula. Where the total fall from inlet to outlet remains the same, a broken-slope flow line reduces the outlet velocity only slightly. The initial steeper slope will bring about a lesser depth and greater velocity at the break in grade, followed by a small increase in depth in the lesser slope section. In supercritical

flow, the total loss of energy by resistance will be somewhat greater with the steeper and then flatter slope because a lesser depth is produced over a greater portion of the barrel length. This increased loss due to resistance will be small, however, as will the reduction in outlet velocity. Formation of a hydraulic jump in the lower barrel is rare, as the downstream depth required to force a jump will seldom be encountered. If this type of design is attempted, water surface profile calculations must be made to insure that the hydraulic jump relationship is fulfilled.

For culverts on slopes greater than critical, rougher material will cause greater depth of flow and less velocity in equal size pipes. Velocity varies inversely with resistance; therefore, using a corrugated metal pipe instead of a concrete pipe will reduce velocity approximately 40 percent, and substitution of a structural plate c.m. pipe for concrete will result in about 50 percent reduction in velocity. Barrel resistance is obviously an important factor in reducing velocity at the outlets of culverts on steep slopes. Chapter VII contains detailed discussion and specific design information for increasing barrel resistance.

III-1. Federal Highway Administration, DESIGN CHARTS FOR OPEN-CHANNEL FLOW, U.S. Government Printing Office, Washington, D.C., 1961, 105 pp. (Hydraulic Design Series No. 3).

III-2. Simons, D. B., Stevens, M. A., Watts, F. J., FLOOD PROTECTION AT CULVERT OUTLETS, Colorado State University, Fort Collins, Colorado, CER 69-70 DBS-MAS-FJW4, 1970.

III-3. U.S. Department of the Interior, Bureau of Reclamation, DESIGN OF SMALL CANAL STRUCTURES, 1974, pp. 127-130.

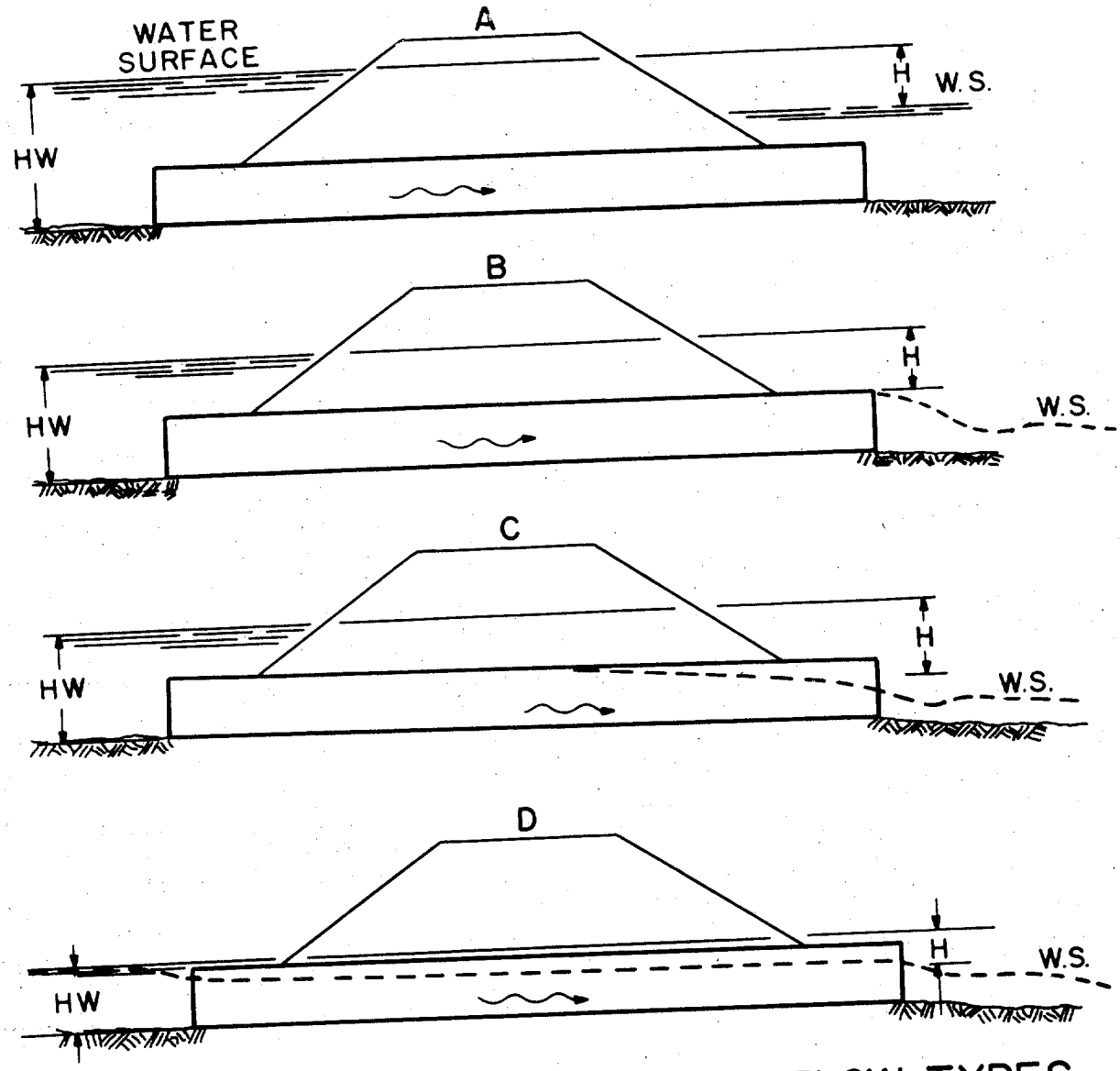


FIGURE III-I OUTLET CONTROL FLOW TYPES

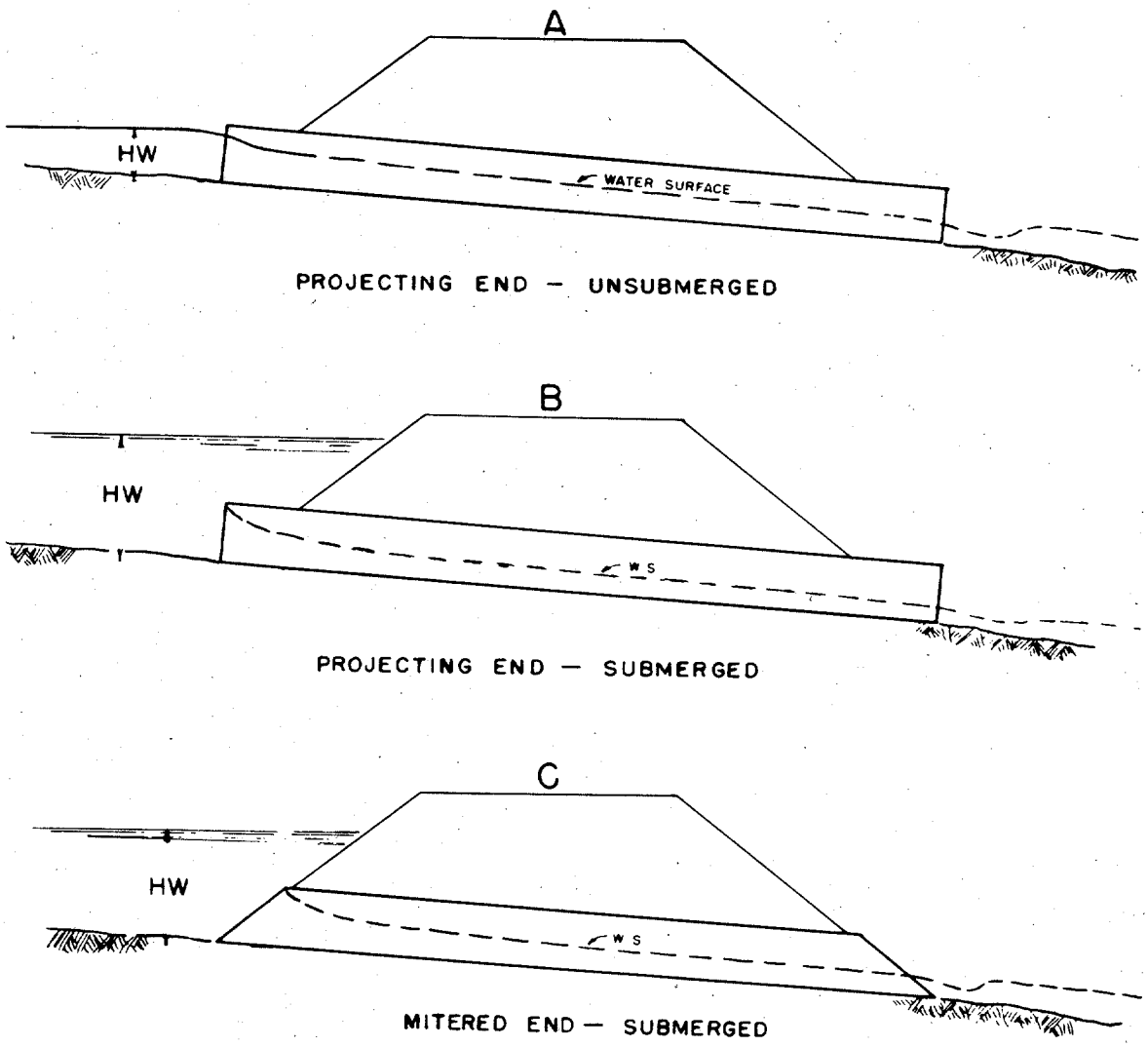
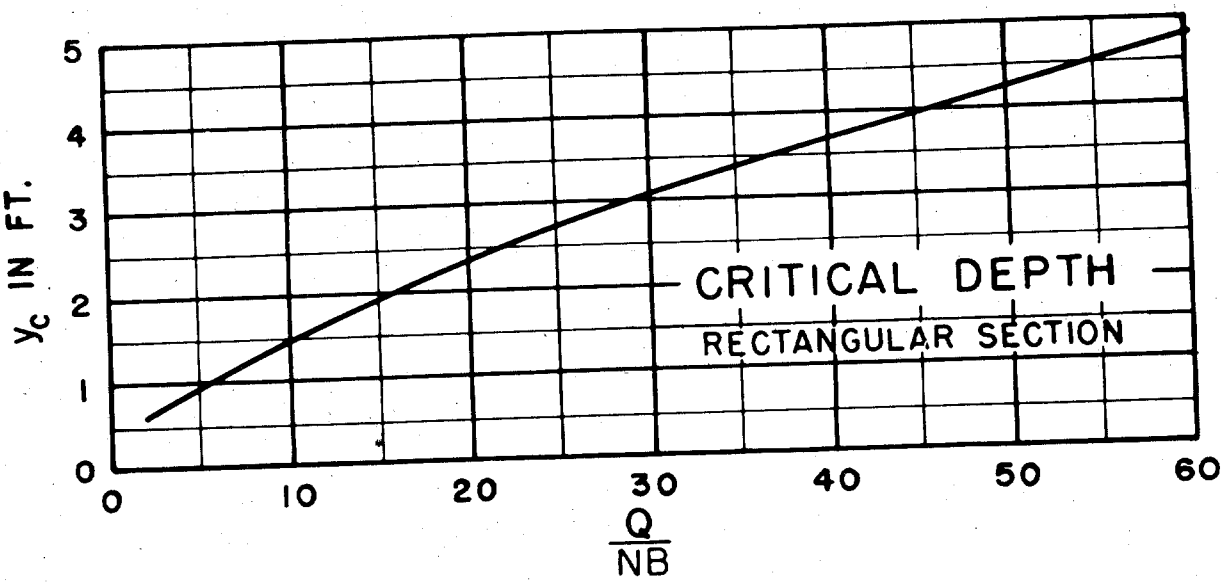


FIGURE III-2 INLET CONTROL FLOW TYPES



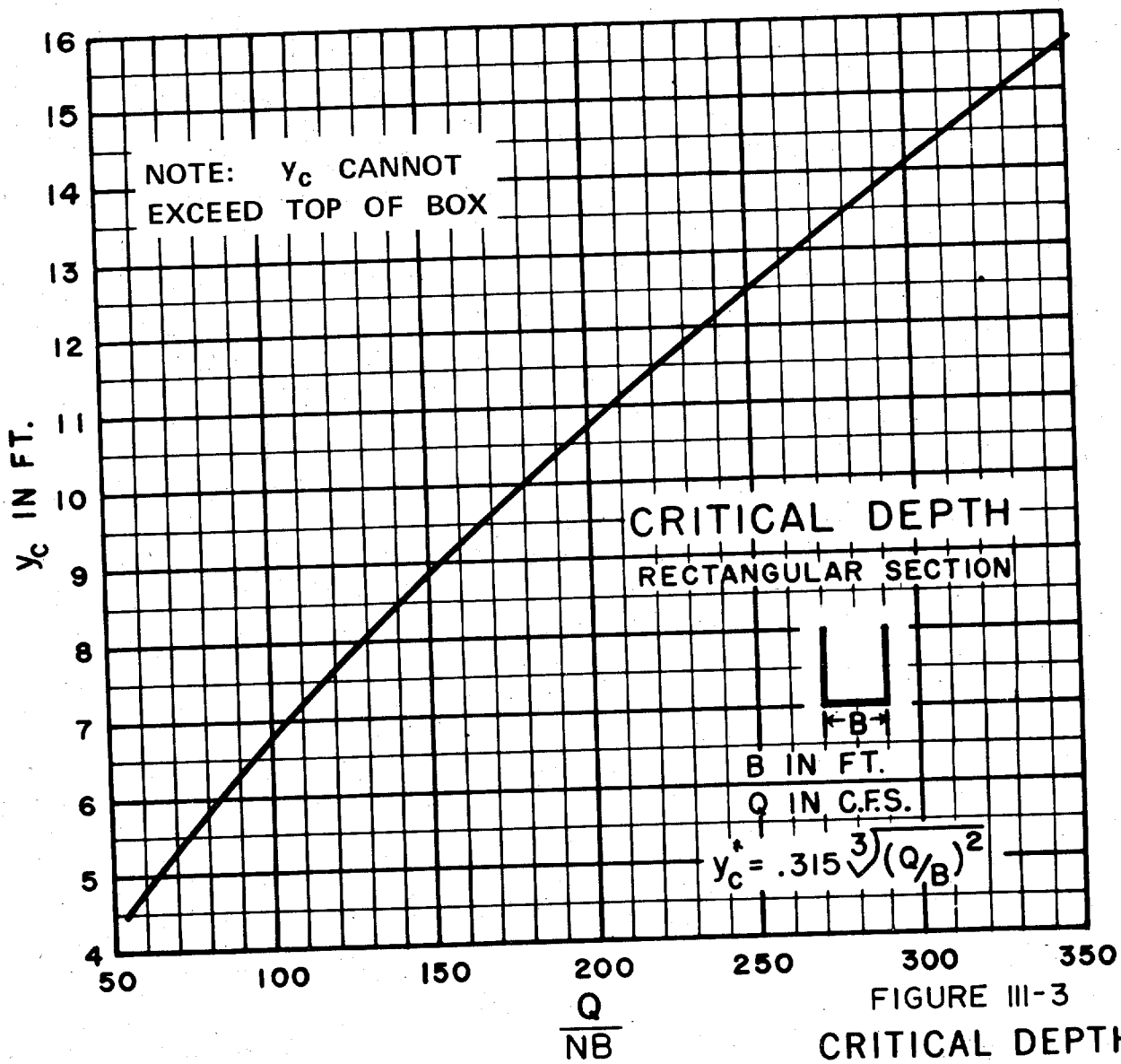


FIGURE III-3

CRITICAL DEPTH
RECTANGULAR SECTION

BUREAU OF PUBLIC ROADS JAN. 1963

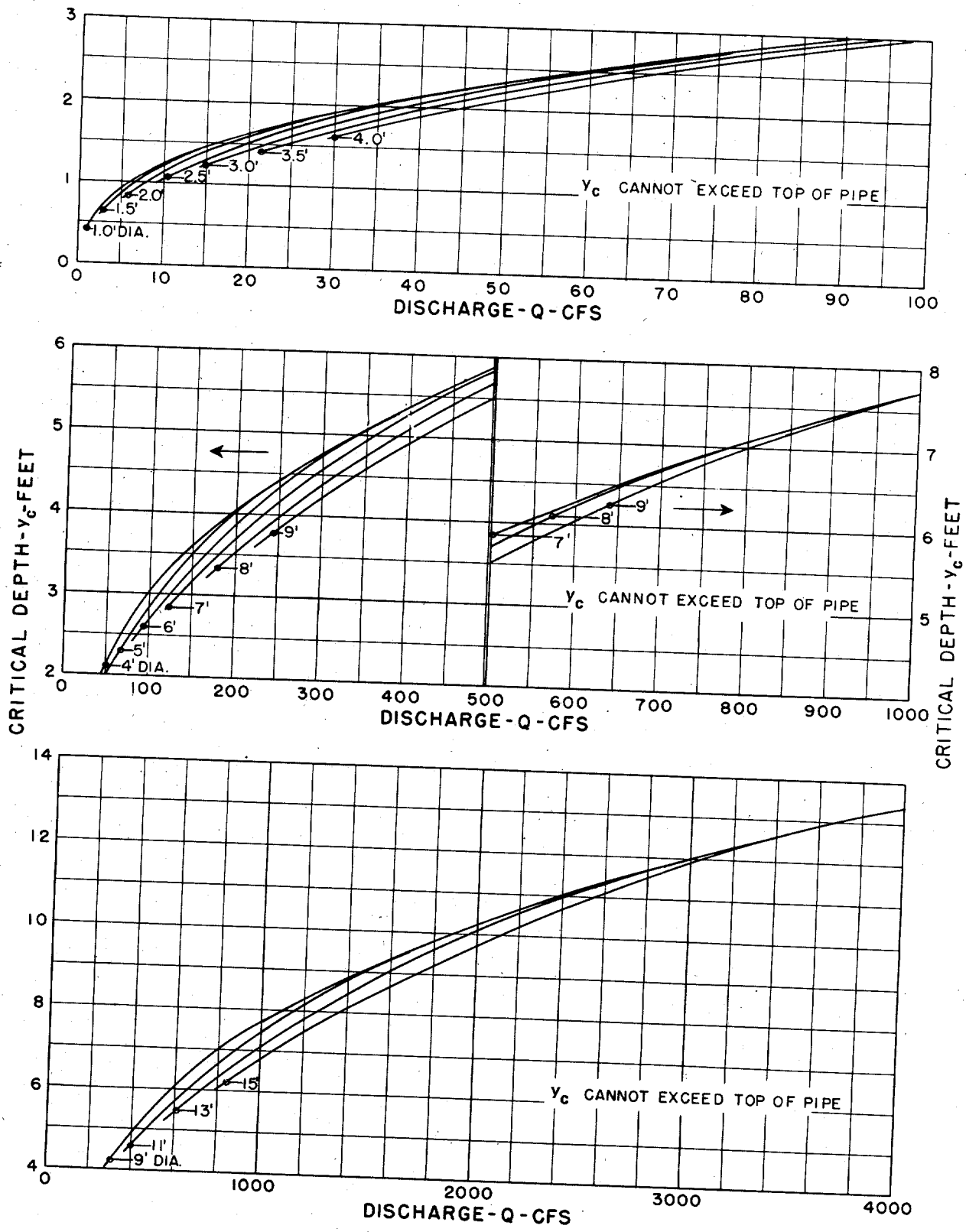


FIGURE III-4

CRITICAL DEPTH
CIRCULAR PIPE

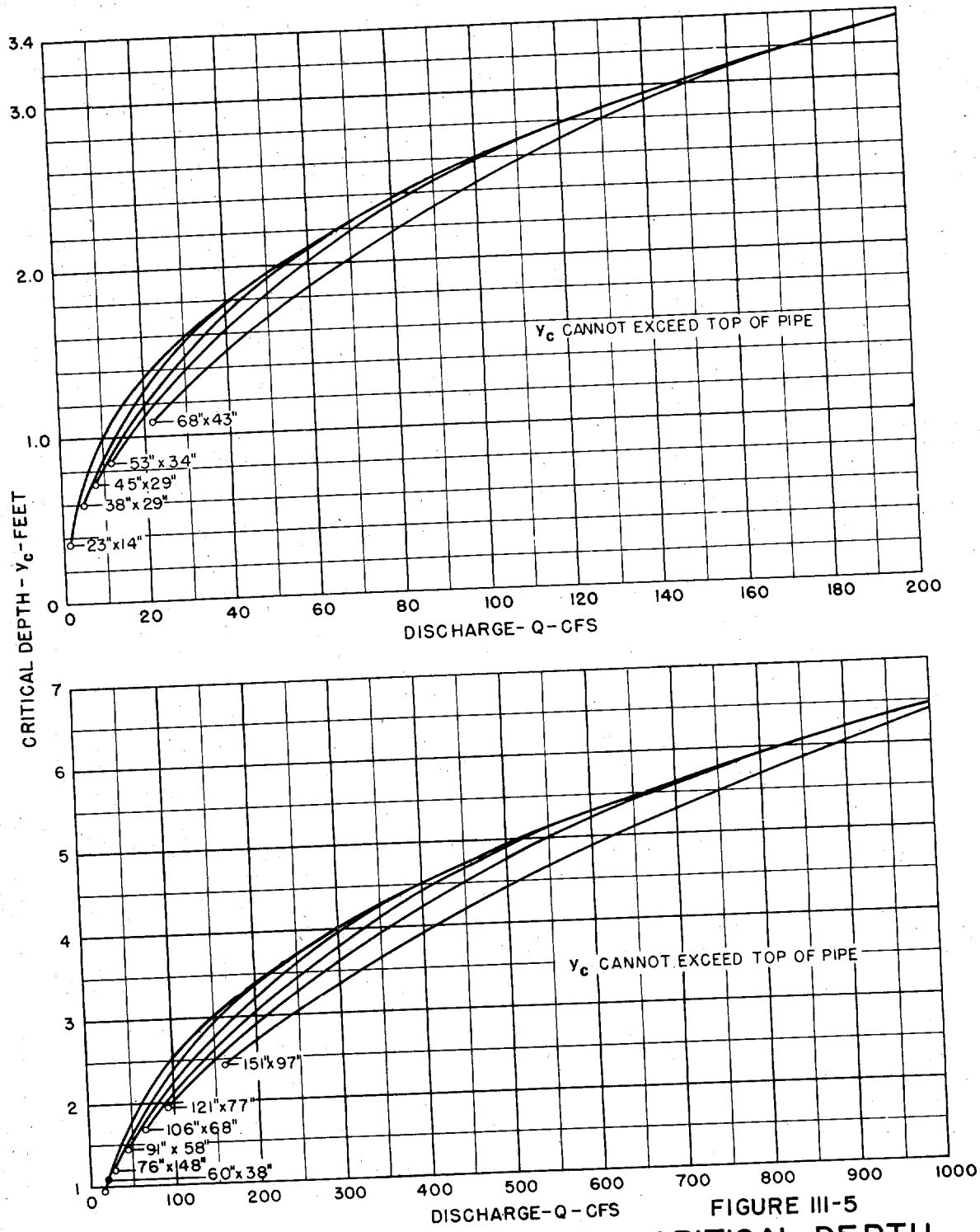


FIGURE III-5

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL

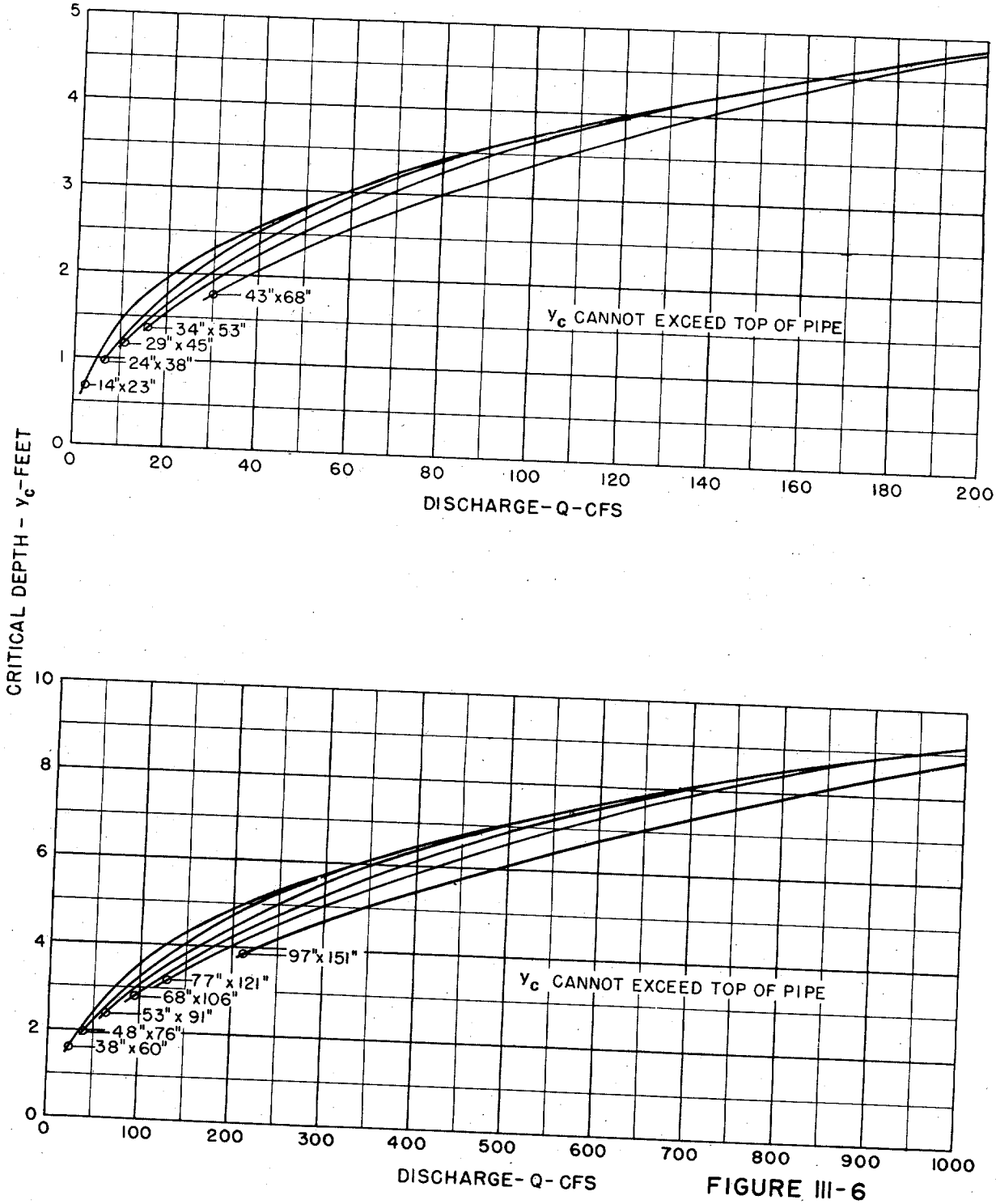
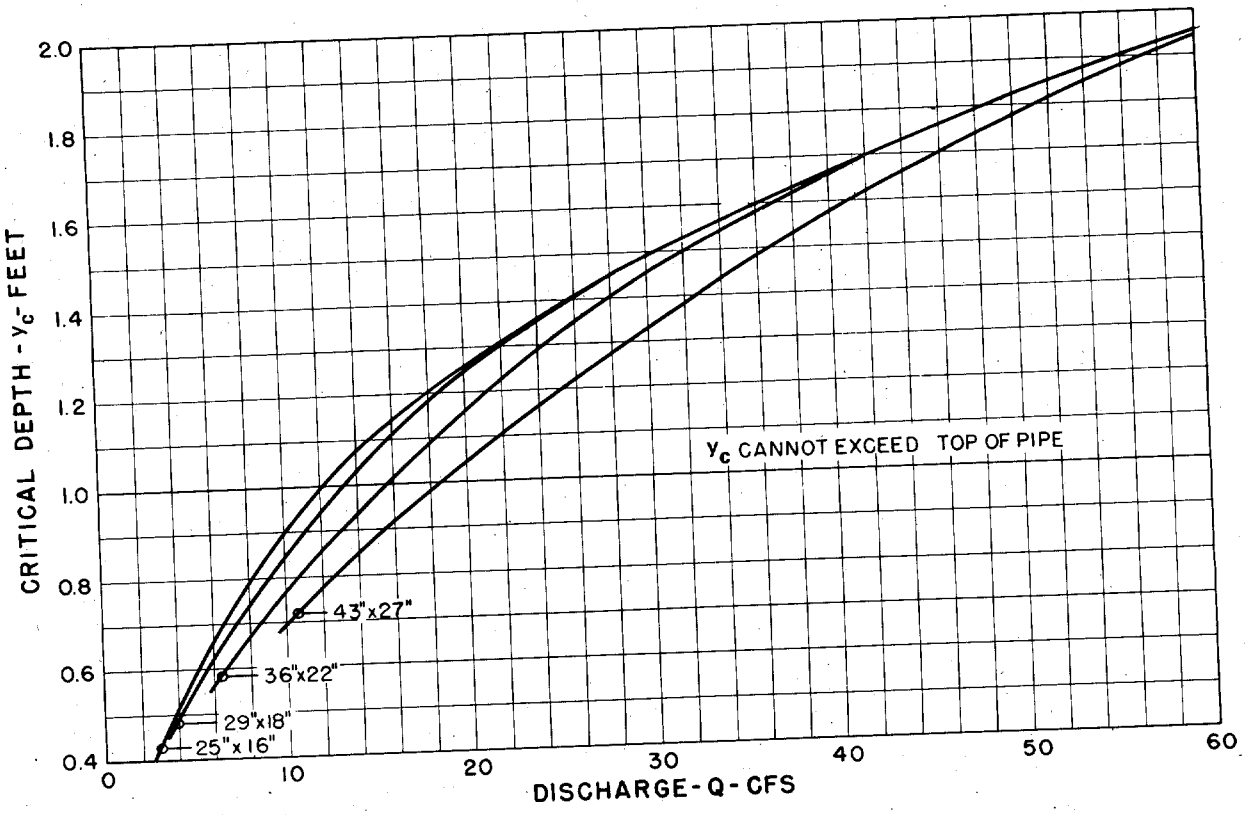


FIGURE III-6

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL

BUREAU OF PUBLIC ROADS JAN. 1964



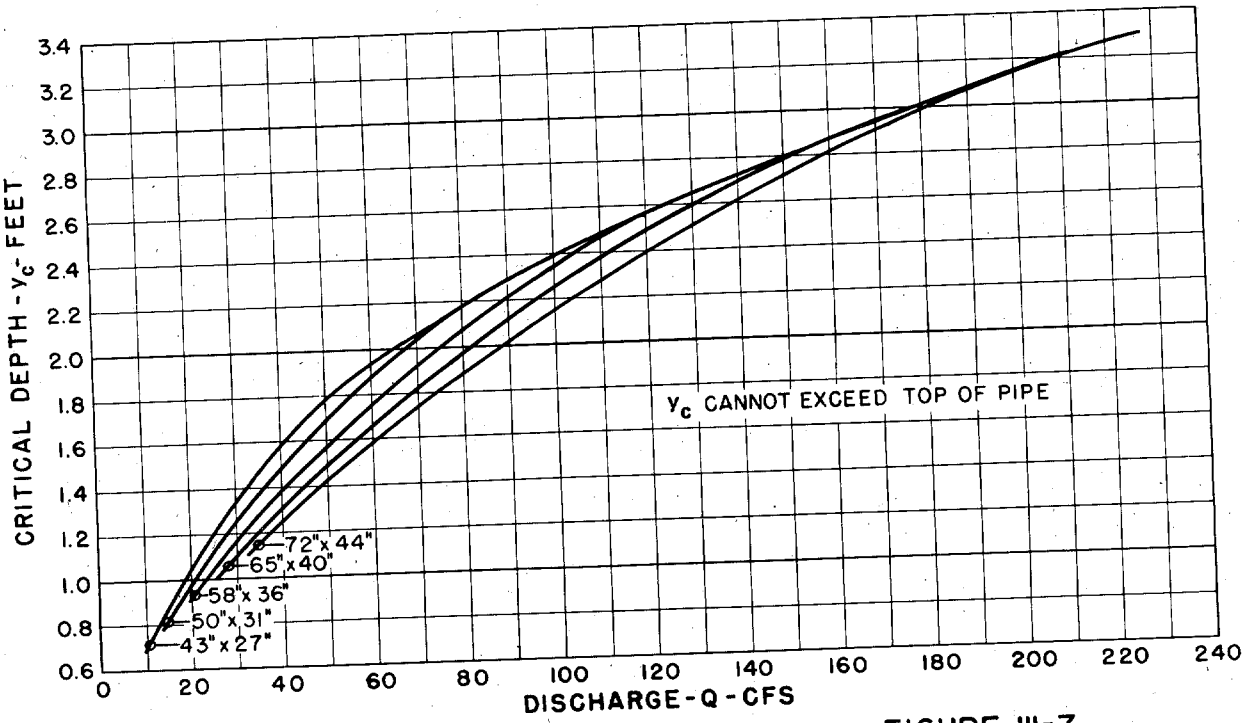
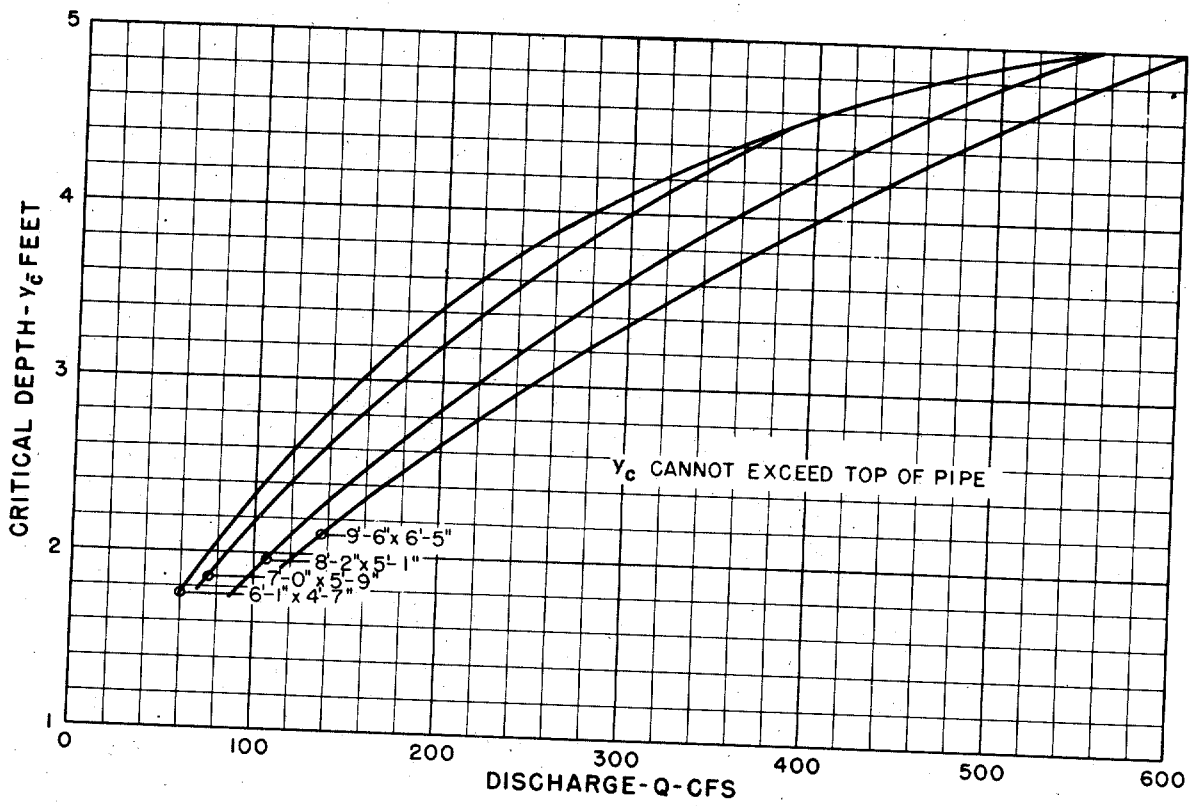


FIGURE III-7

CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH



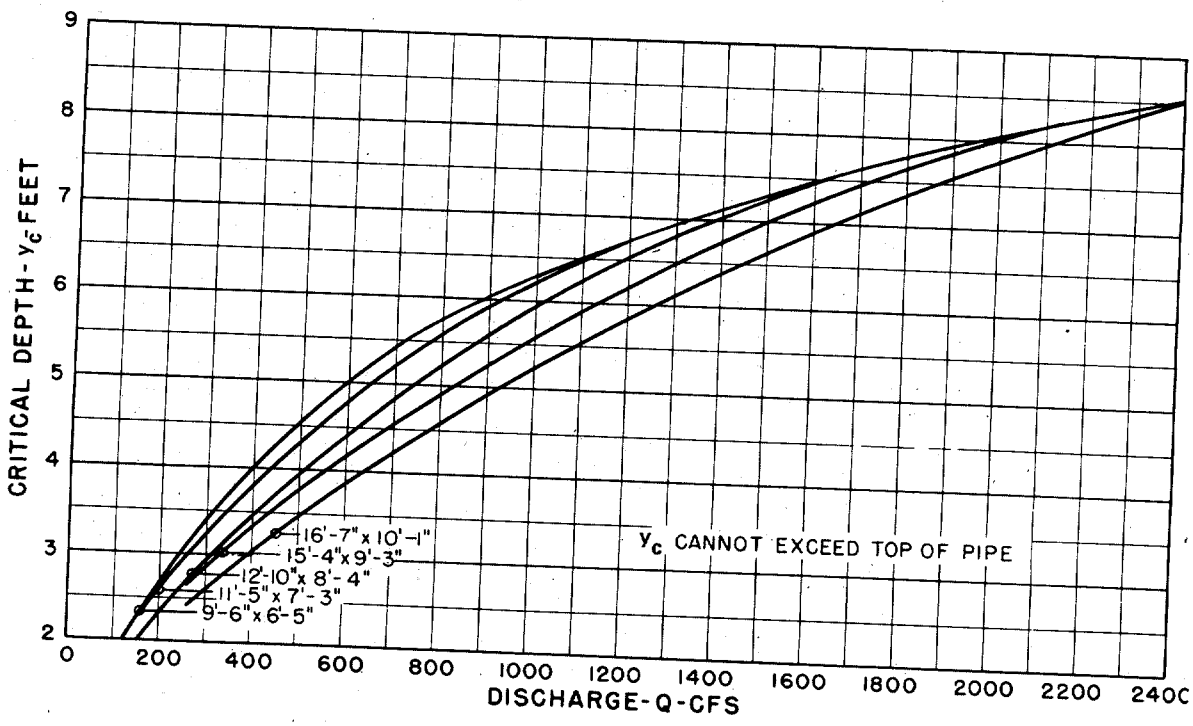


FIGURE III-8

CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH

BUREAU OF PUBLIC ROADS JAN. 1964.

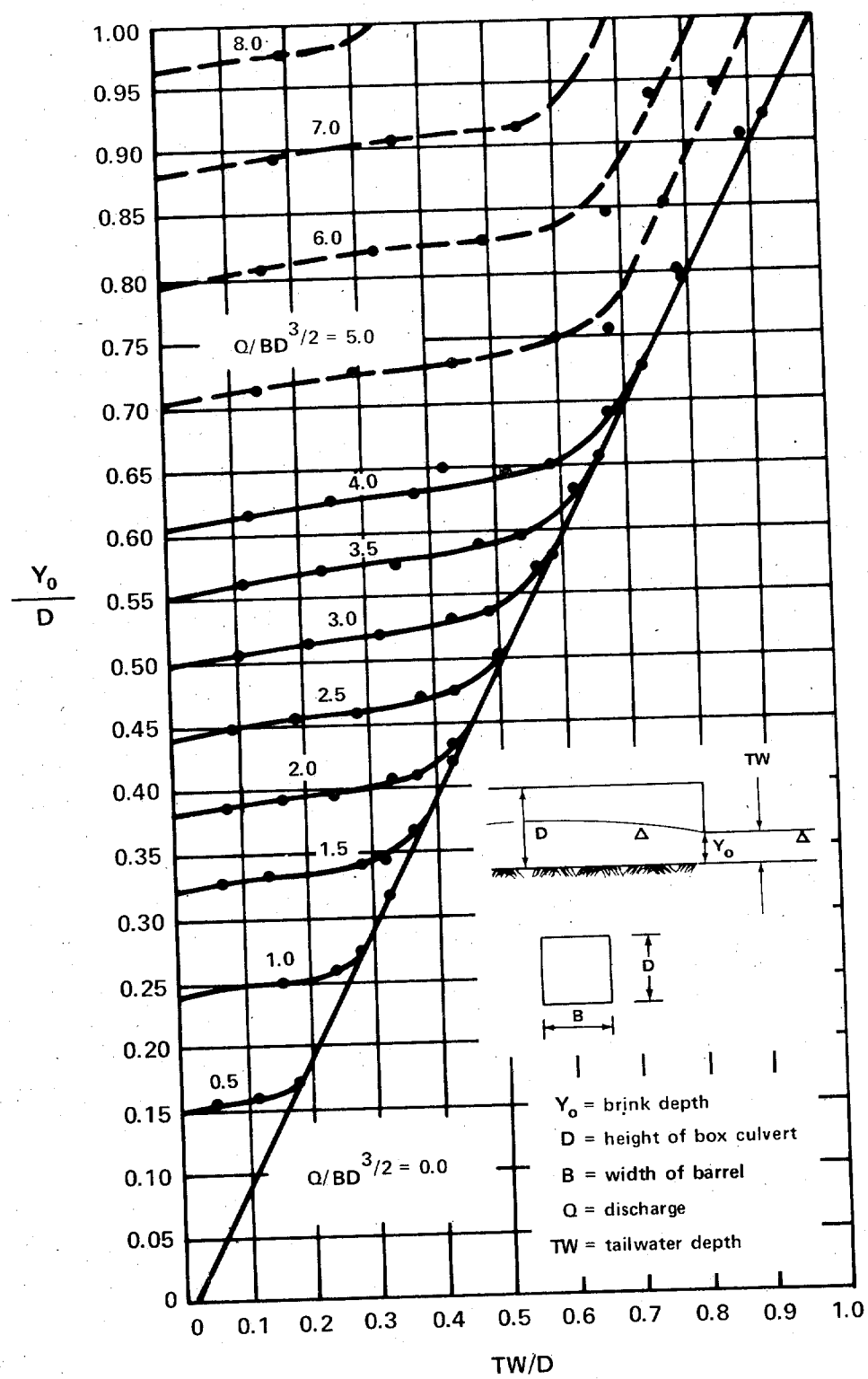


Figure III-9 Dimensionless Rating Curves for the Outlets of Rectangulars Culverts on Horizontal and Mild Slopes from Reference III-2

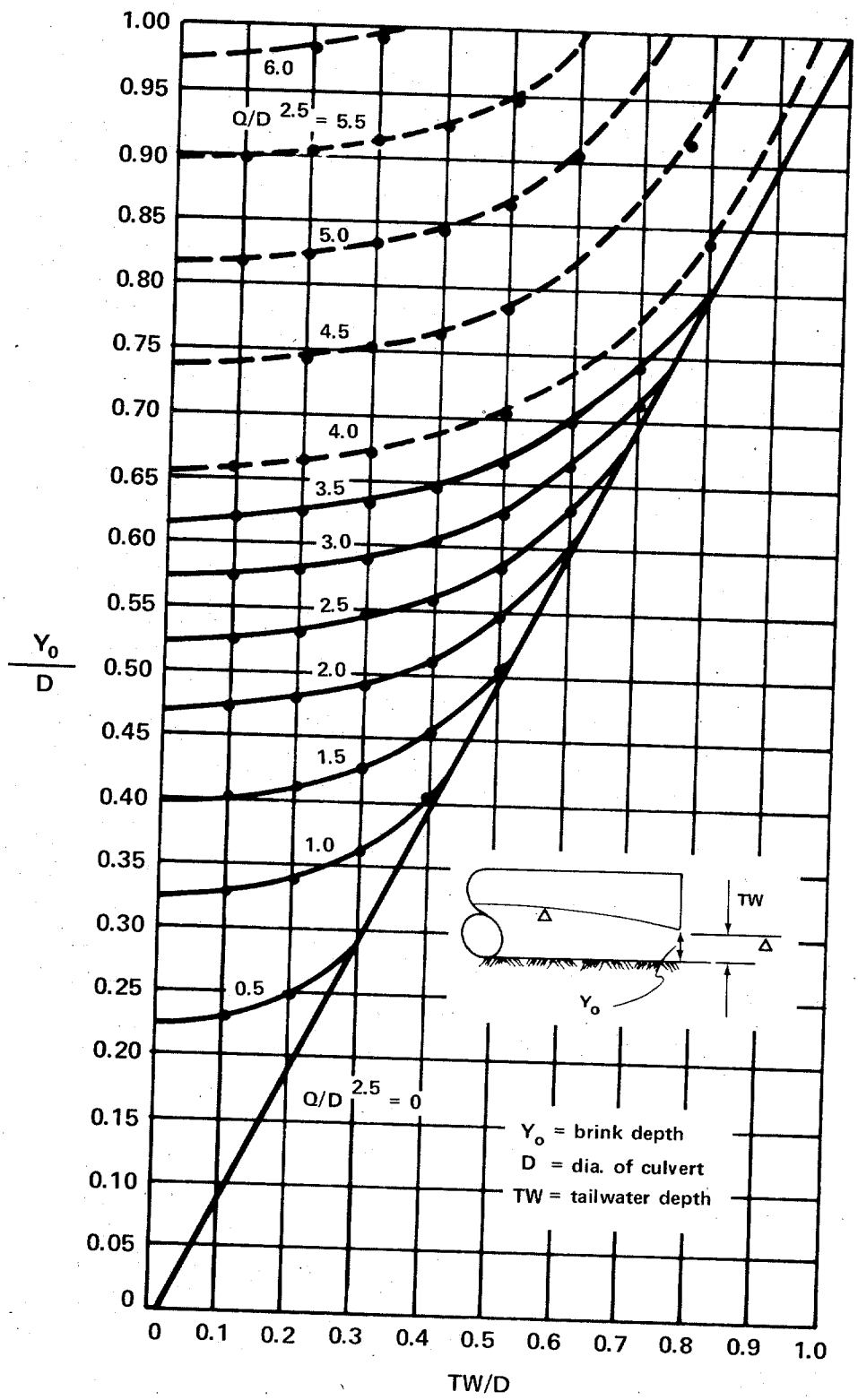
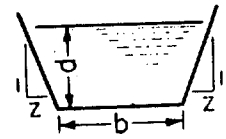


Figure III-10 Dimensionless Rating Curve for the Outlets of Circular Culverts on Horizontal and Mild Slopes from Reference III-2

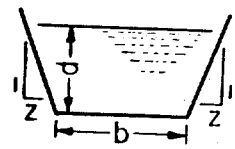
Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.* **From Reference III-3.**



d/b[1]	Values of $\frac{Qn}{b^{8/3}S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.02	.00213	.00215	.00216	.00217	.00218	.00219	.00220	.00220	.00221	.00223
.03	.00414	.00419	.00423	.00426	.00429	.00431	.00433	.00434	.00437	.00443
.04	.00661	.00670	.00679	.00685	.00690	.00696	.00700	.00704	.00707	.00722
.05	.00947	.00964	.00980	.00991	.0100	.0101	.0102	.0103	.0103	.0106
.06	.0127	.0130	.0132	.0134	.0136	.0137	.0138	.0140	.0141	.0145
.07	.0162	.0166	.0170	.0173	.0176	.0177	.0180	.0182	.0183	.0190
.08	.0200	.0206	.0211	.0215	.0219	.0222	.0225	.0228	.0231	.0240
.09	.0240	.0249	.0256	.0262	.0267	.0271	.0275	.0279	.0282	.0296
.10	.0283	.0294	.0305	.0311	.0318	.0324	.0329	.0334	.0339	.0358
.11	.0329	.0342	.0354	.0364	.0373	.0380	.0387	.0394	.0400	.0424
.12	.0376	.0393	.0408	.0420	.0431	.0441	.0450	.0458	.0466	.0497
.13	.0425	.0446	.0464	.0480	.0493	.0505	.0516	.0527	.0537	.0575
.14	.0476	.0501	.0524	.0542	.0559	.0573	.0587	.0599	.0612	.0659
.15	.0528	.0559	.0585	.0608	.0628	.0645	.0662	.0677	.0692	.0749
.16	.0582	.0619	.0650	.0676	.0699	.0720	.0740	.0759	.0776	.0845
.17	.0638	.0680	.0717	.0748	.0775	.0800	.0823	.0845	.0867	.0947
.18	.0695	.0744	.0786	.0822	.0854	.0883	.0910	.0936	.0961	.105
.19	.0753	.0809	.0857	.0900	.0936	.0970	.100	.103	.106	.117
.20	.0813	.0875	.0932	.0979	.102	.106	.110	.113	.116	.129
.21	.0873	.0944	.101	.106	.111	.115	.120	.123	.127	.142
.22	.0935	.101	.109	.115	.120	.125	.130	.134	.139	.155
.23	.0997	.109	.117	.124	.130	.135	.141	.146	.151	.169
.24	.106	.116	.125	.133	.139	.146	.152	.157	.163	.184
.25	.113	.124	.133	.142	.150	.157	.163	.170	.176	.199
.26	.119	.131	.142	.152	.160	.168	.175	.182	.189	.215
.27	.126	.139	.151	.162	.171	.180	.188	.195	.203	.232
.28	.133	.147	.160	.172	.182	.192	.201	.209	.217	.249
.29	.139	.155	.170	.182	.193	.204	.214	.223	.232	.267
.30	.146	.163	.179	.193	.205	.217	.227	.238	.248	.286
.31	.153	.172	.189	.204	.217	.230	.242	.253	.264	.306
.32	.160	.180	.199	.215	.230	.243	.256	.269	.281	.327
.33	.167	.189	.209	.227	.243	.257	.271	.285	.298	.348
.34	.174	.198	.219	.238	.256	.272	.287	.301	.315	.369
.35	.181	.207	.230	.251	.270	.287	.303	.318	.334	.392
.36	.190	.216	.241	.263	.283	.302	.319	.336	.353	.416
.37	.196	.225	.251	.275	.297	.317	.336	.354	.372	.440
.38	.203	.234	.263	.289	.311	.333	.354	.373	.392	.465
.39	.210	.244	.274	.301	.326	.349	.371	.392	.412	.491
.40	.218	.254	.286	.314	.341	.366	.389	.412	.433	.518
.41	.225	.263	.297	.328	.357	.383	.408	.432	.455	.545
.42	.233	.279	.310	.342	.373	.401	.427	.453	.478	.574
.43	.241	.282	.321	.356	.389	.418	.447	.474	.501	.604
.44	.249	.292	.334	.371	.405	.437	.467	.496	.524	.634

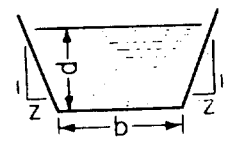
[1]For d/b less than 0.04, use of the assumption R = d is more convenient and more accurate than interpolation in the table.

Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.*—**Continued, from Reference III-3.**



d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.45	.256	.303	.346	.385	.422	.455	.487	.519	.548	.665
.46	.263	.313	.359	.401	.439	.475	.509	.541	.574	.696
.47	.271	.323	.371	.417	.457	.494	.530	.565	.600	.729
.48	.279	.333	.384	.432	.475	.514	.552	.589	.626	.763
.49	.287	.345	.398	.448	.492	.534	.575	.614	.652	.797
.50	.295	.356	.411	.463	.512	.556	.599	.639	.679	.833
.52	.310	.377	.438	.496	.548	.599	.646	.692	.735	.906
.54	.327	.398	.468	.530	.590	.644	.696	.746	.795	.984
.56	.343	.421	.496	.567	.631	.690	.748	.803	.856	1.07
.58	.359	.444	.526	.601	.671	.739	.802	.863	.922	1.15
.60	.375	.468	.556	.640	.717	.789	.858	.924	.988	1.24
.62	.391	.492	.590	.679	.763	.841	.917	.989	1.06	1.33
.64	.408	.516	.620	.718	.809	.894	.976	1.05	1.13	1.43
.66	.424	.541	.653	.759	.858	.951	1.04	1.13	1.21	1.53
.68	.441	.566	.687	.801	.908	1.01	1.10	1.20	1.29	1.64
.70	.457	.591	.722	.842	.958	1.07	1.17	1.27	1.37	1.75
.72	.474	.617	.757	.887	1.01	1.13	1.24	1.35	1.45	1.87
.74	.491	.644	.793	.932	1.07	1.19	1.31	1.43	1.55	1.98
.76	.508	.670	.830	.981	1.12	1.26	1.39	1.51	1.64	2.11
.78	.525	.698	.868	1.03	1.18	1.32	1.46	1.60	1.73	2.24
.80	.542	.725	.906	1.08	1.24	1.40	1.54	1.69	1.83	2.37
.82	.559	.753	.945	1.13	1.30	1.47	1.63	1.78	1.93	2.51
.84	.576	.782	.985	1.18	1.36	1.54	1.71	1.87	2.03	2.65
.86	.593	.810	1.03	1.23	1.43	1.61	1.79	1.97	2.14	2.80
.88	.610	.839	1.07	1.29	1.49	1.69	1.88	2.07	2.25	2.95
.90	.627	.871	1.11	1.34	1.56	1.77	1.98	2.17	2.36	3.11
.92	.645	.898	1.15	1.40	1.63	1.86	2.07	2.28	2.48	3.27
.94	.662	.928	1.20	1.46	1.70	1.94	2.16	2.38	2.60	3.43
.96	.680	.960	1.25	1.52	1.78	2.03	2.27	2.50	2.73	3.61
.98	.697	.991	1.29	1.58	1.85	2.11	2.37	2.61	2.85	3.79
1.00	.714	1.02	1.33	1.64	1.93	2.21	2.47	2.73	2.99	3.97
1.05	.759	1.10	1.46	1.80	2.13	2.44	2.75	3.04	3.33	4.45
1.10	.802	1.19	1.58	1.97	2.34	2.69	3.04	3.37	3.70	4.96
1.15	.846	1.27	1.71	2.14	2.56	2.96	3.34	3.72	4.09	5.52
1.20	.891	1.36	1.85	2.33	2.79	3.24	3.68	4.09	4.50	6.11
1.25	.936	1.45	1.99	2.52	3.04	3.54	4.03	4.49	4.95	6.73
1.30	.980	1.54	2.14	2.73	3.30	3.85	4.39	4.90	5.42	7.39
1.35	1.02	1.64	2.29	2.94	3.57	4.18	4.76	5.34	5.90	8.10
1.40	1.07	1.74	2.45	3.16	3.85	4.52	5.18	5.80	6.43	8.83
1.45	1.11	1.84	2.61	3.39	4.15	4.88	5.60	6.29	6.98	9.62

Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.*—**Continued, from Reference III-3.**



d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
1.50	1.16	1.94	2.78	3.63	4.46	5.26	6.04	6.81	7.55	10.4
1.55	1.20	2.05	2.96	3.88	4.78	5.65	6.50	7.33	8.14	11.3
1.60	1.25	2.15	3.14	4.14	5.12	6.06	6.99	7.89	8.79	12.2
1.65	1.30	2.27	3.33	4.41	5.47	6.49	7.50	8.47	9.42	13.2
1.70	1.34	2.38	3.52	4.69	5.83	6.94	8.02	9.08	10.1	14.2
1.75	1.39	2.50	3.73	4.98	6.21	7.41	8.57	9.72	10.9	15.2
1.80	1.43	2.62	3.93	5.28	6.60	7.89	9.13	10.4	11.6	16.3
1.85	1.48	2.74	4.15	5.59	7.01	8.40	9.75	11.1	12.4	17.4
1.90	1.52	2.86	4.36	5.91	7.43	8.91	10.4	12.4	13.2	18.7
1.95	1.57	2.99	4.59	6.24	7.87	9.46	11.0	12.5	14.0	19.9
2.00	1.61	3.12	4.83	6.58	8.32	10.0	11.7	13.3	14.9	21.1
2.10	1.71	3.39	5.31	7.30	9.27	11.2	13.1	15.0	16.8	23.9
2.20	1.79	3.67	5.82	8.06	10.3	12.5	14.6	16.7	18.7	26.8
2.30	1.89	3.96	6.36	8.86	11.3	13.8	16.2	18.6	20.9	30.0
2.40	1.98	4.26	6.93	9.72	12.5	15.3	17.9	20.6	23.1	33.4
2.50	2.07	4.58	7.52	10.6	13.7	16.8	19.8	22.7	25.6	37.0
2.60	2.16	4.90	8.14	11.6	15.0	18.4	21.7	25.0	28.2	40.8
2.70	2.26	5.24	8.80	12.6	16.3	20.1	23.8	27.4	31.0	44.8
2.80	2.35	5.59	9.49	13.6	17.8	21.9	25.9	29.9	33.8	49.1
2.90	2.44	5.95	10.2	14.7	19.3	23.8	28.2	32.6	36.9	53.7
3.00	2.53	6.33	11.0	15.9	20.9	25.8	30.6	35.4	40.1	58.4
3.20	2.72	7.12	12.5	18.3	24.2	30.1	35.8	41.5	47.1	68.9
3.40	2.90	7.97	14.2	21.0	27.9	34.8	41.5	48.2	54.6	80.2
3.60	3.09	8.86	16.1	24.0	32.0	39.9	47.8	55.5	63.0	92.8
3.80	3.28	9.81	18.1	27.1	36.3	45.5	54.6	63.5	72.4	107
4.00	3.46	10.8	20.2	30.5	41.1	51.6	61.9	72.1	82.2	122
4.50	3.92	13.5	26.2	40.1	54.5	68.8	82.9	96.9	111	164
5.00	4.39	16.7	33.1	51.5	70.3	89.2	108	126	145	216

Table III-2.—*Uniform flow in circular sections flowing partly full.* **From Reference III-3.**

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius

Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

TABLE III-3

VALUES OF $BD^{3/2}$

B x D	$BD^{3/2}$	B x D	$BD^{3/2}$	B x D	$BD^{3/2}$
4 x 4	32.0	7 x 7	129.6	10 x 10	316.2
5 x 4	40.0	8 x 7	148.2	12 x 10	379.4
6 x 4	48.0	9 x 7	166.7	14 x 10	442.7
7 x 4	56.0	10 x 7	185.2	16 x 10	505.9
8 x 4	64.0	12 x 7	222.2		
		14 x 7	259.3	12 x 12	498.8
5 x 5	55.9			14 x 12	582.0
6 x 5	67.1	8 x 8	181.0	16 x 12	665.1
7 x 5	78.3	9 x 8	203.7	18 x 12	748.3
8 x 5	89.4	10 x 8	226.3		
9 x 5	100.6	12 x 8	271.6	14 x 14	733.3
10 x 5	111.8	14 x 8	316.8	16 x 14	838.1
				18 x 14	942.8
6 x 6	88.2	9 x 9	243.0		
7 x 6	102.9	10 x 9	270.0		
8 x 6	117.6	12 x 9	324.0		
9 x 6	132.3	14 x 9	378.0		
10 x 6	147.0				
12 x 6	176.4				

VALUES OF $D^{3/2}$

D	$D^{3/2}$	D	$D^{3/2}$	D	$D^{3/2}$
4	8.0	8	22.6	12	41.6
5	11.2	9	27.0	13	46.9
6	14.7	10	31.6	14	52.4
7	18.5	11	36.5	15	58.1

VALUES OF $D^{5/2}$

D	$D^{5/2}$	D	$D^{5/2}$	D	$D^{5/2}$
1.0	1.0	5.0	55.9	9.0	243.0
1.5	2.8	5.5	70.9	9.5	278.2
2.0	5.7	6.0	88.2	10.0	316.2
2.5	9.9	6.5	107.7	10.5	357.3
3.0	15.6	7.0	129.6	11.0	401.3
3.5	22.9	7.5	154.0	11.5	448.5
4.0	32.0	8.0	181.0	12.0	498.8
4.5	43.0	8.5	210.6	12.5	552.4

IV. FLOW TRANSITIONS

A flow transition, as discussed here, is a change of open channel flow cross section designed to be accomplished in a short distance with a minimum amount of flow disturbance. The types of transitions are shown in figure IV-1. Of these the abrupt (headwall) and the straight line (wingwall) are the most common.

Specially designed open channel flow inlet transitions (contractions) are normally not required for highway culverts. The economical culvert is designed to operate with an upstream headwater pool which dissipates the channel approach velocity and, therefore, negates the need for an approach flow transition. The side and slope tapered inlets (see reference II-A-3), are designed as submerged transitions and do not fall within the intended limits of open channel transitions discussed in this chapter.

Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater considerations such as an irrigation structure in subcritical flow (section IV-A-2), or where it is desirable to maintain a small cross section with supercritical flow in a steep channel (section IV-B-1).

Outlet transitions (expansions) must be considered in the design of all culverts, channel protection, and energy dissipators. Of interest to the highway engineer are the standard wingwall-apron combinations which are abrupt expansions and expansions upstream of dissipator basins (chapter VII).

Transition designs fall into two general categories: Those applicable to culverts in outlet control (subcritical flow) or those applicable to culverts in inlet control (supercritical). For design, use section IV-A for culverts in outlet control and section IV-B for culverts in inlet control.

IV-A. CULVERTS IN OUTLET CONTROL

Two types of design problems apply to culverts in outlet control: abrupt expansions and gradial transitions.

ABRUPT EXPANSION

As a jet of water, which is not laterally constrained, leaves a culvert flowing in outlet control, the water surface plunges or drops very rapidly (see figure IV-A-1). As

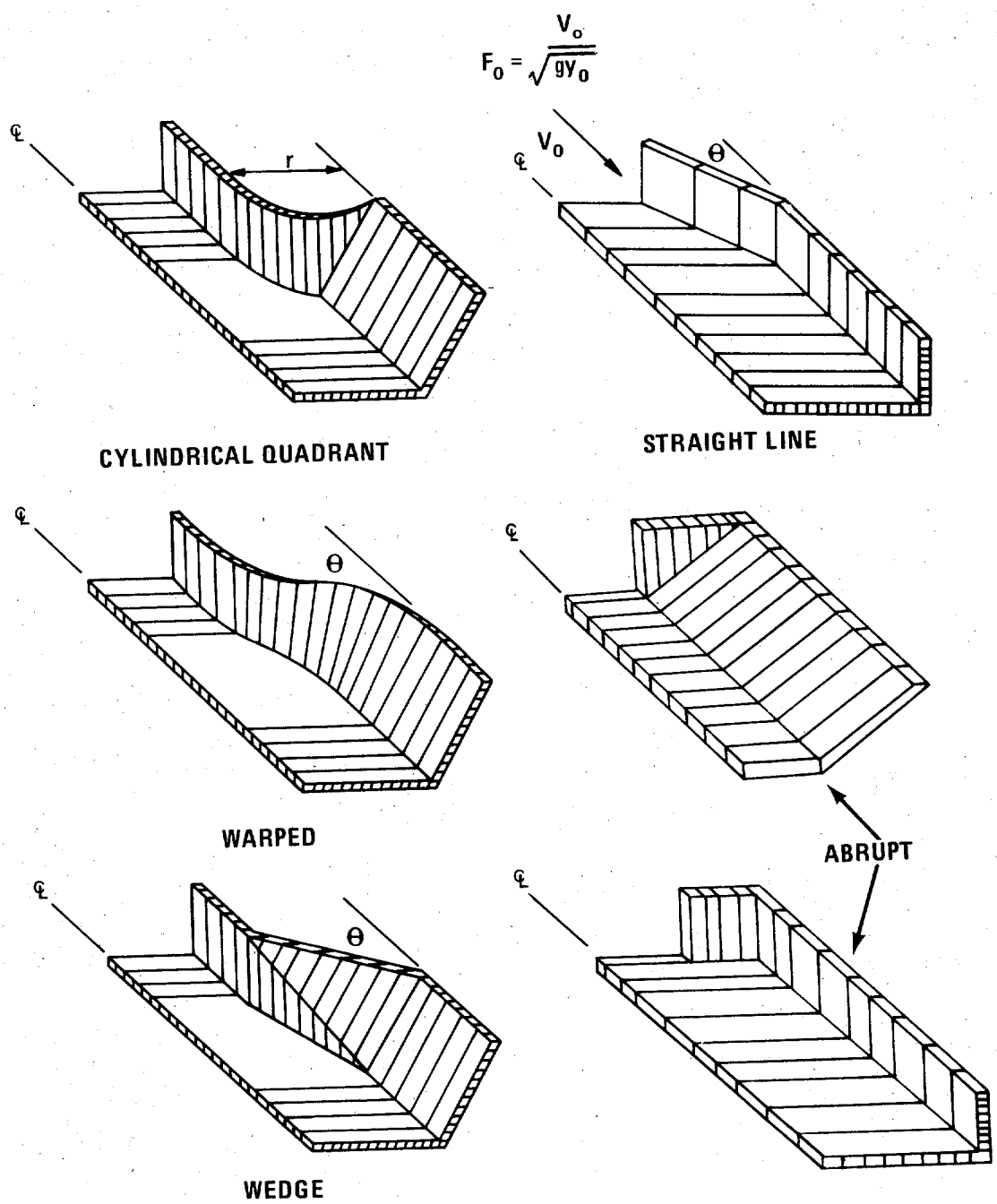


FIGURE IV-1. TRANSITION TYPES

the water surface drops and the flow spreads out, the potential energy stored as depth is converted to kinetic energy or velocity. Therefore, the velocity leaving the wingwall apron can be higher than the culvert outlet velocity and must be considered in determining outlet protection. The straight line transition may also be considered an abrupt transition if the tanθ is greater than 1/3Fr.

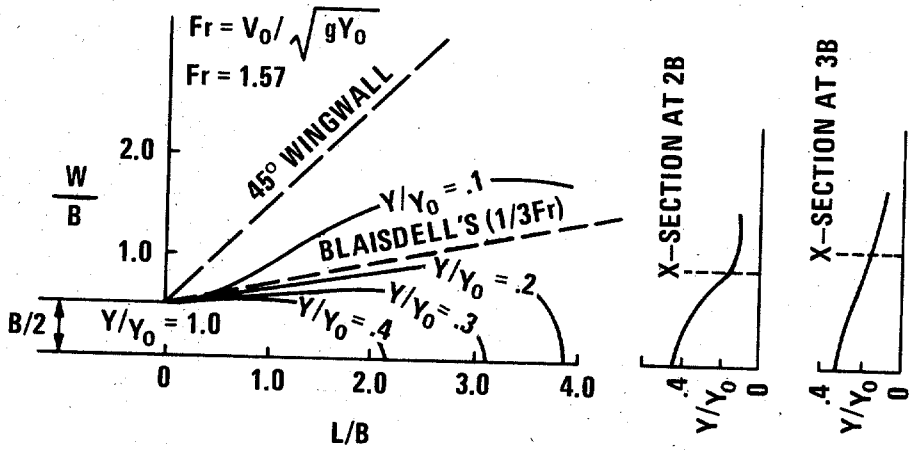


FIGURE IV-A-1 DIMENSIONLESS WATER SURFACE CONTOURS FROM REFERENCE IV-A-1

Design Considerations

A reasonable estimate of transition end velocity can be obtained by using the energy equation and assuming the losses to be negligible. By neglecting friction losses, a higher velocity than actually occurs is predicted making the error on the conservative side.

A more accurate way to determine apron end flow conditions is that developed by Watts (IV-A-1). Watts' experimental data have been converted to a family of curves relating the Froude number (Fr) to the average depth--brink depth ratio (y_A/y_o), figures IV-A-4 and 5 and Fr or $Q/\sqrt{gD^5}$ to V_A/V_o figures IV-A-2 and 3. These curves were developed for Fr from 1 to 2.5. This is the applicable Froude number range for most abrupt outlet transitions. Normally, low tailwater is encountered at the culvert outlet and flow is supercritical on the outlet apron.

Water cannot expand to completely fill the section between the wingwalls in an abrupt expansion. The majority of the flow will stay within an area whose boundaries are defined by $\tan\theta = 1/3Fr$. As shown in figure IV-A-1, flaring the wingwall more than 1/3Fr--45° for example--provides unused space which is not completely filled with water.

Design Procedure

1. Determine the flow conditions at the culvert outlet: (V_o) and (y_o) (see chapter III).

2. Calculate the Froude number (Fr) = $V_o/\sqrt{gy_o}$ at the culvert outlet.

3. Find the optimum flare angle (θ) using $\tan\theta = 1/3Fr$. If the chosen wingwall flare (θ_W) is greater than (θ), consider reducing θ_W to θ.

4. Use figure IV-A-4 for boxes and IV-A-5 for pipes to find the average depth downstream. The ratio y_A/y_o is obtained knowing the Froude number (Fr) and the desired distance downstream (L) expressed in culvert diameters (D).

5. Use figure IV-A-2 for boxes and IV-A-3 for pipes to find average velocity (V_A).

6. Calculate the downstream width (W_2) using:

$$W_2 = W_o + 2L\tan\theta \quad \ldots\ldots\ldots\ldots \quad \text{IV-A-1}$$
$$\tan\theta = 1/3Fr \quad \ldots\ldots\ldots\ldots \quad \text{IV-A-2}$$

if $\theta_W > \theta$ use θ_W in equation IV-A-1.

7. If θ was used in equation IV-A-1, calculate downstream depth y_2 using W_2 and V_A. This depth will be larger than y_A since the flow prism is now laterally confined.

If θ_W was used, $y_2 = y_A$ and the average flow width is $(W_A) = Q/V_A y_A$. If $W_A > W_2$, use W_2 to calculate $Y_2 = Q/V_A W_2$.

Example Problem

Given:

5 x 5 RCB	Q = 270 cfs
L = 200 ft.	$Q/BD^{3/2}$ = 4.83
S_o = 0.002 ft/ft.	d_c = 4.5

Wingwall flare $\theta_W = 45°$ with 10 foot apron

Find: Flow condition at end of apron - y and v.

Solution:

1. Find outlet velocity from figure III-9 with

   $Q/BD^{3/2}=4.83$ and $TW/D \simeq 0$
   $Y_O/D = 0.68$
   $Y_O=0.68(5)=3.40$
   $V_O=Q/A=270/3.4(5)=15.88$ fps

2. Find outlet Froude number

   $Fr = V_O/\sqrt{gY_O} = 15.88/\sqrt{109.48} = 1.52$

3. Find θ

   $\tan\theta = 1/3Fr = 1/3(1.52) = 0.22$
   $\theta = 12.37$

4. Apron Length/Diameter = 10/5 = 2
   Use figure IV-A-4 for average depth, Y_A.

   $Y_A = 0.26(3.4) = 0.88$ feet

5. From figure IV-A-2 the average velocity V_A is:

   $V_A/V_O = 1.2$
   $V_A = 15.88(1.2)$
   $V_A = V_2 = 19.1$ fps

6. $\theta_W > \theta$ use θ_W

   $W_2 = W_O + 2L \tan(\theta_W)$
   $W_2 = 5 + 2(10)(1.0) = 25$ feet

7. θ_W was used:

   $Y_2 = Y_A = 0.88$ feet
   $W_A = Q/V_A Y_A = 270/(19.1)0.88$
   $W_A = 16.1$ ft.< 25 ft.

Alternate Solutions Using Energy Equation

(1) Assume W_2 = full width between wingwalls at the end of the apron

   $W_2 = W_O + 2L \tan 45° = 25$ feet
   $A_2 = W_2 Y_2 = 25\ Y_2$, $V_2 = Q/A_2 = 270/25Y_2 = 10.8/Y_2$

$z_o + y_o + V_o^2/2g = z_2 + y_2 + V_2^2/2g + H_f$

$H_f = 0$ and $z_o = z_2$

$3.4 + (15.88)^2/64.4 = y_2 + (10.8/y_2)^2/64.4$

$3.4 + 3.92 = y_2 + 1.81/y_2$

$7.32 = y_2 + 1.81/y_2^2$

$y_2 = 0.516$ feet, which is 41% lower than first solution

$V_2 = 10.8/0.516 = 20.9$ fps 10%, which is higher than first solution

2. Another depth approximation can be obtained if W_2 is based on θ where $\tan\theta = 1/3Fr$

$W_2 = W_o + 2L \tan 12.41$

$= 5 + 20(.22) = 9.4$ feet

$A_2 = 9.4y_2$, $V_2 = 270/9.4y_2 = 28.7/y_2$

$7.32 = y_2 + 12.8/y_2^2$

$y_2 = 1.48$ feet, which is 68% higher than first solution

$V_2 = 28.7/1.48 = 19.4$, which is 2% higher than first solution

DESIGN OF SUBCRITICAL FLOW TRANSITIONS

Subcritical flow can be transitioned into and out of highway structures without causing adverse effect if subcritical flow is maintained throughout the structure. The flow cannot approach or pass through critical depth (y_c). The range of depths to avoid is $.9y_c$ to $1.1y_c$. In this range, slight changes in specific energy are reflected in large changes in depth, i.e., wave problems develop.

The straight line or wedge transition should be used if conservation of flow energy is required; such as in irrigation canal structures which traverses the highway. Warped and cylindrical transitions are more efficient, but the additional construction cost can only be justified for structures where backwater is critical.

Design Considerations

Figure IV-A-6 illustrates the design problem. Starting upstream of section (1) where some backwater exists due to the culvert, the flow is transitioned from a canal into then out of the highway culvert. The flare angle (θ_W) should be 12.5°, (4.5 to 1 or flatter)(VI-A-3). This criteria provides a gradually varied transition which can be analyzed using the energy equation.

As the flow transitions into the culvert the water surface approaches y_c. To minimize waves, y should be equal to or greater than $1.1y_c$. In the culvert, the depth will

increase and will reach y_n if the culvert is long enough. In the expansion (section 3 to 4), the depth increases to y_n of the downstream channel, section 4.

Associated with both transitions are energy losses which are proportional to the change in velocity head in the transitions. The energy loss in the contraction (H_{LC}) is:

$$H_{LC} = C_c(V_2^2/2g-V_1^2/2g) \quad \text{IV-A-3}$$

and in the expansion

$$H_{Le} = C_e(V_3^2/2g-V_4^2/2g) \quad \text{IV-A-4}$$

where C_c and C_e are found from table IV-A-1.

TABLE IV-A-1. TRANSITION LOSS COEFFICIENTS (IV-A-4)

Transition Type	Contraction C_c	Expansion C_e
Warped	0.10	0.20
Cylindrical quadrant	0.15	0.25
Wedge	0.30	0.50
Straight line	0.30	0.50
Square end	0.30	0.75

The depth in the culvert y_3 can be found by trial and error using the energy equation with $y_4=y_n$ in the downstream channel and assuming $H_{f2}=0$ (see figure IV-A-6). The stream-bed elevation is equal to z.

$$z_4+y_4+V_4^2/2g+H_{Le}+H_{f2}=z_3+y_3+V_3^2/2g$$

$$H_{f2}\simeq 0, \quad V_3=Q/W_3y_3, \quad V_4= Q/W_4y_4$$

$$z_4+y_4+V_4^2/2g+C_e(V_3^2/2g-V_4^2/2g)=z_3+y_3+V_3^2/2g$$

$$z_4+y_4+(1-C_e)V_4^2/2g=z_3+y_3+(1-C_e)V_3^2/2g$$

$$z_4-z_3+y_4+(1-C_e)(Q/W_4y_4)^2/2g=y_3+(1-C_e)(Q/W_3y_3)^2/2g. \quad \text{IV-A-5}$$

When known values are used in equation IV-A-5, the equation reduces to

$$C_1=y_3+C_2/y_3^2$$

which has two constants C_1 and C_2 and can be quickly solved by trail and error.

In a similar manner, y_1 can be determined by assuming $y_2=y_3$ and $H_{f1}=0$.

$$z_2+y_2+V_2^2/2g+H_{Lc}+H_{f1}=z_1+y_1+V_1^2/2g$$

$$z_2+y_2+V_2^2/2g+C_c(V_2^2/2g-V_1^2/2g)=z_1+y_1+V_1^2/2g$$

$$z_2+y_2+(1+C_c)V_2^2/2g=z_1+y_1+(1+C_c)V_1^2/2g$$

$$z_2-z_1+y_2+(1+C_c)(Q/W_2y_2)^2/2g=y_1+(1+C_c)(Q/W_1y_1)^2/2g \quad \text{IV-A-6}$$

These depths are approximate since friction loss were neglected. They should be checked by computing the water surface profile through the structure. Since the channel width is changing, the standard step method (IV-A-5) should be used.

Standard Step Method of Water Surface Profile Computation

The standard step method is a trial and error procedure for computing the water surface profile. The energy equation is used for the energy balance. Manning's equation in the following form:

$$S_f=[n^2V^2/2.22R^{4/3}] \quad \text{IV-A-7}$$

is used to calculate the friction slope (S_f) at each section. The friction loss (H_f) can then be approximated over a small distance (ΔL) by calculating S_f at both ends of the section and using the average (S_f).

$$H_f=S_f\Delta L \quad \text{IV-A-8}$$

The head loss (H_L) due to the contraction or expansion is normally calculated for the entire length of the transition (L_T) and then proportioned equally over the length L_T:

$$H_L= H_{Le}(\Delta L/L_T) \text{ or } H_{Lc}(\Delta L/L_T) \quad \text{IV-A-9}$$

To aid in this computational procedure, the elevation of the water surface is designated (Z) where:

$$Z=z+y \quad \text{IV-A-10}$$

and the total head (H) at a section is equal to

$$H=z+y+V^2/2g=Z+V^2/2g \quad \text{IV-A-11}$$

An energy balance is written between section (K) of known y and V and a section (x), a small distance (ΔL) upstream for subcritical flow or downstream for supercritical flow.

$$H_k+H_f+H_L=H_x \quad \text{. . . . . IV-A-12}$$

To find H_x, choose ΔL and assume a Z_x slightly larger than Z_k for subcritical or slightly smaller for supercritical. This defines

$$z_x=z_k+S_o\Delta L \quad \text{. . . . . IV-A-13}$$

and

$$y_x=Z_x-z_x \quad \text{. . . . . IV-A-14}$$

With y_x known, V_x can be found by the continuity equation $Q=AV$. H_x is determined from equation IV-A-11. Using y_x and V_x in equation IV-A-7, S_f is found at section X, $(S_f)_x$. Since $(S_f)_k$ was previously found for the known section, the average S_f is calculated and used in equation IV-A-8 to find H_f. Equation IV-A-9 provides H_L. If we assumed Z_x correctly, H_x (equation IV-A-12) should be equal to $H_k+H_f+H_L$. If not, chose another Z_x and repeat the procedure. When equation IV-A-12 has been balanced, and y and V are known at section X , the water surface computation proceeds to the next section.

Design Procedure

1. Find y4, and V4 knowing S_o, n, and approach geometry using Manning's equation or table III-3.

2. Calculate critical depth (y_c) using $y_c=0.315(Q/W)^{2/3}$ which is valid for rectangular channel or use figure III-3. Compare y_c with y_n to insure subcritical flow.

3. Choose transition type and C_c and C_e from table IV-A-1.

4. Determine the minimum culvert width by assuming y_c in the culvert and using equation IV-A-5.

5. Knowing y_c for the minimum width choose $y=1.1y_c$ to provide a culvert that will have a flow depth conservatively above y_c. Recalculate W_3, round to nearest even dimension and recalculate y_3.

6. Calculate y_1 by assuming $y_2=y_3$ in equation IV-A-6.

7. Backwater is equal to y_1-y_n. If the backwater exceeds canal freeboard choose a larger culvert width and calculate y_3 using equation IV-A-5 as reduced in step 3. Then use y_3 in equation IV-A-6 as reduced in step 5.

8. With the flow conditions known, calculate the transition length (L_T) using a 4.5:1 flare, $L_T=4.5(W_4-W_3)/2$ or $4.5(W_1-W_2)/2$

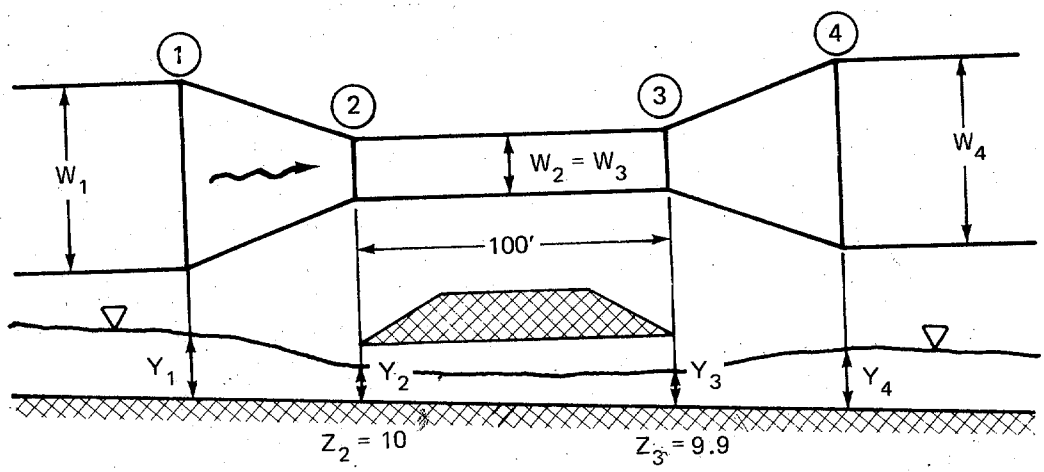
9. Calculate the water surface profile through the structure using the standard step method which includes an evaluation of friction losses.

Example Problem

Given: Ten-foot wide rectangular irrigation canal on a slope (S_o) of .001 ft/ft. The canal is designed to convey 300 cfs with a Manning's roughness (n) = .02. A rural highway will cross the canal requiring a 100 ft. long culvert.

Find: The culvert and transition dimensions.

Solution:



1. Assume normal depth at section 4

   $y_n=y_4=6.41$ ft. from Manning Eq. by trial and error
   $V_n=V_4=4.68$ fps
   $H_n=H_4=y_4+V_4^2/2g=6.75$ ft.

2. Determine critical depth (y_c) for section 4

   $y_c=0.315(Q/\bar{W})^{2/3}=.315(300/10)^{2/3}=3.04$ ft.

3. Use straight line transition $C_e=.5$, $C_c=.3$

4. Calculate minimum W_3 using equation IV-A-5 and $y_c=.315(Q/W_3)^{2/3}=y_3$ also $z_4-z_3=S_oL_T\approx 0$ even if $L_T=50$ ft., S_oL_T would only be .05 feet.

$$z_4-z_3+y_4+(1-C_e)(Q/W_4y_4)^2/2g=y_3+(1-C_e)(Q/W_3y_3)^2/2g$$
$$0+6.4+.5[300/10(6.4)]^2/2g=y_3+.5(300)^2/(W_3y_3)^2/(2g)$$
$$6.4+.17=6.6=y_3+699/(W_3y_3)^2$$

Try $W_3=4$ ft.

$$y_3=.315(300/4)^{2/3}=5.6$$
$$y_3+699/(W_3y_3)^2=5.6+699/[4(5.6)]^2=7 \text{ ft.} >6.6 \text{ ft.}$$

Try $W_3=5$ ft.

$$y_3=.315(300/5)^{2/3}=4.83$$
$$y_3+699/(W_3y_3)^2=4.8+699/[5(4.8)]^2=6 \text{ ft.} <6.6 \text{ ft.}$$

Try $W_3=4.4$ ft. since 4 was .4 high and 5 was .6 low

$$y_3=.315(300/4.4)^{2/3}=5.26$$
$$y_3+699/(W_3y_3)^2=5.3+699/[4.4(5.3)]^2=6.6 \text{ ft. ok}$$
$$y_3=5.3 \text{ ft.}$$

5. Choose $y_3=1.1y_c$ to insure subcritical flow and recalculate W_3, $y_3=1.1(5.3)=5.8$ ft.

$$6.6=5.8+699/[W_3(5.8)]^2$$
$$.8W_3^2(33.64)=699$$
$$W_3=5.1 \text{ ft.}$$

Use $W_3=5$ ft.

$$6.6=y_3+699/(5y_3)^2=y_3+28/y_3^2$$
$$y_3=5.75 \text{ ft.}$$
$$V_3=10.43 \text{ fps}$$

6. Assume $y_2=y_3$ calculate y_1 using equation IV-A-6

$$z_2-z_1+y_2+(1+C_c)(Q/W_2y_2)^2/2g=y_1+(1+C_c)(Q/W_1y_1)^2/2g$$
$$0+5.75+1.3[300/5(5.75)]^2/64.4=y_1+1.3(300/10y_1)^2/2g$$
$$5.75+2.2=7.95=y_1+18.2/y_1^2$$
$y_1=7.6$ ft. which is 1.2 ft. above y_n

7. Backwater of 1.2 ft. is all right since 2 feet of freeboard is available. If backwater was too high, return to step (4), choose another W_3 to use in $6.6=y+699/(W_3y_3)^2$, solve for y_3 then use equation IV-A-6 in step (5) $y_2+1.3(300/W_2y_2)^2/64.4=y_1+18.2/y_1^2$

For example try $W_3=6$ feet:

$$6.6=y_3+699/(6y_3)^2=y_3+19.42/y_3^2$$
$$y_3=6.07 \text{ feet}$$
$$6.07+1.3[300/6(6.07)]^2/64.4=y_1+18.2/y_1^2$$
$$7.44=y_1+18.2/y_1^2$$
$y_1=7.08$ feet or .67 feet of backwater

8. Transition length use 4.5:1

$L_T=4.5(W_1-W_2)/2=4.5(10-5)/2=11.25$ ft. use 11 ft.

9. Calculate the water surface profile.

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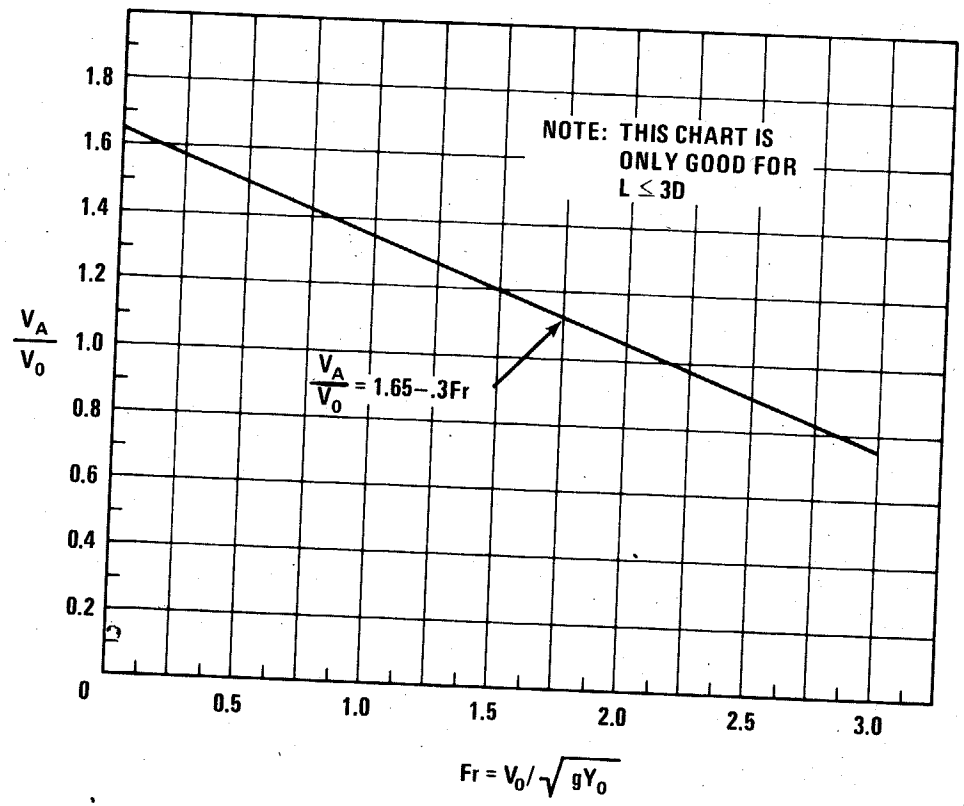
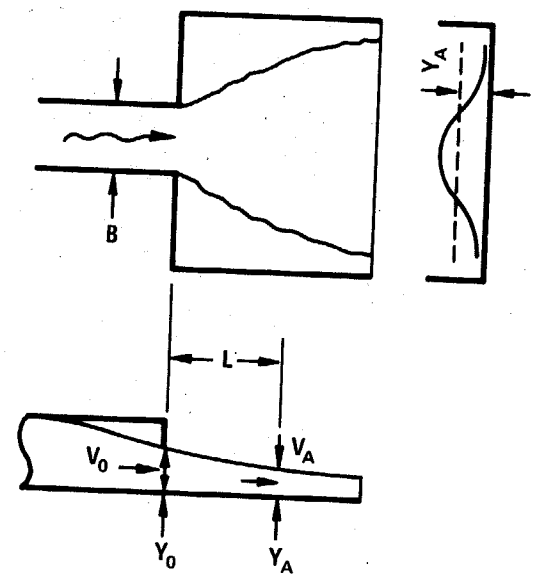
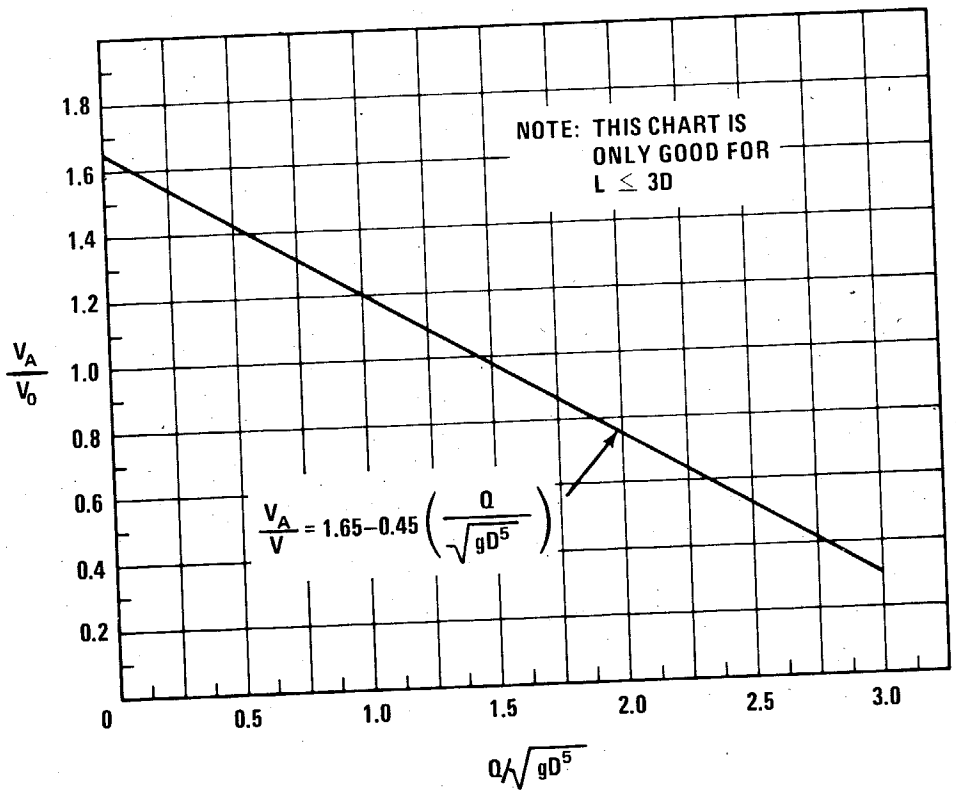
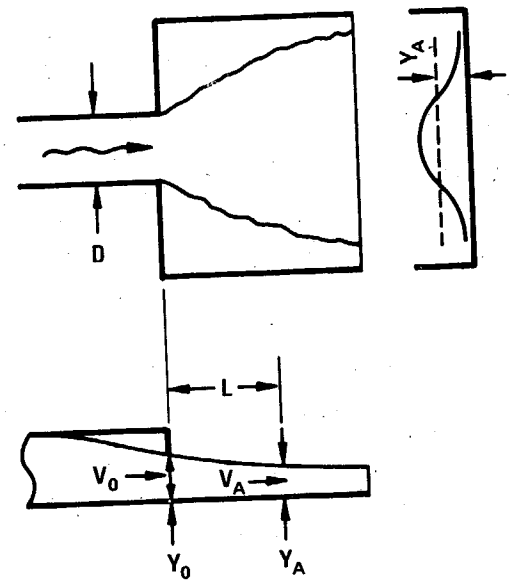




L = DISTANCE TO DESIRED VELOCITY
B = WIDTH OF CULVERT
Y_0 = BRINK DEPTH
Y_A = AVERAGE DEPTH OF FLOW
V_0 = VELOCITY AT CULVERT OUTLET
V_A = AVERAGE VELOCITY IN EXPANSION

FIGURE IV-A-2. AVERAGE VELOCITY FOR ABRUPT EXPANSION BELOW RECTANGULAR OUTLET FROM REFERENCE IV-A-1





L = DISTANCE TO DESIRED VELOCITY
D = DIAMETER OF CULVERT (FT)
Y_0 = BRINK DEPTH
Y_A = AVERAGE DEPTH OF FLOW
V_0 = VELOCITY AT CULVERT OUTLET
V_A = AVERAGE VELOCITY IN EXPANSION

FIGURE IV-A-3. AVERAGE VELOCITY FOR ABRUPT EXPANSION BELOW CIRCULAR OUTLET FROM REFERENCE IV-A-1

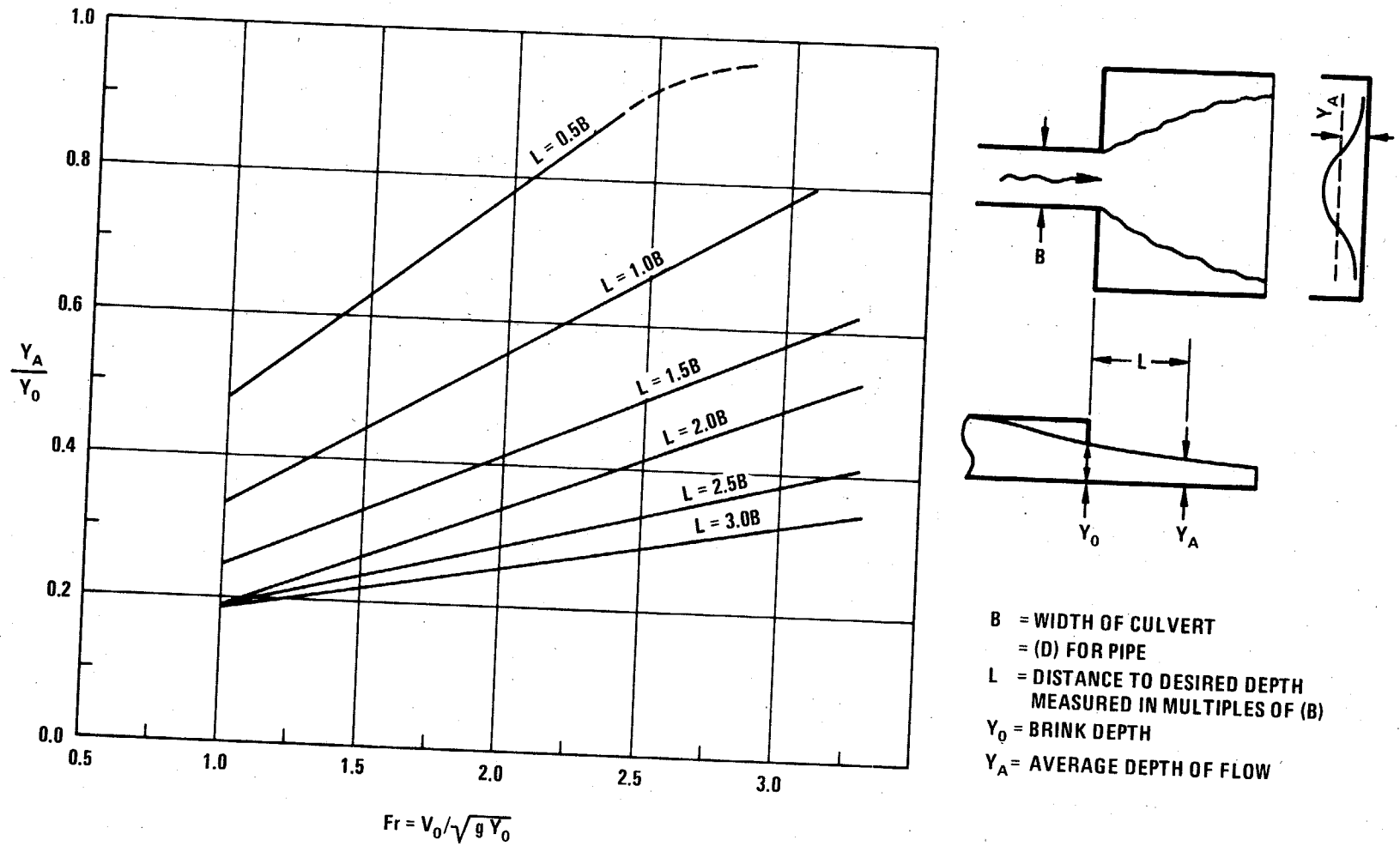


FIGURE IV-A-4. AVERAGE DEPTH FOR ABRUPT EXPANSION BELOW RECTANGULAR CULVERT OUTLET

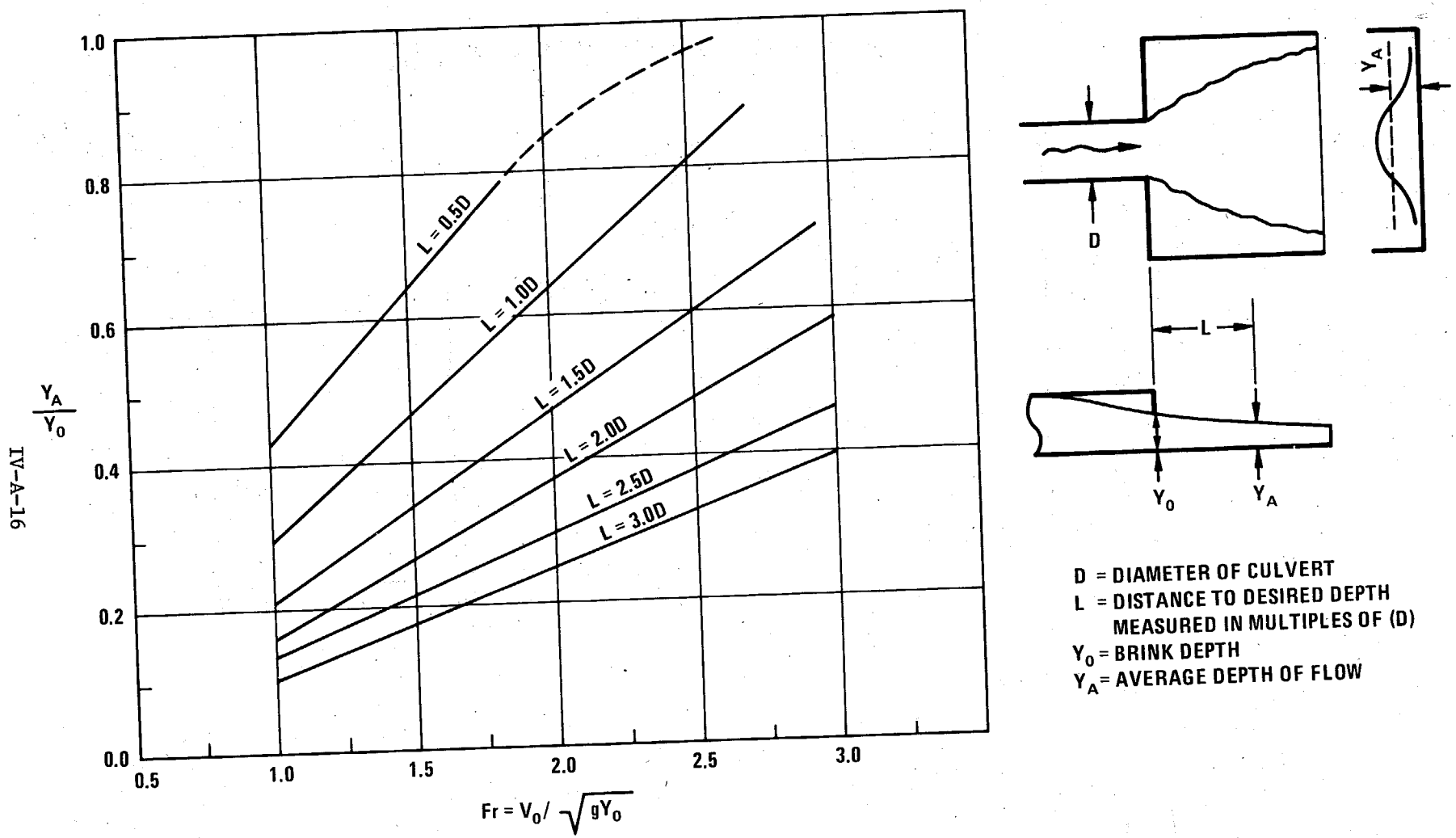


FIGURE IV-A-5. AVERAGE DEPTH FOR ABRUPT EXPANSION BELOW CIRCULAR CULVERT OUTLET

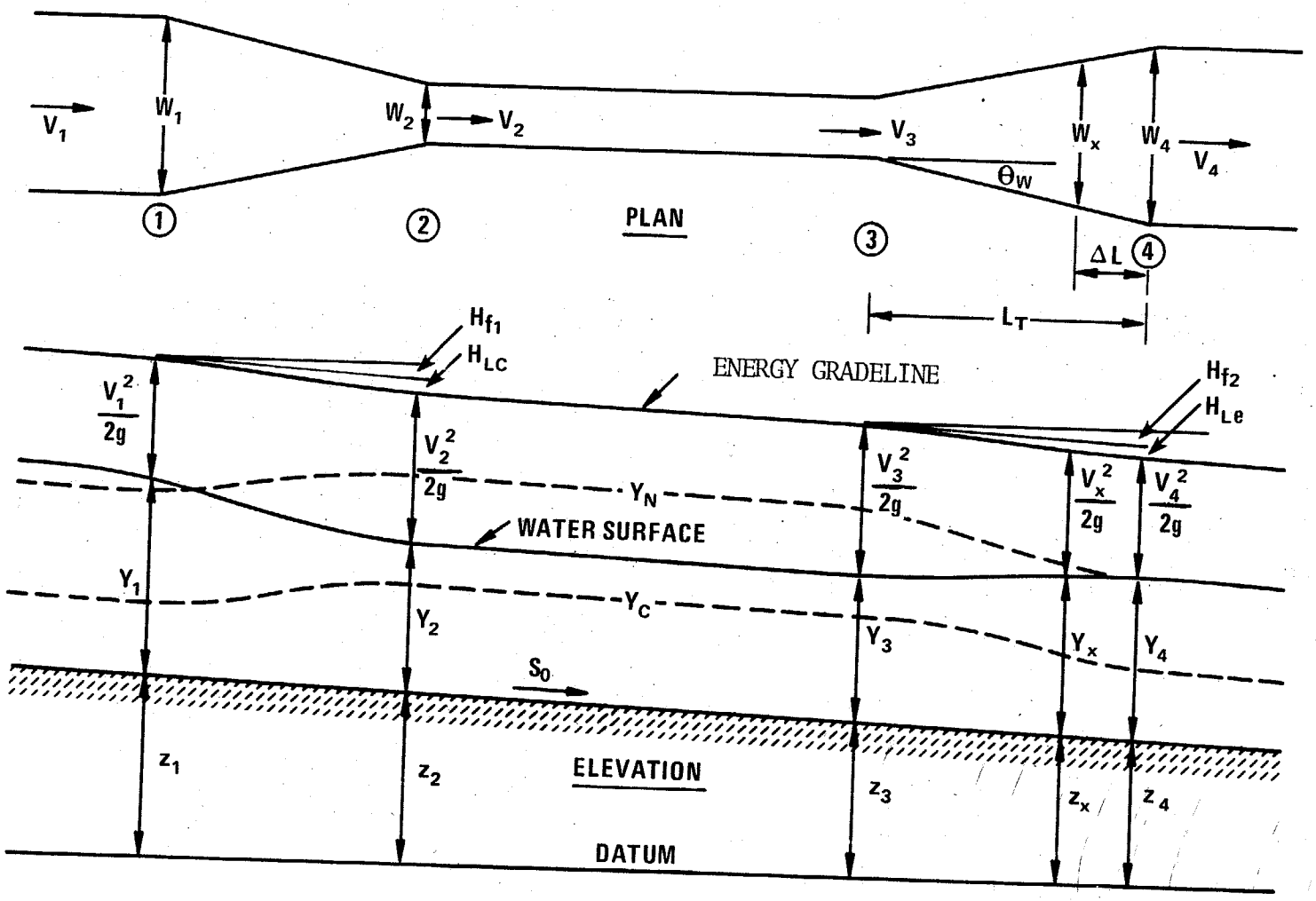


FIGURE IV-A-6. DEFINITION SKETCH

IV-B. CULVERTS IN INLET CONTROL

The design of transitions for culverts in inlet control requires transitioning supercritical flow. Supercritical flow is difficult to manage without causing a hydraulic jump or other surface irregularity; therefore, the full flow area should be maintained if at all possible. Both contractions and expansions are discussed in this section including an expansion design used to accelerate flow into a stilling basin.

SUPERCRITICAL FLOW CONTRACTION

To smoothly transition supercritical flow, requires a long structure and should not be attempted unless the structure is of primary importance. A model study should be used to determine transition geometry where a hydraulic jump is not desired. If a hydraulic jump is acceptable, the inlet structure can be designed as shown in figure IV-B-1. This design, which must be accomplished in a rectangular channel, yields a long transition. The design approach is outlined in reference IV-A-4 and 5.

The length (L) is defined by (W_1-W_2), the channel contraction, and the wall deflection angle (θ_W):

$$L=(W_1-W_2)/2\tan\theta_W \quad \text{IV-B-1}$$

To minimize surface disturbances, L should also equal L_1+L_2 where

$$L_1=W_1/2\tan\beta_1 \quad \text{IV-B-2}$$

$$L_2=W_2/2\tan(\beta_2-\theta_W) \quad \text{IV-B-3}$$

$$\tan\theta_W=\frac{\tan\beta_1(\sqrt{1+8Fr_1^2\sin^2\beta_1}-3)}{2\tan^2\beta_1+\sqrt{1+8Fr_1^2\sin^2\beta_1}-1} \quad \text{IV-B-4}$$

The transition design requires assuming a trial θ_W which fixes L as defined by equation IV-B-1. This length is then checked by finding L_1+L_2. To determine L_1, β_1 is found from equation IV-B-4 by trial and error and then substituted into equation IV-B-2. L_2 is calculated from equation IV-B-3 with β_2 determined from equation IV-B-4 by substituting β_2 for β_1 and Fr_1 for Fr_2. To find Fr_2 first calculate:

$$y_2/y_1=[\sqrt{1+8Fr_1^2\sin^2\beta_1}-1]/2 \quad \text{IV-B-5}$$

$$\text{then } Fr_2^2=(y_1/y_2)[Fr_1^2-(y_1/2y_2)(y_2/y_1-1)(y_2/y_1+1)^2] \quad \text{IV-B-6}$$

If the trial θ_W was chosen correctly $L=L_1+L_2$. If not, choose another trial θ_W and repeat the process until the lengths match. The depth (y_3) and Fr_3 in the culvert can now be calculated using equation IV-B-5 and 6 if the subscripts are increased by 1;.i.e, y_2/y_1 is now y_3/y_2. To aid in the above calculation, figures IV-B-2 and 3 are a graphical solution of equations IV-B-4, 5, and 6.

The above design approach assumes that the width of the channel (W_1) and the width of the culvert (W_2) are known and L is found by trial and error. If W_2 has to be determined, the design problem is complicated by another trial and error process.

Design Procedure

1. The flow conditions (y_n, V_n, Fr) in the approach channel should be computed using table III-1 or other design aids. If the channel is irregular, choose the trapezoidal section which best matches.

2. If the approach section is not rectangular, transition the section to a rectangular section with a bottom width (W_1) approximately equal to the average of the water surface top width (T) and the trapezoidal section base width (b): $W=(T+b)/2$. Compute the flow conditions (y_n, V_n, Fr) for this rectangular section.

3. Assume a trial culvert width W_2.

4. Determine the contraction length (L) required to reduce W_1 to W_2 by varying the contraction wingwall angle (θ_W) until L from equation IV-B-1 is equal to L_1+L_2 from equation IV-B-2 and 3.

   Select trial θ_W

   a. Calculate L using equation IV-B-1

   b. Find β_1, y_2/y_1, and Fr_1 from figure IV-B-2 or 3. If greater accuracy is desired use equations IV-B-4, 5, and 6.

   c. Calculate L_1 using equation IV-B-2.

d. Find β_2, y_3/y_2, and Fr_2 from figures IV-B-2 or 3 by increasing the subscripts shown on the figure by 1. Again equations IV-B-4, 5, and 6 can be used.

e. Calculate L_2 using equation IV-B-3.

f. Find the sum L_1+L_2 and compare with L: if L is smaller decrease θ_W; if L is larger increase θ_W. Select a new trial θ_W and repeat steps a through f until $L=L_1+L_2$.

g. Calculate y_3 by multiplying the depth ratios: $y_3=y_1(y_2/y_1)(y_3/y_2)$.

5. Compare the depth y_3 and width W_2 to see if a culvert of regular dimension (i.e., 6x6, 7x6) results. If not, return to step 3, assume another W_2, and repeat the process until a more favorable combination of y_3 and W_2 is found.

Example Problem

Given: Q=300 cfs in a 6 foot bottom trapezoidal channel with 2:1 side slopes, S_o=.02 ft/ft and n=0.012.

Find: Culvert size and transition dimensions

Solution:

1. $Qn/b^{8/3}S^{1/2}=300(.012)/6^{8/3}.02^{1/2}=.2141$
   $d/b=y_n/b=.278$ from table III-3
   $y_n=1.67$ ft., $V_n=19.2$ fps
   $Fr=V/\sqrt{g(A/T)}=19.2/\sqrt{g(15.6/12.7)}=3.05$

2. $(T+b)/2=(12.7+6)/2=9.4$ ft.
   use $W_1=10$ ft. rectangular channel
   $Qn/b^{8/3}S^{1/2}=300(.012)/10^{8/3}.02^{1/2}=.0548$
   $d/b=y_n/b=.154$ from table III-3
   $y_n=1.54$, $V_n=19.5$ fps
   $Fr_1=V/\sqrt{gy}=19.5/\sqrt{g1.54}=2.77$

3. Assume W_2=5 ft.

4. Try $\theta_W=15°$ for $Fr_1=2.8$
   A. $L=(10-5)/2\tan 15°=9.3$ ft.
   B. $\beta_1=36°$, $y_2/y_1=1.9$, $Fr_2=1.8$
   C. $L_1=10/2\tan 36°=6.9$ ft.
   D. $\beta_2=58°$, $y_3/y_2=1.7$, $Fr_3=1$
   E. $L_2=5/2\tan(58°-15°)=2.7$ ft.
   F. $L_1+L_2=9.6$ ft.> L

Try $\theta_W=10°$ for Fr =2.8

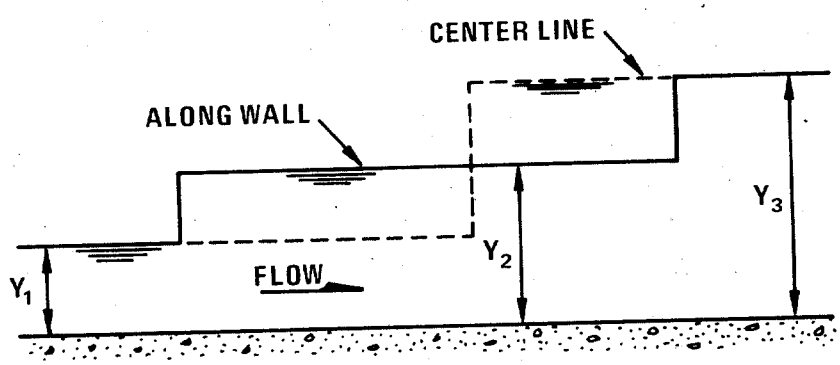
A. $L=(10-5)/2\tan 10°=14.2$ ft.
B. $\beta_1=31°$, $y_2/y_1=1.6$, $Fr_2=2.1$
C. $L_1=10/2\tan 31°=8.3$ ft.
D. $\beta_2=39°$, $y_3/y_2=1.5$, $Fr_3=1.5$
E. $L_2=5/2\tan(39°-10°)=4.5$
F. $L_1+L_2=12.8$ ft. $<$ 14.2 ft.
G. $y_3=1.54(1.5)1.6=3.7$ ft.

Try $\theta_W=14°$ for Fr =2.8

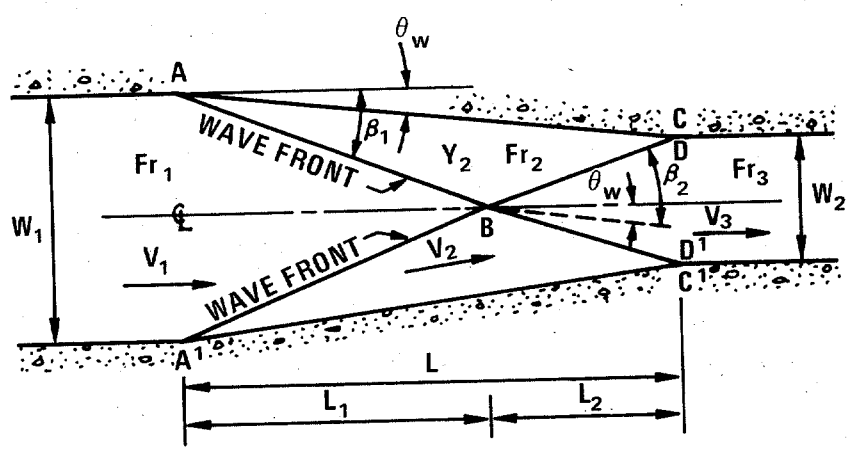
A. $L=(10-5)/2\tan 14°=10$ ft.
B. $\beta_1=35°$, $y_2/y_1=1.8$, $Fr_2=1.8$
C. $L_1=10/2\tan 35°=7.1$ ft.
D. $\beta_2=55°$, $y_3/y_2=1.6$, $Fr_3=1.1$
E. $L_2=5/2\tan(55°-14°)=2.9$ ft.
F. $L_1+L_2=10$ ft.=L, O.K.
G. $y_3=1.54(1.6)1.8=4.4$ ft.

Use $\theta_W=14°$, $y_3=4.4$ ft., $V_3=13.6$ fps, $Fr_3=1.1$
L=10 ft.

5. Since $y_3=4.4$ ft. and $W_3=5$ ft., a 5x5 box culvert will be satisfactory.



a. SCHEMATIC PROFILE



b. PLAN

FIGURE IV-B-1. SUPERCRITICAL INLET TRANSITION FOR RECTANGULAR CHANNEL FROM REFERENCE IV-A-4

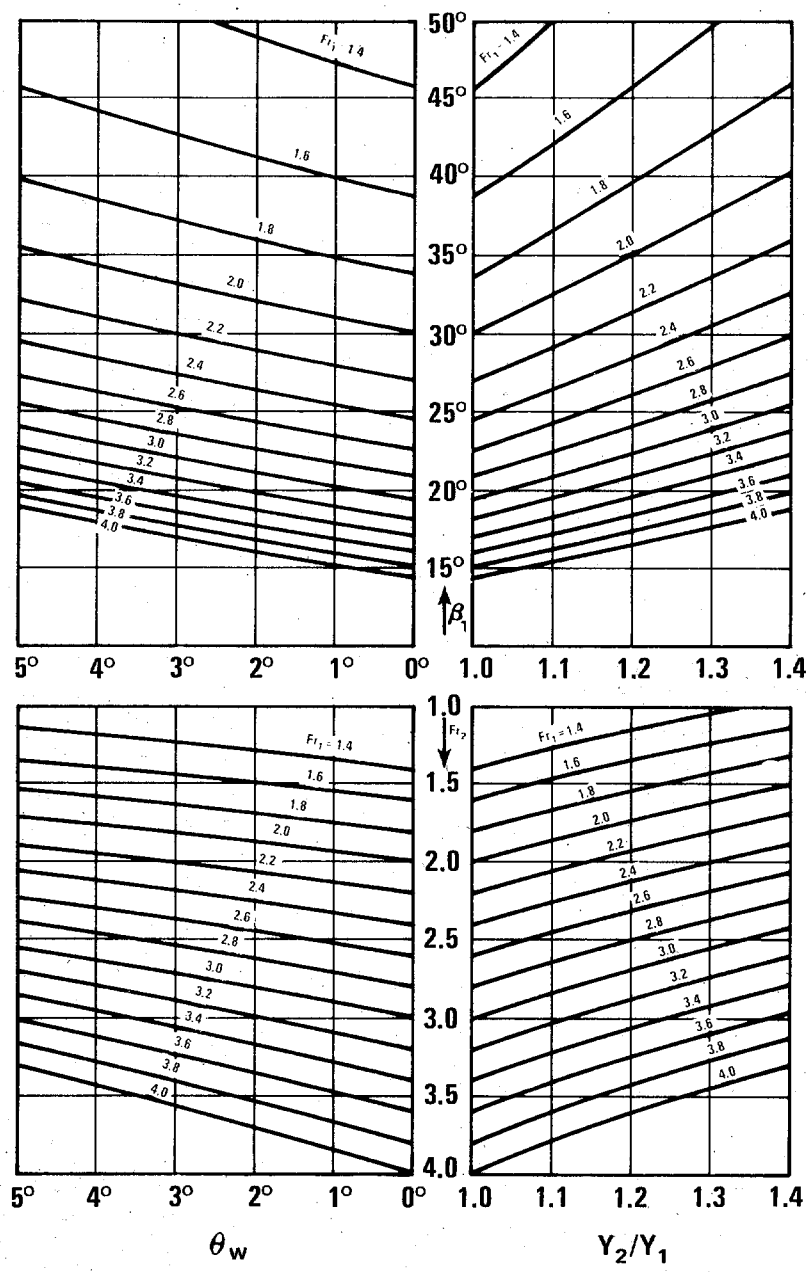


FIGURE IV-B-2 SUPERCRITICAL INLET TRANSITION DESIGN CURVES FOR RECTANGULAR CHANNELS FROM REFERENCE IV-A-4

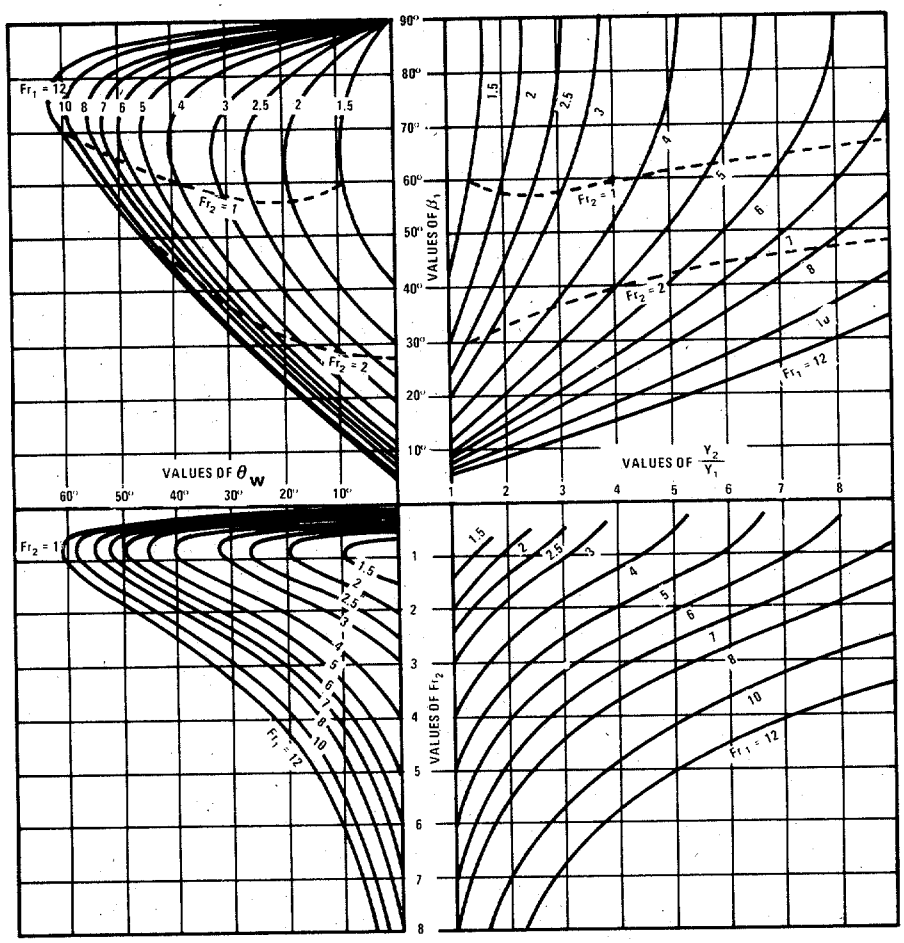


FIGURE IV-B-3. SUPERCRITICAL INLET TRANSITION DESIGN CURVES FOR RECTANGULAR CHANNELS FROM REFERENCE IV-A-5

SUPERCRITICAL FLOW EXPANSION

Supercritical expansion design has in part been discussed in section IV-A. The procedure outlined in that section should be used to determine apron or expansion flow conditions if the culvert exit Froude number (Fr) is less than 3, if the location where the flow conditions are desired is within 3 culvert diameters of the outlet, and if S_O is less than 10 percent.

For expansions outside these limits, the energy equation can be used to determine flow conditions leaving the transition. Normally, these parameters would then be used as the input values for a basin design.

Expansions Into Hydraulic Jump Basins

The expansion shown in figure IV-B-4 is used to convert depth or potential energy at the culvert outlet to kinectic energy by allowing the flow to expand, drop, or both. The result is the depth decreases, the velocity increases, and the Froude number increases. The higher Froude number Fr results in a more efficient jump and a shorter basin.

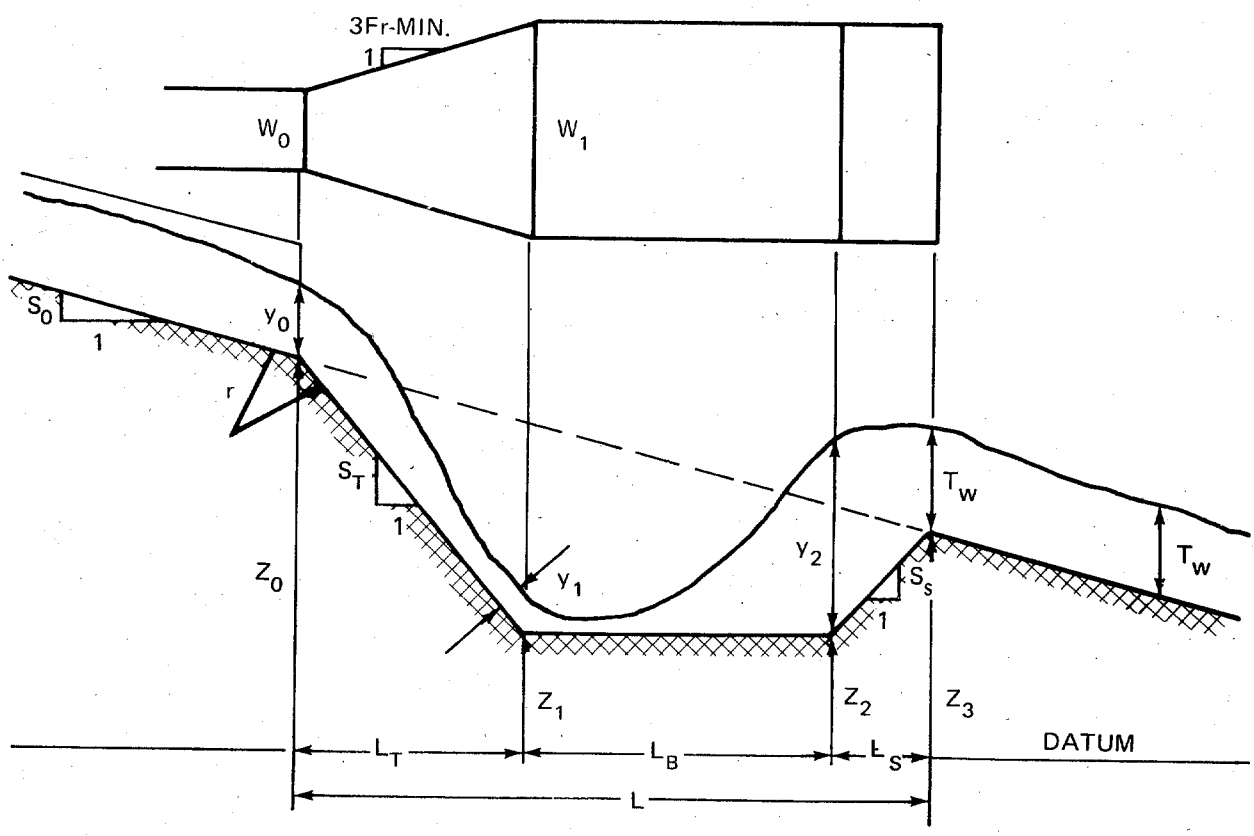


FIGURE IV-B-4. DEFINITION SKETCH BASIN TRANSITION

The energy balance is written from the culvert outlet to the basin, section 1. Substituting Q/y_1W_B for V_1 and solving for Q results in:

$$Q=y_1W_B[2g(z_o-z_1+y_o-y_1)+V_o^2]^{1/2} \quad \text{IV-B-7}$$

This expression has three unknowns y_1, W_B, and z_1. The depth y_1 can be determined by trial and error if W_B and z_1 are assumed. W_B should be limited to the width that a jet would flare naturally in the slope distance L .

$$W_B<W_o+2L_T\sqrt{S_T^2+1}/3Fr_o \quad \text{IV-B-8}$$

Since the flow is supercritical, the trial y_1 value should start near zero and increase until the design Q is reached. This depth y_1 is used to find the sequent depth, y_2 using the hydraulic jump equation:

$$y_2=C_1y_1[\sqrt{1+8Fr^2}-1]/2 \quad \text{IV-B-9}$$

where C_1=TW/y_2 ratio. For USBR basins, C_1 is found on figure VII-D-2; for the hydraulic jump, C_1=1.0; and for the SAF basin, C_1 varies with Fr (see section VII-H for the expressions). The above value of y_2+z_2 must be equal to or less than $TW+z_3$ for the jump to occur. In order to perform this check, z_3 is obtained graphically or by using the following expressions:

$$L_T=(z_o-Z_1)/S_T \quad \text{IV-B-10}$$

$$L_S=(z_3-z_2)/S_S \quad \text{IV-B-11}$$

$$L_B=f(y_1,Fr_1) \quad \text{IV-B-12}$$

$$L=L_T+L_B+L_S=(z_o-z_3)/S_o$$

Solving for z_3 yields

$$z_3=[z_o-(L_T+L_B-z_2/S_S)S_o]/(S_o/S_S+1) \quad \text{IV-B-13}$$

This expression is valid only if z_2 is less than or equal to z_3.

If z_2+y_2 is greater than z_3+TW, the basin must be lowered and the trial and error process repeated until sufficient tailwater exists to force the jump.

Design Procedure

1. Calculate culvert brink depth y_o using figure III-9 or 10, velocity V_o, and $Fr_o=V_o/\sqrt{gy_o}$.

2. Determine y_n (tailwater, TW) in downstream channel with the aid of table III-1.

3. Find y_2 using equation IV-B-9.

4. Compare y_2 and TW. If $y_2 <$ TW, the jump will form. If $Y_2 >$ TW, lower the basin to provide additional tailwater.

5. Determine the elevation of the basin by trial and error.

   Choose trial basin elevation, z_1

   A. Choose basin width, W_B and basin slopes S_T and S_S. A slope of 0.5(2:1) or 0.33(3:1) is satisfactory for either S_T or S_S.
   B. Check W_B using equation IV-B-8
   C. Calculate y_1 by trial and error using equation IV-B-7 and calculate V_1
   D. Calculate $Fr_1=V_1/\sqrt{gy_1}$
   E. Determine y_2 using equation IV-B-9 with C_1 corresponding to basin type
   F. Find z_3 using equation IV-B-13
   G. Calculate y_2+z_2 and z_3+TW. If y_2+z_2 is greater than z_3+TW, choose another z_1 and repeat steps 4A through G until balance is reached.

6. Calculate L_T, L_S, and L_B using equation IV-B-10, 11, and 12. The horizontal distance downstream to the sill crest, L, is $L_T+L_S+L_B$.

7. Determine radius to use between culvert and transition from figure IV-B-5.

IV-B-1. Meshgin, K., Moore, W. L., DESIGN ASPECTS AND PERFORMANCE CHARACTERISTICS OF RADIAL FLOW ENERGY DISSIPATORS, University of Texas at Austin, Research Report 116-2F, August 1970.

Example Problem

Given: 10x6RCB, $Q=417$ cfs, $S_o=6.5\%$
Elevation outlet invert $z_o=100$ feet
$V_o=27.8$ fps, $y_o=1.5$ feet
Downstream channel is a 10 ft. bottom trapezoidal channel with 2:1 side slopes and $n=.03$

Find: Dimensions for hydraulic jump basin

Solution:

1. $V_o=27.8$ fps, $y_o=1.5$ ft.

   $Fr_o=27.8/\sqrt{g1.5}=4$

2. $Qn/b^{8/3}S^{1/2}=417(.03)/10^{8/3}(.065)^{1/2}=.1057$

   $d/b=y_n/b=.19$, $y_n=TW=1.9$ ft., $V_n=15.9$ fps

3. $y_2=C_1y_1[\sqrt{1+8Fr^2}-1]/2=1.5[\sqrt{1+8(4)^2}-1]/2=7.8$ ft.

4. Since $y_2>TW$, $7.7>1.9$, The basin is too high.

5. Try $z_1=94$ ft. since $y_2-TW=6$ ft.
   A. $W_B=10$ ft., $S_T=S_s=.5$

   B. $W_B=W_o+2(z_o-z_1)\sqrt{S_T^2+1}/3Fr_oS_T$

      $W_B=10+2(100-94)\sqrt{.5^2+1}/3(4).5=12.2$ ft.>10 ft. o.k.
   C. $Q=y_1 10[2g(100-94+1.5-y_1)+27.8^2]^{1/2}$
      $Q=10y_1[64.4(7.5-y_1)+772.8]^{1/2}$
      Try $y_1=1$ ft., $Q=345$-low
      $y_1=2$ ft., $Q=671$-high
      $y_1=1.22$ ft., $Q=418$-OK
      $V=417/1.22(10)=34.2$ fps
   D. $Fr_1=34.2/\sqrt{g1.22}=5.45$
   E. for $C_1=1$, $y_2=1.22[\sqrt{1+8(5.45)^2}-1]/2=8.81$
   F. $L_B=43(1.22)=53$ ft. figure VI-11
      $L_T=(z_o-z_1)/S_T=(100-94)/.5=12$ ft.
      $z_3=[100-(12+53-94/.5).065]/(.065/.5+1)$
      $z_3=[100+8]/1.13=95.6$ ft.
   G. $y_2+z_2=94+8.8=102.8$ ft.
      $z_3+TW=95.6+1.9=97.5$ ft.
      $102.8>97.5$ try $z_1=90$ ft.

   Try $z_1=90$ ft.
   A. $W_B=10$ ft., $S_T=S_s=.5$
   B. $W_B=10$ ft. OK
   C. $Q=10y_1[64.4(11.5-y_1)+772.8]^{1/2}$
      $y_1=1.1$ ft., $V_1=37.9$ fps

D. $Fr_1=37.9/\sqrt{g1.1}=6.37$

E. $y_2=1.1[\sqrt{1+8(6.37)^2}-1]/2=9.37$

F. $L_B=53(1.1)=58$ ft.

$L_T=(100-90)/.5=20$ ft.

$z_3=[100-(20+58-90/.5).065]/1.13$

$z_3=94.4$ ft.

G. $y_2+z_2=9.37+90=99.4$ ft.

$z_3+TW=94.4+1.9=96.3$ ft.

since $99.4>96.3$ try $z_1=85$ ft.

Try $z_1=85$

A. $W_B=10$ ft., $S_T=S_s=.5$

B. $W_B=10$ ft. OK

C. $Q=10y_1[64.4(16.5-y_1)+772.8]^{1/2}$

$y_1=.99$ ft., $V_1=42.1$ fps

D. $Fr_1=42.1/\sqrt{g.99}=7.46$

E. $y_2=.99[\sqrt{1+8(7.46)^2}-1]/2=9.96$

F. $L_B=64(0.99)=63$ ft.

$L_T=(100-85)/.5=30$ ft.

$z_3=[100-(30+63-85/.5).065]/1.13$

$z_3=92.9$

G. $y_2+z_2=95$ ft.

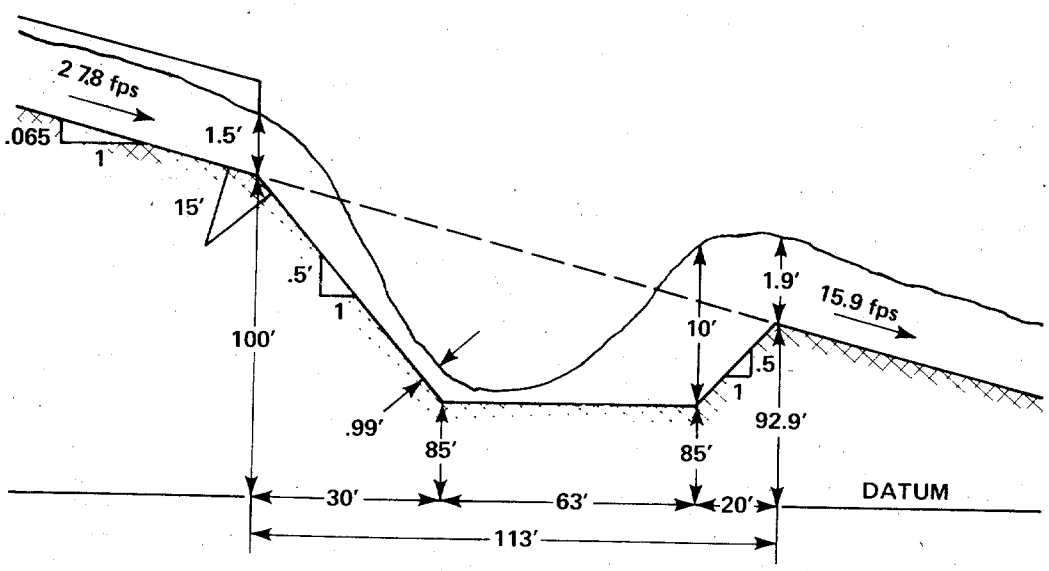
$z_3+TW=94.8$ ft. OK use $z_1=85$ ft.

5. $L_T=30$ ft., $L_B=63$ ft.

$L_s=(z_3-z_2)/S_s=(94.8-85)/.5=19.6$ ft.

$L=30+63+19.6=112.6$ ft.

6. $Fr_o=4$ from figure IV-B-5 $y_o/r=.1$



EXAMPLE PROBLEM SKETCH (IV-B)

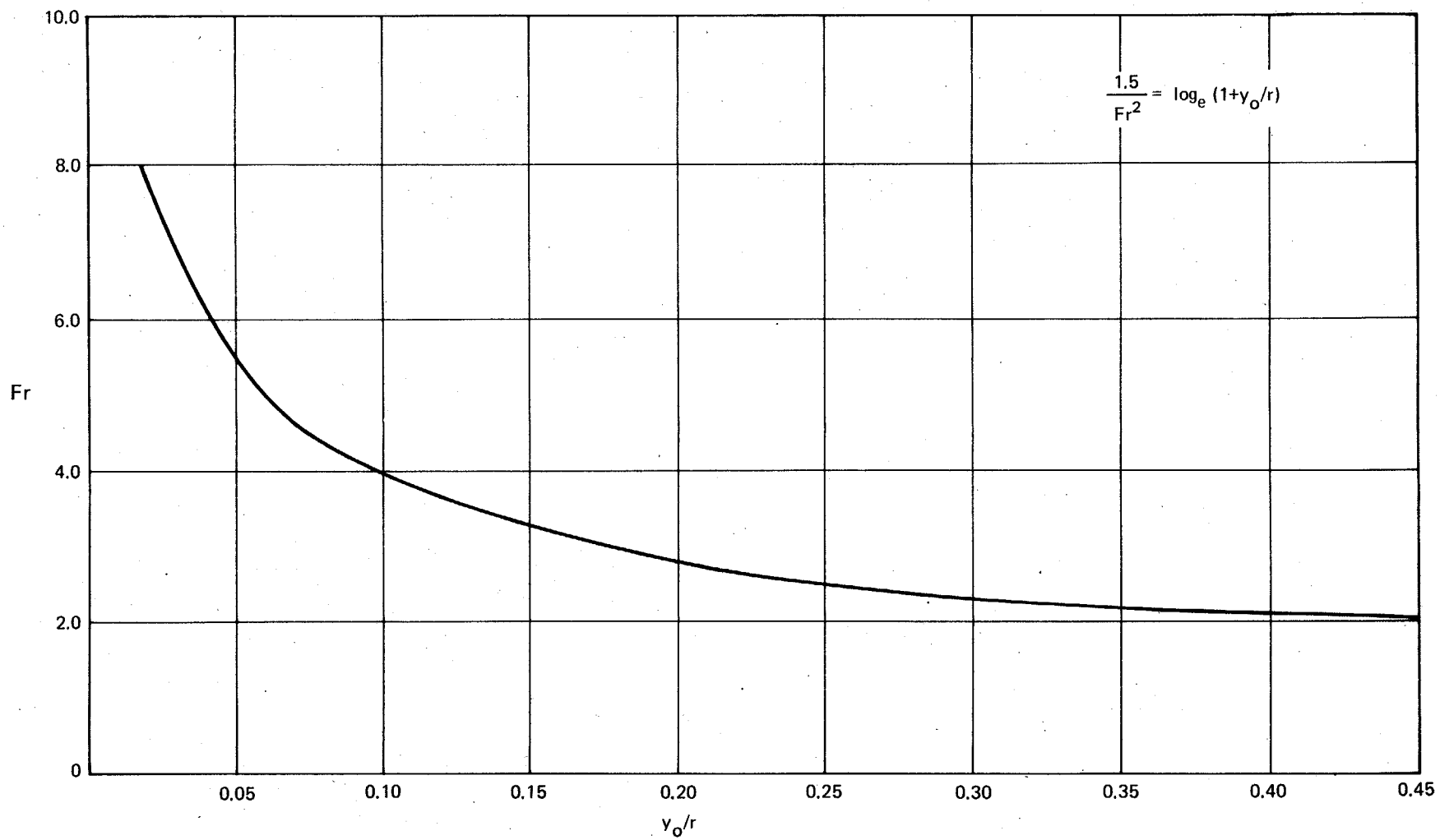


FIGURE IV-B-5 Fr Vs. y_o/r FOR TRANSITION FROM REFERENCE IV-B-1

CHAPTER V

ESTIMATING EROSION AT CULVERT OUTLETS

Estimating erosion at culvert outlets is difficult because of the many complex factors affecting erosion. Some of these factors are the discharge, culvert diameter, soil type, duration of flow and tailwater depth. In addition, the magnitude of the total erosion can consist of local scour and channel degradation, the two types of erosion discussed in Chapter II-B. Maintenance history, site reconnaissance and data on soils, flows and flow duration provide the best estimate of the potential erosion hazard at a culvert outlet.

The objective of this chapter is to present a method for predicting local scour at the outlet of structures based on soil and flow data and culvert geometry. This scour prediction is intended to serve together with the maintenance history and site reconnaissance information for determining energy dissipator needs.

Investigations (1), (3), indicate that the scour hole geometry varies with tailwater conditions with the maximum scour geometry occuring at tailwater depths less than half the culvert diameter (1); and that the maximum depth of scour (h_s) occurs at a location approximately 0.4 L_s downstream of the culvert outlet (3) where L_s is the length of scour.

Empirical equations defining the relationship between the culvert discharge intensity, time, and the length, width, depth, and volume of scour hole are presented for the maximum or extreme scour case.

Cohesionless Material

The general expression for determining scour geometry in a cohesionless soil for a circular pipe flowing full is

$$\text{Dimensionless Scour Geometry} = \alpha\left(\frac{Q}{\sqrt{g}\ D^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta} \qquad \text{(V-1)}$$

where:

Dimensionless Scour Geometry is $\frac{h_s}{y_e}$, $\frac{W_s}{y_e}$, $\frac{L_s}{y_e}$, or $\frac{V_s}{y_e}$

h_s, W_s, L_s, and V_s are depth, width, length and volume of scour respectively.

D is the diameter of the culvert

Q is the discharge, g is the acceleration of gravity

t is the time in minutes

t_o is a base time used in the experiments to derive coefficients (316 minutes unless specified otherwise).

For noncircular or part full culverts, the diameter D can be replaced by a n equivalent depth y_e, where y_e is defined as

$$y_e = (A/2)^{1/2}$$

and A is the cross sectional area of flow. Modifying Equation (V-1) to include the equivalent depth results in the general expression.

$$\text{Dimensionless Scour Geometry} = \alpha_e \left(\frac{Q}{\sqrt{g}\, y_e^{5/2}}\right)^{\beta} \left(\frac{t}{t_o}\right)^{\theta} \qquad \text{(V-2)}$$

where:

$$\alpha_e = \alpha\, 0.63^{2.5\,\beta-1} \text{ for } h_s,\ W_s, \text{ and } L_s$$

$$\alpha_e = \alpha\, 0.63^{2.5\,\beta-3} \text{ for } V_s$$

The values of the coefficients α_e, β, and θ in Equations V-1 and V-2 are given in Table V-1.

Gradation

The cohensionless bed materials presented in Table V-1 are categorized as either uniform (U) or graded (G). The grain size distribution is determined by performing a sieve analysis (ASTM DA22-63). The standard deviation (σ) is computed as:

$$\sigma = \left(\frac{d_{84}}{d_{16}}\right)^{1/2}$$

where the values of d_{84} and d_{16} are extracted from the grain size distribution. If ≤ 1.5, the material is considered to be uniform; if ≥ 1.5, the material is classified as graded.

Cohesive Soils

If the cohesive soil is a sandy clay similar to the one tested at Colorado State University by Abt et al (8), Equation (V-1) or (V-2) and the appropriate coefficients in Table V-1 can be used to estimate the scour hole dimensions. The sandy clay tested had 58 percent sand, 27 percent clay, 15 percent silt and 1 percent organic matter; had a mean grain size of 0.15 mm and had a plasticity index, PI, of 15.

Since Equations V-1 and V-2 do not include soil characterisitcs, they can only be used for soils similar to the ones tested. Shear number expressions, that related scour to the critical shear stress of the soil, were derived to have a wider range of applicability for cohesive soils besides the one specific sandy clay that was tested. The shear number expressions for circular culverts are:

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{ or } \frac{V_s}{D}\right] = \alpha\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta} \qquad \text{(V-3)}$$

and for other shaped culverts:

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{ or } \frac{V_s}{y_e}\right] = \alpha_e\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta} \qquad \text{(V-4)}$$

where: $\frac{\rho V^2}{\tau_c}$ is the modified shear number

V = outlet mean velocity

τ_c = critical tractive shear stress

ρ = fluid density

$\alpha_e = \frac{\alpha}{.63}$ for h_s, W_s, and L_s

$\alpha_e = \frac{\alpha}{(.63)^3}$ for V_s

The values of the coefficients α, β, θ, and α_e in Equations V-4 and V-5 are presented in Table V-1. The critical tractive shear stress (2) is defined as

$$\tau_c = 0.0001\ (S_v + 180)\ \tan\ (30 + 1.73\ PI) \qquad \text{(V-5)}$$

where S_v is the saturated shear strength in pounds per square inch and PI is the Plasticity Index from the Atterberg Limits.

It is recommended that Equations V-3 and V-4 be limited to sandy clay soils with a plasticity index of 5-16.

Time of Scour

The time of scour is estimated based upon a knowledge of peak flow duration. Lacking this knowledge, it is recommended that a time of 30 minutes be used in Equations V-1, V-2, V-3, and V-4. The tests indicate that approximately 2/3 to 3/4 of the maximum scour occurs in the first 30 minutes of the flow duration.

It should be noted that the exponents for the time parameter in Table V-1 reflect the relatively flat part of the scour-time relationship and are not applicable for the first 30 minutes of the scour process.

Headwalls

Installation of headwalls (6) flush with the culvert outlet moves the scour hole downstream. However, the magnitude of the scour geometries remain essentially the same as for the case without the headwall. If the culvert is installed with a headwall, the headwall should extend to a depth equal to the maximum depth of scour.

SUMMARY

The prediction equations presented in this chapter are intended to serve along with field reconnaissance as guidance for determining the need for energy dissipators at culvert outlets. It should be remembered that the equations do not include long-term channel degradation of the downstream channel. The equations are based on tests which were conducted to determine maximum scour for the given condition and therefore represent what might be termed worst case scour geometries. The equations were derived from tests conducted by the Corps of Engineers (1), and Colorado State University (5), (6), (7), (8) and (9).

Design Procedure

1. Perform a hydrologic analysis of the drainage in which the culvert is located or to be placed. Estimate the magnitude and duration of the peak discharge. Express the discharge in cfs and the duration in minutes.

   The discharge intensity is

$$D.I. = \frac{Q}{\sqrt{g}\ D^{5/2}} \text{ for circular culverts flowing full}$$

$$D.I. = \frac{Q}{\sqrt{g}\ y_e^{5/2}} \text{ for other shapes}$$

where $y_e = \left(\frac{A}{2}\right)^{1/2}$

FOR COHESIONLESS MATERIALS, OR THE 0.15mm SANDY CLAY

2. Compute the discharge intensity when the culvert is flowing at the peak discharge.

3. Determine scour coefficients from Table V-1.

4. Compute the scour hole dimensions from

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{ or } \frac{V_s}{D^3}\right] = \alpha\left(\frac{Q}{\sqrt{g}\ D^{5/2}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta} \qquad \text{(V-1)}$$

or

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{ or } \frac{V_s}{y_e}\right] = \alpha_e\left(\frac{Q}{\sqrt{g}\ y_e^{5/2}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta} \qquad \text{(V-2)}$$

FOR OTHER COHESIVE MATERIALS WITH PI FROM 5 TO 16

a. Compute the culvert outlet velocity in feet/sec.

b. Obtain a soil sample at the proposed culvert location.

c. Perform Atterberg limits tests and determine the plasticity index, PI (ASTM D423-36).

d. Saturate a sample and perform an unconfined compressive test (ASTM D211-66-76) to determine the saturated shear stress, Sv, in pounds per square inch.

e. Compute the critical tractive shear strength, τ_c, from equation V-5.

f. Compute the modified shear number $\frac{\rho V^2}{\tau_c}$

3. Determine scour coefficients from Table V-1.

4. Compute the desired scour hole dimensions from

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{ or } \frac{V_s}{D}\right] = \alpha \left(\frac{V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{316}\right)^{\theta}$$

for circular culvert

or

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \frac{V_s}{y_e^3}\right] = \alpha_e \left(\frac{V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{316}\right)^{\theta}$$

for noncircular culverts.

Example Problem Cohesionless Material

Determine the scour geometry--maximum depth, width, length and volume of scour--for a proposed circular 30-inch C.M.P. discharging an estimated 50 cfs when flowing full. The downstream channel is composed of a graded gravel material.

1. The duration of the peak discharge of 50 cfs is not known. Therefore, a peak flow duration of 30 minutes will be estimated.

2. The circular, 30-inch C.M.P. at 50 cfs will have a discharge intensity of

$$D.I. = \frac{50}{\sqrt{g}\,\left(\frac{30}{12}\right)^{5/2}} = \frac{50}{(5.67)(2.5)^{5/2}} = 0.89$$

3. The coefficients of scour obtained from Table V-1 are:

	α	β	θ
Depth of Scour	1.49	.50	.03
Width of Scour	8.76	0.89	.10
Length of Scour	13.09	0.62	.07
Volume of Scour	42.31	2.28	.17

4. Scour hole dimensions:

depth: $$\frac{h_s}{D} = \alpha\left(\frac{Q}{\sqrt{g}\,D^{2.5}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta}$$

$$= 1.49\ (0.89)^{0.50}\ (0.09)^{.03};\ h_s = 3.27\ \text{ft}$$

width: $$\frac{W_s}{D} = 8.76(0.89)^{0.89}\ (.09)^{.10};\ W_s = 15.5\ \text{ft}$$

Length: $$L_s = 13.09(0.89)^{0.62}\ (.09)^{.07};\ L_s = 25.72\ \text{ft}$$

Volume: $$V_s = 42.31(0.89)^{2.28}\ (.09)^{.17};\ V_s = 335.79\ \text{ft}^3$$

5. The location of the maximum scour (Figure V-2)

$0.4\ (L_s) = .4\ (25.72) = 10.3$ ft downstream of the culvert outle

Example Problem Cohesive Material

Determine the scour geometry-maximum depth, width, length and volume of scour for an existing circular 24-inch C.M.P. discharging an estimated 40 cfs when flowing full. The downstream channel is composed of a sandy-clay material.

1. The duration of the peak discharge of 40 cfs is not known. Therefore, a peak flow duration of 30 minutes will be estimated.

2. a. The average velocity at the culvert outlet is:

$$V = \frac{Q}{A} = \frac{40.0}{3.14} = 12.74 \text{ fps}$$

b-e. The sandy-clay material was tested and found to have a Plasticity Index (PI) of 12 and a saturated shear strength (Sv) of 240 psi.

The critical tractive shear can be estimated by substituting into Equation V-5

$$\tau_c = 0.001\ (240 + 180)\ \tan\ (30 + 1.73(12))$$

$$0.001(420)\ \tan\ (50.76) = 0.51 \text{ lb/ft}^2$$

f. The modified shear number $S_{n_{mod}} = \frac{(\rho V^2)}{\tau_c}$ is:

$$S_{n_{mod}} = \frac{1.94\ (12.74)^2}{0.51} = 617.4$$

3. The experimental coefficients α, β and θ from Table V-1 are

	α	β	θ
Depth	.86	.18	.10
Width	3.55	.17	.07
Length	2.82	.33	.09
Volume	.62	.93	.23

4. The scour hole dimensions are:

$$\frac{h_s}{D} = \alpha \left(\frac{\rho V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{316}\right)^{\theta}$$

$$= .86(617.4)^{.18}\ (.09)^{.10}; \quad h_s = 2.14 \times 2 = 4.30 \text{ ft}$$

$$\frac{W_s}{D} = 3.55(617.4)^{.17}\ (.09)^{.07}; \quad W_s = 8.94 \times 2 = 27.9 \text{ ft}$$

$$\frac{L_s}{D} = 2.82(617.4)^{.33}\ (.09)^{.09}; \quad L_s = 18.92 \times 2 = 37.8 \text{ ft}$$

$$\frac{V_s}{D^3} = .62(617.4)^{.93}\ (.09)^{.23}; \quad V_s = 140.3 \times 23 = 1122.5 \text{ ft}^3$$

5. Location of maximum depth of scour (Figure V-2)

$0.4\ L_s = 0.4(37.8) = 15.1$ ft downstream of culvert outlet

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Table V-1. Experimental Coefficients for Culvert Outlet Scour

MATERIAL	NOMINAL GRAIN SIZE d_{50} (mm)	SCOUR EQUATION	DEPTH h_s				WIDTH W_s				LENGTH L_s				VOLUME V_s			
			α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e
Uniform Sand	0.20	V-1 or V-2	2.72	.375	0.10	2.79	11.73	0.92	.15	6.44	16.82	0.71	0.125	11.75	203.36	2.0	0.375	80.71
Uniform Sand	2.0	V-1 or V-2	1.86	0.45	0.09	1.76	8.44	0.57	0.06	6.94	18.28	0.51	0.17	16.10	101.48	1.41	0.34	79.62
Graded Sand	2.0	V-1 or V-2	1.22	0.85	0.07	.75	7.25	0.76	0.06	4.78	12.77	0.41	0.04	12.62	36.17	2.09	0.19	12.94
Uniform Gravel	8.0	V-1 or V-2	1.78	0.45	0.04	1.68	9.13	0.62	0.08	7.08	14.36	0.95	0.12	7.61	65.91	1.86	0.19	12.15
Graded Gravel	8.0	V-1 or V-2	1.49	0.50	0.03	1.33	8.76	0.89	0.10	4.97	13.09	0.62	0.07	10.15	42.31	2.28	0.17	32.82
Cohesive Sandy Clay 60% Sand PI 15	0.15	V-1 or V-2	1.86	0.57	0.10	1.53	8.63	0.35	0.07	9.14	15.30	0.43	0.09	14.78	79.73	1.42	0.23	61.84
Clay PI 5-16	Various	V-3 or V-4	0.86	0.18	0.10	1.37	3.55	0.17	0.07	5.63	2.82	0.33	0.09	4.48	0.62	0.93	0.23	2.48

EQUATIONS:

V-1. FOR CIRCULAR CULVERTS. Cohesionless material or the 0.15mm cohesive sandy clay

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{ or } \frac{V_s}{D^3}\right] = \alpha\left(\frac{Q}{\sqrt{g}\,D^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-2. FOR OTHER CULVERTS SHAPES. Same material as above.

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{ or } \frac{V_s}{y_e^3}\right] = \alpha_e\left(\frac{Q}{\sqrt{g}\,y_e^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-3. FOR CIRCULAR CULVERTS. Cohesive sandy clay with PI = 5-16

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{ or } \frac{V_s}{D^3}\right] = \alpha\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where: t_o = 316 min.

V-4. FOR OTHER CULVERT SHAPES. Cohesive sandy clay with PI = 5-16

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{ or } \frac{V_s}{y_e^3}\right] = \alpha_e\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where: t_o = 316 min.

VI. HYDRAULIC JUMP

NATURE OF THE HYDRAULIC JUMP

The hydraulic jump is a natural phenomenon which occurs when supercritical flow changes to subcritical flow. This abrupt change in flow condition is accompanied by considerable turbulence and loss of energy. Within certain flow ranges, the hydraulic jump is an effective energy dissipation device which is often employed to control erosion at hydraulic structures.

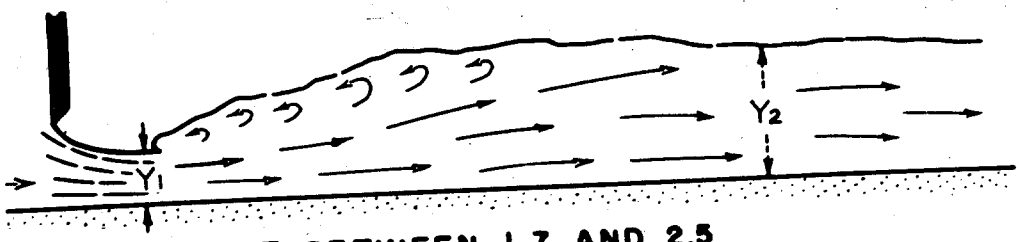
The Bureau of Reclamation (VI-1) has related the jump form and flow characteristics to the Froude number, figure VI-1. The design and evaluation of stilling basins are based on these relationships.

When the upstream Froude number (Fr) is 1.0, the flow is at critical and a jump cannot form.

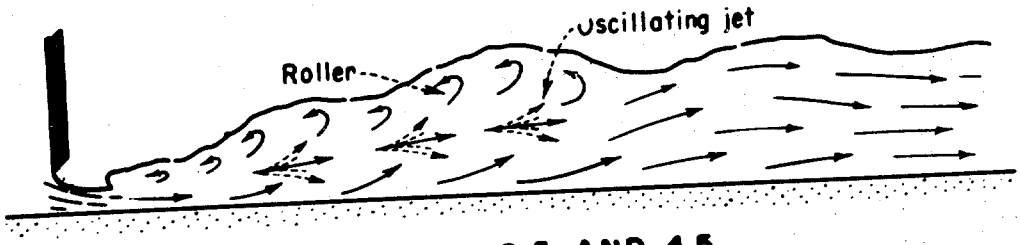
For Froude numbers greater than 1.0, but less than 1.7, the upstream flow is only slightly below critical depth and the change from supercritical to subcritical flow will result in only a slight disturbance of the water surface. On the high end of this range, Fr approaching 1.7, the downstream depth will be about twice the incoming depth and the exit velocity about half the upstream velocity.

When the upstream Froude number is between 1.7 and 2.5, a roller begins to appear, becoming more intense as the Froude number increases. This is the prejump range with very low energy loss. In this range, there are no particular stilling basin problems involved. The only requirement is that the proper length of basin, which is rather short, be provided. The water surface is quite smooth, the velocity throughout the cross section uniform, and the energy loss in the range of 20 percent. The exit Froude number for many culverts falls within the range 1.5 to 4.5.

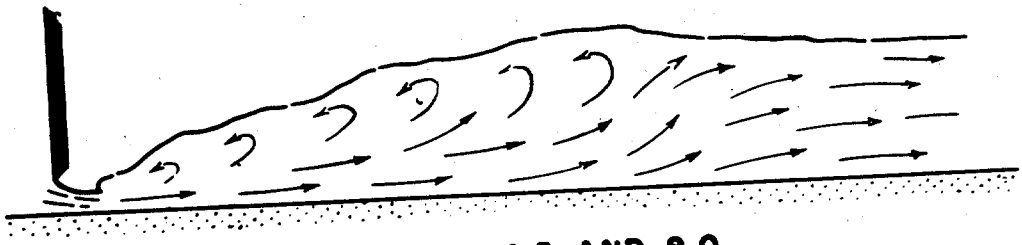
An oscillating form of jump occurs for Froude numbers between 2.5 and 4.5. The incoming jet alternately flows near the bottom and then along the surface. This results in objectionable surface waves which can cause erosion problems downstream.



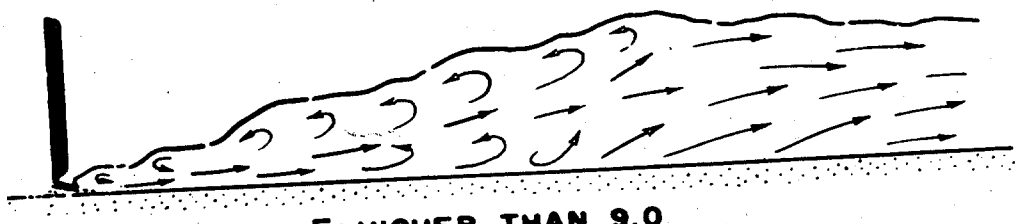
Fr BETWEEN 1.7 AND 2.5
A - PREJUMP STAGE



Fr BETWEEN 2.5 AND 4.5
B - TRANSITION STAGE



Fr BETWEEN 4.5 AND 9.0
C - RANGE OF WELL-BALANCED JUMPS



Fr HIGHER THAN 9.0
D - EFFECTIVE JUMP BUT
ROUGH SURFACE DOWNSTREAM

FIGURE VI-1 JUMP FORMS
RELATED TO Fr FROM REFERENCE VI-1

Q = TOTAL DISCHARGE IN C.F.S.

W = WIDTH OF FLUME IN FEET

q = DISCHARGE IN C.F.S. PER FOOT OF WIDTH

E_1 = ENERGY ENTERING JUMP

E_2 = ENERGY LEAVING JUMP

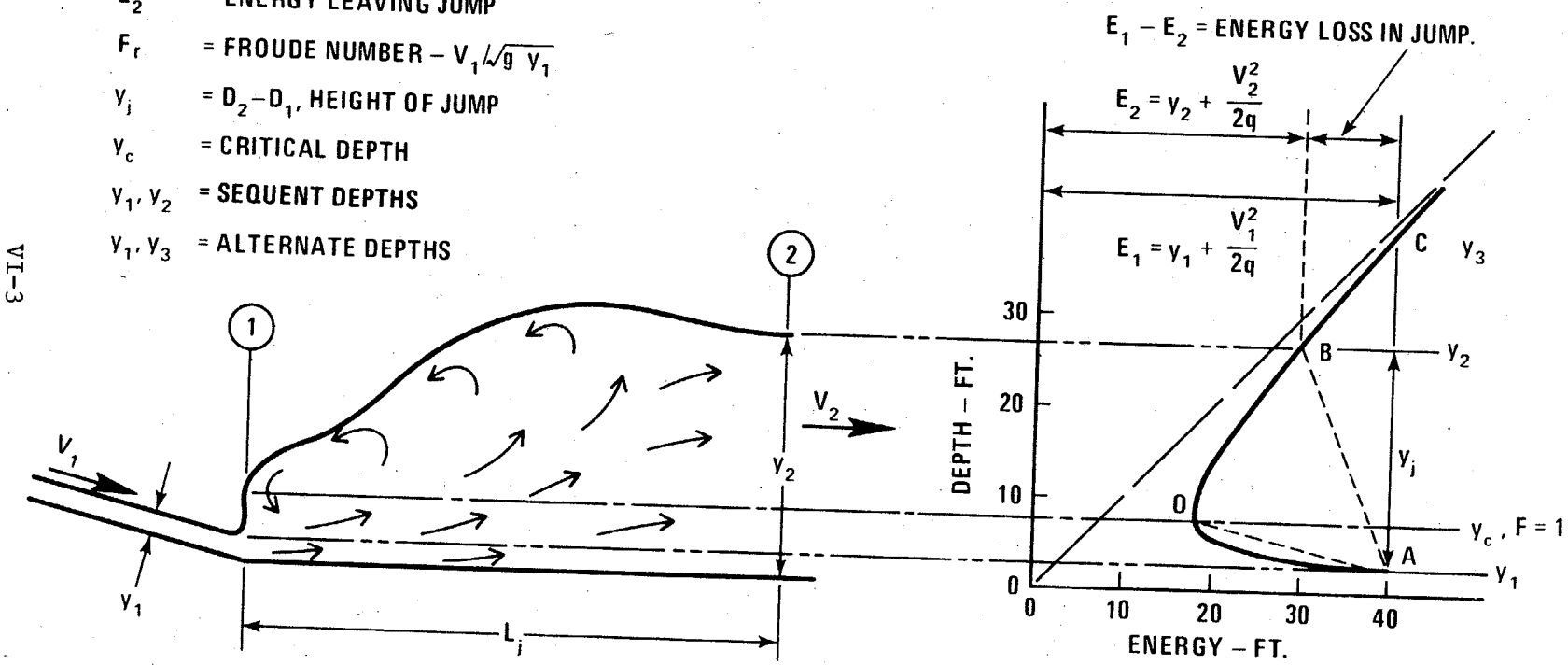
F_r = FROUDE NUMBER – $V_1/\sqrt{g\ y_1}$

y_j = $D_2 - D_1$, HEIGHT OF JUMP

y_c = CRITICAL DEPTH

y_1, y_2 = SEQUENT DEPTHS

y_1, y_3 = ALTERNATE DEPTHS



HYDRAULIC JUMP – ON HORIZONTAL FLOOR

RELATION OF SPECIFIC ENERGY TO DEPTH OF FLOW

FIGURE VI-2. HYDRAULIC JUMP.

A well balanced and stable jump occurs where the incoming flow Froude number is greater than 4.5. Fluid turbulence is mostly confined to the jump, and for Froude numbers up to 9.0 the downstream water surface is comparatively smooth. Jump energy loss of 45 to 70 percent can be expected.

With Froude numbers greater than 9.0, a highly efficient jump results but the rough water surface may cause downstream erosion problems.

The nature of the hydraulic jump may be illustrated by use of the specific energy diagram, figure VI-2. The flow entering the jump at supercritical velocity V_1, and depth y_1, has a specific energy of $E = y_1 + V_1^2/2g$, the kinetic energy term, $V^2/2g$, is predominant. As the depth of flow increases through the jump, the specific energy decreases. Flow leaves the jump at subcritical velocity with the potential energy y, predominant.

The hydraulic jump commonly occurs with natural flow conditions and with proper design can be an effective means of dissipating energy at hydraulic structures. In designing energy dissipators which includes a hydraulic jump, expressions for computing the before and after jump depth ratio (conjugate depths), and the length of jump are needed.

HYDRAULIC JUMP EXPRESSIONS--HORIZONTAL CHANNELS

The hydraulic jump in any shape of horizontal channel is relatively simple to analyze (VI-2). Figure VI-3 indicates the control volume used and the forces involved. Control section one is before the jump where the flow is undisturbed, and control section two is after the jump, far enough downstream for the flow to be again taken as parallel. Distribution of pressure in both sections is assumed hydrostatic. The change in momentum of the entering and exiting stream is balanced by the resultant of the forces acting on the control volume, i.e., pressure and boundary frictional forces. Since the length of the jump is relatively short, the external energy losses (boundary frictional forces) may be ignored without introducing serious error. The momentum principle provides for solution of the sequent depth, y_2, and downstream velocity, V_2. Once these are known, the internal energy losses and jump efficiency can be determined by application of the energy principle.

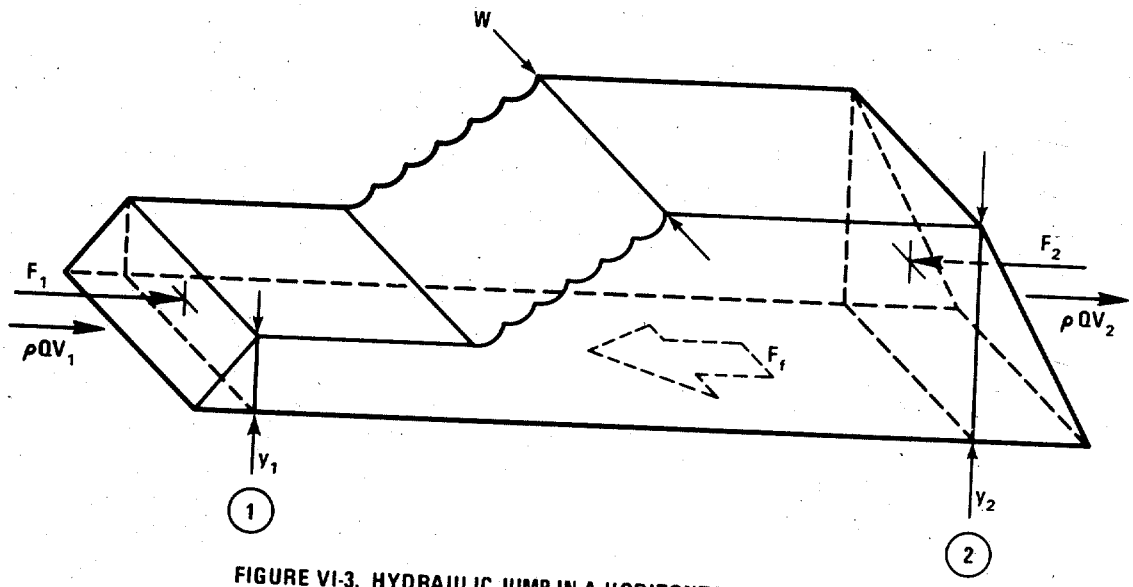


FIGURE VI-3. HYDRAULIC JUMP IN A HORIZONTAL CHANNEL.

The momentum function can be used in a general format for the solution of the hydraulic jump sequent-depth relationship in any shape of channel with a horizontal floor.

The momentum function is, $M = Q^2/gA + Ay$.

At section 1 and 2 respectively,

$$Q^2/gA_1 + A_1y_1 = Q^2/gA_2 + A_2y_2 \quad \text{. . . . . .Vl-1}$$

or

$$A_1y_1 - A_2y_2 = (1/A_2 - 1/A_1)Q^2/g.$$

Letting the distance to the centroid from the water Surface = Ky gives: $A_1K_1y_1 - A_2K_2y_2 = (1/A_2 - 1/A_1)Q^2/g.$

Rearranging and using $Fr_1^2 = V_1^2/gy_1 = Q^2/A_1^2gy_1$

gives: $A_1K_1y_1 - A_2K_2y_2 = Fr_1^2A_1y_1(A_1/A_2-1).$

Dividing this by A_1y_1 provides:

$$K_2A_2y_2/A_1y_1 - K_1 = Fr_1^2(1 - A_1/A_2) \quad \text{. . . . . .VI-2}$$

This is a general expression for the hydraulic jump in a horizontal channel. For various channel shapes, the constants K_1 and K_2 and the ratio A_1/A_2 may be evaluated:

For rectangular channels:

$$K_1 = K_2 = 1/2 \text{ and } A_1/A_2 = y_1/y_2$$

and

$$y_2^2/y_1^2 - 1^2 = 2Fr_1^2(1 - y_1/y_2).$$

Defining $y_2/y_1 = J$ the expression for a hydraulic jump in a horizontal, rectangular channel is obtained:

$$J^2 - 1 = 2Fr_1^2(1 - 1/J) \quad \ldots\ldots \quad \text{VI-3}$$

Figure VI-4 is a plot of Equation VI-3.

For triangular channels:

$$K_1 = K_2 = A_1/3A_2 = y_1^2/y_2^2$$

and

$$y_2^3/y_1^3 - 1 = 3Fr_1^2[1 - y^2_1/y^2_2]$$

or

$$J^3 - 1 = 3Fr_1^2(1 - 1/J^2)$$

This gives:

$$Fr_1 = J^2(J^3 - 1)/3(J^2 - 1)$$

or

$$Fr_1 = (J^4 + J^3 + J^2)/3(J + 1) \quad \ldots\ldots \quad \text{VI-4}$$

Figure VI-5 is a design curve for hydraulic jumps in horizontal triangular channels. Two scales for the Froude number are indicated on this figure. The upper scale defines the Froude number as: $Fr_m = V/\sqrt{gy_m}$, where y_m is the hydraulic depth, area/top width.

For a triangular channel $y_m = y/2$ so the upper scale is offset 1.414 units to the left of the lower scale which defines the Froude number as $Fr = V/\sqrt{gy}$.

The reason for using the hydraulic depth is that at a Froude number of 1.0, $y_2/y_1 = 1.0$, irrespective of sectional shape. The hydraulic depth and the actual depth in a rectangular section are identical so the additional scale is omitted from figure VI-4.

For Parabolic channel:

$K_1 = K_2 = 2/5$, and the area ratio $A_1/A_2 = (y_1/y_2)^{1.5}$

This results in:

$$(y_2/y_1)^{2.5} - 1 = 2.5\ Fr^2[1 - (y_1/y_2)^{1.5}]$$

or

$$Fr = [.4(J^4 - J^{1.5})/(J^{1.5} - 1)]^{.5} \qquad .VI\text{-}5$$

Using the hydraulic depth y_m in the Froude number expression gives:

$$Fr = [.6(J^4 - J^{1.5})/(J^{1.5} - 1)]^{.5} \qquad .VI\text{-}6$$

Figure VI-6 is a plot of these relationships, again using a scale adjustment for the hydraulic depth.

For circular channels:

Figures VI-7 and VI-8 are the design charts for horizontal circular channels using the hydraulic depth and actual depth in computing the Froude numbers.

For circular channels, it is necessary to consider two cases, where y_2 is greater than D, and where y_2 is less than D.

For y_2 less than D

$$(K_2y_2C_2/y_1C_1)-K_1 = Fr^2(1-C_1/C_2) \ldots \qquad .VI\text{-}7$$

and for y_2 greater than or equal to D

$$(y_2C_2/y_1C_1)-.5(C_2D/C_1y_1)-K_1 = Fr^2(1-C_1/C_2) \qquad .VI\text{-}8$$

C and K are functions of y/D and may be evaluated from the following table:

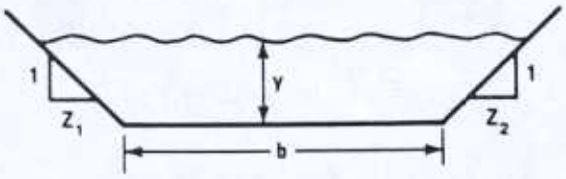
TABLE VI-1

y/D	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
K	.410	.413	.416	.419	.424	.432	.445	.462	.473	.500
C	.041	.112	.198	.293	.393	.494	.587	.674	.745	.748
C'	.600	.800	.917	.980	1.0	.980	.917	.800	.600	

The C' values are used in converting the Froude number $Fr = V/\sqrt{gy}$, to $Fr_m = V/\sqrt{gy_m}$. Where $y_m = (C/C')D$.

For trapezoidal channels:

In these channels the A and K values take on a more complex form:



$$z = (z_1+z_2)/2, \quad t = b/z(y_1) \text{ or } z(y_1) = b/t$$
$$J = y_2/y_1 \text{ or } y_2 = J(y_1)$$

$$\text{Area (1)} = z(y_1)^2+b(y_1) = b(y_1)(1+t)/t$$

$$\text{Area (2)} = z(y_2)^2+b(y_2) = Jb(y_2)(J+t)/t$$

$$6(K_1) = [2z(y_1)+3b]/[z(y_1)+b] = (2+3t)/(1+t)$$

$$6(K_2) = [2z(y_2)+3b]/[z(y_2)+b] = (2J+3t)/(J+t)$$

Using these relationships in the hydraulic jump equation,

$K_2(A_2/A_1)(y_2/y_1)-K_1 = Fr^2(1-A_1/A_2)$ gives:

$$J[2J+3t)/(J+t)][J(J+t)/(1+t)]-(2+3t)/(1+t) = 6Fr^2[1-(1+t)/J(J+t)]$$

Simplifying and expanding yields:

$$Fr^2 = J(J+t)[3t(J+1)+2(J^2+J+1)]/[6(J+t+1)(1+t)] \quad \text{....VI-9}$$

and using the hydraulic depth:

$$A/T = [z(y_1)^2+b(y_1)]/[2z(y_1)+b]$$

$$A/T = [(y_1)\ b/t + b(y_1)]/(2b/t + b)$$

$$A/T = y_1(1+t)/(2+t)$$

$$(Fr_m)^2 = Fr^2(1+t)/(2+t) \quad \text{.....VI-10}$$

Figure VI-9 and VI-10 represent plots of equations VI-9 and VI-10. These figures represent ranges of Froude numbers, shape factors $b/z(y_1)$, and depth ratios sufficient for most highway design problems.

GENERAL EQUATION FOR A HYDRAULIC JUMP IN A HORIZONTAL CHANNEL

$$K_2 \left(\frac{A_2\ Y_2}{A_1\ Y_1}\right) - K_1 = Fr^2 \left(1 - \frac{A_1}{A_2}\right)$$

$$K = \frac{\bar{Y}}{Y};\ J = \frac{Y_2}{Y_1};\ t = \frac{b}{zy_1}$$

TABLE VI-2 AREA RATIOS AND K FACTORS

Channel Shape	Area Ratios		K Factors	
	A_2/A_1	A_1/A_2	K_1	K_2
Rectangular	J	$1/J$	1/2	1/2
Triangular	J^2	$1/J^2$	1/3	1/3
Parabolic	$J^{3/2}$	$1/J^{3/2}$	2/5	2/5
Circular	See Table VI-1			
Trapezoidal	$\frac{J\ (J + t)}{(1 + t)}$	$\frac{(1 + t)}{J\ (J + t)}$	$1/6 \left[\frac{(2 + 3t)}{(1 + t)}\right]$	$1/6 \left[\frac{(2J + 3t)}{J + t}\right]$

Length of Jump Horizontal Channels. The length of the hydraulic jump is generally measured to the downstream section at which the mean water surface attains the maximum depth and becomes reasonably level. Errors may be introduced in determining Lj since the water surface is rather flat near the end of the jump. This is undoubtedly one of the reasons so many empirical formulas for determining jump length are found in the literature.

The jump length for rectangular basins has been extensively studied and can be reasonably well defined for Froude numbers up to 20. This is not the case for nonrectangular channels. In these channels, there are side areas, roughly triangular in shape, which are not directly influenced by the upstream jet. The flow must expand in the lateral direction as well as vertical. This lateral expansion results in the formation of wings and as the channel side slopes increase (become flatter), return or upstream flow becomes stronger. The reverse circulation results in increased energy dissipation but longer jump lengths. It will, however, eventually prevent the formation of a stable jump and convert the flow into an oscillating jet.

The jump length curves presented for nonrectangular sections should not be extrapolated. The curves and recommendations presented in this circular are based on a large number of observations by a number of investigators. While the length of the hydraulic jump in nonrectangular channels is still being actively debated, the recommendations presented are considered conservative if used within the indicated limits.

Figures VI-11, 12, and 13 may be used for the determination of jump lengths in rectangular, trapezoidal, parabolic, triangular, and circular channels, respectively. The circular channel curve (figure VI-13) is for the case where y_2 is less than D. For the case where y_2 is greater than D, it is suggested that the length be taken as seven times the difference in depths, i.e., $Lj = 7(y_2-y_1)$.

Free jump basins can be designed for any flow conditions; but because of economic and performance characteristics they are, in general, only employed in the lower range of Froude numbers. At higher Froude numbers, the use of baffles and sills make it possible to reduce the basin length and stabilize the jump over a wider range of flow situations.

Flows with Froude numbers below 1.7 may not require stilling basins but may require protection such as riprap and wingwalls and apron. For Froude numbers between 1.7 and 2.5, the free jump basin may be all that is required. In this range, loss of energy is less than 20 percent; the conjugate depth is about three times the incoming flow depth; and, the length of basin required is less than about 5 times the conjugate depth. Many highway culverts operate in this flow range.

Example Problem

As an example, consider a 7-foot wide box culvert discharging 400 cfs on a 0.2 percent slope. Outlet flow conditions are:

$$V_1 = 19 \text{ fps}$$
$$y_1 = 3.0 \text{ feet}$$
$$Fr = 1.9$$

For these conditions, in a rectangular basin, the conjugate depth required is:

$$J = y_2/y_1 = 2.2,$$
$$y_2 = 2.2(3.0) = 6.6 \text{ feet}$$

Length of jump, Lj: (Figure VI-11)

$$Lj/y_1 = 9.0$$
$$Lj = 3.0(9.0) = 27.0 \text{ feet}$$

After jump velocity,

$$V_2 = Q/A = 400/7(6.6) = 8.7 \text{ fps.}$$

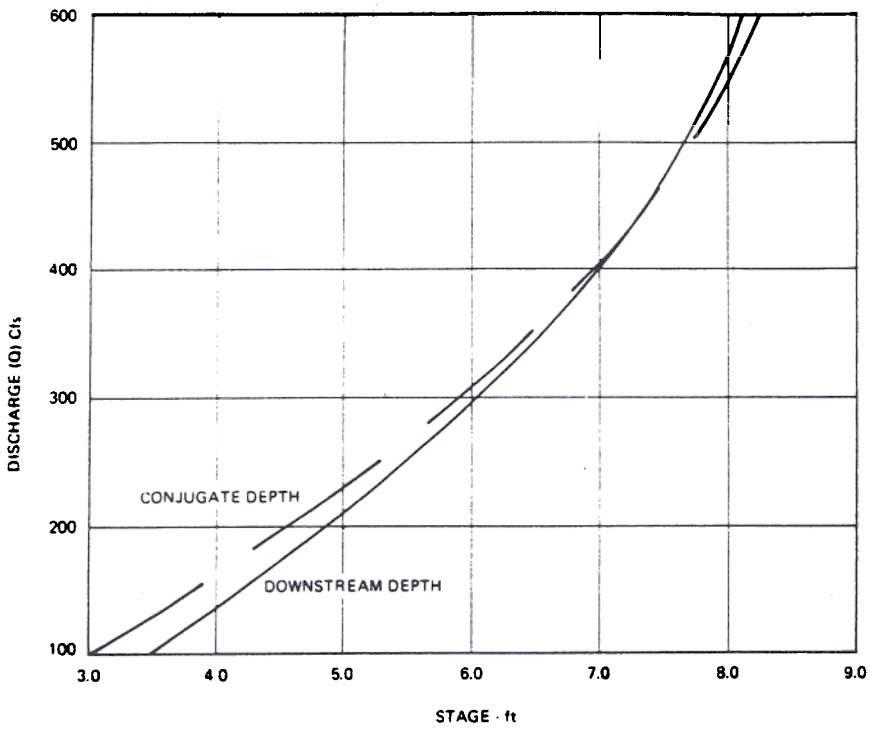
Velocity reduction is more than 54%.

$$V(in) = 19 \text{ fps}$$
$$V(out) = 8.7 \text{ fps}$$

These answers could also be obtained using figure VI-21.

It is always advisable to investigate the operating characteristics of the dissipator selected over the range of flows expected. This involves developing a downstream rating curve

for the natural channel (Q vs. stage curve) and comparing this with the sequent depth requirements. In the previous example, the downstream channel has a 10-foot bottom, is trapezoidal with 2 to 1 side slopes, and n = 0.03. The bottom slope changes to 0.0004 ft/ft at the culvert exit. The dissipator is on this new slope. The normal depth values for the various discharges can be readily obtained from HDS No. 3 (III-A-1) or table III-1.



NORMAL CHANNEL DEPTH - CONJUGATE DEPTH RELATIONSHIP

The sequent depth values are obtained by applying the same process used to determine the design sequent depth above. These values are plotted in the figure above. This plot indicates excess tailwater depth is available in the downstream channel for discharges up to approximately 480 cfs. Beyond this point, the jump would begin to move downstream out of the basin.

HYDRAULIC JUMP EXPRESSIONS--SLOPING CHANNELS

Figure VI-14 from reference (VI-3), indicates a method of delineating hydraulic jumps in horizontal and sloping channels. Case A was analyzed in the previous section, and

Cases B and D will be considered in this section. Case C is not included since it is assumed that a horizontal floor begins at the end of the jump for Case D, making C and D, for practical purposes, the same.

If the channel bottom is selected as a datum, the momentum equation becomes:

$$Q(V_2-V_1)/g = .5b(y_1^2-y_2^2)\cos\theta + w\sin\theta \quad \ldots\ldots \text{VI-11}$$

The momentum formula used for the horizontal channels cannot be applied directly to hydraulic jumps in sloping channels since the weight of water (w), within the jump, must be considered. The difficulty encountered is in defining the water surface profile to determine the volume of water within the jumps for various channel slopes. This volume may be neglected for slopes less than 10 degrees and the jump analyzed as case A.

The Bureau of Reclamation (VI-3) conducted extensive model tests on Case B and C type jumps to define the length and depth relationships.

The procedures presented in this section are from those tests and apply to hydraulic jumps in sloping rectangular channels, only. Other channel shapes are not included because of their limited use and the difficulties involved in analysis. Model tests should be considered where other channel shapes are involved.

Figure VI-15 indicates the relation between the Froude number, tailwater and upstream depth for various slopes, Case D. The small inset provides a relationship between tailwater depth for a continuous slope and the conjugate depth for a jump on a horizontal apron. The inset indicates the additional depth required for a jump to form in a sloping channel.

Case B is the more common jump encountered in sloping channels. In this case, the jump forms on both the sloping and horizontal parts of the channel.

Sufficient tailwater depth should be provided for the front of the jump to be positioned at Section 1, Figure VI-14. Figure 16 indicates what occurs when the tailwater is increased a vertical increment. When the tailwater is increased Δy, the front of the jump moves up the slope several times Δy until the tailwater depth approaches $1.3(y_2)$. At this point, the relationship becomes geometric; an increase in tailwater moves the front of the jump an equal vertical distance. Figure VI-17 provides the depth relationship for the Case B-type jump. Design rules are provided at the end of this section.

Jump Length. The length of jump for both Case B and Case D can be obtained from Figure VI-18. This figure is for Case D type jump but it can be applied to Case B with negligible error. Figure VI-19 may also be used to determine the length of Case D type jumps.

Tailwater-Jump Height. The major design concern should be to determine an apron slope which will provide minimum excavation and require minimum concrete for the maximum discharge and tailwater condition.

Once this condition is established, then the jump height-tailwater relationship for intermediate flow condition can be checked. Generally, the tailwater for intermediate flows will be excessive for the jump requirements. This will not cause difficulty but will result in a submerged jump which provides a smoother water surface downstream and greater jump stability. Where the tailwater is found insufficient for the intermediate flows, the depth of the apron will have to be increased. It is not necessary that the jump form at the upstream end of the apron for intermediate flows as long as the length of basin is considered adequate. The slope itself has little effect on the stilling basin performance; therefore, using this design approach gives the designer freedom to choose the slope he desires.

The design of sloping hydraulic jump basins required greater individual judgment than for the more standardized horizontal jump basins. The length of basin is judged on the basis of the downstream channel bed while the slope and shape of aprons are determined from economic reasoning.

DESIGN RECOMMENDATIONS

The following recommendations from the Bureau of Reclamation should be followed in the design of the sloping aprons:

1. Determine an apron arrangement which will give the greater economy for the maximum discharge condition. This is the governing factor and the only justification for using a sloping apron.

2. Position the apron so that the front of the jump will form at the upstream end of the slope for maximum discharge and tailwater condition by means of the information

on Figure VI-15 and 17. Several trials will usually be required before the slope and location of the apron are compatible with the hydraulic requirement. It may be necessary to raise or lower the apron, or change the original slope entirely.

3. The length of the jump for maximum or partial flows can be obtained from Figure VI-18. The portion of the jump to be confined on the stilling basin apron is a decision for the designer. The average overall apron averages 60 percent of the length of jump for the maximum discharge condition. The apron may be lengthened or shortened, depending upon the quality of the rock in the channel and other local conditions. If the apron is set on loose material and the downstream channel is in poor condition, it may be advisable to make the total length of apron the same as the length of jump.

4. With the apron designed properly for the maximum discharge condition, it should then be determined that the tailwater depth and length of basin available for energy dissipation are sufficient for, say, 1/4, 1/2, and 3/4 capacity. If the tailwater depth is sufficient or in excess of the jump height for the intermediate discharges, the design is acceptable. If the tailwater depth is deficient, it may then be necessary to try a flatter slope or reposition the sloping portion of the apron for partial flows. In other words, the front of the jump may remain at Section 1 (Figure VI-14), move upstream from Section 1, or move down the slope for partial flows, providing the tailwater depth and length of apron are considered sufficient for these flows.

5. Horizontal and sloping aprons will perform equally well for high values of the Froude number if the velocity distribution and depth of flow are reasonably uniform on entering the jump.

6. The slope of the chute upstream from a stilling basin has little effect on the hydraulic jump when the velocity distribution and depth of flow are reasonably uniform on entering the jump.

7. A small solid triangular sill, placed at the end of the apron, is the only appurtenance needed in conjunction with the sloping apron. It serves to lift the flow as it leaves the apron and thus acts to control scour. Its dimensions are not critical; the most effective height is between $0.05(y_2)$ and $0.10(y_2)$ with a face slope of 3:1 to 2:1.

8. The approach should be designed to insure symmetrical flow into the stilling basin. (This applies to all stilling basins.) Asymmetry produces large horizontal eddies that can carry riverbed material onto the apron. This material, circulated by the eddies, can abrade the apron and appurtenances in the basin at a very surprising rate. Eddies can also undermine wing walls and riprap.

9. A model study is advisable when the discharge exceeds 500 cfs per foot of apron width, where there is any form of asymmetry involved, and for higher values of the Froude number where stilling basins become increasingly costly and the performance less acceptable.

JUMP EFFICIENCY

A general expression for the energy loss (H_L/H_1 in any shape channel is:

$$H_L/H_1 = 2-2(y_2)+Fr^2[1-A_1{}^2/A_2{}^2]/(2+Fr^2) \qquad \text{.VI-12}$$

Where Fr is the upstream Froude number at section one:

$Fr^2 = V^2/gy_m$, y_m is the hydraulic depth

This equation is plotted for the various channel shapes as Figure VI-20.

Even though this figure indicates that the nonrectangular sections are more efficient for the higher Froude Numbers, it should be remembered that these sections also involve longer jumps, stability problems, and a rough downstream water surface.

VI-1. U.S. Bureau of Reclamation, DESIGN OF SMALL DAMS, Second Edition 1973, GPO

VI-2. Sylvester, R., HYDRAULIC JUMP IN ALL SHAPES OF HORIZONTAL CHANNELS, Jourl. Hyd. Div. ASCE, Vol. 90, HY1, Jan 1964, pp. 23-55.

VI-3. Bradley, J. N. and Peterka, A. J., STILLING BASINS WITH SLOPING APRON, Symposium on Stilling Basins and Energy Dissipators, ASCE Series No. 5., June 1961, pp. 1405-1.

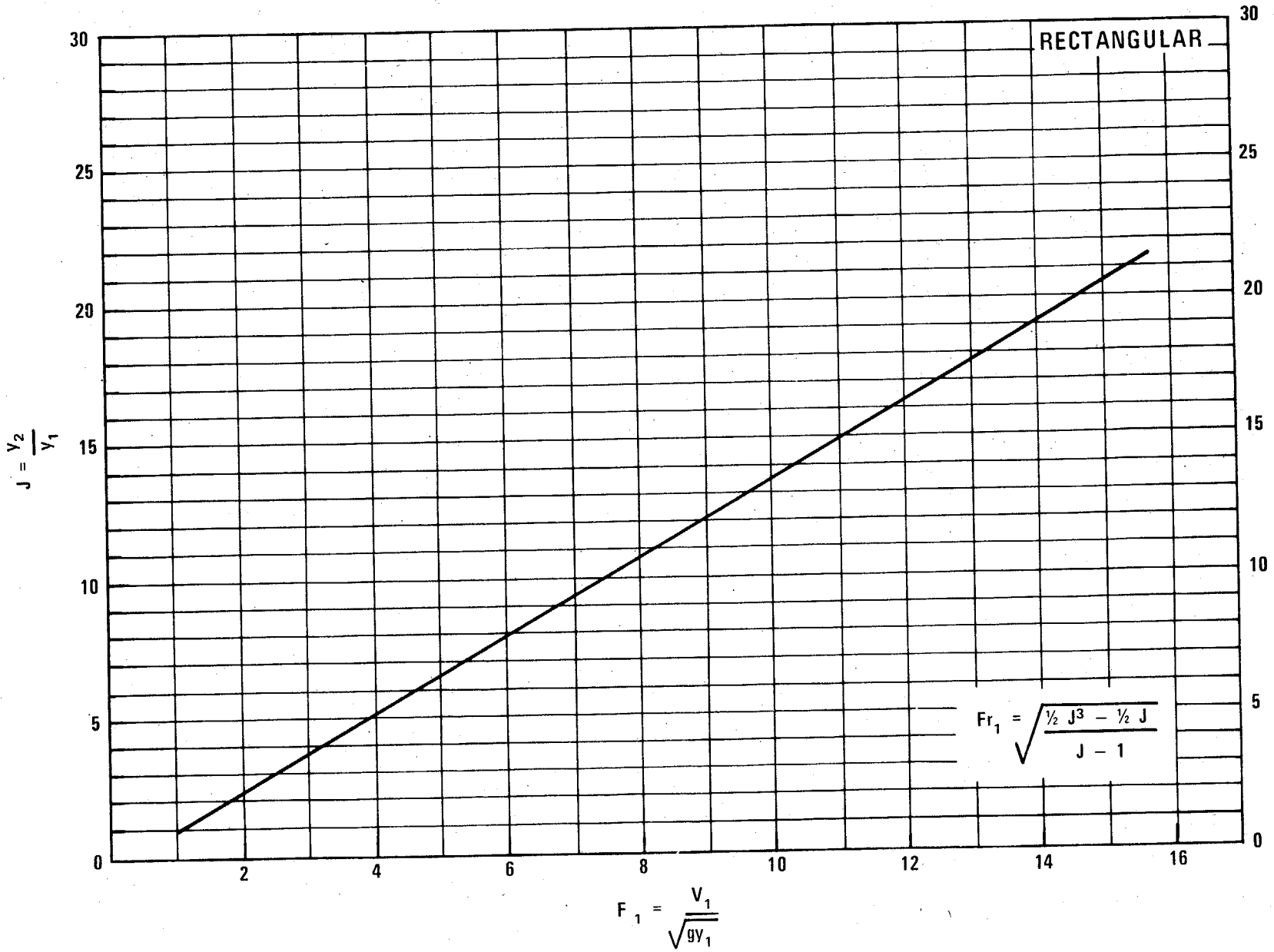


FIGURE VI-4 HYDRAULIC JUMP – HORIZONTAL, RECTANGULAR CHANNEL

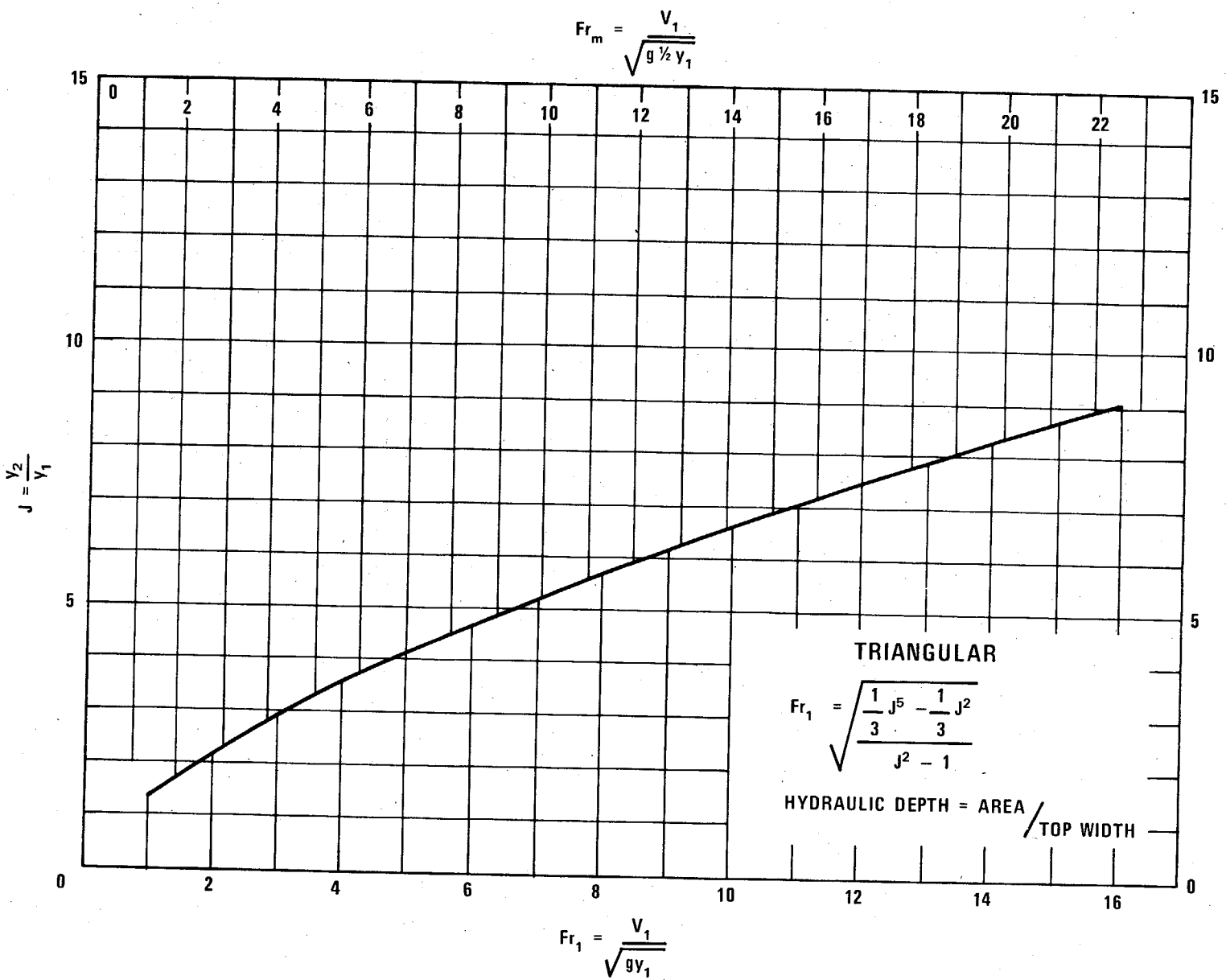


FIGURE VI-5 HYDRAULIC JUMP – HORIZONTAL, TRIANGULAR CHANNEL

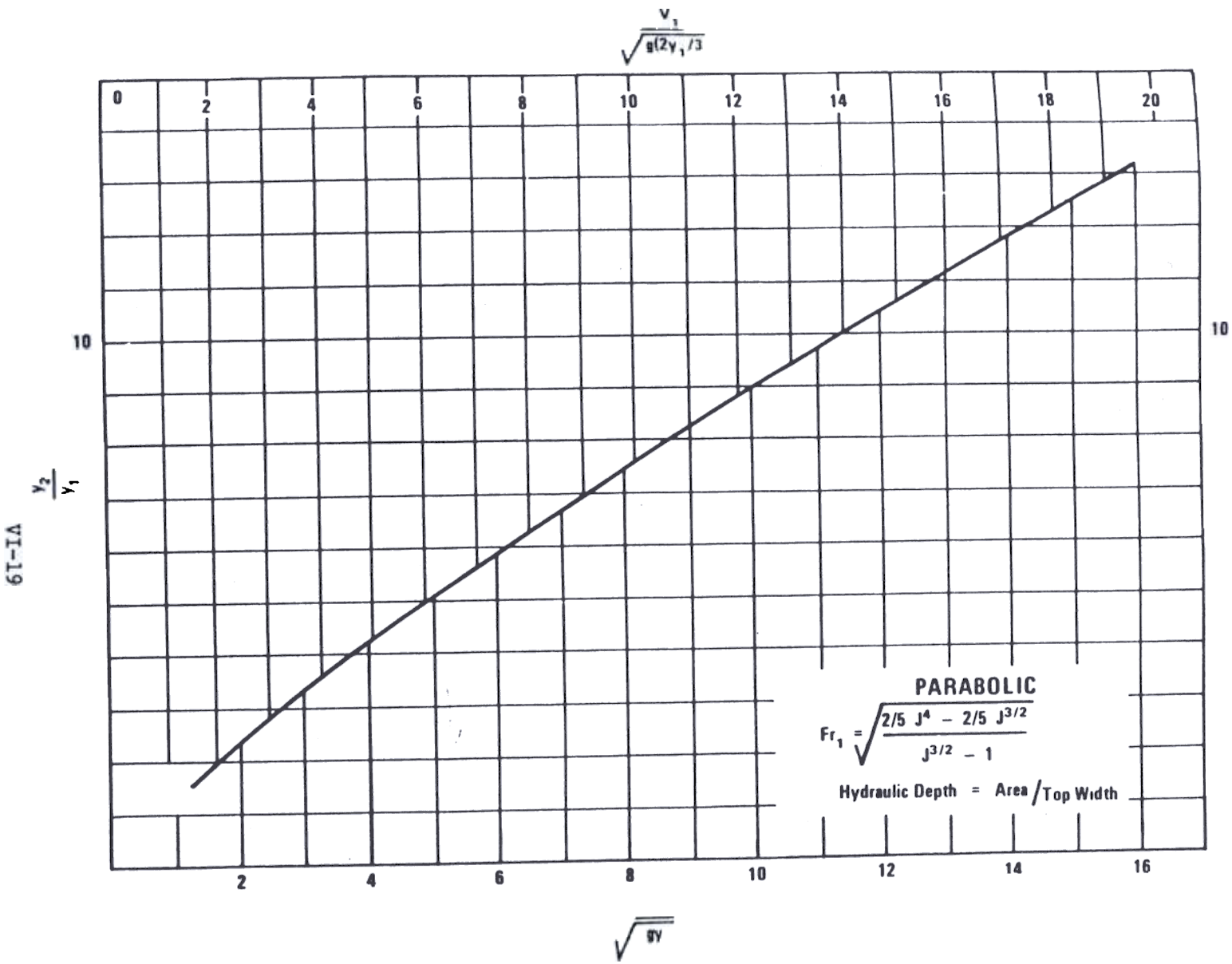


FIGURE HYDRAULIC JUMP HORIZONTAL, ARABOLIC CHANNE

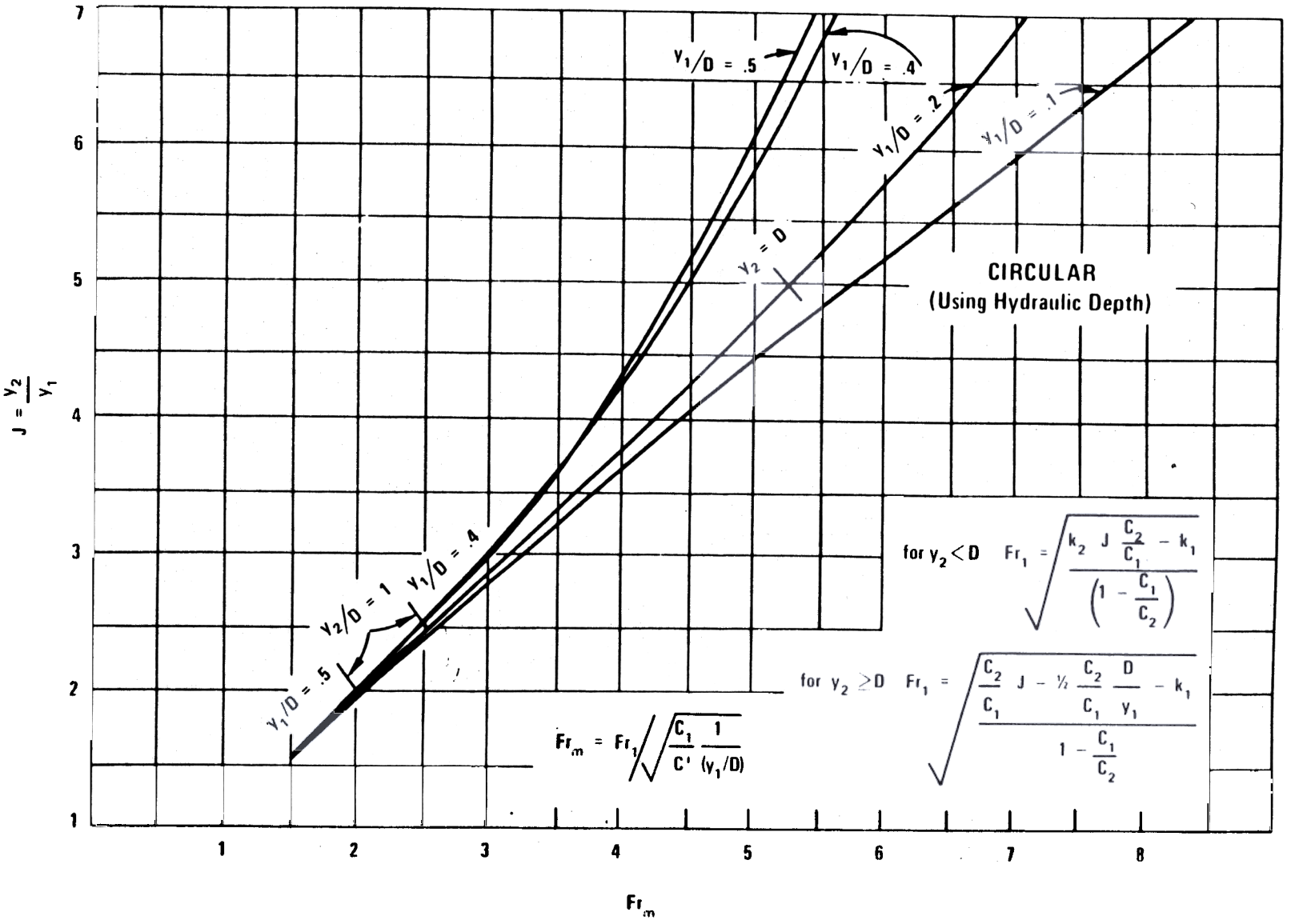


FIGURE VI-7 HYDRAULIC JUMP – HORIZONTAL, CIRCULAR CHANNEL (HYDRAULIC DEPTH)

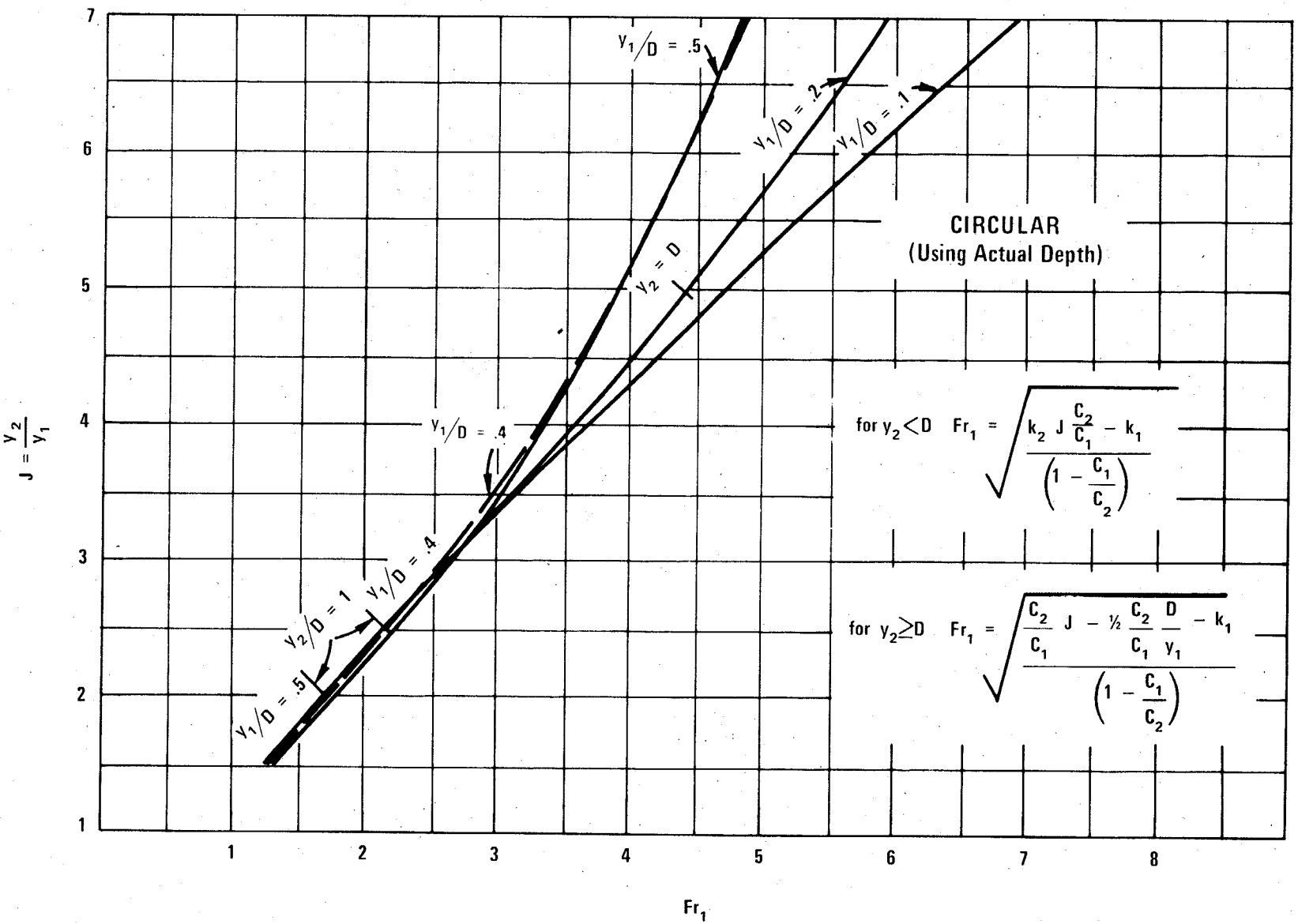


FIGURE VI-8 HYDRAULIC JUMP – HORIZONTAL, CIRCULAR CHANNEL (ACTUAL DEPTH)

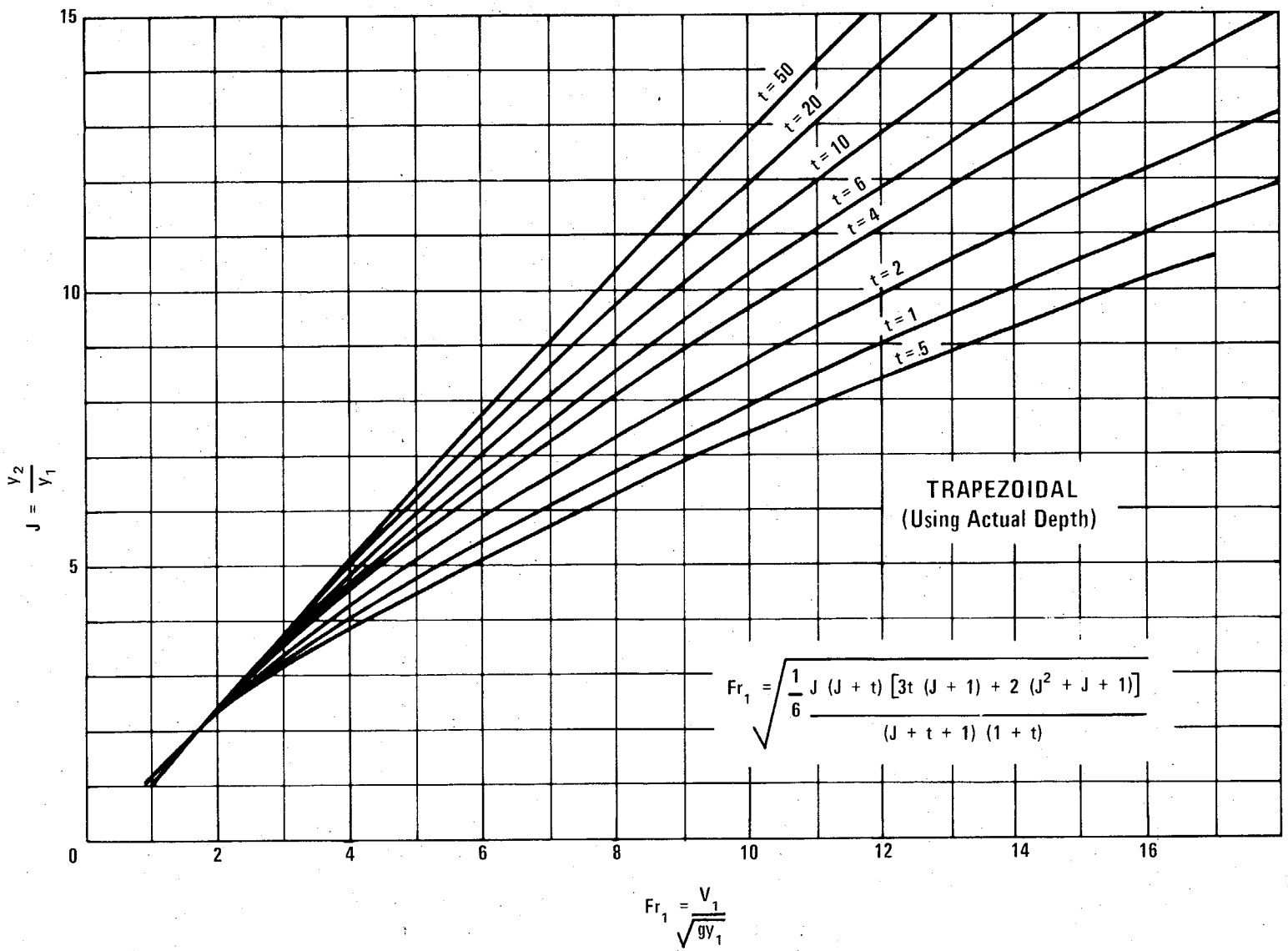


FIGURE VI-9 HYDRAULIC JUMP – HORIZONTAL, TRAPEZOIDAL CHANNEL (ACTUAL DEPTH)

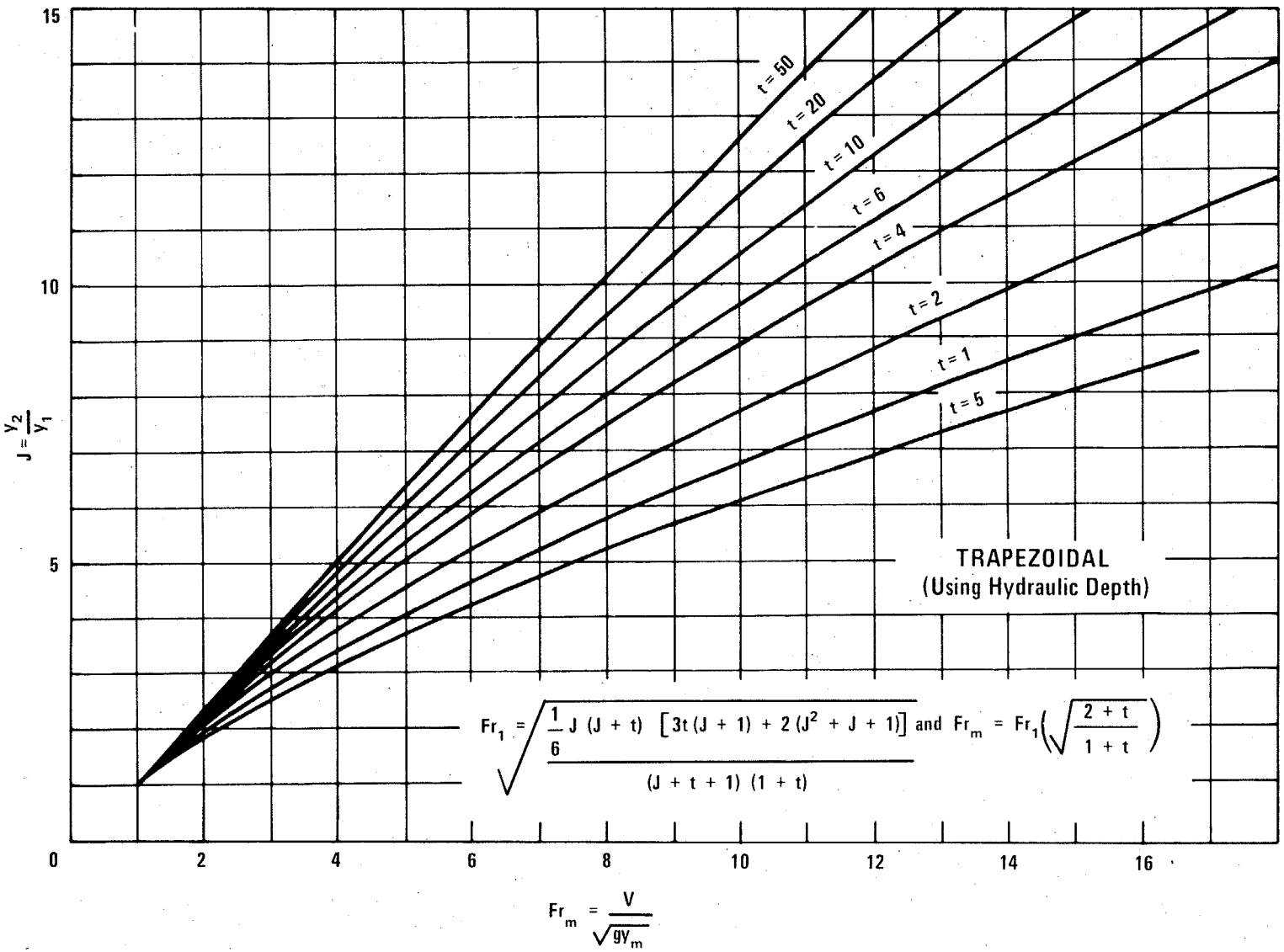


FIGURE VI-10 HYDRAULIC JUMP – HORIZONTAL, TRAPEZOIDAL CHANNEL (HYDRAULIC DEPTH)

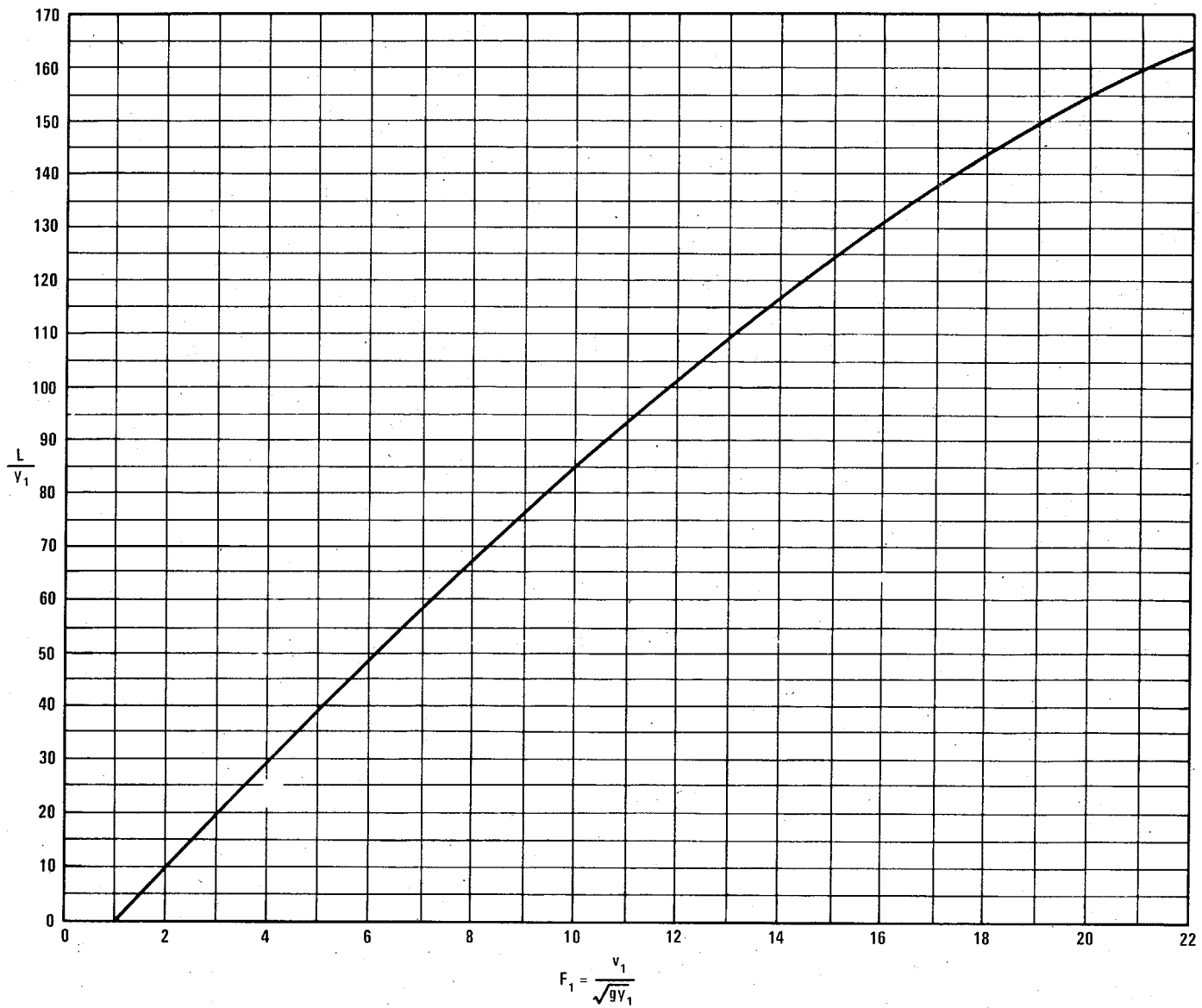


FIGURE VI-11. LENGTH OF JUMP IN TERMS OF y_1, RECTANGULAR CHANNEL

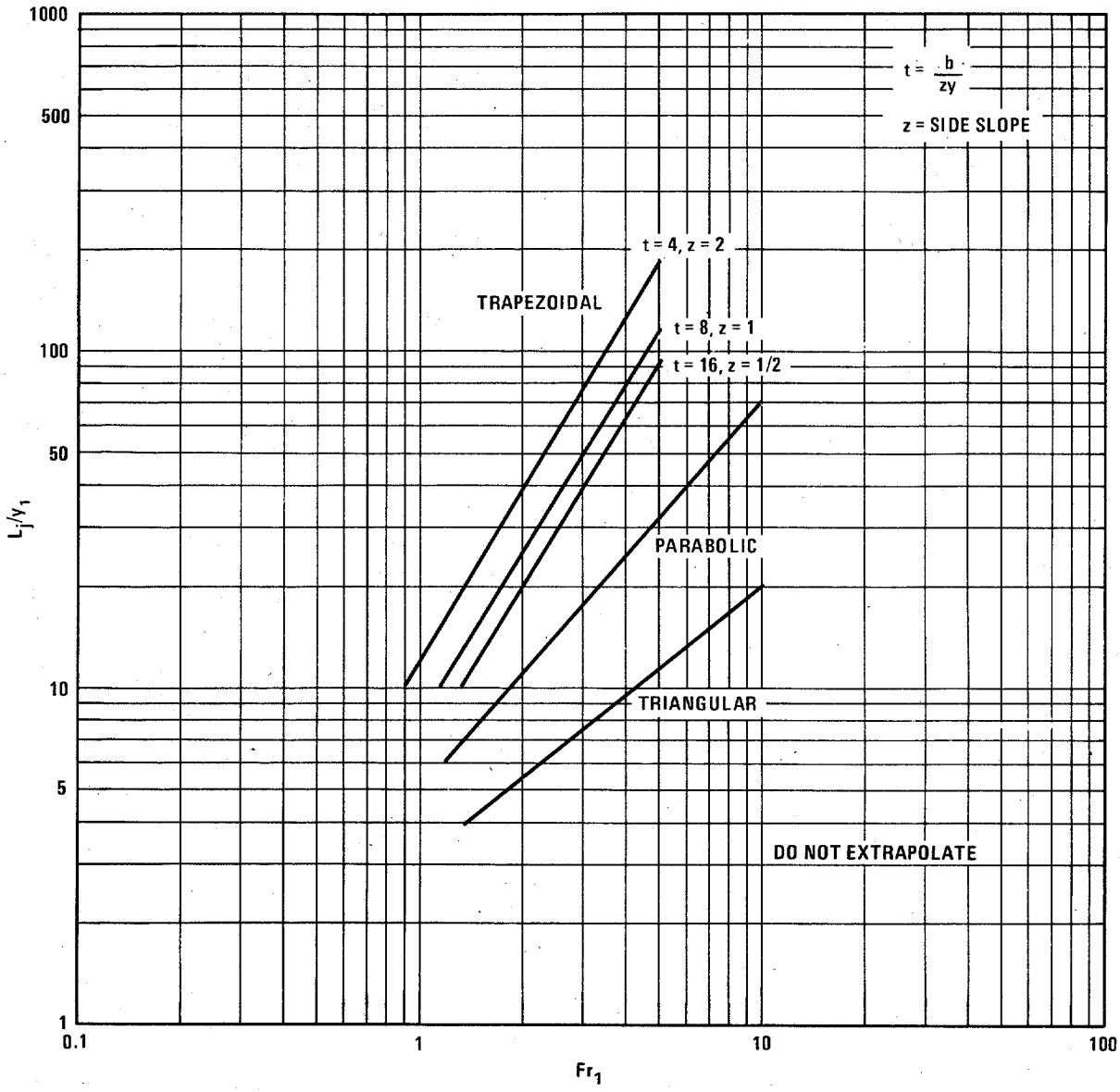


FIGURE VI-12. HYDRAULIC JUMP LENGTH FOR NONRECTANGULAR CHANNELS

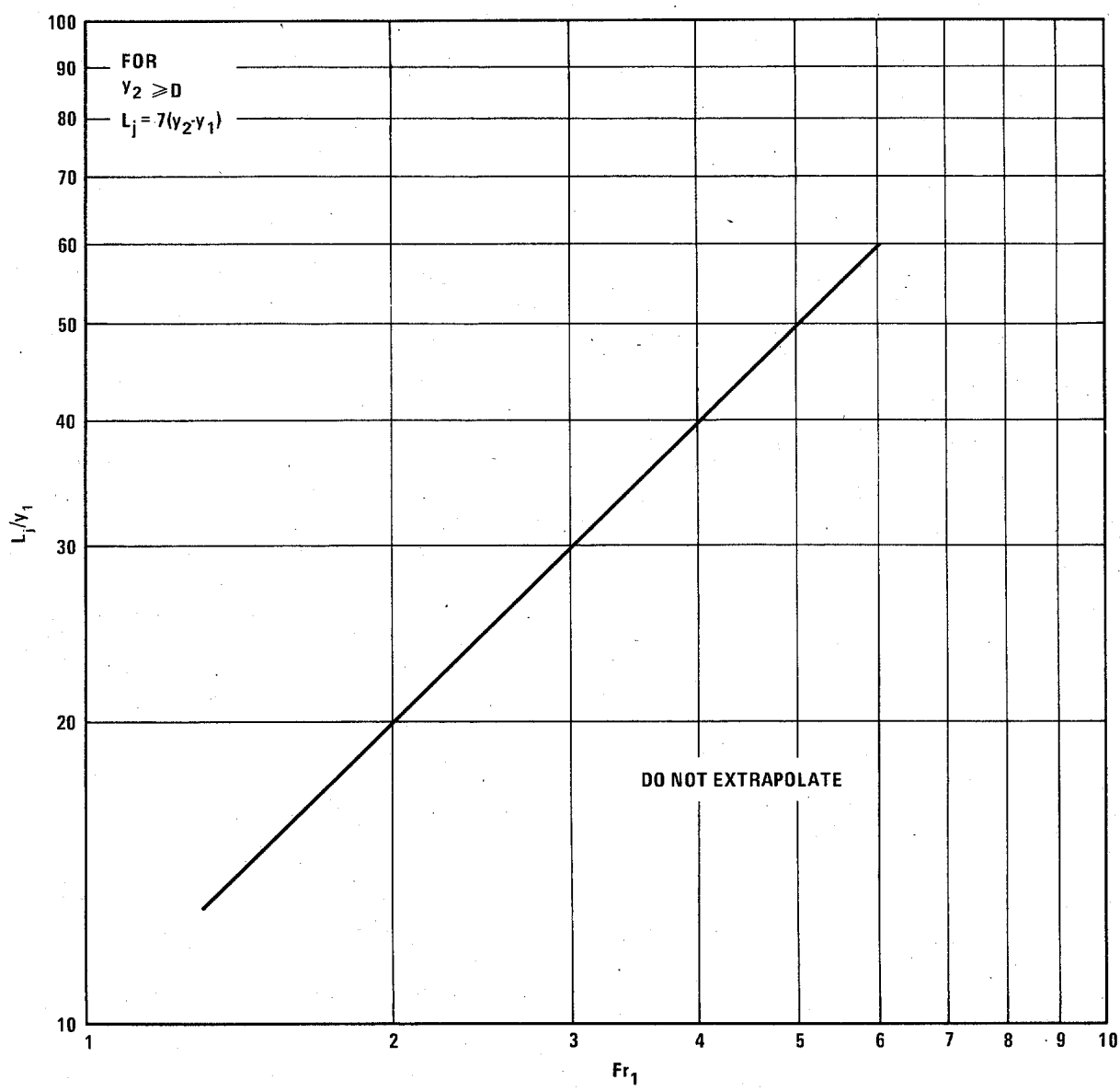


FIGURE VI-13. JUMP LENGTH CIRCULAR CHANNEL WITH $y_2 < D$.

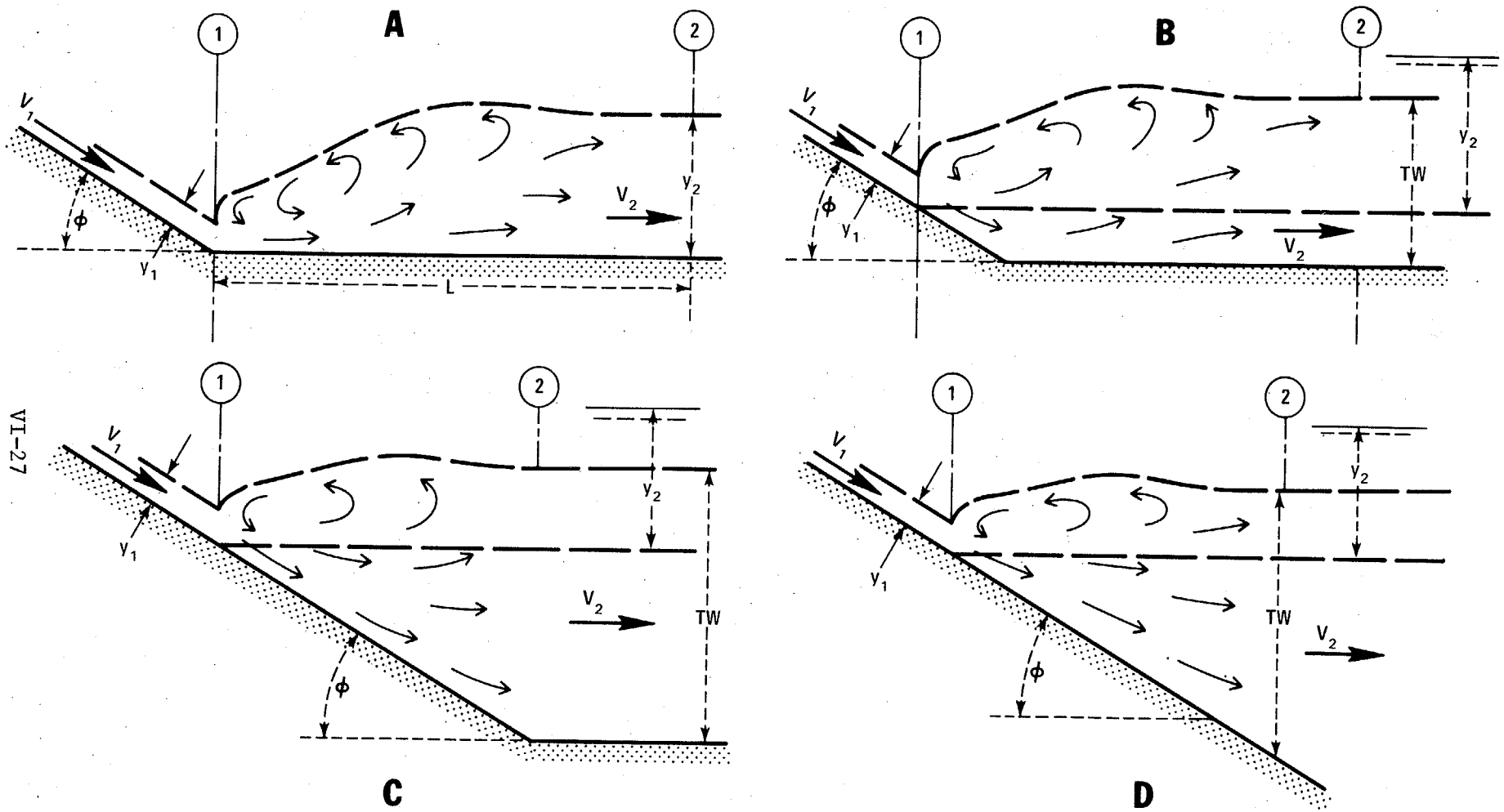


FIGURE VI-14. HYDRAULIC JUMP TYPES SLOPING CHANNELS, FROM REFERENCE VI-2

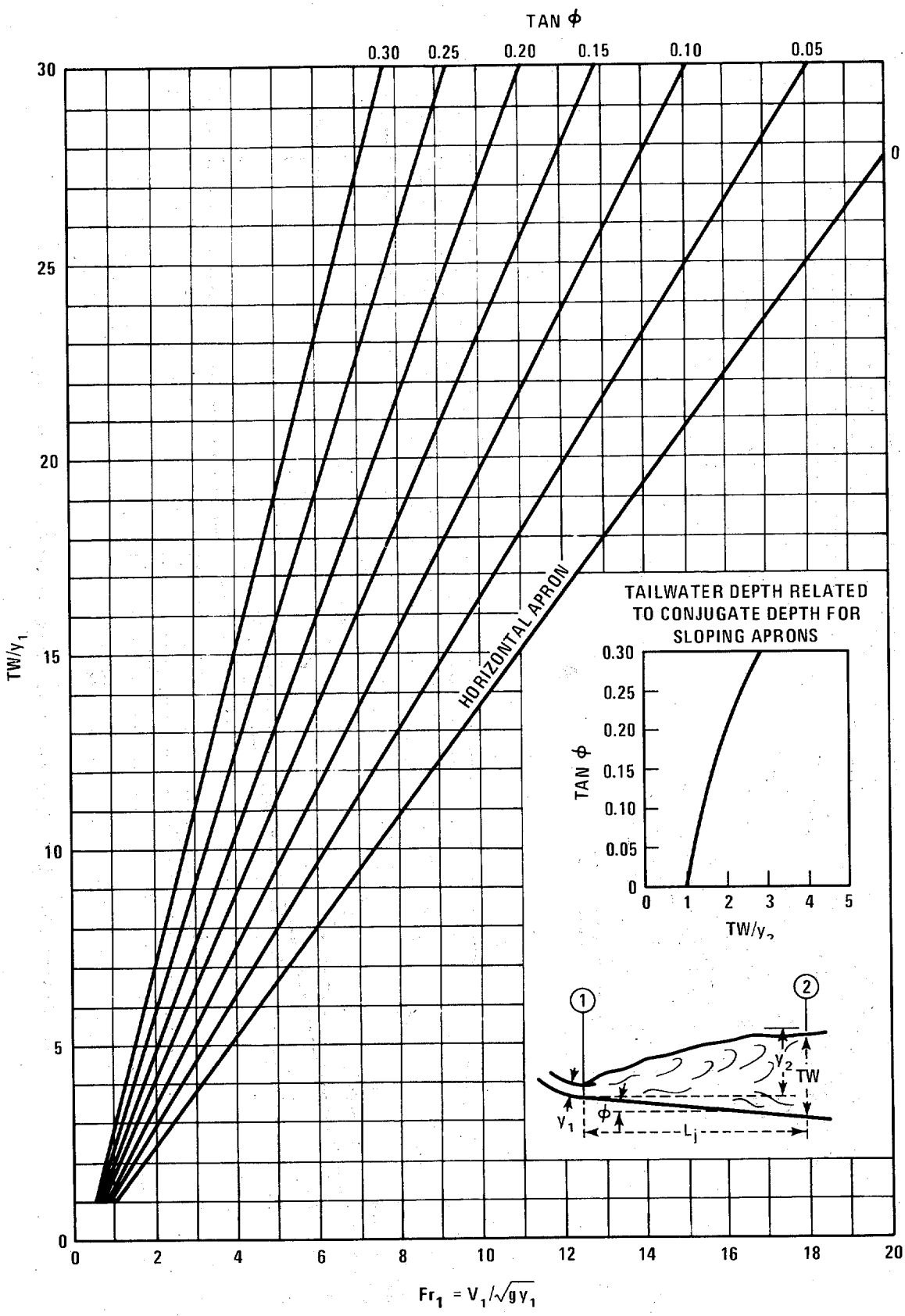


FIGURE VI-15. STILLING BASIN, CASE D, HYDRAULIC JUMP ON SLOPING APRON RATIO OF TAILWATER DEPTH TO y_1, FROM REFERENCE VI-2

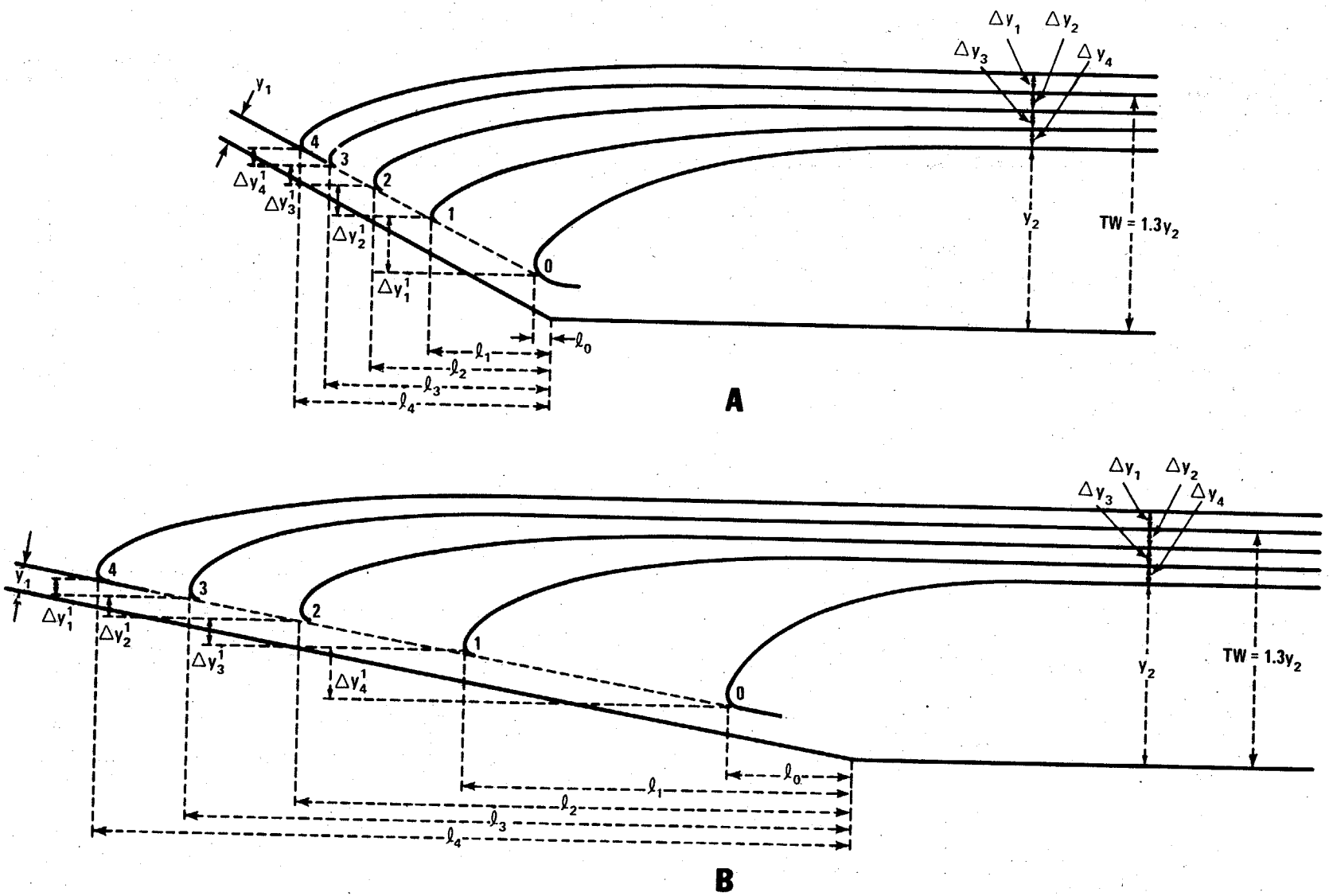


FIGURE VI-16. STILLING BASIN, CASE B, PROFILE CHARACTERISTICS, FROM REFERENCE VI-2

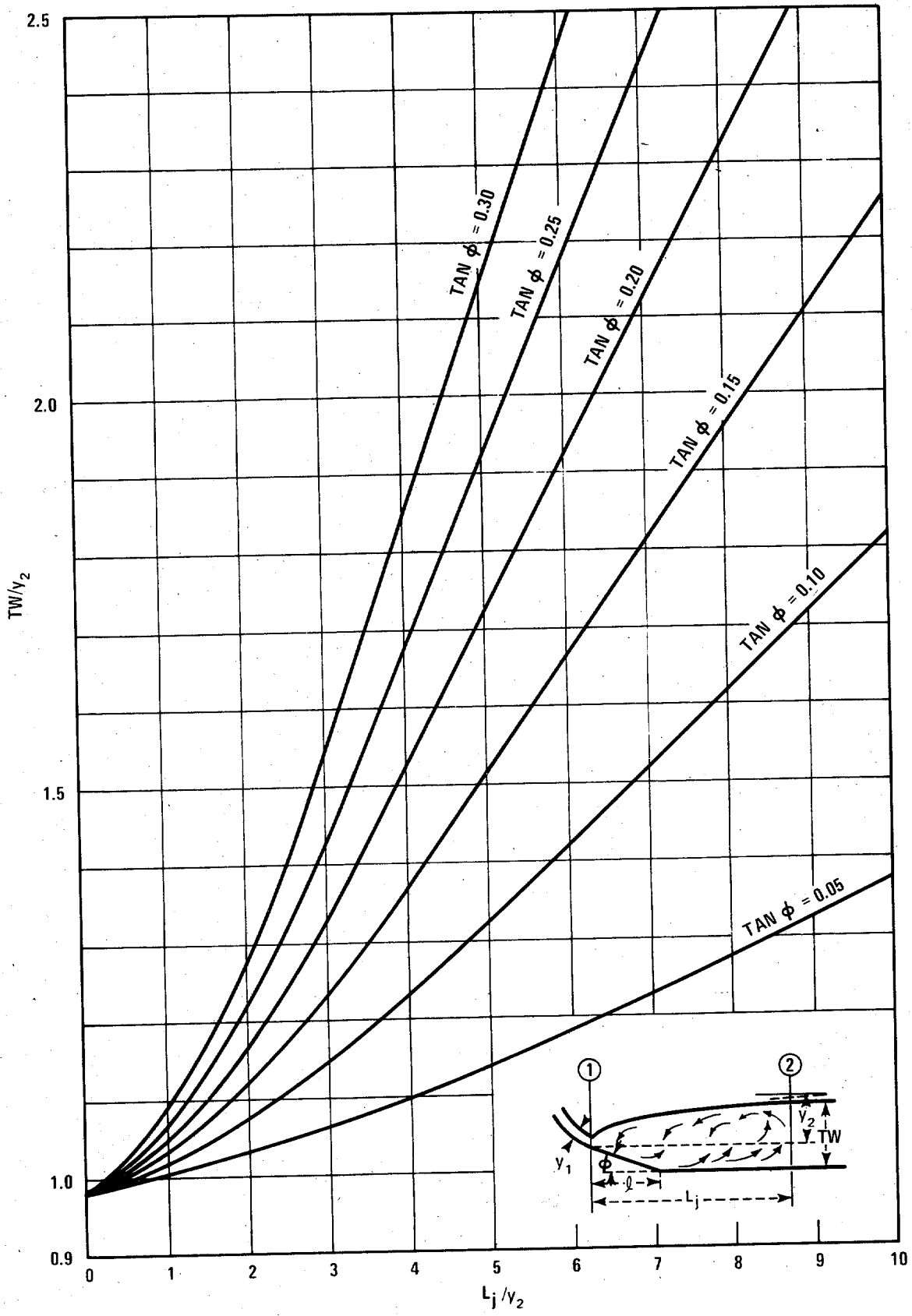


FIGURE VI-17. STILLING BASIN TAILWATER REQUIREMENT FOR SLOPING APRONS – CASE B. FROM REFERENCE VI-2

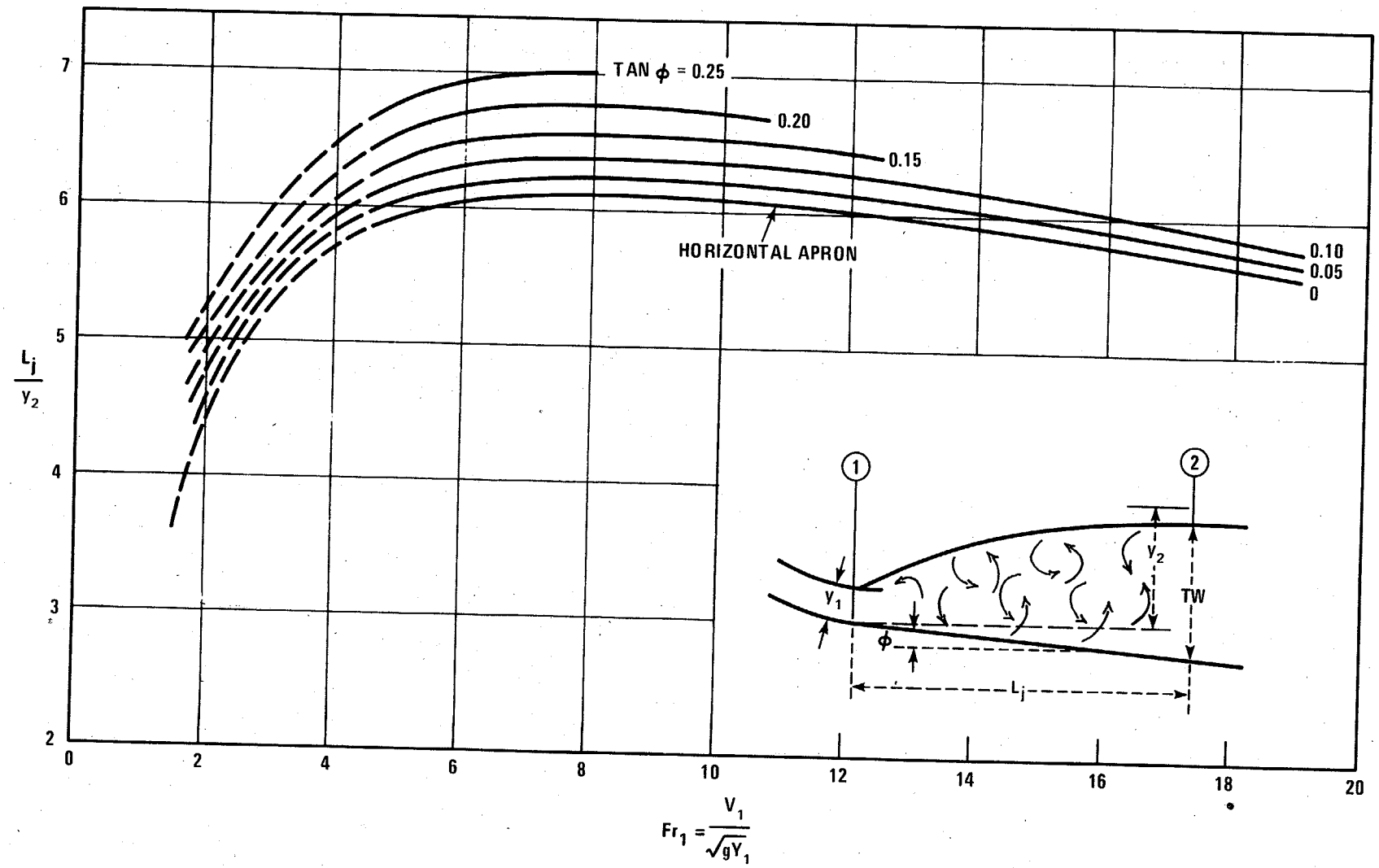


FIGURE VI-18. STILLING BASIN, CASE D, LENGTH OF JUMP IN TERMS OF CONJUGATE DEPTH, y_2, FROM REFERENCE VI-2

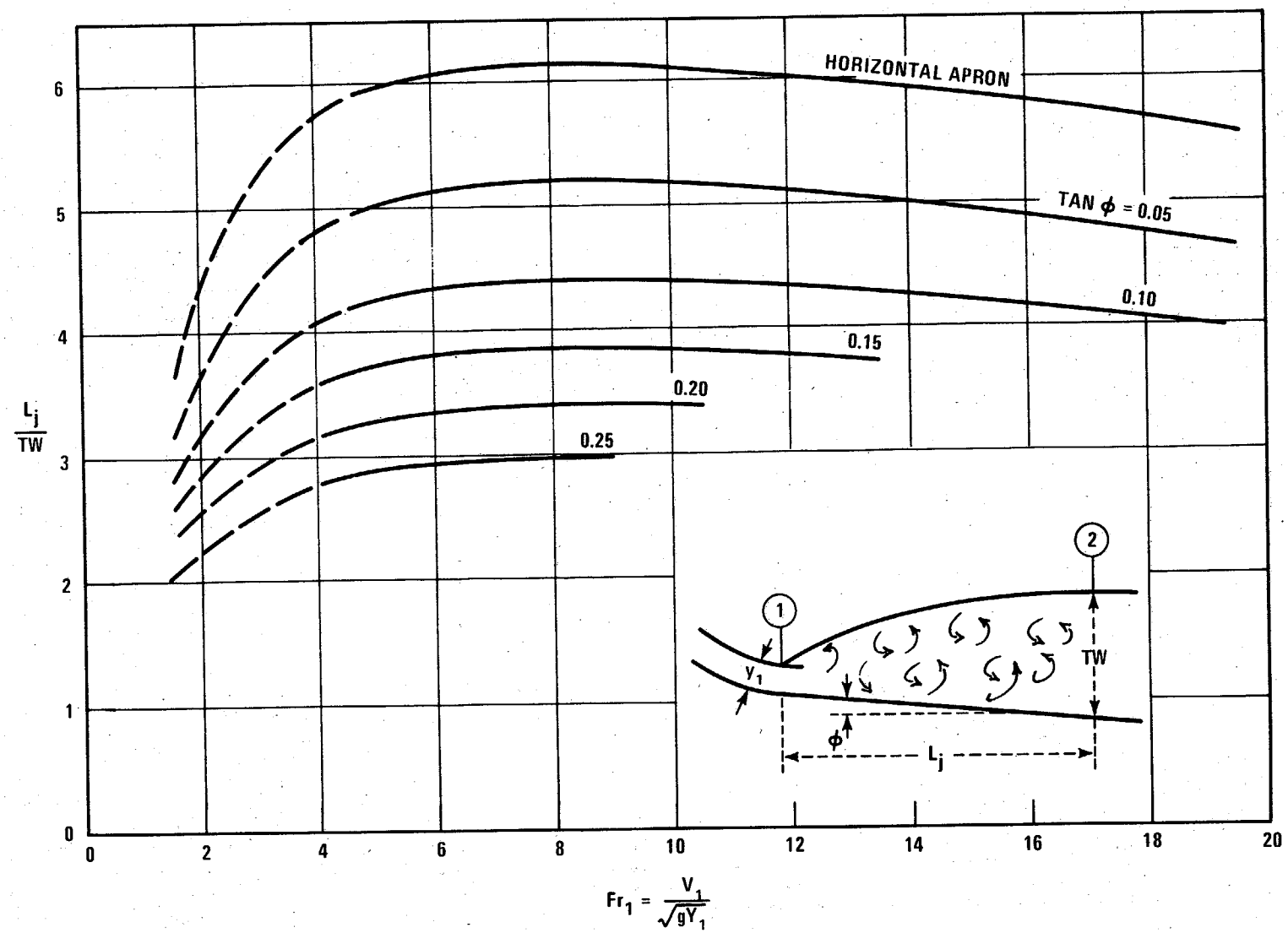


FIGURE VI-19. STILLING BASIN, CASE D, LENGTH OF JUMP IN TERMS OF TW DEPTH FOR REFERENCE VI-2

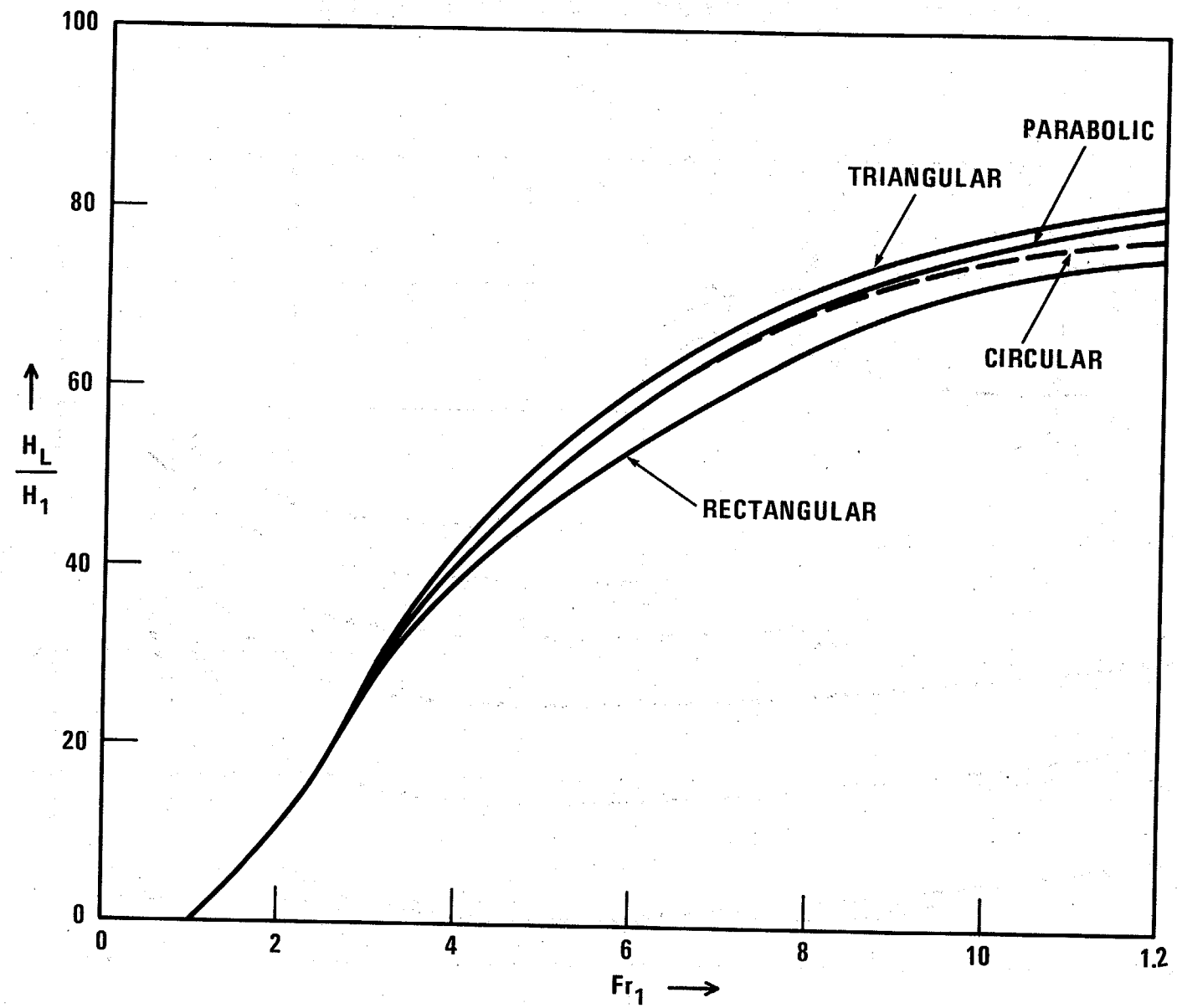


FIGURE VI-20. RELATIVE ENERGY LOSS FOR VARIOUS CHANNEL SHAPES.

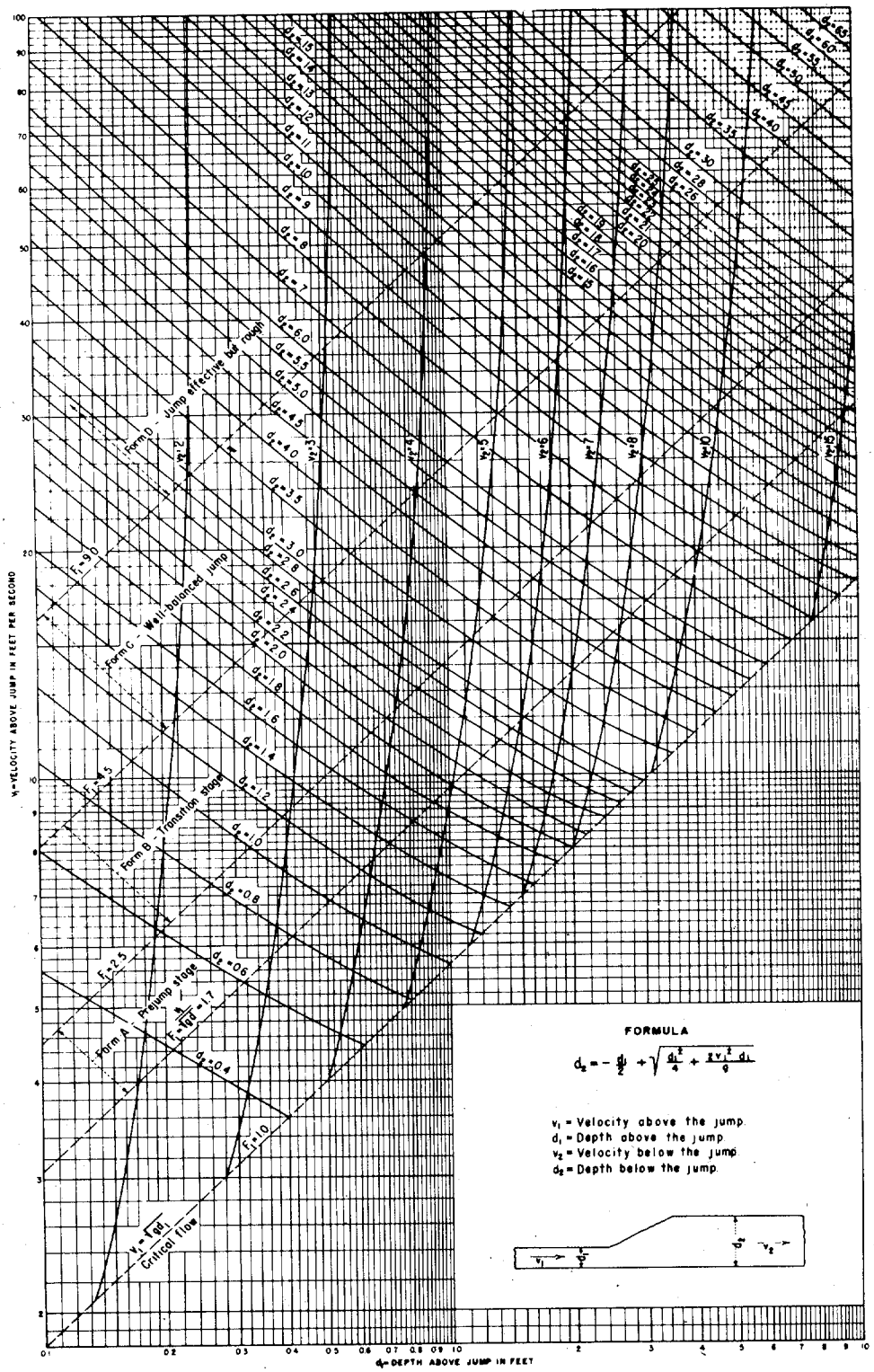


Figure VI-21. **Relations between variables in hydraulic jump for rectangular channel. From Reference III-3.**

VII - FORCED HYDRAULIC JUMP BASINS

There are a number of energy dissipator designs which utilize blocks, sills, or other roughness elements to impose exaggerated resistance to flow. Roughness elements provide the designer with a versatile tool in that they may be utilized in forcing and stabilizing the hydraulic jump and shortening the hydraulic jump basin. They may also be employed inside the culvert barrel, at the culvert exit or in open channels.

This section contains information which enables the designer to evaluate the effect of roughness elements and, within limits, "tailor-make" an energy dissipator. A number of "formal or fixed" designs are also presented. Each design section discusses limitations, provides design guidance and sample problem solutions.

DRAG FORCE ON ROUGHNESS ELEMENTS

Roughness elements must be anchored sufficiently to withstand the drag forces on the elements. The fluid dynamic drag equation is:

$$F_D = C_D A_F \rho V_a^2/2$$

Horner's (VII-1) maximum coefficient of drag, C_D, for a structural angle or a rectangular block is 1.98. Using 1.935 slugs/ft.^{3} for the density of water, the drag force becomes:

$$F_D = 1.94 A_F V_a^2 \quad \ldots\ldots\ldots\ldots\ldots \text{VII-1}$$

In the CSU rigid boundary basin, the USBR basins, and the SAF basin, design all of the roughness elements for the worst case using the approach velocity at the first row for V_a. In cases of tumbling flow or increased resistance on steep slopes, use the normal velocity of the culvert without roughness elements for V_a.

The force may be assumed to act at the center of the roughness element as shown in figure VII-1.

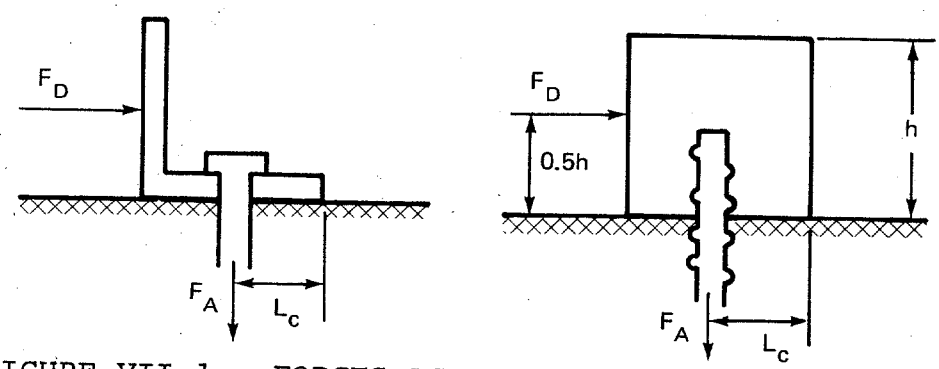


FIGURE VII-1. FORCES ACTING ON A ROUGHNESS ELEMENT

The anchor forces necessary to resist overturning can be computed as follows:

$$F_A = hF_D/2L_c = 0.97(h/L_c)A_F V_a^2 \quad \ldots\ldots\ldots\ldots \text{VII-2}$$

where: F_A = total force on anchors
F_D = drag force on roughness element
h = height of roughness element
L_c = distance from downstream edge of roughness element to the centroid of the anchors
A_F = frontal area of roughness element
V_a = approach velocity acting on roughness element

VII-A. C.S.U. RIGID BOUNDARY BASIN

The Colorado State University rigid boundary basin (VII-A-1) which uses staggered rows of roughness elements is illustrated in figure VII-A-1.

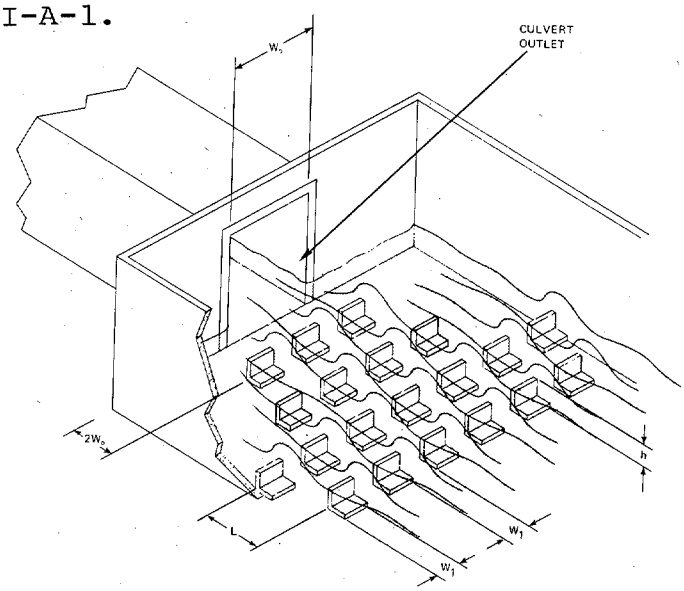


FIGURE VII-A-1. SKETCH OF C.S.U. RIGID BOUNDARY BASIN

CSU tested a number of basins with different roughness configurations to determine the average drag coefficient over the roughened portion of the basins. The effects of the roughness elements are reflected in a drag coefficient which was derived empirically for each roughness configuration. The experimental procedure was to measure depths and velocities at each end of the control volume illustrated in figure VII-A-2, and compute the drag coefficient from the momentum equation by balancing the forces acting on the volume of fluid.

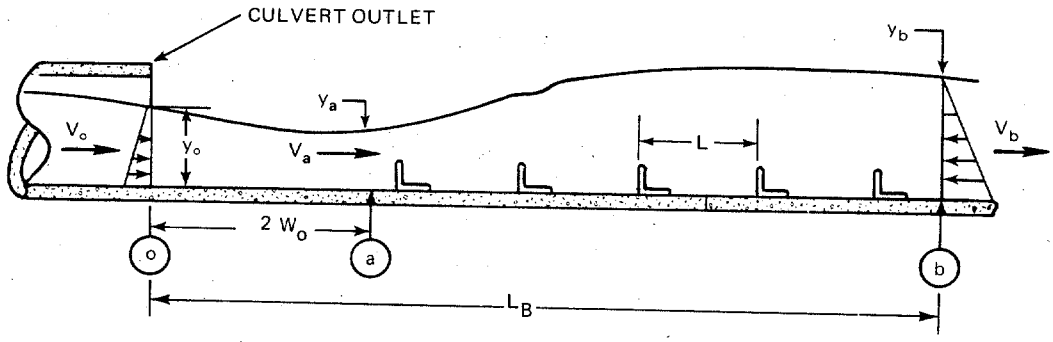


FIGURE VII-A-2. DEFINITIVE SKETCH FOR THE MOMENTUM EQUATION

The CSU test indicate several design limitations. The height, h, of the roughness elements must be between 0.31 and 0.91 of the approach flow average depth (y_A); and, the relative spacing, L/h, between rows of elements, must be either 6 or 12. The latter is not a severe restriction since relative spacing is normally a fixed parameter in a design procedure and other tests (VII-A-2) have shown that the best range for energy dissipation is from 6 to 12.

Although the tests were made with abrupt expansions, the configurations recommended for use are the combination flared-abrupt expansion basins shown in figure VII-A-5. These basins contain the same number of roughness elements as the abrupt expansion basin.

The flare divergence, u_e, is a function of the longitudinal spacing between rows of elements, L, and the culvert barrel width, W_o:

$$u_e = 4/7 + (10/7)L/W_o$$

The values of the basin drag, C_B, for each basin configuration are given in figure VII-A-5. The C_B values listed are for expansion ratios (W_B/W_o) from 4 to 8. They are also valid for lower ratios (2 to 4) if the same number of roughness elements, N, are placed in the basin. This requires additional rows of elements for basins with expansion ratios less than 4. The arrangements of the elements for all basins is symmetrical about the basin centerline. All basins are flared to the width W_B of the corresponding abrupt expansion basin.

The basic design equation is:

$$\rho V_o Q + C_p \gamma (y_o^2/2) W_o = C_B A_F N \rho V_A^2/2 + \rho V_B Q + \gamma Q^2/2V_B^2 W_B \quad \text{. . .VII-A-1}$$

Where: C_p is the momentum correction coefficient for the pressure at the culvert outlet (figure VII-A-4).

γ & ρ are unit weight 62.4 lb/ft and density (1.94) lb-sec /ft of water, respectively

y_o, V_o, and W_o are depth, velocity and culvert width, respectively, at the culvert outlet.

V_a = the approach velocity at two culvert widths downstream of the culvert outlet.

V_B & W_B = exit velocity and basin width just downstream of the last row of roughness elements.

N = total number of roughness elements in the basin

A_f = frontal area of one full roughness element.

C_B = basin drag coefficient.

This equation is applicable for basins on less than 10 percent slopes. For basins with greater slopes, the weight of the water within the hydraulic jump must be considered in the expression. Equation VII-A-2 includes the weight component by assuming a straight-line water surface profile across the jump:

$$C_p \gamma Y_o^2 W_o/2+\rho V_o Q+w(\sin\theta)=C_B A_f N \rho V_A^2/2+\gamma Q^2/2V_B^2 W_B+\rho V_B Q. \quad . . .\text{VII-A-2}$$

where: w = weight of water within the basin.

$$\text{Approximate Volume} = (y_o W_o+y_A W_A)W_o+(.75LQ/V_B)[(N_r-1) -(W_B/W_o-3)(1-W_A/W_B)/2]$$

Weight = (Volume)γ

θ = arc tan of the channel slope, S_o.

N_r= Number of rows of roughness elements

L = longitudinal spacing between rows of elements.

The velocity V_A and depth y_A at the beginning of the roughness elements can be determined from figures IV-A-2 through 5. These figures are also based on slopes less than 10 percent. Where slopes are greater than 10 percent, V_A and y_A can be computed using the energy equation written between the end of the culvert (section o) and two culvert widths downstream (section A).

$$2W_o S_o+y_A+(0.25)(Q/W_A y_A)^2/2g=y_o+0.25(V_o^2/2g) \quad . . . .\text{VII-A-3}$$

where $W_A=W_o[4/3Fr+1]$
(Equation VII-A-3 adapted from Equation IV-A-5)

There will be substantial splashing over the first row of roughness elements if the elements are large and if the approach velocity is high. This problem can be handled by providing sufficient freeboard or by providing some type of splash plate. If feasible from the standpoint of culvert design, both structural and hydraulic, one solution to potential splash problems is to locate the dissipator partially or totally within the culvert barrel. Such a design might also result in economic, safety, and aesthetic advantages.

The necessary freeboard (F.B.) can be obtained from:

$$F.B. = h+y_1+0.5(V_A \sin\phi)^2/32.2 \quad \text{. . . . . VII-A-4}$$

The value of ϕ is a function of y_A/h and the Froude number, $V_A/\sqrt{gy}$. It is suggested that $\phi=45°$ be used in design, since no relationship has been derived.

Another solution is a splash shield, which has been investigated in the laboratory (VII-A-3). This involves suspending a plate with a stiffener between the first two rows of roughness elements as shown in figure VII-A-3. The height to the plate was selected rather arbitrarily as a function of the critical depth since flow usually passed through critical in the vicinity of the large roughness elements.

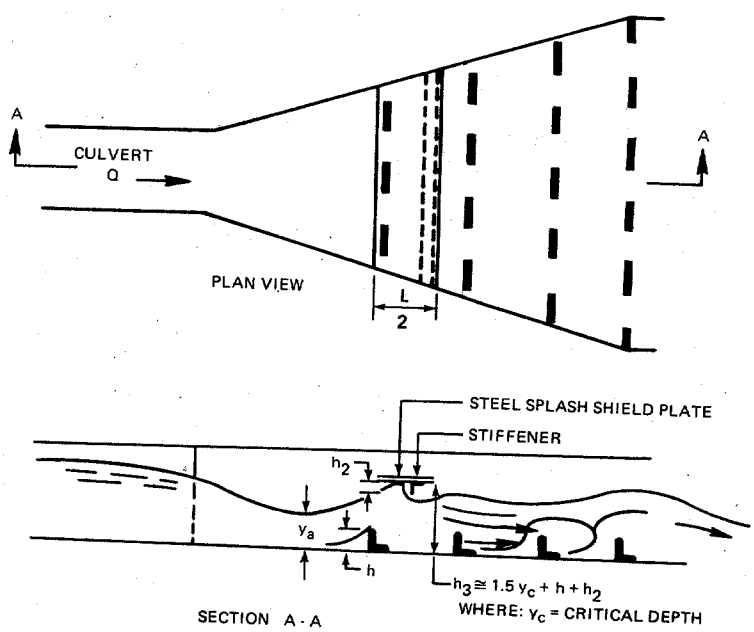


FIGURE VII-A-3. SPLASH SHIELD

DESIGN DISCUSSION

The inital design step is to compute the flow parameters at the culvert outlet or, if the basin is partially or totally located within the culvert barrel, at the beginning of the flared portion of the barrel. Compute the velocity, V_o, depth, y_o, and Froude number, Fr.

Select a trial basin from figure VII-A-5 based on the W_B/W_o expansion ratio which best matches the site geometry or satisfies other constraints.

Determine the flow condition V_A and y_A at the approach to the roughness element field--two culvert widths downstream.

For basins on slopes less than 10 percent with expansion ratios, W_B/W_o, between 4 and 8, use figure IV-A-2 or 3 to find V_A and figure IV-A-4 or 5 to find y_A. For basins with expansion ratios between 2 and 4, use figures IV-A-2 or 3 to determine V_A and compute y_A based on the actual width of the basin two culvert widths downstream.

For basins with slopes greater than about 10 percent, use equation VII-A-3 to determine both V_A and y_A.

Select the trial roughness height to depth ratio h/y_A from figure VII-A-5 and determine: roughness element height, h; longitudinal spacings between rows of elements, L; width of basin, W_B; number of rows, N_r; number of elements, N; element width, W_1; divergence, u_e; basin drag, C_B; frontal area of element, $A_f=W_1h$; and C_p from figure VII-A-4.

Total basin length is $L_1=2W_o+LN_r$. This provides a length downstream of the last row of elements equal to the length between rows, L.

Solve equation VII-A-1 or 2 and if the width of the basin matches the downstream channel and the normal flow conditions, V_n and y_n for the channel are known, solve equation VII-A-1 or 2 for C_BA_fN.

Using the C_B, A_f, and N values found in figure VII-A-5 also compute C_BA_fN. This last value should be equal to or larger than the C_BA_fN value obtained from equation VII-A-1 or 2. If the value is less, select a new roughness configuration.

If the basin width is less than the downstream channel width--widths larger than the natural channel are not recommended--solve equation VII-A-1 or 2 for V_B. This is a trial and error process and will result in three solutions. The negative root may be discarded and the correct positive root determined from the downstream condition. If the downstream depth is subcritical, the smaller root (V_B) is the solution providing the tailwater depth is less than y_B. If y_B is smaller than the tailwater--tailwater controls the outlet flow. If the downstream flow is supercritical, the larger root (V_B) is the proper choice; however, when the tailwater depth is larger than y_B, tailwater may again control.

The basin layout is indicated on figure VII-A-5. The elements are symmetrical about the basin centerline and the spacing between elements is approximately equal to the element width. In no case should this spacing be made less than 75 percent of the element width.

The W_1/h ratio must be between 2 and 8 and at least half the rows of elements should have an element near the wall to prevent high velocity jets from traversing the entire basin length. Alternate rows are staggered.

Riprap may be needed for a short distance downstream of the dissipator. Chapter II contains a section on "Riprap Protection" and figure II-C-1 may be used to size the required riprap.

DESIGN PROCEDURE

1. Compute

   a. V_o

   b. y_o

   c. Fr

2. Select a basin from figure VII-A-5 that fits site geometry. Choose W_B/W_o, number of rows, N_r, N, L/y_A and L/h.

3. Determine:

   a. V_A

   b. y_A

   Use figures IV-A-2, 3, 4, and 5 for $4<W_B/W_o< 8$

For $W_B/W_O<4$ use figures IV-A-2 or 3 for V_A and compute y_A by equation VII-A-3.

For slopes > 10 percent use equation VII-A-3 to find both V_A and y_A

4. Determine dissipator parameters

   a. h--element height

   b. L--length between rows

   c. W_B--basin width

   d. W_1--element width=element spacing

   e. u_e--divergence

   f. C_B--basin drag

   g. $A_F=W_1h$--element frontal area

   h. C_p--from figure VII-A-4

   i. $L_B=2(W_O)+LN_r$

5. a. If the downstream channel width is approximately equal to W_B, compute from equation VII-A-1 or 2,

$$C_BA_fN$$

   Also compute C_BA_fN from values in step 4. If latter value is equal to or greater than value from VII-A-1 or 2, design is satisfactory. If less, return to step 2 and select new design.

   b. If channel width is greater than W_B, compute V_B from equation VII-A-1 or 2 and compare with downstream flow to determine controlling V_B. Compute y_B and compare with TW. If TW>y_B TW controlls.

6. Sketch the basin:

   a. Elements are symmetrical about centerline

   b. Lateral spacing approximately equal to element width

   c. W_1/h ratio between 2 and 8

   d. Minimum of half of the rows with elements near walls

   e. Stagger rows

7. Determine riprap protection requirement downstream of basin. Chapter II provides guidance and figure II-C-1 design information.

Example Problem

Given: 8x8' Box culvert: length = 235', slope = 0.02, Q Design = 1400 cfs. Assumed n = 0.013, computed critical depth, y_c = 9.8' normal depth, y_n = 6.0'.

Natural channel: width = 41', Q = 1400 cfs. slopes and cross sections vary but all slopes are subcritical for the channel discharge so channel water surface profiles must be computed from downstream controls. In this case normal depth several hundred feet downstream of the basin could be assumed. Using the standard step method, a backwater profile was plotted and the tailwater depth determined as TW = 3.3 feet.

Find: Design a CSU basin to provide a transition from the 8-foot wide culvert to the 41-foot wide natural channel and reduce velocities to approximately the downstream level.

1. Culvert outlet flow conditions

   a. $Y_o=Y_n=6.0$ feet Reference III-1.

   b. $V_o=V_n=29.1$ fps

   c. $Fr=V_o/\sqrt{gy_o}=29.1/\sqrt{32.2(6.0)}=2.1$

2. Select basin configuration figure VII-A-5.

   Channel Width/Culvert Width=41/8=5.14

   Try expansion ratio, $W_B/W_o=5$,
   $W_1/W_o=0.63$, $N_r=4$, N=15
   $h/y_A=0.71$
   $L/h=6$

3. Flow conditions at beginning of roughness field; $2W_o$ or 2x8=16 feet from culvert exit.

   a. $V_A/V_o=1.05$ from Figure IV-A-2

b. $y_A/y_o=0.33$ Figure IV-A-4
$V_A=29.1(1.05)=30.6$ fps.
$y_A=6.0(0.33)=1.98$ feet

4. Determine dissipator parameters,

a. $h/y_A=0.71$; $h=0.71(1.98)=1.4$ feet

b. $L/h=6$; $L=6(1.4)=8.4$ feet

c. $W_B/W_o=5$; $W_B=5(8)=40$ feet

d. $W_1/W_o=0.63$; $W_1=0.63(8)=5.04$ feet; use 5 feet

e. $u_e=4/7+10L/7W_o=4/7+10(8.4)/(8)7=2.07$ use 2

f. $C_B=0.42$

g. $A_f=(5)(1.4)=7$ sq. ft.

h. $C_p=0.7$

i. $L_B=2(8)+4(8.4)=49.6$ feet

5. Since channel and dissipator are approximately equal in width, 41 versus 40 feet, the C_BA_fN value will be computed directly from Equation VII-A-1.

y_n Downstream=3.3 feet

$V_B=1400/40(3.3)=1400/136=10.6$ fps

$\rho V_oQ+C_1\gamma y_o{}^2W_o/2=C_BA_fN\rho V_A{}^2/2+\rho V_BQ+\gamma Q^2/2V_B{}^2W_B$

$\rho=\gamma/g=62.4/32.2=1.94$

Terms with V_o and y_o:
$(1.94)(29.1)(1400)+0.7(62.4)(6)^28/2=85,325.5$

Terms with V_B:
$(1.94)(10.6)(1400)+62.4(1400)^2/2(10.6)^240=42,395.9$

$C_BA_fN(1.94)(30.6)^2/2=908.3C_BA_fN$
$85,325.5-42,395.9=908.3C_BA_fN$
$C_BA_fN=47.3$

From step 4

$C_B=0.42$
$N=15$
$A_f=7$

so $C_BNA_f=44.1<47.3$
try 5-rows same h/y_A and return to step 4.

4.a. $h=1.4$ feet

b. $L=8.4$ feet

c. $W_B=40$ feet

d. $W_1=5$ feet

e. $u_e=2$

f. $C_B=0.38$

g. $A_f=7$ sq. feet

h. $C_p=0.7$

i. $L_B=58$ feet

$C_BA_fN=7(19)(0.38)=50.5>47.7$ o.k.

6. Sketch basin and distribute roughness elements.

$W_1/h=5/1.4=3.57$ between 2 and 8 o.k.

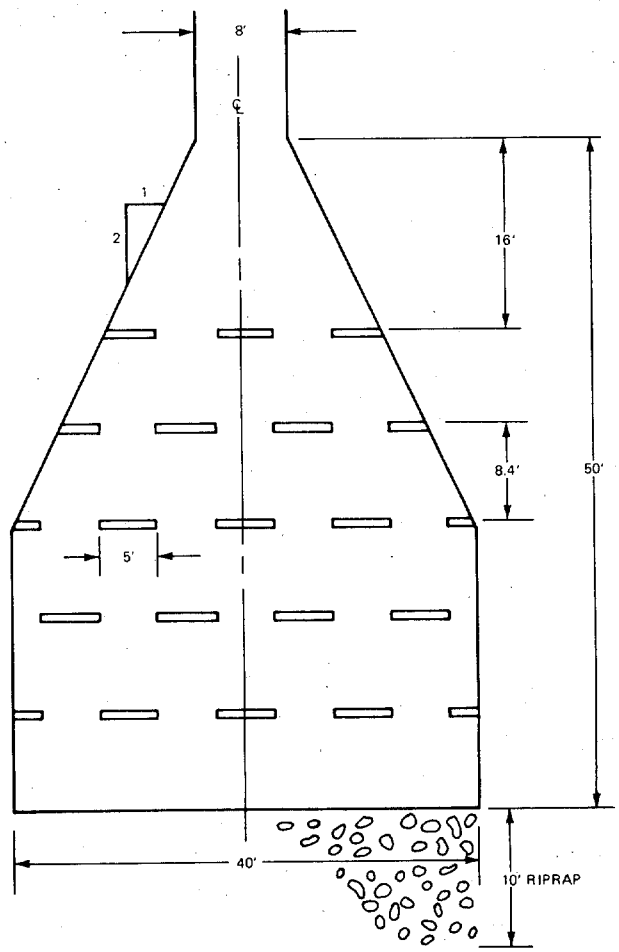
7. Since the design matches the downstream conditions, minimum riprap will be required. From figure II-C-1, place stone with 0.7 foot mean diameter in a 1.5 foot layer for 10 feet downstream of dissipator exit. Design required filter from reference III-4.

VII-1. Horner, S. F., FLUID DYNAMIC DRAG, published by author, 2 King Lane, Greenbriar, Bricktown, N.J. 08723, 1965.

VII-A-1. Simons, D. B., Stevens, M. A., Watts, F. J. , FLOOD PROTECTION AT CULVERT OUTLETS, Colorado State University, Fort Collins, Colorado, CER69-70-DBS-MAS-FJW4, 1970.

VII-A-2. Morris, H. M., HYDRAULICS OF ENERGY DISSIPATION IN STEEP ROUGH CHANNELS, VPI Bulletin 19, VPI and SU, Blacksburg, Virginia, November 1968.

VII-A-3. Jones, J. S., FHWA IN-HOUSE RESEARCH



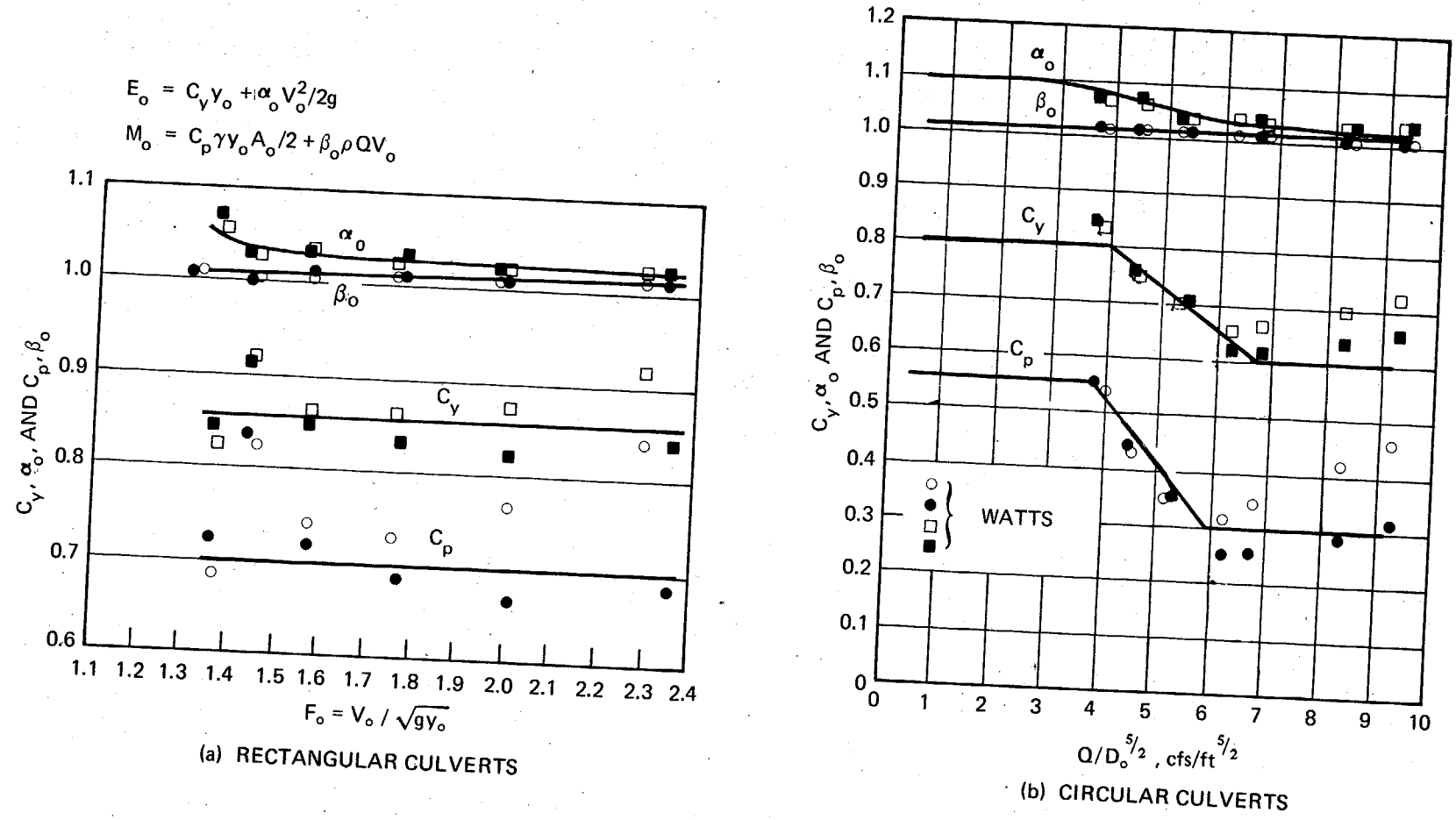
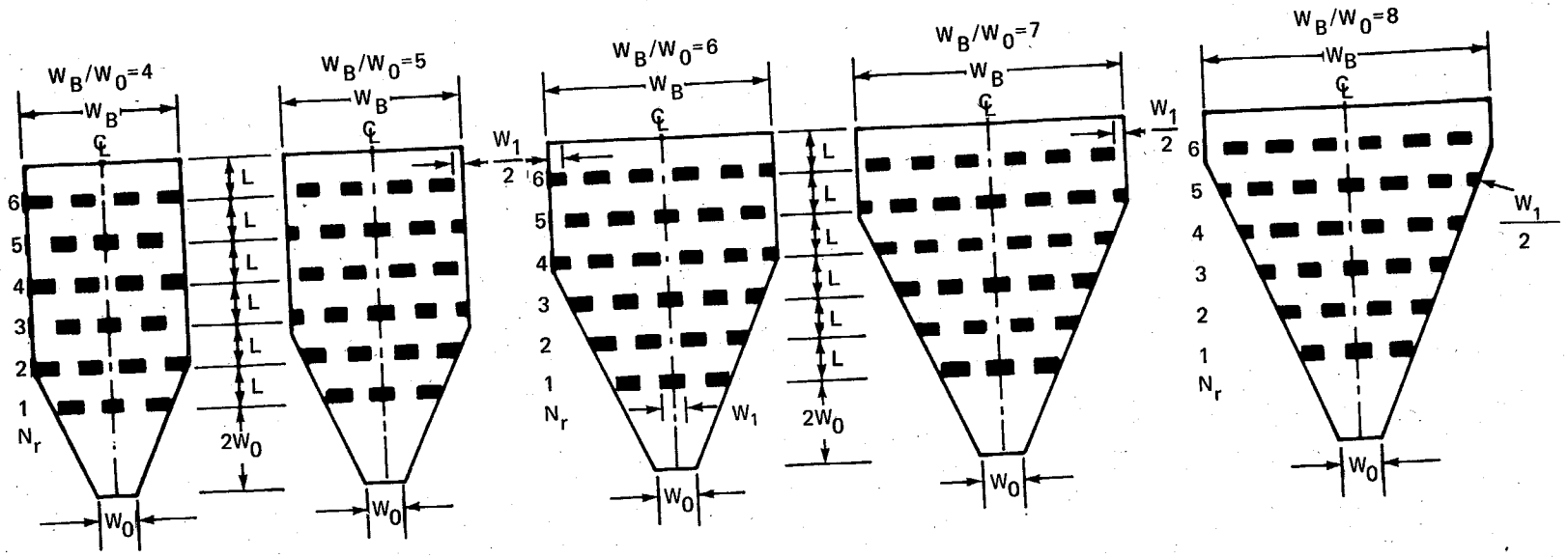


Figure VII - A - 4 ENERGY AND MOMENTUM COEFFICIENTS (FROM WATTS AND SIMONS REFERENCE VII-A-1)



	W_B/W_0		2 TO 4			5			6			7		8
	W_1/W_0		.57			.63			.6			.58		.62
	NO. ROWS (N_r)		4	5	6	4	5	6	4	5	6	5	6	6
	NO. ELEMENTS (N)		14	17	21	15	19	23	17	22	27	24	30	30
	h/Y_A	L/h	C_B FOR ROUGHNESS ELEMENT DISSIPATORS											
RECTANGULAR	0.91	6	0.32	0.28	0.24	0.32	0.28	0.24	0.31	0.27	0.23	0.26	0.22	0.22
	0.71	6	0.44	0.40	0.37	0.42	0.38	0.35	0.40	0.36	0.33	0.34	0.31	0.29
	0.48	12	0.60	0.55	0.51	0.56	0.51	0.47	0.53	0.48	0.43	0.46	0.39	0.35
	0.37	12	0.68	0.66	0.65	0.65	0.62	0.60	0.62	0.58	0.55	0.54	0.50	0.45
CIRCULAR	0.91	6	0.21	0.20	0.48	0.21	0.19	0.17	0.21	0.19	0.17	0.18	0.16	
	0.71	6	0.29	0.27	0.40	0.27	0.25	0.23	0.25	0.23	0.22	0.22	0.20	
	0.31	6	0.38	0.36	0.34	0.36	0.34	0.32	0.34	0.32	0.30	0.30	0.28	
	0.48	12	0.45	0.42	0.25	0.40	0.38	0.36	0.36	0.34	0.32	0.30	0.28	
	0.37	12	0.52	0.50	0.18	0.48	0.46	0.44	0.44	0.42	0.40	0.38	0.36	

FIGURE VII-A-5 DESIGN VALUES FOR ROUGHNESS ELEMENT DISSIPATORS

VII-B. LARGE ROUGHNESS ELEMENTS ON STEEP SLOPES

In situations where there is limited right-of-way for an energy dissipator at the culvert outlet and where the culvert barrel is not used to capacity due to inlet control, roughness elements are somtimes a convenient way of controlling outlet velocities. Roughness elements placed in the culvert barrel may be used to decrease velocities by creating a series of hydraulic jumps in a phenomenon known as tumbling flow (VII-B-1).

TUMBLING FLOW IN BOX CULVERTS AND OPEN CHUTES

The tumbling flow phenomenon was investigated as a means of dissipating energy in highway culverts and embankment chutes at Virginia Polytechnic Institute (VPI), (VII-B-2, 3, 4, and 5). Slopes up to 20 percent were tested at VPI and up to 35 percent in subsequent tests by the Federal Highway Administration (VII-B-6).

Drainage chutes on highway cut and fill slopes are candidate sites for roughness element energy dissipators. Use of roughness elements is reasonable for slopes up to 10 or 15 percent. Beyond this, flow separation and the trajectory of the flow which is out of contact with the channel bed are so exaggerated that provisons must be incorporated to counter splashing.

Tumbling flow is an optimum dissipator on steep slopes. It is essentially a series of hydraulic jumps and overfalls that maintain the predominate flow paths at approximately critical velocity even on slopes that would otherwise be characterized by high supercritical velocities.

One of the major limitations of tumbling flow as an energy dissipator is that the required height of the roughness elements is closely related to the unit discharge (discharge per unit width of channel). Conversely, the required element height is relatively insensitive to the culvert slope. For a given slope and culvert width, doubling the discharge increases the required height of roughness elements by approximately 50 percent; whereas, for a given discharge, increasing the slope from 2 percent to 4 percent increases the required element height by less than 3 percent. There will be many situations where the element height may have to be half the culvert height to maintain tumbling flow. Practical applications of tumbing flow are likely to be limited to low-discharge per unit width, high-velocity culverts.

Tumbling flow is uniform flow in a cyclical sense, with the same patterns of depth and velocity repeated at each roughness element. It is not necessary to line the entire length of the culvert with roughness elements to get outlet velocity control. Four or five rows of roughness elements are sufficient to establish the cyclical uniform flow pattern.

Tumbling flow can be established rather quickly by using either a very large leading element, or a smaller leading element and a splash shield to reverse the flow jet between the first and second rows. The first alternative is not considered to be a practical solution since the element size is likely to be excessive.

The splash shield has merit since it deflects the so-called "rooster tail" jet against the channel bed and brings the flow under control very quickly without using a large leading roughness element. For open chutes a splash plate such as the one sketched in figure VII-B-1 must be used, but for culverts the top of the culvert can serve as the shield. In either case, there should be a top baffle to help redirect the flow. The top baffle need not be the same size as the bed elements. It is not unreasonable to expect to provide additional culvert height in the roughened region of culverts.

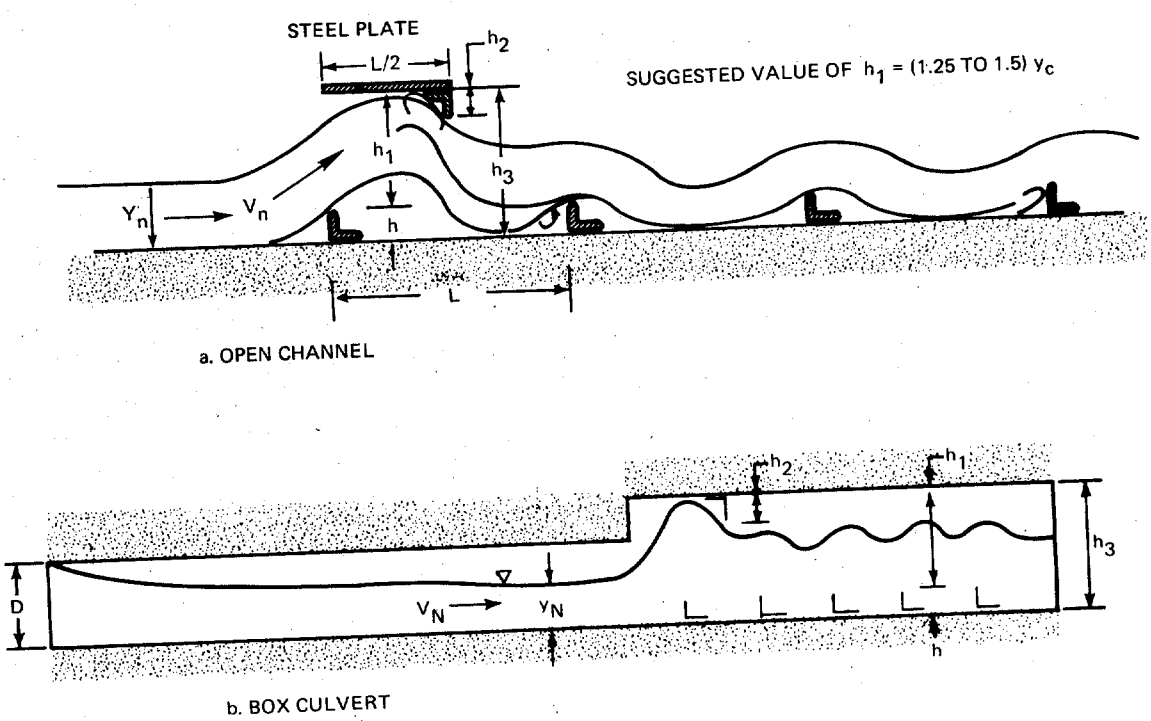


FIGURE VII-B-1. SPLASH SHIELD DEFINITIVE SKETCH FOR TUMBLING FLOW

The basic premise of the tumbling flow regime is that it will maintain essentially critical flow even on very steep slopes. The last element is located a distance L/2 upstream of the outlet so the flow reattaches to the channel bed right at the outlet. Outlet velocity will approach critical velocity, unless backwater exists.

DESIGN PROCEDURE

1. Check culvert control. If inlet control governs, tumbling flow may be a good choice for dissipating energy.

2. Compute the initial condition:

   a. The discharge intensity--discharge per unit width $Q/W=q$

   b. Critical velocity V_c and depth y_c--see chapter III

   c. Normal depth y_n and velocity V_n--table III-1 or 2

3. Select type of roughness configuration

   a. The recommended configuration is to use 5 rows of elements all the same height (h).

$$h = y_c/(3-3.7S_o)^{2/3} \quad \text{VII-B-1}$$

   b. The alternate method is to use a large initial roughness element followed by four smaller elements. In this case it is necessary to compute the sequent depth (y_2) required for the hydrualic jump:

$$y_2=(Fr)y_n(S_o+0.153)/0.133 \quad \text{VII-B-2}$$

   where y_n and Fr are the approach conditions at the toe of the jump.

   In order for tumbling flow to occur the large initial element height should be:

$$h_i>y_2-y_c \quad \text{VII-B-3}$$

   This element is followed by four smaller elements with a height computed by equation VII-B-1.

4. Compute the longitudinal spacing:

a. for the small elements, an L/h ratio of either 8.5 or 10 is selected and L computed

b. for the alternate design where a large leading roughness element is used, the spacing between this element and the first small element L is

$$L_1 = 2h + (q^{2/3}/10.1)(\cos\psi/\cos\theta)\{\tan\psi\cos\theta + \sin\theta + [(\tan\psi\cos\theta + \sin\theta)^2 + (20.3/q^{2/3})(\cos\theta/\cos\psi)^2 h_1]^{1/2}\}$$

where $\psi = \phi - \theta$ and $\phi = 45°$ is used.

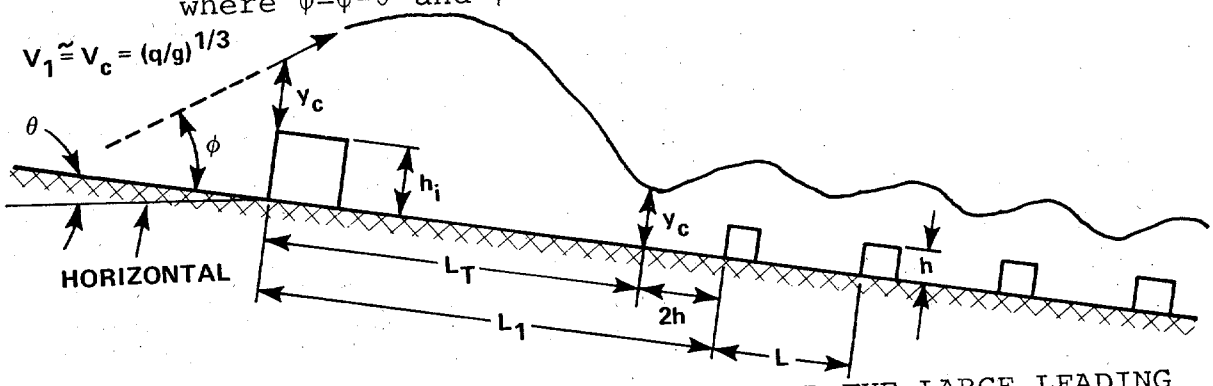


FIGURE VII-B-2. TRAJECTORY FOR FLOW OVER THE LARGE LEADING ELEMENT

5. Slots may be provided in the roughness elements as shown in figure VII-B-3 to pass fine sediment and to allow for drainage during low flow. The slot width (W_2) should be:

$$W_2 = 0.5h \quad \text{. . . . . .VII-B-4}$$

where h is the height of the small elements.

The slot configuration shown in figure VII-B-2 is recommended.

$$W_1 = (W - N_s W_2)/3 \quad \text{. . . . . .VII-B-5}$$

where N_s is the number of slots.

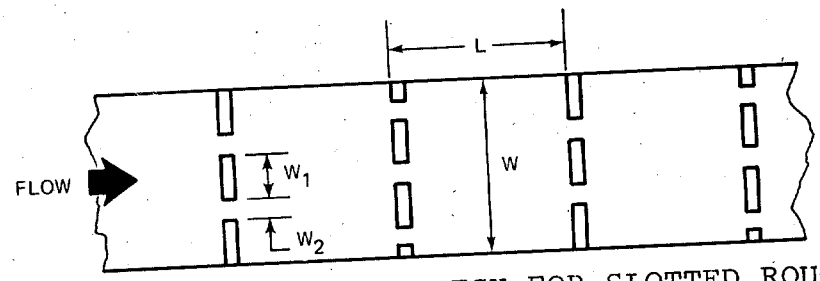


FIGURE VII-B-3. DEFINITION SKETCH FOR SLOTTED ROUGHNESS ELEMENTS

6. When the recommended roughness configuration is used, step 3A, a splash shield may be desirable.

   a. For open channels use the splash shield as indicated on figure VII-B-1 with the splash guard height (h_2) equal to 2 inches or as dictated by structural requirements. The length, position and height are shown on figure VII-B-1.

   b. For culverts, the jet should just clear the culvert top. The jet height (h_1) is:

$$h_1=1.25y_c \quad \text{. . . . . .VII-B-6}$$

   If $D<(h_1+h)$ an enlarged culvert height (h_3) equal to h_1+h is required.

   If $D>(h_1+h)$ an element, to redirect the flow, with height (h_2) should be located downstream as shown on figure VII-B-1b.

$$h_2=1.5(D-h_1-h) \quad \text{. . . . . .VII-B-7}$$

   No splash shield is necessary where the large initial roughness element design is used.

7. The outlet velocity can be computed by:

$$V_c=(qg)^{1/3} \quad \text{. . . . . .VII-B-8}$$

DESIGN PROCEDURE SUMMARY

1. Check culvert control

2. Compute: $q=Q/W$, V_c, y_c, y_n, and V_n

3. Compute h using equation VII-B-1
   Compute y_2 for alternate design and $h_i>y_2-y_c$

4. Select L/h = 8.5 or 10 and compute L
   Compute L_i if alternate design is used

5. Calculate W_1 and W_2 from Equations VII-B-4 and 5 and arrange elements as indicated in figure VII-B-3

6. Splash shields

   a. For open channels h_2 = 2 inches minimum, $h_1 = 1.25y_c$ and $h_3 = h+h_1$. The splash shield length equals L/2.

b. For culverts check jet clearance. Find h_1 from equation VII-B-6. If $D<h_1+h$, make $h_3=h_1+h$. If $D>h_1+h$ use equation VII-B-7 to find h_2 required.

7. Calculate V_c using equation VII-B-8

EXAMPLE PROBLEM

Design a tumbling flow energy dissipator for:

2 foot wide concrete channel
Manning's n = 0.015
Q = 20 cfs
S_o = 10%

1. Inlet control check not applicable for open channel flow.

2. $q=Q/W=20/2=10$ cfs/ft
$y_c=(q^2/g)^{1/3}=(10^2/32.2)^{1/3}=1.46$ feet
$V_c=6.85$ FPS
$y_n=.61$ feet
$V_n=16.4$ fps

3. Use 5 rows of elements all the same height.

$h=y_c/(3.-3.7S_o)^{2/3}$
$=1.46/1.91=0.76$ use .75 feet

4. Spacing L, using $L/h=8.5$
$L=8.5(0.75)=6.38$ feet, use 6.5 feet

5. Slots W_2
$W_2=0.5(h)=0.5(0.75)=0.38$, use 0.4 feet

Segment width W_1
1st, 3rd, and 5th rows $W_1=[2-2(0.38)]/3=0.4$ feet
2nd, and 4 rows $W_1=[2-3(0.38)]/3=0.29$ feet, use 3 feet

6. $h_2=2$ inches
$h_1=1.25y_c=1.25(1.46)=1.83$ feet
$h_3=h+h_1=0.75+1.83=2.58$ feet

Space shield length=L/2=6.38/2=3.19 ft.

7. V_c at end of dissipator
$V_c=(qg)^{1/3}=(10x32.2)^{1/3}=6.85$ fps

TUMBLING FLOW IN CIRCULAR CULVERTS

Tumbling flow in circular culverts can be attained by inserting circular rings inside the barrel, figure VII-B-4. Geometrical considerations are more complex, but the phenomenon of tumbling flow is the same as for box culverts.

In the previous section, primarily bottom roughness elements were considered, whereas in circular culverts the elements are complete rings. The culvert is treated as an open channel which greatly simplifies the discussion, and the diameter is varied to obtain vertical clearance for free surface flow.

Design procedures have been described by Wiggert and Erfle. (VII-B-7,8) Their experiments for tumbling flow in circular culverts were run with a 6-inch plexiglass model and an 18-inch concrete prototype culvert. Slopes ranged from 0 to 25 percent, h/D_1 ranged from 0.06 to .15 and L/D_1 ranged from 0.3 to 3.0 (L/h from 5 to 20). The experimental variables are illustrated in figure VII-B-4.

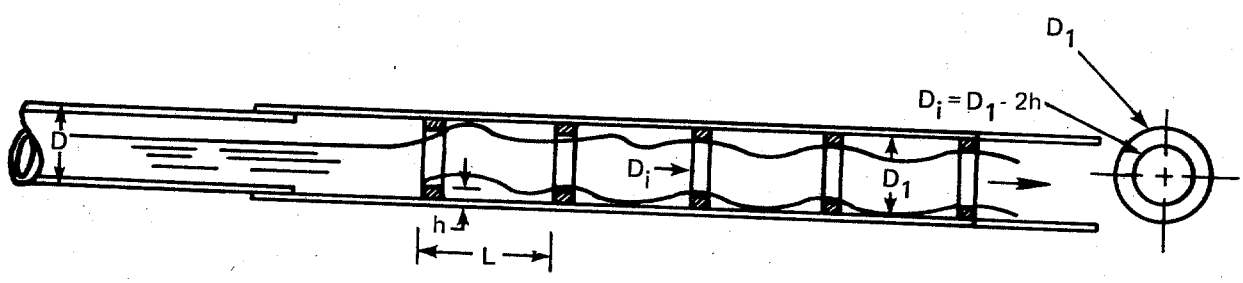


FIGURE VII-B-4. DEFINITION SKETCH FOR TUMBLING FLOW IN CIRCULAR CULVERTS

The variables that determine whether or not tumbling flow will occur are roughness height, h, spacing, L, slope, S_o, discharge, Q, and the diameter, D_1. A functional relationship for the roughness height can be described as

$$h=f(L,S,Q,D_1,g) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{VII-B-9}$$

Establishing dimensionless groupings yields

$$h/D_1=f(L/D_1,S,Q/(g\ D_1^5)^{1/2} \quad \ldots\ldots\ldots\ldots \text{VII-B-10}$$

Practical design limits can be assigned to h/D_1 and L/D_1 to further simplify the functional relationship. Based on qualitative laboratory observations, tumbling flow is easiest to maintain when L/D_1 is between 1.5 and 2.5 and when h/D_1 is between 0.10 and 0.15. Assigning these limits for circular culverts is analogous to assigning values for L/h in the design procedure for box culverts. The functional relationship in Equation VII-B-10 can be rewritten

$$\text{Constant} = f(S,\ Q/g\ D_1^{\ 5})^{1/2}$$

or

$$Q/(g\ D_1^{\ 5})^{1/2} = f(S) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{VII-B-11}$$

Theoretically f(S) in Equation VII-B-11 could be any function involving the slope term. Empirically f(S) was found to be approximately a constant. The slight observed dependence of f(S) on slope is considered to be much less significant than the inaccuracies associated with measuring flow characteristics over the large roughness elements. Based on model and prototype data, f(S) can be defined by

$$0.21 < f(S) < 0.32,$$

if the slope is between 4 percent and 25 percent. For slopes less than 4 percent, the culvert should be designed for full flow rather than tumbling flow. See section VII-B "Roughness Elements for Increased Flow Resistance." Equation VII-B-11 can be rewritten to yield

$$0.21 < Q/(g\ D_1^{\ 5})^{1/2} < 0.32 \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{VII-B-12}$$

and

$$1.6\,(Q^2/g)^{1/5} < D_1 < 1.9\,(Q^2/g)^{1/5}$$

Equation VII-B-12 is the basic design equation for tumbling flow in steep circular culverts. If the <u>diameter of the roughened section</u> of the culvert is sized according to this equation, tumbling flow will occur and the outlet velocity will be approximately critical velocity. Equation VII-B-12 is limited to the following conditions:

$L/D_1 \simeq 2.0$ (tolerance±25%)
$h/D_1 \simeq 0.125$ (Tolerance=±20%), and
slope greater than 4% and less than 25%

Since tumbling flow is an open channel phenomenon, gravity forces prevail and the Froude number, $V/(gy)^{1/2}$, should be used as the basis for design (or interpretation of model results.) Watts (VII-B-9) established, by reference to several publications, that h/y is an important scaling parameter for roughness elements in open channel flow. In both of these dimensionless terms, y is a characteristic flow depth. The validity of using D in lieu of a characteristic flow depth in $Q/(gD_1^5)^{1/2}$ must be carefully examined for culverts flowing less than full. The characteristic depth for tumbling flow, however, is critical depth which is uniquely defined by Q and D_1; so D_1 can be substituted for y in this special case of partially full culverts.

Furthermore, the higher coefficient in Equation VII-B-12 resulted from the 6" model data rather than from the 18" prototype. Differences in model and prototype data were attributed to experimental difficulties with the prototype; nevertheless, if there are scaling errors, they appear to be on the conservative side.

A major concern is that silt may accumulate in front of the roughness elements and render them ineffective. This is perhaps unwarranted as the element enhances sediment transport capacity and tend to be self-cleansing. In their original list of possible applications, Peterson and Mohanty (VII-B-1) noted that by "using roughness elements to induce greater turbulence, the sediment-carrying capacity of a channel may be increased."

Water trapped between elements may cause difficulties during dry periods due to freezing and thawing and insect breeding. Narrow slots in the roughness rings (less than 0.5h) can be used to allow complete drainage without changing the design criteria.

Five roughness rings at the outlet end of the culvert are sufficient to establish tumbling flow. The diameter computed from equation VII-B-12 is for the roughened section only, and will not necessarily be the same as the rest of the culvert. The American Concrete Pipe Association (VII-B-7) introduced the telescoping concept in which the main section of the culvert is governed by the usual design parameters (presumably inlet control) and the roughened section is designed by Equation VII-B-12. They suggest telescoping the larger diameter pipe over the smaller "for at least the length of a normal joint and using normal sealing materials in the annular space.

Velocity Prediction at the Culvert Outlet

The outlet velocity for tumbling flow is approximately critical velocity. It can be computed by determining the critical depth, d_c, for the inside diameter of the roughness rings. Critical flow for an open channel of any shape will occur when

$$Q^2T/gA_c^3=1 \quad \text{VII-B-13}$$

Referring to figure VII-B-5, the following additional relationships can be written:

For $y_c>r$:

$y_1=y_c-r$

$\beta=2 \text{ Arc cos } (y_1/r)$

$A_c=\pi r^2(1-\beta/360)+y_1(r^2-y_1^2)^{1/2}$

$T=2(r^2-y_1^2)^{1/2}$

For $y_c<r$:

$y_1=r-y_c$

$\beta=\text{same}$

$A_c=\pi r^2\beta/360-y_1(r^2-y_1^2)^{1/2}$

$T=2(r^2-y_1^2)^{1/2}$

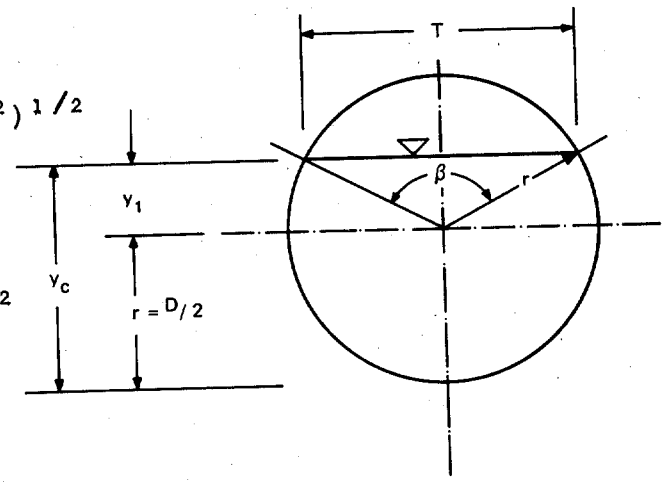


FIGURE VII-B-5. DEFINITION SKETCH FOR CRITICAL FLOW IN CIRCULAR PIPES

Figure III-4 can be used to determine critical depth using D_i.

Table III-2 can be used to determine the critical area, A_c. Outlet velocity can be computed from

$$V_c=Q/A_c$$

Design Procedure:

1. Check culvert control. If inlet control governs tumbling flow may be a good choice for dissipating energy.

2. Determine the Diameter, D_1, of the roughened section of pipe to sustain tumbling flow.
Use Equation VII-B-12,

$$1.6(Q^2/g)^{1/5} < D_1 < 1.9(Q^2/g)^{1/5}$$

3. Compute h and L from:
$h/D_1 = 0.125 \pm 20\%$
$L/D_1 = 2.0 \pm 25\%$

4. Compute the internal diameter of the roughness rings
$D_i = D_1 - 2h$

5. Determine the critical depth, y_c, from figure III-4, using the design discharge for Q and D_1 for diameter.

6. Compute y_c/D_i

7. Determine A_c from table III-2 use y_c/D_i for d/D and read A/D^2 which equals A_c/D_i^2
$A_c = (A_c/D_i^2)D_i^2$

8. Compute the outlet velocity
$V_o \simeq V_c = Q/A_c$

<u>Example Problem:</u>

Given: 48 inch diameter culvert, 200 feet long,
n=0.012, 6% slope
Q-design=80 c.f.s.
V_o=24 ft/sec
The culvert is governed by inlet control.

Req'd: Determine size and spacing of roughness elements for tumbling flow.

Solution: 1. Inlet control governs.

2. Equation VII-B-12

$$1.6\ (Q^2/g)^{1/5} < D_1 < 1.9(Q^2/g)^{1/5}$$
$$4.6' < D_1 < 5.5'$$
Use $D_1 = 5'$

3. Compute h and L
$h/D_1 = 0.125 \pm 20\%$
$h = 0.125(5) = 0.625'$
$.5 < h < .75$ Use h=.58 ft. or 7 inches

$L/D_1=2+25\%$
$L=2.0(5)=10'$
$7.5<L<12.5$ Use $L=10$

4. Compute D_i
   $D_i=D_1-2h=5-1.2=3.8'$

5. Determine y_c from figure III-4
   $y_c=2.8'$

6. Compute y_c/D_i
   $y_c/D_i=0.737=d/D$ for table III-2

7. Determine A_c from table III-2
   $A/D^2=A_c/D_i^2=.623$
   $A_c=.623(3.8)^2=9.07$ ft.^{2}

8. Compute the outlet velocity
   $V_c=Q/A_c=80/9.07=8.8$ ft/sec
   This is a reduction from V=24 fps in the original culvert of:
   $100(24-8.8)/24=63\%$

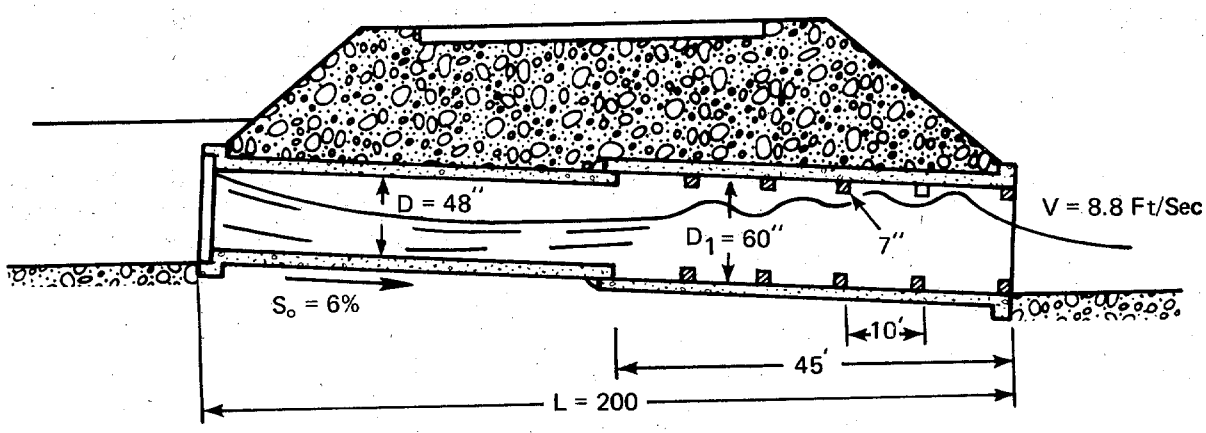


FIGURE VII-B-6. SKETCHED SOLUTION FOR THE TUMBLING FLOW DESIGN EXAMPLE FOR CIRCULAR CULVERTS

VII-B-1. Peterson, D. F. & P. K. Mohanty, FLUME STUDIES OF FLOW IN STEEP ROUGH CHANNELS, ASCE Hydraulics Journal, HY-9, Nov. 1960.

VII-B-2. Morris, H. M., HYDRAULICS OF ENERGY DISSIPATION IN STEEP ROUGH CHANNELS, Virginia Polytechnical Institute Bulletin 19, VPI & SU, Blacksburg, VA., Nov. 1968.

VII-B-3. Morris, H. M., DESIGN OF ROUGHNESS ELEMENTS FOR ENERGY DISSIPATION IN HIGHWAY DRAINAGE CHUTES, H.R.R. #261, pp. 25-37, TRB, Washington, D.C., 1969.

VII-B-4. Mohanty, P. K., THE DYNAMICS OF TURBULENT FLOW IN STEEP ROUGH, OPEN CHANNELS, Ph.D. Dissertation, Utah State University, Logan, Utah, 1959.

VII-B-5. Morris, Bulletin 19, pp. 50, 54 & 56.

VII-B-6. Jones. J. S., FHWA IN-HOUSE RESEARCH

VII-B-7. Wiggert, J. M. and P. D. Erfle, ROUGHNESS ELEMENTS AS ENERGY DISSIPATORS OF FREE SURFACE FLOW IN CIRCULAR PIPES, HPR #373, pp. 64-73, TRB, Washington, D.C., 1971.

VII-B-8. CULVERT VELOCITY REDUCTION BY INTERNAL ENERGY DISSIPATORS, Concrete Pipe News, pp. 87-94, American Concrete Pipe Association, Arlington, VA, Oct. 1972.

VII-B-9. Watts, F. J., HYDRAULICS OF RIGID BOUNDARY BASINS, p. 157, Ph.D. Dissertation, Colorado State University, Fort Collins, Colorado, August 1968.

VII-C. ROUGHNESS ELEMENTS TO INCREASE CULVERT RESISTANCE NEAR THE OUTLET

INCREASED RESISTANCE IN CIRCULAR CULVERT

The methodology described in this section involves using roughness elements to increase resistance and induce velocity reductions. Increasing resistance may cause a culvert to change from partial flow to full flow in the roughened zone. Velocity reduction is accomplished by increasing the wetted surfaces as well as by increasing drag and turbulence by the use of roughness elements.

Tumbling flow, as described in VII-B, is the limiting design condition for roughness elements on steep slopes. Tumbling flow essentially delivers the outlet flow at critical velocity. If the requirement is for outlet velocities between critical and the normal culvert velocity, designing increased resistance into the barrel is a viable alternative.

The most obvious situation for application of increased barrel resistance is a culvert flowing partially full with inlet control. The objective is to force full flow near the culvert outlet without creating additional headwater.

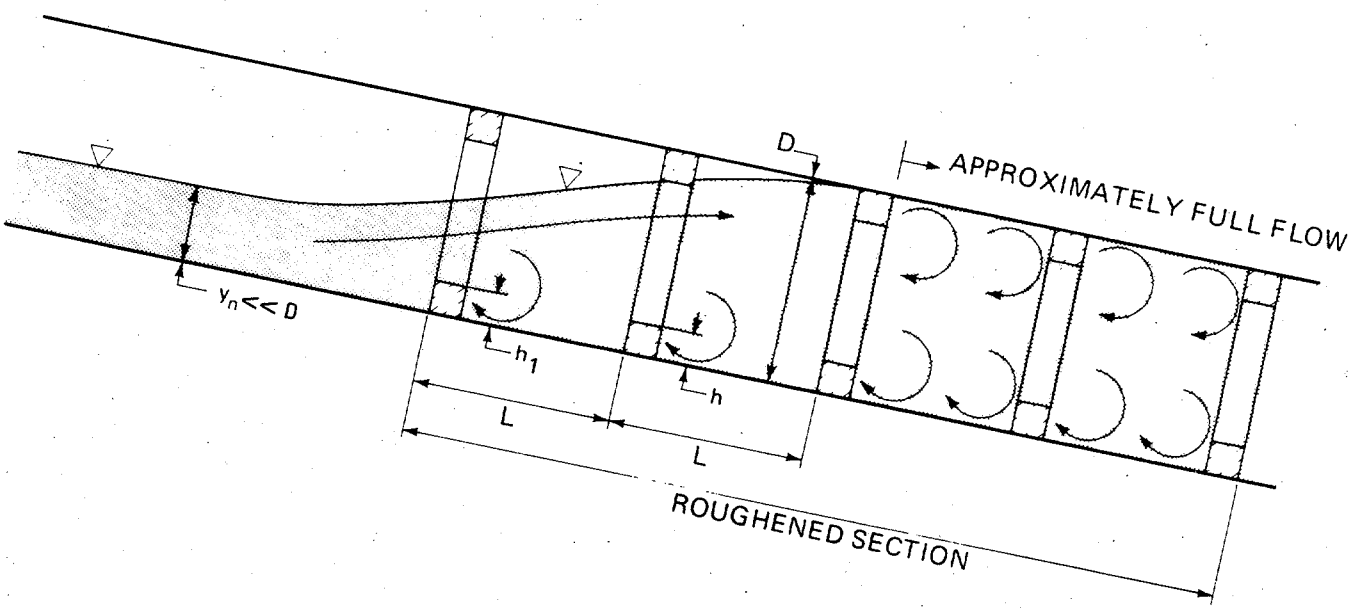


FIGURE VII-C-1. CONCEPTUAL SKETCH OF ROUGHNESS ELEMENTS TO INCREASE RESISTANCE

Based on experience with large elements used to force tumbling flow, five rows of roughness elements with heights ranging from 5 to 10 percent of the culvert diameter are sufficient.

Much of the literature relative to large roughness elements in circular pipes expresses resistance in terms of the friction factor, "f." Although there is some merit in using the friction factor, all resistance equations are converted to Manning's "n" expressions for this manual.

The Manning equation for a circular culvert flowing full is

$$Q=AV=(\pi D^2/4)(1.49/n)(D/4)^{2/3}S_f^{1/2}$$
$$=(0.46/n)D^{8/3}S_f^{1/2} \quad \text{. . . . . .VII-C-1}$$

Assuming normal flow near the outlet and inlet control allows substitution of the bottom slope, S_o, for the friction slope, S_f. If the culvert flows less than full, it is usually expedient to compute full flow and to use a hydraulic elements graph, figure VII-C-3, to compute partial flow parameters. Designing roughness elements is basically a matter of manipulating equation VII-C-1 and figure VII-C-3 in conjunction with empirical graphs for determining "n" in roughened pipes.

Wiggert and Erfle (VII-B-7) studied the effectiveness of roughness rings as energy dissipators in circular culverts. Although their study was primarily a tumbling flow study, they observed in many tests that they could get velocity reductions greater than 50 percent without reaching the roughness level necessary for tumbling flow. They did not derive resistance equations, but they did established approximate design limits.

Best performance was observed when h/D was .06 to .09. Doubling the height, h_1, of the first ring was effective in triggering full flow in the roughened zone. Adequate performance was obtained with four rings but with double spacing between the first two. However, the same pipe length is involved if a constant spacing is maintained and five rings used, with the first double the height of the other four. The additional ring should help establish the assumed full flow condition.

Subsequent experience reported by the American Concrete Pipe Association (VII-B-8) indicated a need to consider lower values of h/D, and to establish approximate resistance

curves for evaluating a design in order to avoid installations that will propagate full flow upstream to the culvert inlet.

Morris' (VII-C-4) studied all pertinent rough pipe flow data available and concluded that there are three flow regimes and each has a different resistance relationship. The three regimes are quasi-smooth flow, hyperturbulent flow and isolated roughness flow. Quasi-smooth flow occurs only when there are depressions or when roughness elements are spaced very close ($L/h \approx 2$). Quasi-smooth flow is not important for this discussion. Hyperturbulent flow occurs when roughness elements are sufficiently close so each element is in the wake of the previous element and rough-surface vortices are the primary source of the overall friction drag. Isolated roughness flow occurs when roughness spacing is large and overall resistance is due to drag on the culvert surface plus form drag on the roughness elements. The three regimes are illustrated in figure VII-C-2.

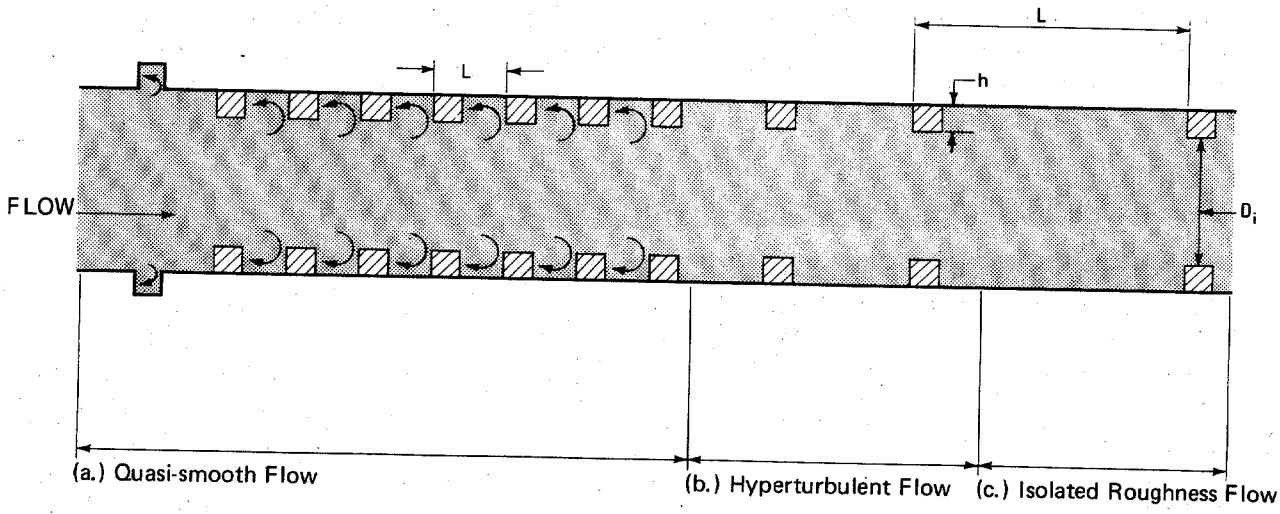


FIGURE VII-C-2. FLOW REGIMES IN ROUGH PIPES

Isolated-Roughness Flow

The overall friction or resistance, f_{IR}, is made up of two parts:

$$f_{IR}=f_s+f_d$$

where: f_s = friction on the culvert surface
f_d = friction due to form drag on the roughness elements.

The friction due to form drag is a function of the drag coefficient for the particular shape, the percentage of the wetted perimeter that is roughened, the roughness dimensions and spacings and the velocity impinging on the roughness elements. Morris related the velocity to surface drag and derived the following equation

$$f_{IR}= f_s[1+67.2C_D(L_r/P)(h/r_i)(r_i/L)] \quad \text{. . . .VII-C-3}$$

where: C_D = drag coefficient for the roughness shape
L_r/P = ratio of total peripheral length of roughness elements to total wetted perimeter
r_i = pipe radius based on the inside diameter of roughness rings measured from crest to crest.

Throughout Morris' work, he used measurements from crest to crest of a roughness element ring as the effective diameter, D_i. Equation VII-C-3 can be converted to a Manning's "n" expression as follows:

$$f_s=184.19(n/D^{1/6})^2$$
$$f_{IR}=184.19(n_{IR}/D_i^{1/6})^2$$

$$n_{IR}=n(D_i/D)^{1/6}[1+67.2\ C_D(L_r/P)(h/L)]^{1/2} \quad \text{. . .VII-C-4}$$

where: n_{IR} = overall Manning's "n" for isolated roughness flow.
n = Manning's "n" for the culvert suface without roughness rings
D = nominal diameter of the culvert
D_i = $D - 2h$ = inside diameter of roughness rings

For sharp edge rectangular roughness shapes, a constant value of 1.9 can be used for C_D.

Figure VII-C-4 is a graphical solution to Equation VII-C-4 for sharp edged rectangular roughness shapes and continuous rings. If gaps are left in the roughness rings, L_r/P is less than 1.0 and the equation rather than the figure must be

used to compute the resistance. It is noteworthy that the overall resistance, n_{IR}, decreases as the relative spacing, L/D_i, increases for this regime.

Hyperturbulent Flow

The friction in this regime is independent of friction on the culvert surface

$$1/\sqrt{f_{HT}}=2\ \log_{10}(r_i/L)+1.75+\phi \quad \ldots\ldots\ldots \text{VII-C-5}$$

where: f_{HT} = overall friction for hyperturbulent flow.
ϕ = function of Reynolds number, element shape, and relative spacing.

By restricting application of equation VII-C-5 to sharp-edged roughness rings and to spacings greater than the pipe radius, ϕ can be neglected.

Substituting:

$$D_i^{1/6}/n_{HT}=\sqrt{184.19/f_{HT}}$$

into Equation VII-C-5 and rearranging terms yields

$$n_{HT}=0.0736\ D_i^{1/6}/(1.75-2\ \log_{10}L/r_i) \quad \ldots \text{VII-C-6}$$

The effect of the roughness height, h, is included inherently in D_i. From figure VII-C-5, it can be seen that n_{HT} increases as the spacing increases for this regime.

Regime Boundaries

Since resistance increases when the spacing increases for the hyperturbulent regime and when the spacing decreases for the isolated roughness regime, the boundary between the regimes occurs when the resistance equations are the same. The boundary is determined by equating f_{IR} in equation VII-C-3 to f_{HT} in equation VII-C-5. All the boundary curves in figure VII-C-6 are based on sharp-edged rectangular roughness elements (C_D=1.90) and on full roughness rings (L_r/P=1.0). Smaller values of L_r/P increase the isolated roughness flow zone; so if isolated roughness flow occurs for L_r/P=1.0, it will occur for any value of L_r/P less than 1.0.

Design Procedure

1. Compute $n/D^{1/6}$, where "n" is Mannings coefficient for smooth culvert and "D" is the diameter.

2. Select L/D_i in the range 0.5 to 1.5. (1.0 is suggested as a starting point)

3. Select h/D_i in the range 0.05 to 0.10. Use sharp edged roughness rings.

4. Determine the flow regime from figure VII-C-6. The flow regime will be "isolated roughness" (I.R.) if the point defined by the L/D_i and h/D_i ratios is above the $n/D^{1/6}$ value. If the point is below, the flow is hyperturbulent" (H.T.). Isolated roughness flow is the most common for large culverts.

5. Determine the rough pipe resistance ($n_r=n_{IR}$ or n_{HT})

   (a) For isolated roughness flow obtain (n_{IR}/n) from figure VII-C-4 or from Equation VII-C-4
   $n_r=(n_{IR}/n)n$
   note: If gaps are to be left in the roughness rings so that L_r/P is much less than 1.0, Equation VII-C-4 must be used since figure VII-C-4 is based on $L_r/P=1.0$

   (b) For hyperturbulent flow obtain $n_r=n_{HT}$ from figure VII-C-5 directly or from Equation VII-C-6

6. Compute the crest to crest roughness ring diameter,

   $D_i=D-2h=D/(1+2h/D_i)$

7. Compute full flow characteristics based on D and n:

   $Q(FULL)=(0.46/n_r)D_i^{8/3}S_o^{1/2}$

   and $V(FULL)=(0.59/n_r)D_i^{2/3}S_o^{1/2}$

8. Determine outlet velocities:

   (a) If Q(FULL)=Design Q,

   V(OUTLET)=V(FULL)

   (b) If Q(FULL) is less than Design Q, the culvert is likely to flow full and result in increased head-water requirements. In this case, a complete

hydraulic analysis of the culvert is necessary to compute the outlet velocity which will be greater than V(FULL) from Step 7. To avoid this situation, use an oversize diameter, D_i, for the roughened section of the culvert and repeat steps 1 through 7 above.

(c) If Q(FULL) is greater than Design Q, use figure VII-C-2 to compute the velocity. Enter the figure with

$$Q/Q(FULL) = Q(DESIGN)/Q(FULL)$$

read

$$V/V(FULL)$$

and compute

$$V(OUTLET) = (V/V(FULL))\ V(FULL)$$

9. Evaluate acceptability of outlet velocity and repeat design steps if necessary.

Acceptable outlet velocity is a site determination that must be made by the designer. It is anticipated that one use of roughness rings may be to complement riprap protection.

If the outlet velocity is not acceptable, the recommended order of considerations is:

(a) If Q(FULL) is less than Design Q, increase h/D_i to approach full flow. A solution can usually be attained with one iteration by approximating the resistance from $n = 0.59 D_i^{2/3} S_o^{1/2}/(V(DESIRED)/1.15))$ and using an estimated value of D_i slightly greater than expected. With n_r known, selecting a corresponding h/D_i from figure VII-C-4 or figure VII-C-5 is relatively straightforward.

(b) If V(FULL) is still too high, increase D_i for the roughened section to make possible higher values of h/D_i and correspondingly higher values of n_r; i.e., use an oversized culvert with diameter, D_i, in the rough section and repeat steps 1 through 8 above.

(c) Use a tumbling flow design as described in section VII-B.

(d) Use another type of dissipator either in lieu of or in addition to the roughness rings.

10. Determine the size and spacing of the roughness rings.

(a) $D_i = D/(1+2(h/D_i))$

(b) $h = (h/D_i)D_i$

(c) $L = (L/D_i)D_i$

(d) $h_1 = 2h$
(height of first roughness ring; see figure VII-C-1)

(e) $D = D_i + 2h$
(for oversized sections of rough culverts)

(f) Use five roughness rings including the oversized first ring. If an oversized diameter is used provide an approach length of one diameter before the first ring.

Example Problem:

Given: Culvert flowing under inlet control

Diameter = 48 inches
Design Q = 100 cfs
n = 0.012
Slope = 4%
Length = 200'

Also,

Q(FULL) = 314 cfs
V(FULL) = 25 fps
Q(DESIGN)/Q(FULL) = 100/314 = 0.32
V(DESIGN)/V(FULL) = 0.88 (from figure VII-C-2)
V_o= 22 fps
y_o= 1.5 feet

Find the size and spacing of roughness element and the diameter of an enlarged end section (if required) to reduce the outlet velocity to 15 fps.

Solution

1. Compute $n/D^{1/6}$
$n/D^{1/6} = .012/4^{.1667} = 0.0095$

2. Select $L/D_i = 1.5$

3. Try $h/D_i=0.05$

4. From figure VII-C-6, the regime is isolated roughness flow. ($n/D^{1/6}$ is less than the point defined by the h/D_i and L/D_i ratios)

5. Determine rough pipe resistance from figure VII-C-4

$n_r/n=2.3$
$n_r=(2.3)0.012=0.0275$

6. Determine D_i:

$D_i=D/(1+2h/D_i)=4/1(1+0.10)=3.6'$

7. Full flow computations for the rough pipe:

$Q(FULL)=(0.46/.0275)(3.6)^{8/3}\sqrt{.04}=89.8$ cfs
$V(FULL)=8.83$ fps

8. Compare full flow with design flow

$Q(DESIGN)/Q(FULL)=100/102=0.98$
$V/V(FULL)=1.14$ from figure VII-C-3
$V(OUTLET)=(1.14)8.83=10.1$ FPS<15 which meets design conditons

9. The roughness size could be reduced slightly since velocities up to 15 fps can be tolerated. From figure VII-C-3 it can be seen that

$$\max(V/V(FULL))\approx 1.15;$$

The resistance, n, could be estimated such that

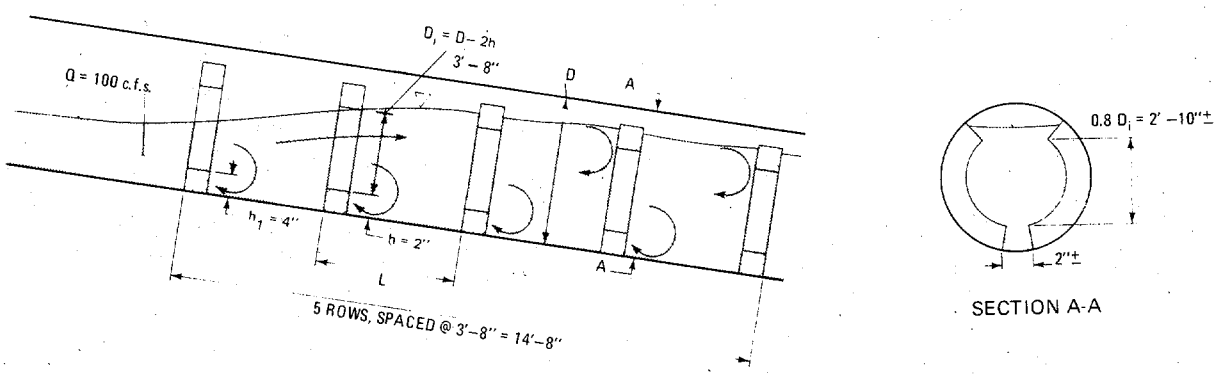
$V = 15$ ft./sec and $V(FULL) = 15/1.15$:
i.e.

$$n_r=0.59D_i^{2/3}S_o^{1/2}/(15/1.15)$$

Using $D_i=3.8$, $n_r=.022$; so $h/D_i=0.025$ would be satisfactory. The inherent assumption in the design procedure may not be valid for h/D_i less than 0.05; use $h/D_i=0.05$ rather than reduce h. A gap should be left at the bottom for dry weather drainage. From figure VII-C-3, the roughened culvert will flow 80 percent full at the design discharge so it is reasonable to also leave a gap in the top of each ring as an additional safeguard against propagating full flow upstream to the inlet.

10. Roughness sizes and spacing:

$D_i = D/(1+2h/D_i) = 4/1.1 = 3.64'$
$h = (h/D_i)D_i = (.05)3.64 = .18'$ use 2"
$L = (L/D_i)D_i = (1.5)3.64 = 5.5'$ use 5'-6"
$h_1 = 2h = 4"$
Use five roughness rings



SKETCH OF DESIGN EXAMPLE FOR INCREASED RESISTANCE IN CIRCULAR CULVERTS

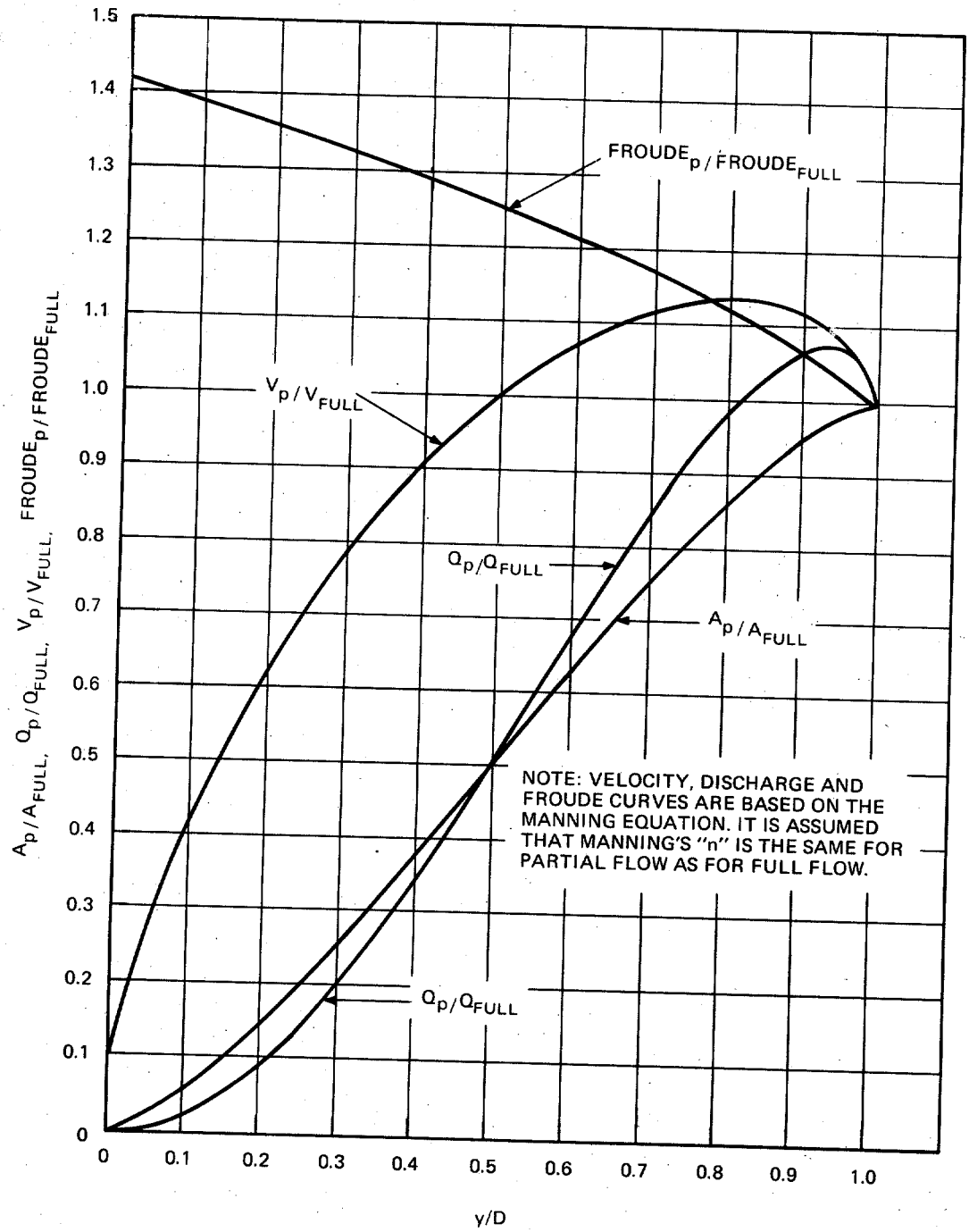


FIGURE VII - C - 3 HYDRAULIC ELEMENTS DIAGRAM FOR CIRCULAR CULVERTS FLOWING PART FULL

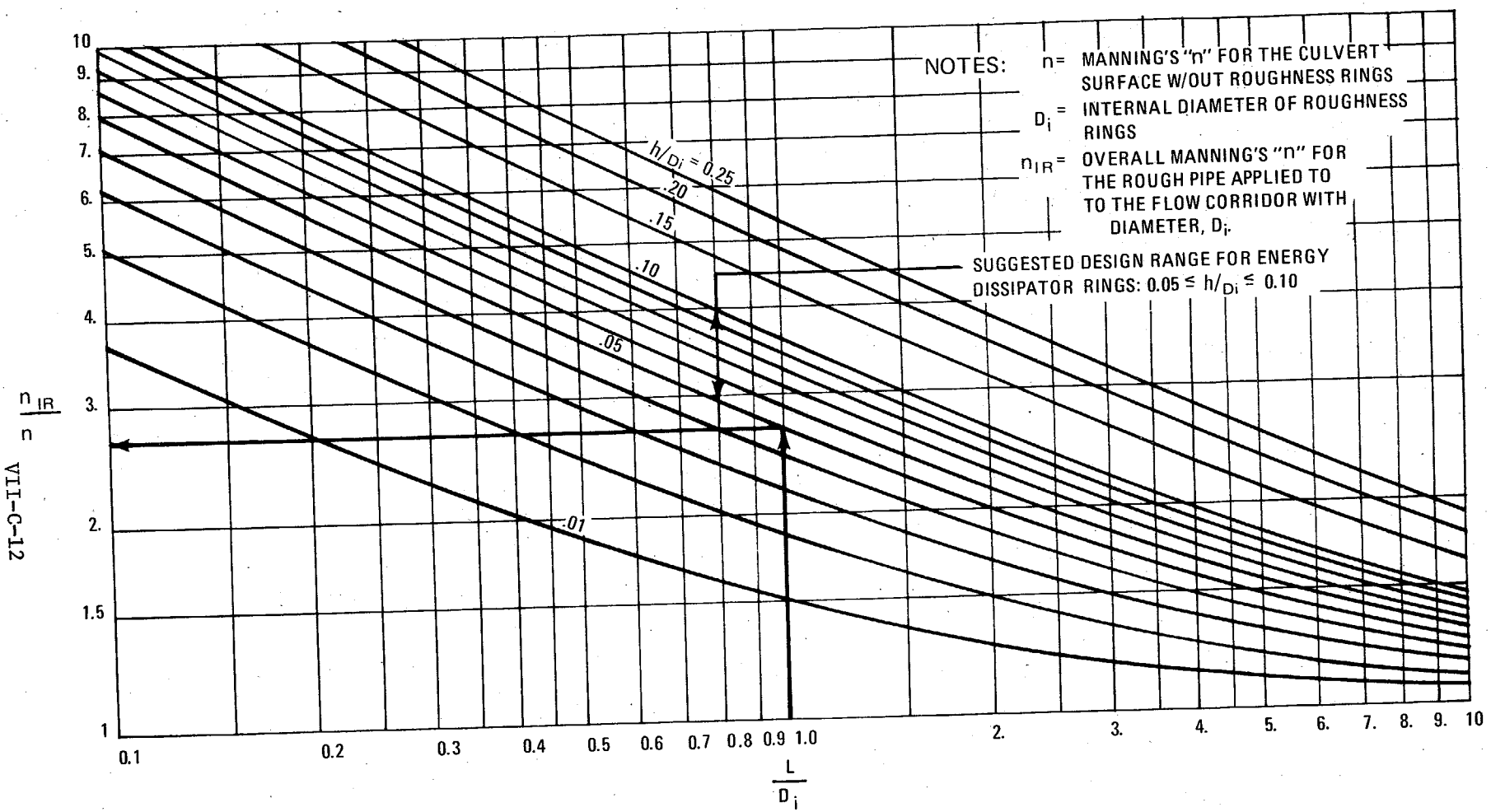


Figure VII-C-4 RELATIVE RESISTANCE CURVES FOR ISOLATED ROUGHNESS FLOW

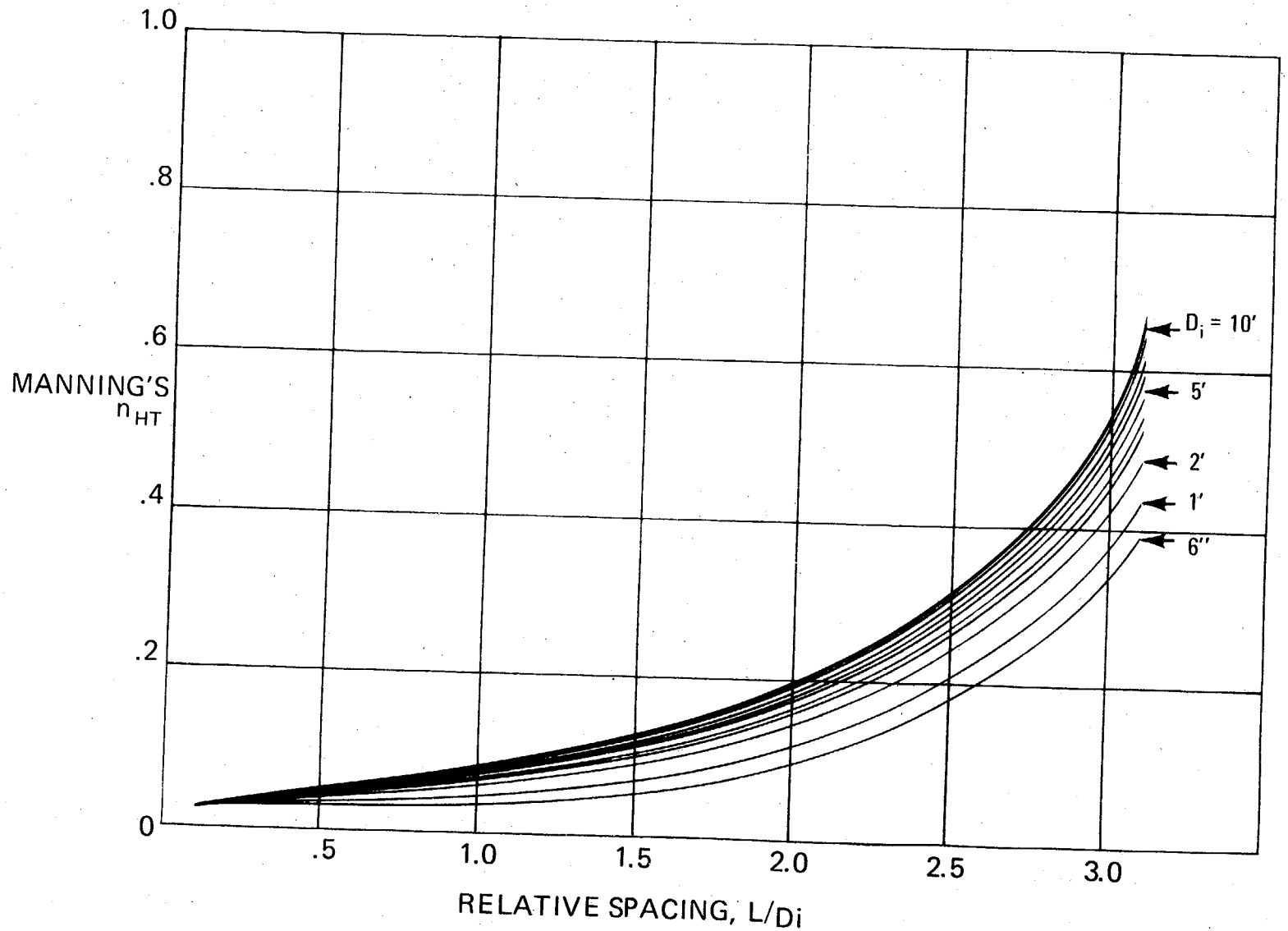


Figure VII-C-5 RESISTANCE CURVES FOR HYPERTURBULENT FLOW

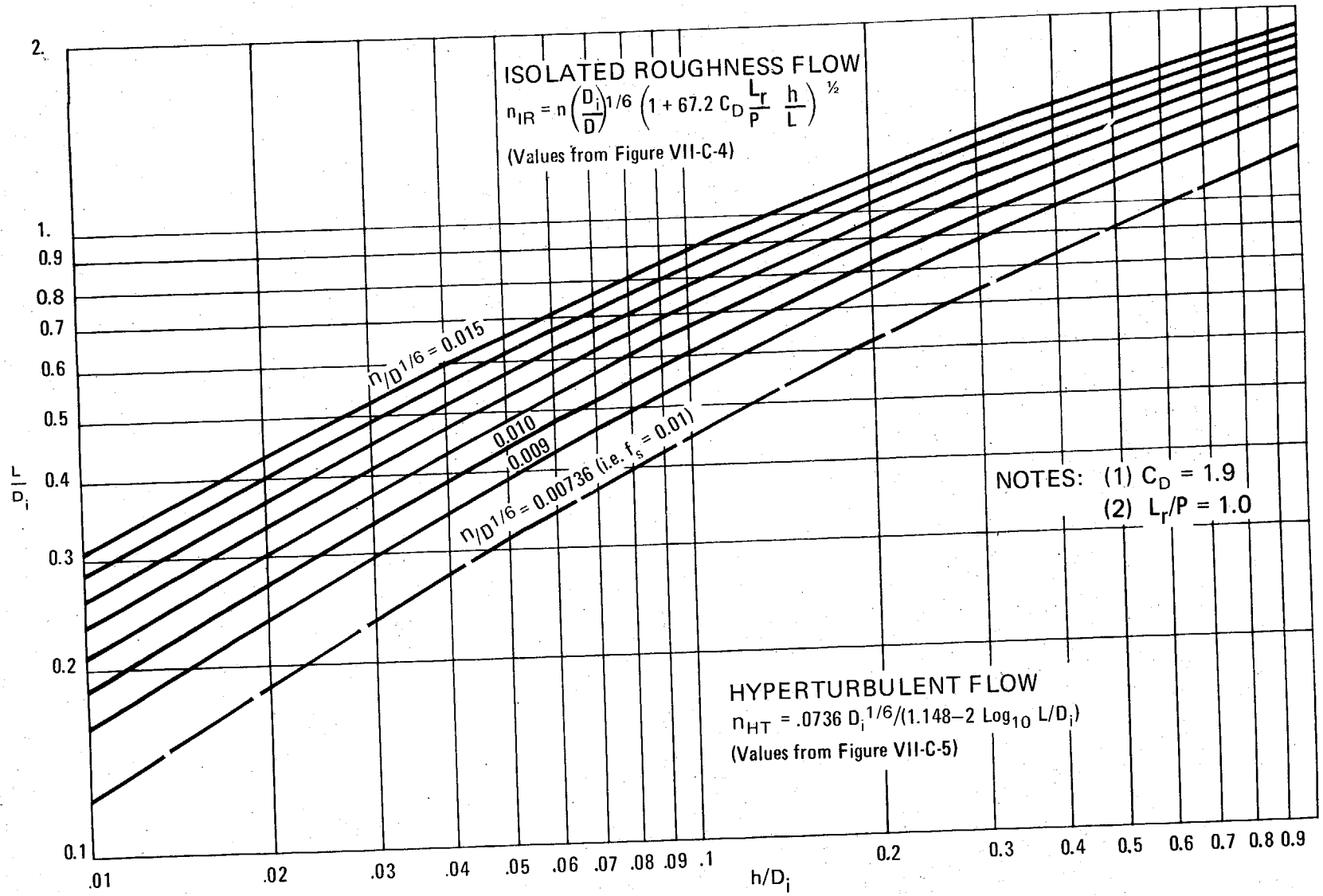


Figure VII-C-6 FLOW REGIME BOUNDARY CURVES

INCREASED RESISTANCE IN BOX CULVERTS

Material for this section was drawn primarily from an preliminary FHWA report on fish baffles in box culverts (VII-C-1). This report used Morris' (VII-C-4) categorization of flow regimes and basic friction equations, but a more representative approach velocity, V_A, in one of the regimes. Experimental data by Shoemaker (VII-C-2) was also utilized to define the transition curves. For several reasons, modifications to the fish baffle development were necessary to better fit energy dissipator needs. In fish baffle design, the interest is in a conservative estimate of resistance in order to size a culvert; whereas, in this manual, a conservative estimate of the outlet velocity is also important. Also, fish baffle design curves involve bottom roughness only.

The use of a representative approach velocity, V_A, allows an opportunity to input culvert parameters that will lean towards either an overprediction or an underprediction of resistance. For this manual, it is appropriate to develop high as well as low resistance curves. Rather than attempt to define the transition between these curves, an abrupt transition is used as the worst condition for the high curves, and a straight line transition is assumed as the mildest condition for the low curves. This is illustrated in figure VII-C-7.

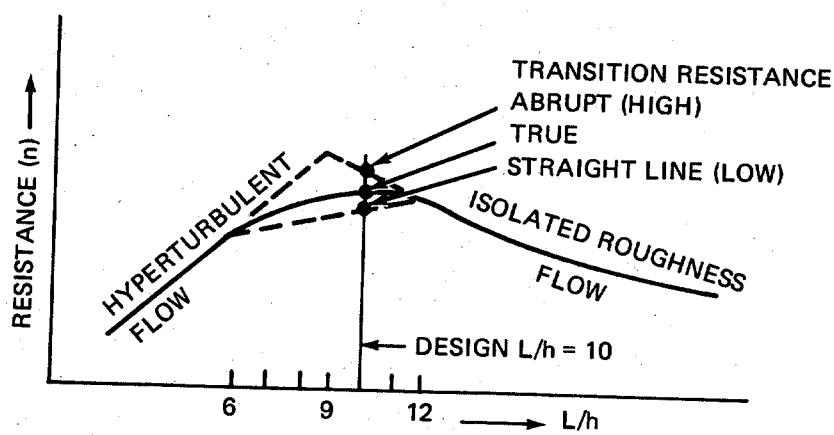


FIGURE VII-C-7. TRANSITION CURVES BETWEEN FLOW REGIMES

Observations by Powell (VII-C-3) are the basis for assuming the 6 to 12 range of L/h for the transition curve. An L/h=10 is chosen for design because it yields the largest n value.

Design Recommendations

The three equations below form the basis for determining, through a procedure simular to that shown in figure VII-C-7, the upper and lower design curves which can be used to conservatively compute resistance for culvert sizing and outlet velocities.

$$(n_{IR}/n)_{Low}=[1+200(h/L)(L_r/P)]^{1/2} \quad \text{VII-C-7}$$

$$(n_{IR}/n)_{High}=[1+390(h/L)(L_r/P)]^{1/2} \quad \text{VII-C-8}$$

$$(n_{HT}/n)=\{(1-L_r/P)+70.6(L_r/P)/[2\log((R_i/h)(h/L))+1.75]^2\}^{1/2} \quad \text{VII-C-9}$$

The equations are based on C_D=1.9, f=0.14 (where f is the Darcy friction factor for the culvert surface without roughness elements), and V_A/V=0.60 or 0.85. The lower value of V_A/V is implicitly included in equation VII-C-7 and the higher value in equation VII-C-8. It is assumed that $(R/R_i)^{1/3}$ is approximately one. R is the hydraulic radius of the culvert proper and R_i is the hydraulic radius taken inside the crests of the roughness elements.

Since the above equations are normal flow equation and since roughness elements may be relatively small using this method, it is necessary to compute the length of culvert to be roughened. The momentum equation, written for the roughened section of culvert, is used to compute the number of rows of roughness element needed. The number of rows should never be less than five. Furthermore, it is recommended that one large element be used at the beginning of the roughened zone to accelerate the asymptotic approach to normal flow. The recommended height of the larger element is twice the height of the regular elements. The spacing is the same for all rows of elements.

The procedure is limited to solid strip roughness elements with sharp upstream edges. Rectangular cross section roughness elements will best fit the assumptions made.

Due to the assumed velocity distribution, application of the procedure must be limited to small roughness heights and to relatively flat slopes. The roughness height should not exceed ten percent of the flow depth. This restriction is inherently included in the suggested range of h/Ri in the design procedure. Slope should <u>not</u> exceed 6 percent.

Design Procedure

Note: Steps 1 through 7 are concerned with computing an outlet velocity to evaluate scouring potential and/or to design additional outlet protection.

1. Use $L/h=10$

2. Select h/R_i from 0.10 to 0.40.

3. Compute L_r/P, where L_r= peripheral roughness length including slots. $L_r/P=1$ when roughness length extents through the flow or when the culvert is flowing full with a roughness length equal to the circumference. Slots are provided for low flow drainage. Their width should not exceed h/2. P is the total wetted perimeter of the culvert. In this step assume the culvert will flow full; so

$$P=2(B+D)$$

and

$$L_r=B \text{ for bottom roughness only}$$

4. Determine (n_r/n) from the lower set of curves of figure VII-C-8. In this ratio, "n" is the Manning resistance coefficient for the culvert without roughness elements or .015, whichever is smaller. Compute n_r which is the overall effective Manning resistance coefficient for the roughened portion of the culvert.

5. Determine the flow depth, y_i, measured from the roughness element crests:

   a. Assume a value of h from $h \simeq (h/R_i)BD/2(B+D)$

   b. Assume a trial value of y_i. Initally assume $y_i=D-h$. Compute $C=1.49S_o^{1/2}/n_r$

   c. Compute A_i and R_i

   $$A_i=By_i$$
   $$R_i=A_i/(B+2y_i)$$

   d. Compute "Q" from the Manning equation

   $$Q=[(1.49/n_r)S_o^{1/2}]A_i R_i^{2/3}$$

   where S_o = bottom slope of the culvert

   e. Compare Q with Q(DESIGN); and increase y_i if Q is less than Q(DESIGN); decrease y_i if Q is greater than Q(DESIGN).

   f. Repeat steps 5c, d, and e until Q and Q(DESIGN) are approximately equal.

6. a. Compute the velocity, V, using A_i from the last iteration

   $$V=Q/A_i$$

b. Compare V with the allowable outlet velocity. If a different value of V is required, select a new h/R_i and repeat steps 2 through 6.

7. Compute the required number of rows of roughness elements from the momentum equation written as follows:

$$0.5\gamma By_n^2+\rho QV_n=(N)C_DA_f\rho V_w^2/2+0.5\gamma By_i^2+\rho QV_i \quad \text{. . . . .VII-C-10}$$

where: y_n and V_n are normal depth and velocity for the smooth culvert.
y_i and V_i are normal depth and velocity for the rough culvert

$C_D=1.9$
A_f=wetted frontal area of a roughness row
$=B(h)$ for bottom roughness
$\rho=1.94$
V_w=average wall velocity acting on the roughness elements
$\approx V_{avg.}/3=(V_n+V_i)/6$
N=required number of rows of roughness elements

Note: Steps 8 through 9 check the height of the culvert for capacity.

8. Determine (n_r/n) from the upper set of curves of figure VII-C-8, using L_r/P from step 3. Compute "n_r."

9. Check adequacy of culvert height and compute flow depth if necessary by trial and error:

a. Compute h using R_i from step 5

$$h=(h/R_i)R_i$$

b. Try $y_i=D-h$
Compute $C=1.49S_o^{1/2}/n_r$

c. Compute A_i and R_i

$$A_i=By_i$$
$$R_i=A_i/2(B+y_i)$$

d. Compute "Q" from the Manning equation

$$Q=[(1.49/n_r)S_o^{1/2}]A_iR_i^{2/3}$$

e. Compare "Q" with "Q(DESIGN)." If "Q" is greater than or equal to Q(DESIGN), the culvert size is adequate. If "Q" is less than "Q(DESIGN)," increase D and repeat steps 9b through e.

10. Use the last value of D as the height of the culvert for the roughened section.

11. Specify dimensions:

    Use h from step 9a
    Compute L=10h
    Use one upstream element twice the height of the others; $h_1=2h$

Design Example

Given: 4'x4' box culvert, 200 feet long
n=.013, 6% slope
Q(DESIGN)100 cfs
Allowable outlet velocity=15 fps
$V_o=V_n=22.3$ ft/sec.
$Y_o=Y_n=1.12$ feet

1. Use L/h=10

2. Select h/R_i =0.10; use bottom roughness only

3. $L_r=B=4$ ft.
   P=2(B+D)=16 ft.
   $L_r/P=.25$

4. $(n_r/n)=2.25$ from the lower set of curves of figure VII-C-8
   $n_r=.225x.013=0.029$

5. Flow depth is:

   a. h≃0.10(4x4)/2(4+4)=0.10 ft.
   b. Try $y_i=4-0.10=3.9'$, C=12.6
   c. $A_i=4(3.9)=15.6$
      $R_i=15.6/(4+7.8)=1.32$
   d. $Q=[(1.49/n_r)S_o^{1/2}]A_iR_i^{2/3}=[12.6](15.6)(1.20)=$
      237 cfs>100 cfs
      try smaller y_i

Trial y_i	A_i	R_i	Q cfs
3.9			237
2.5	10	1.11	135
2.0	8.0	1.0	100.8

6. $V=Q/A_i=100/8.0=12.5$ ft/sec<15 ft/sec OK

7. $.5(62.4)(4)(1.12)^2+1.94(100)(22.3)=$
$N(1.9)(4)(.103)(1.94)[(22.3+12.5)/6]^2/2$
$+62.4(4)(2.0)^2/2+1.94(100)12.5$

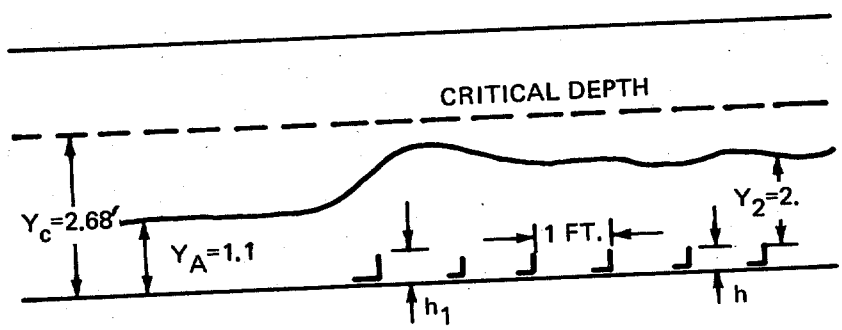
$156.5+4326.2=26.1N+499.2+2425$
$N=59.7$ use 60 rows

8. $(n_r/n)=2.5$ from the upper curves figure VII-C-8
$n_r=2.5x0.013=0.033$

9. a. $h=0.10(1)=0.1$
b. Try $y_i=4-0.1=3.9'$, $C=11.1$
c. $A_i=4(3.9)=15.6$
$R_i=15.6/(4+7.8)=1.32$
d. $Q=[11.1](15.6)(1.32)^{2/3}=208$
e. $Q>Q$(DESIGN); therefore the culvert size is adequate

10. D=4 ft.

11. h=0.1', use 1 1/4" angles
L=10h=1'; use 1'-0'
$h_1=2h$; use 2 1/2" angles



VII-C-1. Normann, J. M., HYDRAULIC ASPECTS OF FISH-LADDER BAFFLES IN BOX CULVERTS, Preliminary FHWA Hydraulics Engineering Circular, Jan. 1974.

VII-C-2. Shoemaker, R. H., HYDRAULICS OF BOX CULVERTS WITH FISH-LADDER BAFFLES, Proceedings of the 35th Ann. Meeting of the HRB, Washington, D.C., 1956, pp. 196-209.

VII-C-3. Powell, R. W., FLOW IN A CHANNEL OF DEFINITE ROUGHNESS, ASCE Transactions, Figure 10, p. 544, 1946.

VII-C-4. MORRIS, H. M., APPLIED HYDRAULICS IN ENGINEERING, The Ronald Press Company, New York, 1963.

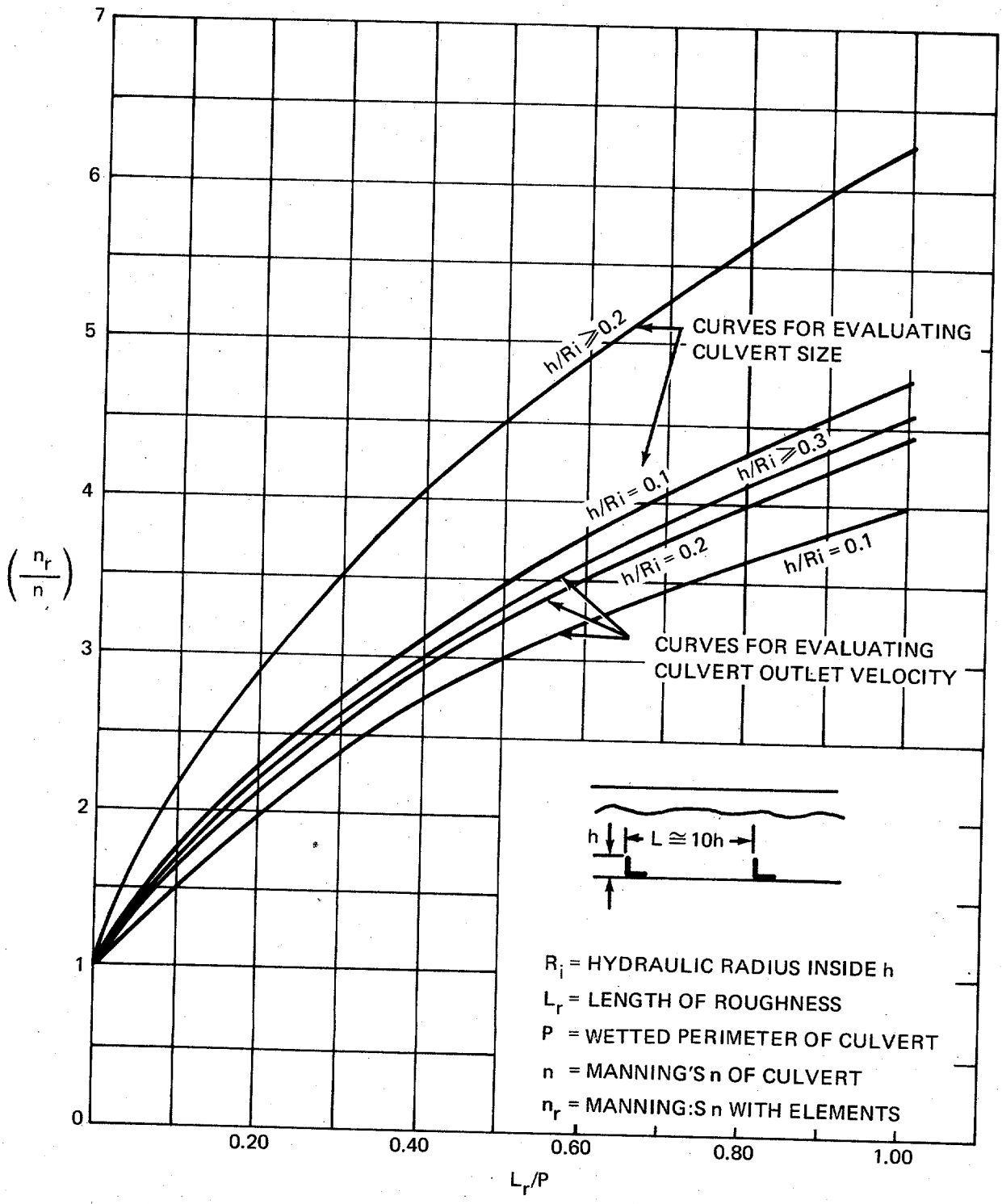


FIGURE VII-C-8 RELATIVE RESISTANCE CURVES FOR SHARP EDGED RECTANGULAR ROUGHNESS ELEMENTS SPACED AT L/h = 10

VII-D. USBR TYPE II BASIN

The Type II basin was developed by the United States Bureau of Reclamation (VII-D-1). The design is based on model studies and evaluation of existing basins.

The basin elements are shown in figure VII-D-1. Chute blocks and a dentated sill are used, but because the useful range of the basin involves relatively high velocities entering the jump, baffle blocks are not employed.

The chute blocks tend to lift part of the incoming jet from the floor, creating a large number of energy dissipating eddies. The blocks also reduces the tendency of the jump to sweep off the apron. Test data and evaluation of existing structures indicated that a chute block height, width, and spacing equal to the depth of incoming flow (y_1) are satisfactory.

The effect of the chute slope was also investigated by USBR. As long as the velocity distribution of the incoming jet is fairly uniform, the effect of the slope on jump performance is insignificant. For steep chutes or short flat chutes, the velocity distribution can be considered uniform. Difficulty will be experienced with long flat chutes where frictional resistance results in center velocities substantially exceeding those on the sides. This results in an asymmetrical jump with strong side eddies. The same effect will result from sidewall divergent angles too large for the water to follow. See chapter IV for details on the design of diverging transition sections.

The design information for this basin is considered valid for rectangular sections only. If trapezoidal or other sections are proposed, a model study is recommended to determine design parameters.

It is also recommended that a margin of safety for tailwater be included in the design. The basin should always be designed with a tailwater 10 percent greater than the conjugate depth. Figure VII-D-2 includes a design curve which incorporates the factor of safety.

Design Recommendation

The basin may be utilized for Froude numbers of 4.0 to 14.

The required tailwater depth is as indicated on figure VII-D-2.

The height of the chute blocks (h_1) is equal to the depth of the incoming flow, y_1, figure VII-D-1. The width (W_1) and spacing (W_2) of the chute blocks also equals y_1. A space $y_1/2$ is preferred along each wall.

The height (h_2) of the dentated sill is $0.2(y_2)$ and the maximum width (W_3) and spacing (W_4) is $0.15(y_2)$. The downstream slope of the sill is 2:1. For narrow basins, the width and spacing may be reduced but should remain proportional.

The chute blocks and end sill do not need to be staggered relative to each other.

The USBR tests indicated that the slope of the incoming chute has no preceptible effect on stilling basin action. Their test slopes varied from 0.6:1 to 2:1. If the chute slope is 2:1 or greater, a reasonable radius curve should be incorporated into the chute design, see figure IV-B-5.

The length of the basin (L_B) may be obtained from figure VII-D-3.

These design recommendations will result in a conservative stilling basin for flows up to 500 cfs per foot of basin width.

Design Procedure

1. Determine basin width (W_B), elevation (z_1), length (L_B), total length (L), incoming depth (y_1), incoming Froude number (Fr_1), and jump height (y_2) by using the design procedure in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E, use C =1.1 or figure VII-D-2 to find y_2. For step 5F, use figure VII-D-3 to find L_B.

2. The chute block height (h_1), width (W_1), and spacing (W_2) are all equal to the incoming depth.

$$W_1 \approx W_2 \approx h_1 = y_1$$

The number of blocks (N_c) is equal to

$$N_c = W_B/2y_1, \text{ rounded to a whole number.}$$

Adjusted $W_1 = W_2 = W_B/2N_c$
Side wall spacing = $W_1/2$

3. The dentated sill height, $(h_2)=0.2y_2$, the block width (W_3) = the spacing width (W_4) which is equal to 0.15 times the jump depth.

$$h_2 = 0.2y_2$$
$$W_3 \approx W_4 \approx 0.15y_2$$

The number of blocks (N_s) plus spaces approximately equals W_B/W_3. Round this to the next lowest odd whole number and adjust $W_3=W_4$ to fit W_B.

Example Problem

Given: Same Conditions as IV-B, Supercritical Expansion
10X6 RCB, Q=417 cfs, S_o=6.5%
Elevation outlet invert z_o=100 ft.
V_o=27.8 fps, y_o=1.5 ft.
Downstream channel is a 10 ft. bottom trapezoidal channel with 2:1 side slopes and n=.03

Find: Dimensions for a USBR Type II basin

Solution:

1. Determine basin elevation using design procedure outlined in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins

   Steps from IV-B:

   1. V_o=27.8 fps, y_o=1.5 ft., Fr_o=4
   2. In channel TW=y_n=1.9 ft., V_n=15.9 fps
   3. $y_2=C_1y_1[\sqrt{1+8Fr^2}-1]/2=1.1(1.5)[\sqrt{1+8(4)^2}-1]/2=8.6$
   4. $y_2>TW$, 8.6>1.9 drop the basin
   5. Use z_1=84.5 ft.=z_2

      A. W_B=10 ft., $S_T=S_s$=.5

      B. W_B-OK no flare

      C. $Q=y_1 10[2g(100-84.5+1.5-y_1)+27.8^2]^{1/2}$
      $Q=10y_1[64.4(17-y_1)+772.8]^{1/2}$
      y_1=.98 OK
      V_1=417/.98(10)=42.6 fps

      D. $Fr_1=42.6/\sqrt{g(.98)}=7.58$

      E. For C_1=1.1, $y_2=1.1(.98)[\sqrt{1+8(7.58)^2}-1]/2$=11 ft.

      F. From figure VII-D-3 L_B/y_2=4.3, L_B=47.5 ft.
      $L_T=(z_o-z_1)/S_T=(100-84.5)/.5$=31 ft.
      $z_3=[100-(47.5+31-84.5/.5).065]/1.13$
      z_3=93.7 ft.

      G. y_2+z_2=95.5 ft.
      z_3+TW=95.6 ft. OK

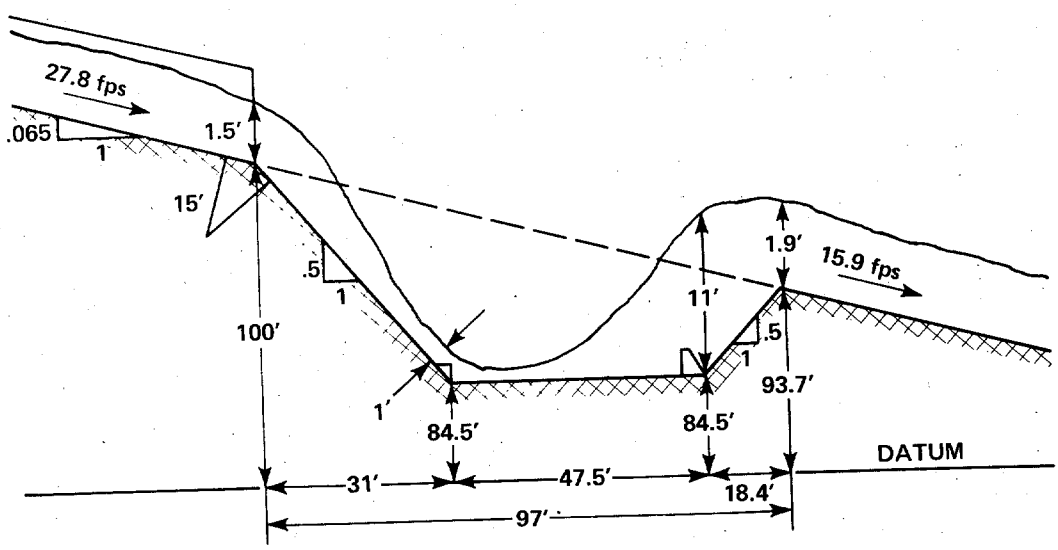
6. L_T=31 ft., L_B=47.5 ft.
$L_S=(z_3-z_2)/S_s=(93.7-84.5)/.5=18.4$ ft.
L=31+47.5+18.4=97 ft.

7. Fr_o=4 from figure IV-B-5 y_o/r=.1
r=1.5/.1=15 ft.

Basin width, W_B=10 ft.
Basin elevation, z_1= 84.5 ft.
Basin length, L_B=47.5 ft.
Total length, L=97 ft.
Incoming depth, $y_1 \approx 1$ ft.
Incoming Froude number, Fr_1=7.6
Jump height, $y_2 \approx 11$ ft.

2. Chute Blocks:
$h_1=W_1=W_2=y_1$=1.0 ft.
N_c=10/2(1)=5-OK whole number
$W_1=W_2$=10/2(.5)=1
Sidewall spacing = .5 ft.

3. Dentated Sill:
$h_2=0.2y_2$=.2(11)=2.2 ft.
$W_3=W_4$=.15y_2=1.65 ft.
$N_s=W_B/W_3$=10/1.65≈6, Use 5
which makes 3 blocks and 2 spaces each 2 ft.



Note: See the USBR and SAF design comparison at the end of section VII-G.

VII-D-1. U.S. Bureau of Reclamation, DESIGN OF SMALL DAMS, U.S. Government Printing Office, 2nd Ed. 1973, pp. 393-439.

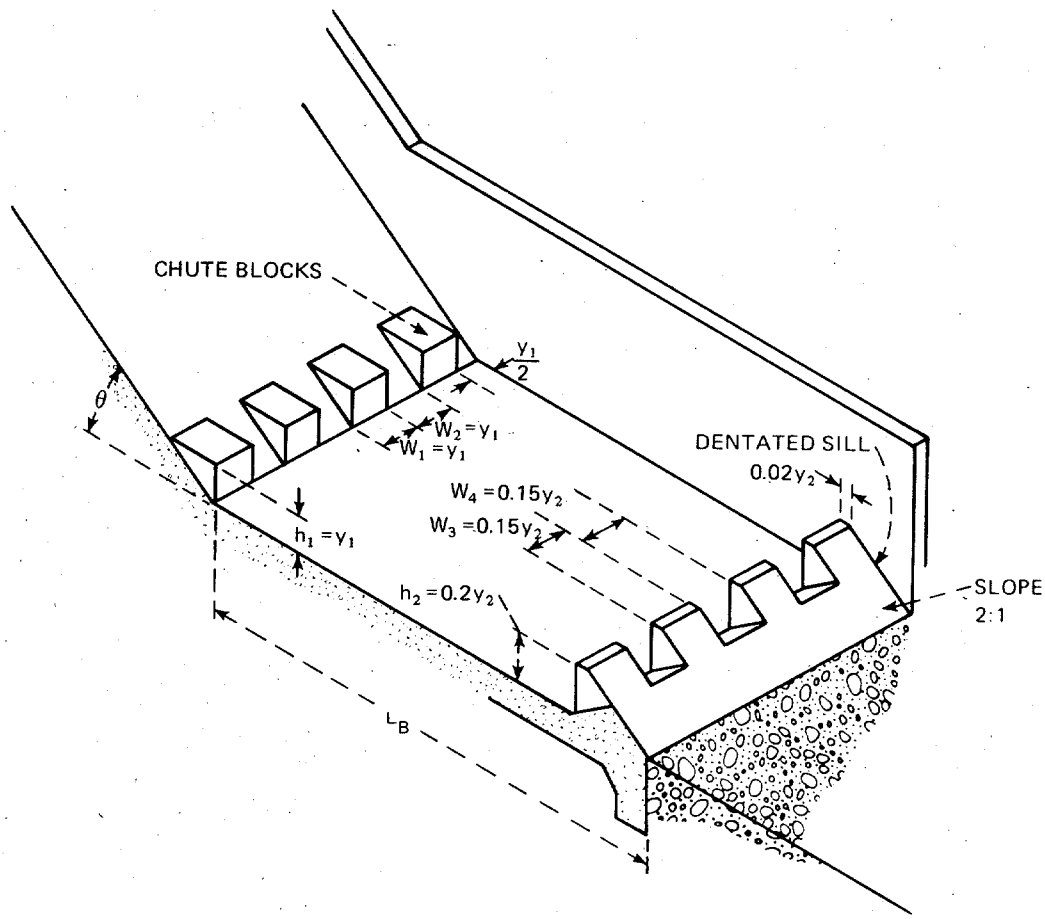


FIGURE VII – D-1 USBR TYPE II BASIN

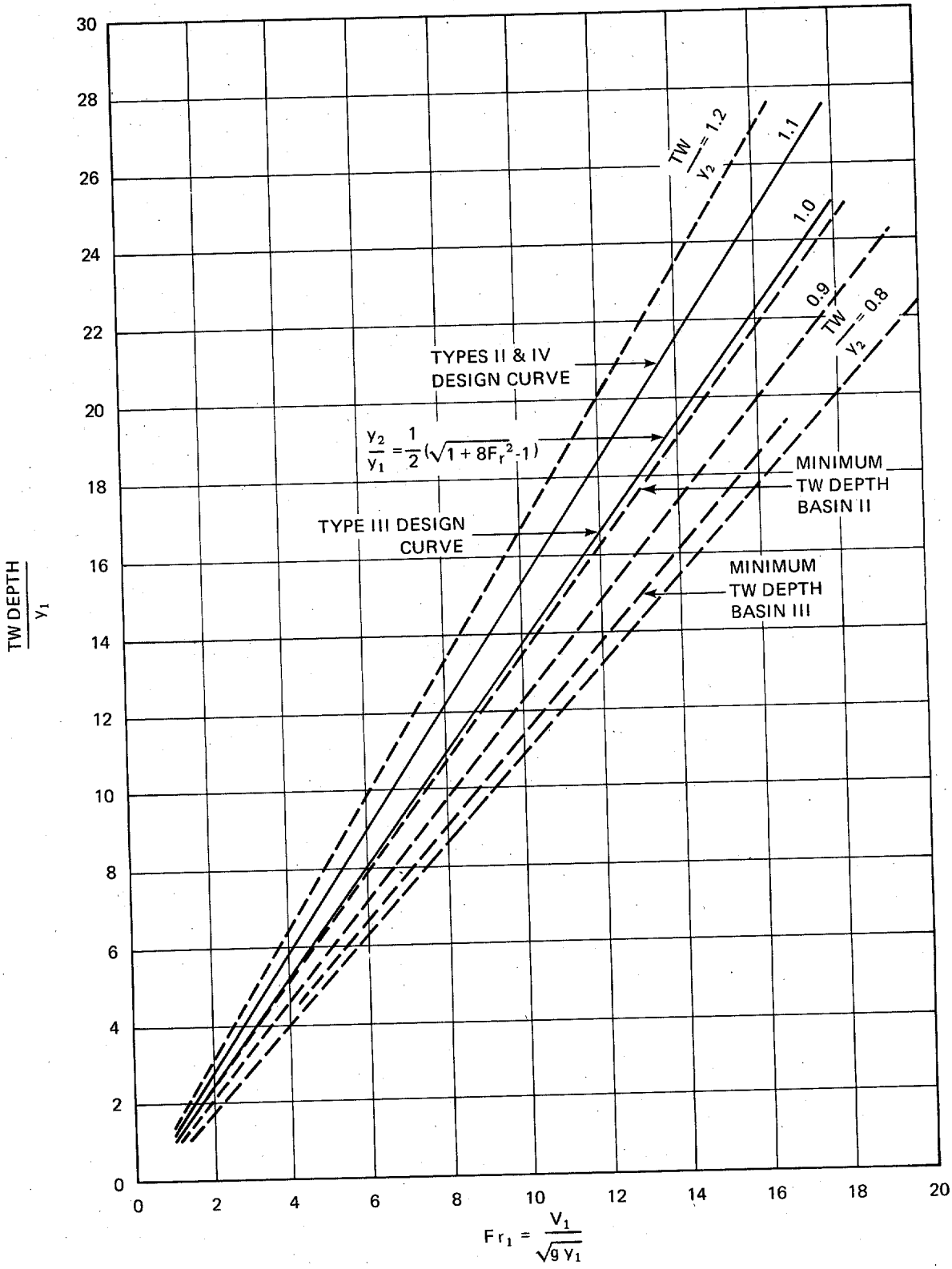


FIGURE VII - D - 2 TAIL WATER DEPTH (BASIN II, III & IV)

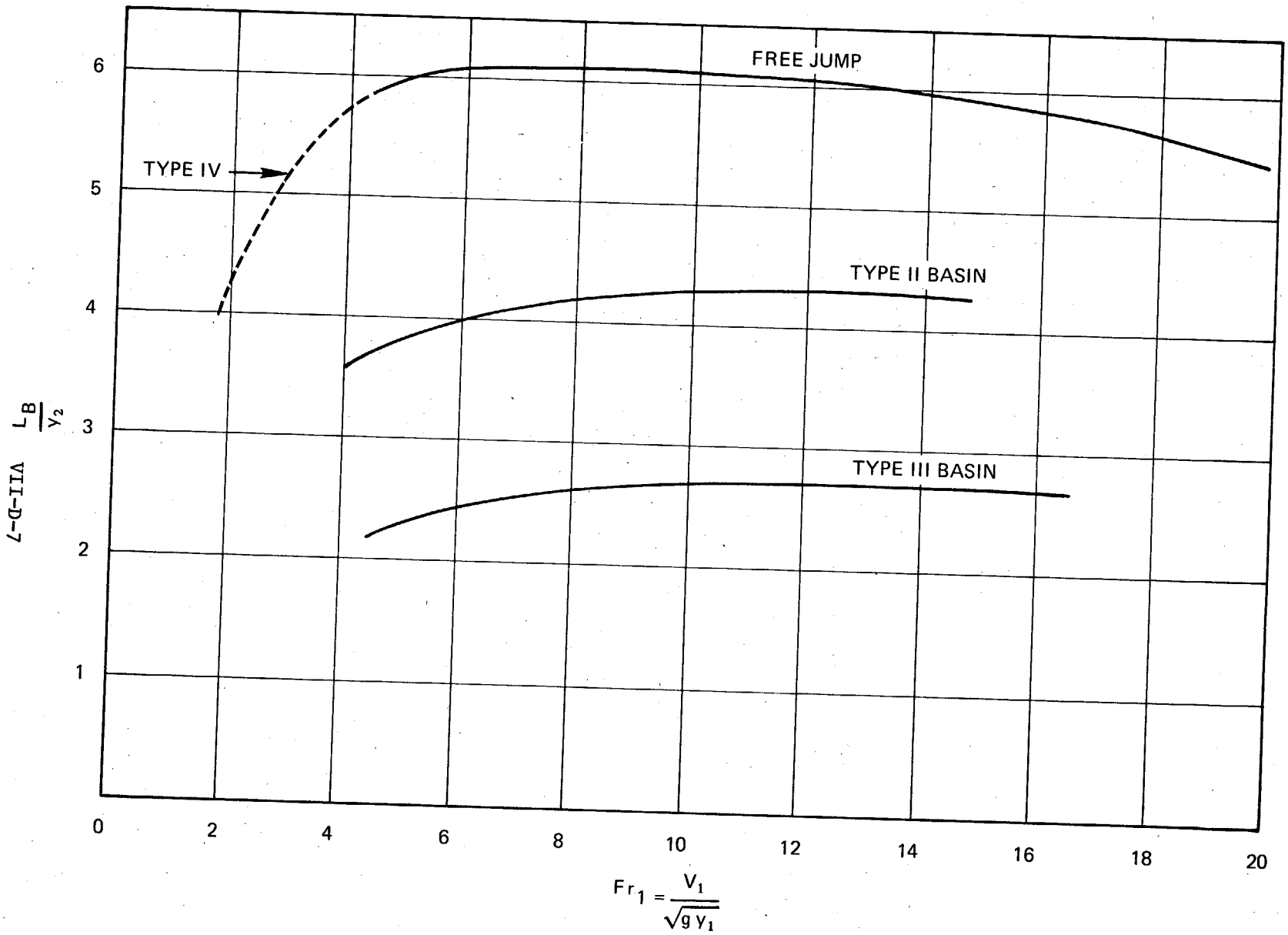


FIGURE VII - D - 3. LENGTH OF JUMP ON HORIZONTAL FLOOR

VII-E. USBR TYPE III BASIN

This basin, which originated with the U.S. Bureau of Reclamation (VII-D-1), is intended for discharges up to 200 cubic feet per second per foot of basin width and velocities up to 50 to 60 feet per second. It operates effectively for Froude numbers ranging from 4.5 to 17.

The design employs chute blocks, baffle blocks, and an end sill, figure VII-E-1.

The basin action is very stable with a steep jump front and less wave action downstream than with either the USBR Type II or the free hydraulic jump. The position, height, and spacing of the baffle blocks as recommended below should be adhered to carefully. If the baffle blocks are too far upstream, wave action will result; if too far downstream, a longer basin will be required; if too high, waves can be produced; and, if too low, jump sweep out or rough water may result.

The baffle piers may be as shown in figure VII-E-1 or cubes; both shapes are effective. The corners should not be rounded as this reduces energy dissipation.

Tailwater depth equal to full conjugate depth is recommended, figure VII-D-2. This provides a 15 to 18 percent factor of safety.

Design Recommendations

Length of basin can be obtained from figure VII-D-3.

Tailwater depth equal to full conjugate depth is essential.

The height, width, and spacing of chute blocks should equal the depth of flow entering the basin, y_1. The width may be reduced provided the spacing is also reduced a like amount. If y_1 is less than 8 inches, make blocks 8 inches high.

The height, width, and spacing of the baffle piers are shown on figure VII-E-1. The width and spacing may be reduced, for narrow structure, provided both are reduced a like amount. A half space should be left adjacent to the walls.

The distance from the downstream face of the chute blocks to the upstream face of the baffle pier should be $0.8(y_2)$.

The height and shape of the solid end sill is given in figure VII-E-2.

If the chute slope is 2:1 or greater, use a circular curve at the intersection with the basin floor. See figure IV-B-5 to determine the radius.

If these recommendations are followed, a short, compact basin with good dissipation action will result. If they cannot be followed closely, a model study is recommended.

Design Procedure

1. Determine basin width (W_B), elevation (z_1), length (L_B), total length (L), incoming depth (y_1), incoming Froude number (Fr_1), and jump height (y_2) by using the design procedure in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E, use $C_1=1.0$ or figure VII-D-2 to find y_2. For step 5F, use figure VII-D-3 to find L_B.

2. The chute block height (h_1), width (W_1), and spacing (W_2) are all equal to the incoming depth

$$W_1 \simeq W_2 \simeq h_1 = y_1$$

The number of blocks (N_C) is equal to

$N_C = W_B/2y_1$ rounded to a whole number.

Adjusted $W_1=W_2=W_B/2N_C$
Space at wall = $W_2/2$.

3. The baffle block height (h_3) is found by using figure VII-E-2 to find h_3/y_1 knowing Fr_1. The width (W_3) and spacing (W_4) equal:

$$W_3=W_4=.75h_3$$

The number of blocks (N_B) is equal to $N_B=W_B/1.5h_3$ rounded to a whole number.
Adjusted $W_3=W_4=W_B/2N_B$.
Space at wall = $W_3/2$.
Length from chute block=$0.8y_2$.

4. The end sill height (h_4) is found by using figure VII-E-2 to determine h_4/y_1, knowing Fr_1.

Example Problem

Given: Same Conditions as VII-D
10X6 RCB, Q=417 cfs, S_o=6.5%
Elevation outlet invert z_o=100 ft.
V_o=27.8 fps, y_o=1.5 ft.
Downstream channel is a 10 ft. bottom trapezoidal channel with 2:1 side slopes and n=.03

Find: Dimensions for a USBR Type III basin

Solution:

1. Determine basin elevation using design procedure outlined in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins

   Steps from IV-B:

   1. V_o=27.8 fps, y_o=1.5 ft., Fr_o=4
   2. Downstream channel TW=y_n=1.9 ft., V_n=15.9 fps
   3. $y_2=C_1y_1[\sqrt{1+8Fr^2}-1]/2=1(1.5)[\sqrt{1+8(4)^2}-1]/2=7.8$ ft.
   4. Since y_2>TW, 7.8>1.9 drop the basin
   5. Use z_1=87.5 ft.

      A. W_B=10 ft., $S_T=S_S$=.5

      B. W_B-OK no flare

      C. $Q=10y_1[2g(100-87.5+1.5-y_1)+27.8^2]^{1/2}$
         $Q=10y_1[64.4(14-y_1)+772.8]^{1/2}$
         y_1=1.04 ft.
         V_1=417/1.04(10)=40.1 fps

      D. $Fr_1=40.1/\sqrt{g(1.04)}=6.93$

      E. For C_1=1, $y_2=1.04[\sqrt{1+8(6.93)^2}-1]/2=9.68$ ft.

      F. From figure VII-D-3 L_B/y_2=2.7, L_B=26 ft.
         $L_T=(z_o-z_1)/S_T=(100-87.5/.5=25$ ft.
         $z_3=[100-(26+25-87.5/.5).065]/1.13=95.6$

      G. y_2+z_2=9.7+87.5=97.2
         TW+z_3=95.6+1.9=97.5 OK

6. L_T=25 ft., L_B=26 ft.
$L_S=(z_3-z_2)/S_s=(95.6-87.5)/.5=16.2$ ft
$L=25+26+16.2=67.2$ ft.

7. Fr_o=4 from figure IV-B-5, $y_o/r=.1$
$r=1.5/.1=15$ ft.

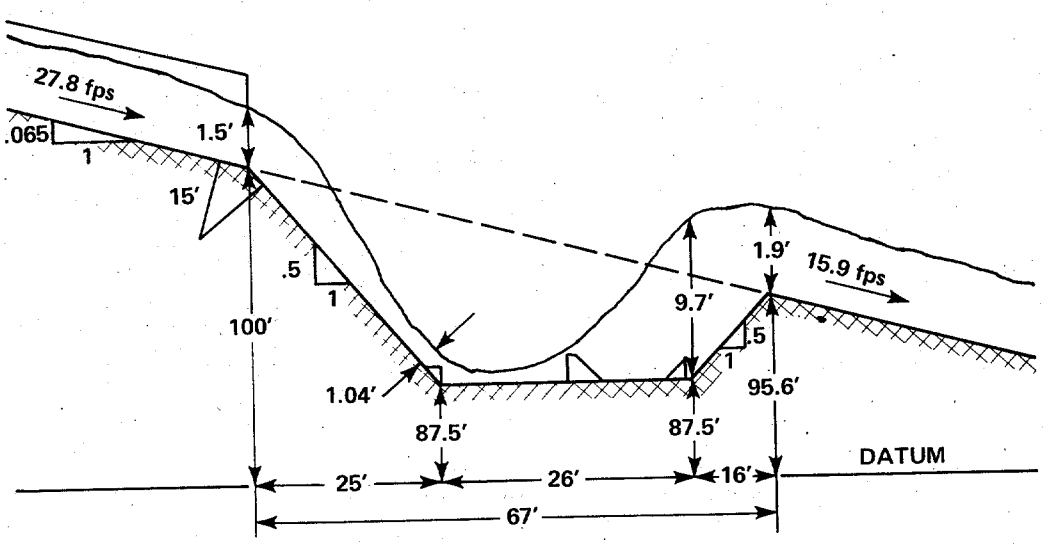
Basin width, W_B=10 ft.
Basin elevation, z_1=87.5 ft.
Basin length, L_B=26 ft.
Total length, L=67.2 ft.
Incoming depth, y_1=1.04 ft.
Jump height, y_2=9.7 ft.
Incoming Froude number, Fr_1=6.9

2. Chute Blocks:
$h_1=W_1=W_2=y_1=1.0$ ft.
$N_C=W_B/2y_1=10/2=5$-OK whole number
Space at wall$=W_2/2=1/2=.5$ ft.

3. Baffle Blocks:
From figure VII-E-2, $h_3/y_1=1.75$ for $Fr_1=6.9$
$h_3=1.75(1)=1.75$ ft.
$W_3=W_4=.75h_3=.75(1.75)=1.3$ ft.
$N_B=W_B/1.5h_3=10/1.5(1.75)=3.8\simeq4$ blocks
adjusted $W_3=W_4=W_B/2N_B=10/2(4)=1.25$ ft.
Space at wall = $W_3/2=.63$ ft.
Length from chute block = $.8y_2=.8(9.7)=7.8$ ft.

4. End Sill:
From figure VII-E-2, $h_4/y_1=1.4$ for $Fr_1=6.9$
$h_4=1.4(1)=1.4$ ft.

Note: See the USBR and SAF design comparison at the end of section VII-G.



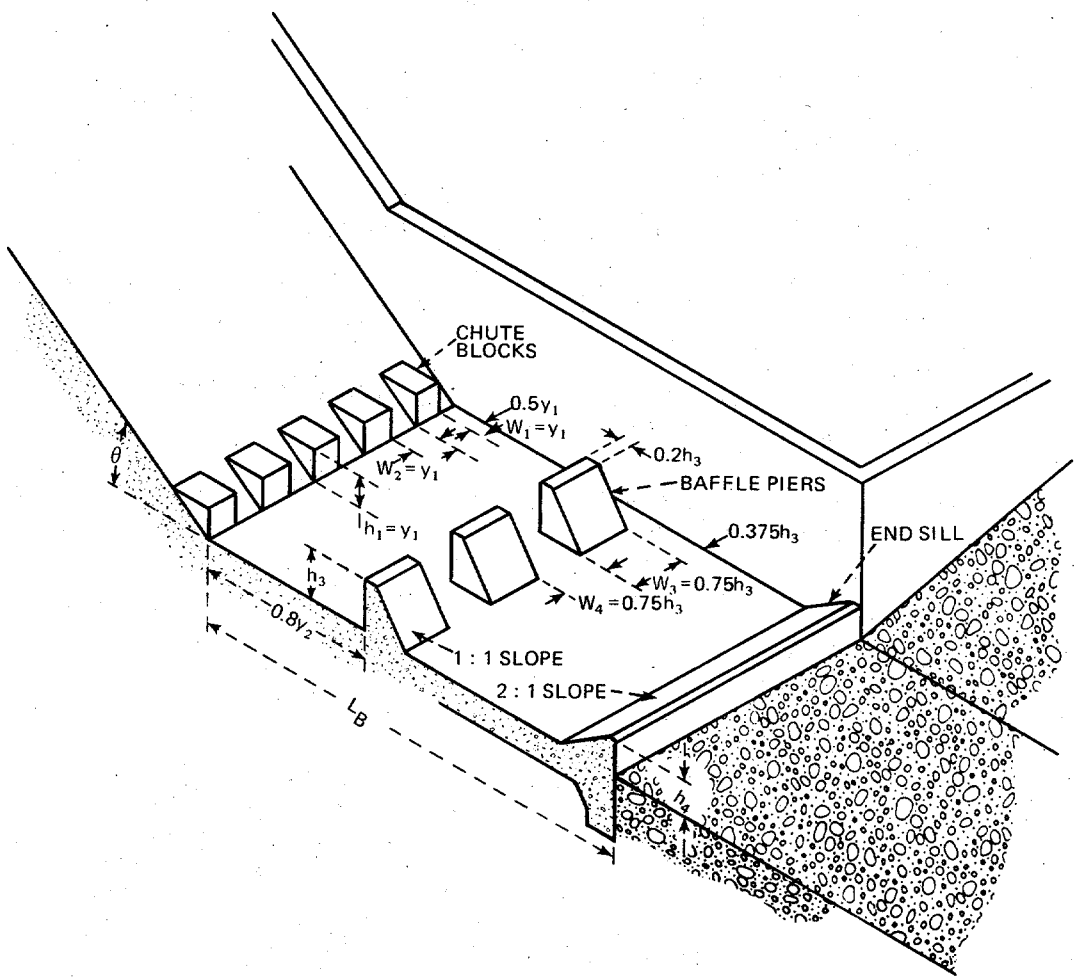


FIGURE VII - E - 1 USBR TYPE III BASIN

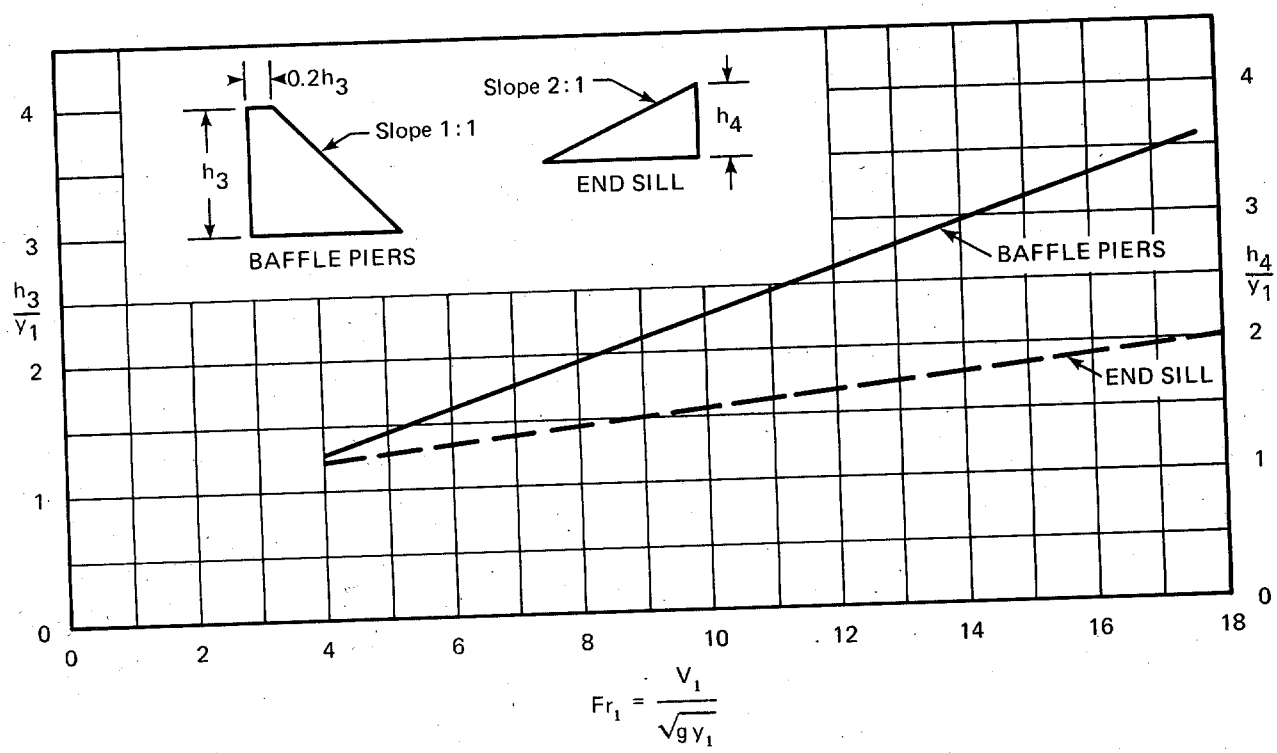


FIGURE VII - E - 2 HEIGHT OF BAFFLE PIERS AND END SILL (BASIN III) FROM REFERENCE VII-D-1

VII-F. USBR TYPE IV BASIN

The Type IV U.S. Bureau of Reclamation basin (VII-D-1) is intended for use in the Froude number range of 2.5 to 4.5. In this low Froude number range, the jump is not fully developed and downstream wave action may be a problem as discussed in chapter VI. For the intermittent flow encountered at most highway culverts, this is not judged to be a severe limitation.

The basin employs chute blocks and an end sill, figure VII-F-1.

Design Recommendations

The maximum width of the chute blocks is equal to the depth of incoming flow, y_1. From a hydraulic standpoint, it is better to construct the blocks narrower than y_1, preferably $0.75(y_1)$. The block width to spacing should be maintained at 1:2.5 with a fractional space at each wall. The top of the blocks is placed $2(y_1)$ above the basin floor and sloped 5 degrees downstream.

It is recommended that a tailwater depth 10 percent greater than the conjugate depth be used for design. The hydraulic jump is very sensitive to tailwater depth at these low Froude numbers. The jump performance is increased and wave action reduced with the higher tailwater, $1.1(y_2)$, figure VII-D-2.

The basin length can be obtained from the dashed portion of the free jump curve of figure VII-D-3.

The end sill used with the type III basin is also recommended for this basin, figure VII-E-2.

The Type IV basin is for use in rectangular channels only. See chapter IV for details on the design of transition sections.

Design Procedure

1. Determine basin width (W_B), elevation (z_1), length (L_B), total length L), incoming depth (y_1), incoming Froude number (Fr_1), and jump height (y_2) by using the design procedure in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E: use C_1=1.0 or figure VII-D-2 to find y_2. For step 5F: use figure VII-D-3 to find L_B.

2. The chute block height $(h_1)=2y_1$
   width $(W_1)=0.75y_1$
   spacing $(W_2)=2.5W_1$

The number of blocks (N_c) is equal to

$N_c = W_B/3.5W_1$ rounded to a whole number

Adjusted $W_1 = W_B/3.5N_c$
$W_2 = 2.5W_1$

Side Wall Spacing = $1.25W_1$

3. The end sill height (h_4) is determined by using figure VII-E-2 to find h_4/y_1 knowing Fr_1.

Example Problem

Given: Same conditions as VII-D
10X6 RCB, Q=417 cfs, S_o=6.5%
Elevation of outlet invert z_o=100 ft.
V_o=27.8 fps, y_o=1.5 ft.
Downstream channel is a 10 ft. bottom trapezoidal channel with 2:1 side slopes

Find: Dimensions for a USBR Type IV basin

Solution:

1. This basin, which is essentially a free hydraulic jump, was designed in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins. The design is summarized below. Even though the incoming Fr_1 is outside the range for this basin, the design procedure is the same. The problem is used so that the different USBR basins can be compared (see design summary in section VII-G).

Steps from IV-B:

1. V_o=27.8 fps, y_o=1.5 ft., Fr_o=4
2. TW=y_n=1.9 ft., V_n=15.9 fps
3. y_2=7.8 ft.
4. Since y_2>TW drop the basin
5. Use z_1=85
   A. W_B=10 ft., S_T=S_S=.5
   B. W_B=10 ft.-OK no flare
   C. y_1=.99 ft., V_1=42.1 fps

D. $Fr_1=7.46$ which exceeds the 2.5 to 4.5 design range

E. $y_2=9.96$ ft.

F. $L_B=63$ ft., $L_T=30$ ft, $z_3=92.9$ ft.

G. $y_2+z_2=95$ ft
$z_3+TW=94.8$ ft. OK

6. $L_T=30$ ft., $L_B=63$ ft., $L_S=19.6$ ft., $L=112.6$ ft.

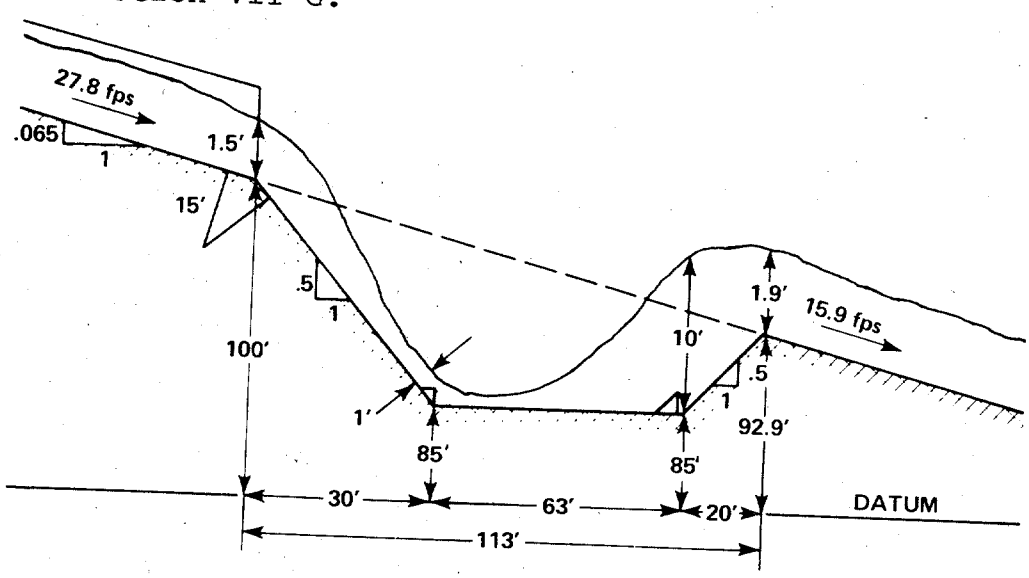
7. $r=15$ ft.

Basin Width, $W_B=10$ ft.
Basin elevation, $z_1=85$ ft.
Basin length, $L_B=63$ ft.
Total length, $L=113$ ft.
Incoming depth, $y_1 \simeq 1$ ft.
Incoming Froude number, $Fr_1 \simeq 7.5$
Jump height, $y_2 \simeq 10$ ft.

2. Chute Blocks:
$h_1=2y_1=2(1)=2$ ft.
$W_1 \simeq .75y_1=.75$ ft.
$W_2 \simeq 2.5W_1=1.9$ ft.
$N_c=W_B/3.5W_1=10/3.5(.75)=3.8$, Use 4
Adjusted $W_1=W_B/3.5N_c=10/3.5(4)=.7$ ft.
$W_2=2.5W_1=2.5(.7)=1.8$ ft.
Side wall spacing = $1.25\ W_1=1.25(.7)=.9$ ft.

3. End Sill: $h_4/y_1=1.4$ from figure VII-E-2 with $Fr_1=7.5$
$h_4=1.4(1)=1.4$ ft.

Note: See USBR and SAF design comparison at the end of section VII-G.



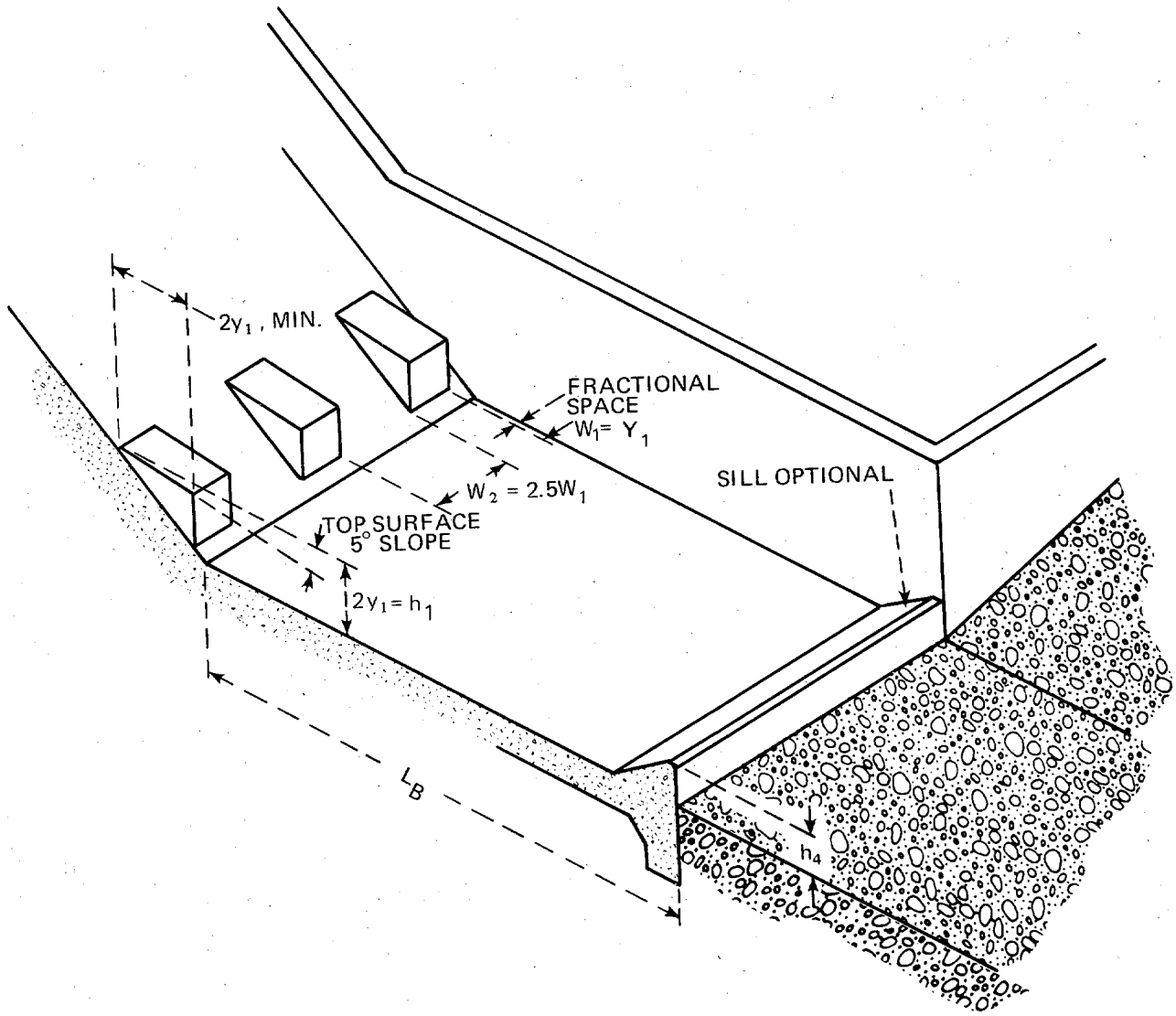


FIGURE VII - F - 1 USBR TYPE IV BASIN

VII-G. SAF STILLING BASIN

The St. Anthony Falls or SAF stilling basin is a generalized design that uses a hydraulic jump to dissipate energy. The design is based on model studies conducted by the Soil Conservation Service at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota (VII-G-1).

The design provides special appurtenances, chute blocks, baffle or floor blocks and an end sill, which allow the basin to be shorter than free hydraulic jump basins. It is recommended for use at small structures such as spillways, outlet works, and canals where Fr = 1.7 to 17. Fr is the Froude number at the dissipator entrance. The reduction in basin length achieved through the use of appurtenances is about 80 percent of the free hydraulic jump length.

At the design flow, the SAF stilling basin provides an economical method of dissipating energy and preventing dangerous stream bed erosion.

Design Recommendations

The width W_B of the stilling basin is equal to the culvert width W_o. For circular conduits, W_B is the larger of D_o or

$$W_B=0.3D(Q/D^{2.5}) \quad \ldots\ldots\ldots\ldots \text{VII-G-1}$$

The basin can be flared to fit an existing channel as indicated on figure VII-G-1. The sidewall flare dimension z should not be smaller than 2, i.e., 2:1, 3:1, or flatter.

The length L_B of the stilling basin for Froude numbers between Fr =1.7 and Fr=17 is proportional to the theoretical sequent depth y_j found from the hydraulic jump equation

$$y_j=y_1(\sqrt{1+8Fr_1^2}-1)/2 \quad \ldots\ldots\ldots\ldots \text{VII-G-2}$$

and

$$L_B=4.5y_j/Fr^{0.76} \quad \ldots\ldots\ldots\ldots \text{VII-G-3}$$

The height of the chute block is y_1, and the width and spacing are approximately $0.75y_1$.

Floor or baffle blocks should be staggered with respect to the chute blocks and should be placed downstream a distance $L_B/3$. They should occupy between 40 and 55 percent of the stilling basin width. 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. No floor block should be placed closer to the side wall than $3y_1/8$.

Height of the end sill is $0.07y_j$, where y_j is the theoretical sequent depth corresponding to y_1.

The depth of tailwater y_2 above the stilling basin floor is:

$Fr=1.7$ to 5.5, $y_2=(1.1-Fr_1^2/120)y_j$ VII-G-4
$Fr=5.5$ to 11, $y_2=0.85y_j$. VII-G-5
$Fr=11$ to 17, $y_2=(1.0-Fr_1^2/800)y_j$ VII-G-6

Wingwalls should be equal in height and length to the stilling basin sidewalls. The top of the wingwall should have a 1:1 slope. Flaring wingwalls are preferred to perpendicular or parallel wingwalls. The best overall conditions are obtained if the triangular wingwalls are located at an angle of 45° to the outlet centerline.

The stilling basin sidewalls may be parallel (rectangular stilling basin) or diverge as an extension of the transition sidewalls (flared stilling basin). The height of the side wall above the maximum tailwater depth to be expected during the life of the structure is given by $y_j/3$.

A cut-off wall of adequate depth should be used at the end of the stilling basin to prevent undermining. The depth of the cut-off wall must be greater than the maximum depth of anticipated erosion at the end of the stilling basin.

Design Procedure

1. Choose basin configuration and flare diemension, z.

2. Determine basin width (W_B), elevation (z_1), length (L_B), total length (L), incoming depth (y_1), incoming Froude number (Fr_1), and jump height (y_2) by using the design procedure in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E: y_2 is found from equation VII-G-4, 5, or 6 and y_j from equation VII-G-2. For step 5F: use equation VII-G-3 for L_B.

3. Chute Block:
   height, $h_1=y_1$
   width, (W_1)=spacing, $(W_2)=.75y_1$
   Number, $N_c \simeq W_B/2W_1$ rounded to a whole number
   Adjusted $W_1=W_2=W_B/2N_c$
   N_c includes the 1/2 block at each wall.

4. Baffle Block:
height, $h_3=y_1$
Width, (W_3)=Spacing, $(W_4)=.75y_1$
Basin width at baffle blocks, $W_{B2}=W_B+2L_B/3z$
Number of blocks, $N_B=W_{B2}/2W_3$ rounded to a whole number
Adjusted $W_3=W_4=W_{B2}/2N_B$
Check total block width to insure that at least 40 to 55 percent of W_{B2} is occupied by blocks

Distance from chute blocks to baffle blocks = $L_B/3$

5. End Sill height, $h_4=.07y_j$

6. Side wall height = $y_2+y_j/3$

Example Problem

Given: Same conditions as VII-D
10X6 RCB, Q=417 cfs, S =6.5%
Elevation of outlet invert z_o=100 ft.
V_o=27.8 fps, y_o=1.5 ft.
Downstream channel is a 10 ft. bottom trapezoidal channel with 2:1 side slopes

Find: Dimensions for a SAF Basin

Solution:

1. Use rectangular basin with no flare

2. Determine basin elevation using design procedure in section IV-B, Supercritical Expansion Into Hydraulic Jump Basins.

Steps from IV-B:

1. V_o=27.8 fps, y_o=1.5 ft., Fr_o=4

2. Downstream channel TW=y_n=1.9 ft., V_n=15.9 fps

3. From equations VII-G-2&4:
$y_j=y_1[(1+8Fr_1^2)^{1/2}-1]/2=1.5[(1+8x16)^{1/2}-1]/2=7.8$ ft.
$y_2=(1.1-Fr_1^2/120)y_j=(1.1-16/120)7.8=7.5$ ft.

4. Since y_2>TW, 7.5>1.9 drop the basin

5. Use z_1=91.5 ft.

A. W_B=10 ft., $S_T=S_S=.5$

B. W_B-OK no flare

C. $Q=10y_1[2g(100-91.5+1.5-y_1)+27.8^2]^{1/2}$
$Q=10y_1[64.4(10-y_1)+772.8]^{1/2}$
$y_1=1.14$ ft.
$V_1=417/1.14(10)=36.6$ fps

D. $Fr_1=36.6/(g1.14)^{1/2}=6.04$

E. $y_j=y_1[\sqrt{(1+8Fr^2)}-1]/2=1.14[\sqrt{1+8(6.04)^2}-1]/2=9.2$ ft.
equation VII-G-5, $y_2=.85y_j=.85(9.2)=7.8$ ft.

F. Equation VII-G-3, $L_B=4.5y_j/Fr^{0.76}$
$=4.5(9.2)/3.9=10.5$ ft.

$L_T=(z_o-z_1)/S_T=(100-91.5)/.5=17$ ft.
$z_3=[100-(10.5+17-91.5/.5).065]/1.13=97.4$ ft.

G. $y_2+z_2=99.3$
$TW+z_3=99.3$ OK

6. $L_T=17$ft., $L_B=10.5$ ft.
$L_S=(z_3-z_2)/S_S=(97.4-91.5)/.5=12$ ft.
$L=17+10.5+12=39.5$ ft.

7. $Fr_o=4$ from figure IV-B-5, $y_o/r=.1$
$r=1.5/.1=15$ ft.

Basin Width, $W_B=10$ ft.
Basin Elevation, $z_1=91.5$ ft.
Basin Length, $L_B\simeq 11$ ft.
Total Length, $L\simeq 40$ ft.
Incoming depth, $y_1=1.14$ ft.
Incoming $Fr_1=6.04$
Theoretical jump height, $y_j=9.2$ ft.
Jump height, $y_2=7.8$ ft.

3. Chute Blocks:
$h_1\simeq 1.2$ ft.
$W_1\simeq .75y_1=.9$ ft.$=W_2$
$N_c=W_B/2W_1=10/2(.9)=5.6$, use 6 blocks
Adjusted $W_1=W_B/2N_c=10/2(6)=.8$ ft.
This gives 5 full blocks, 6 spaces, and a half block at each wall.

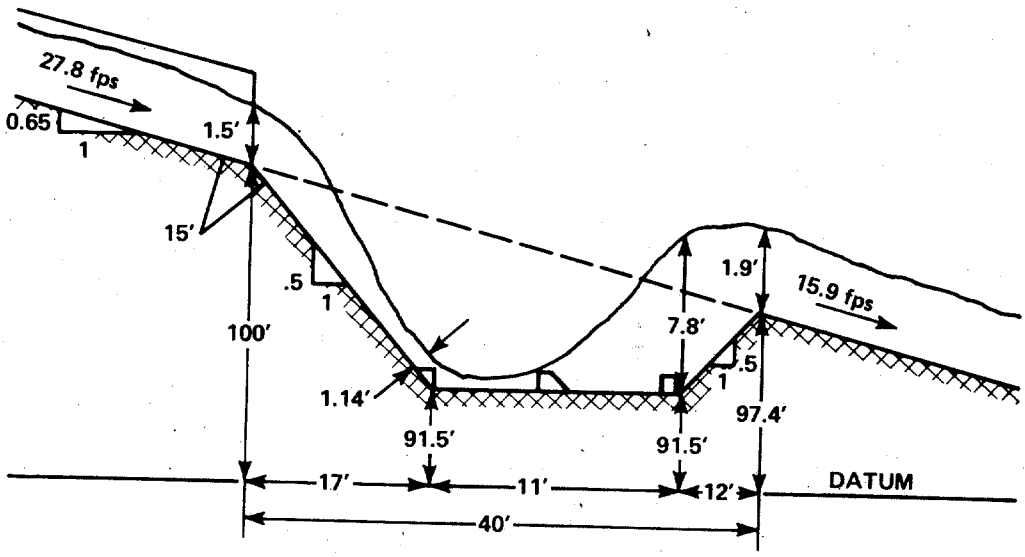
4. Baffle Blocks:
$h_3\simeq y_1=1.2$ ft.
$W_3\simeq .75y_1=.9$ ft.$=W_4$
Basin Width, $W_{B2}=W_B+2L_B/3z=10+0=10$ ft.
$N_B=W_{B2}/2W_3=10/2(9)=5.6$ ft., use 6 blocks
Adjust $W_3=W_4=10/2(6)=.8$ ft.
Total block width $=6(.8)=4.8$ ft.
Check percent $4.8/10=.48$, $.4<.48<.55$ OK
This gives 6 blocks, 5 spaces, and a half space at each wall
Distance from chute block $=L_B/3=11/3=3.7$ ft.

5. End Sill:
$h_4 = .07y_j = .07(9.2) = .6$ ft.

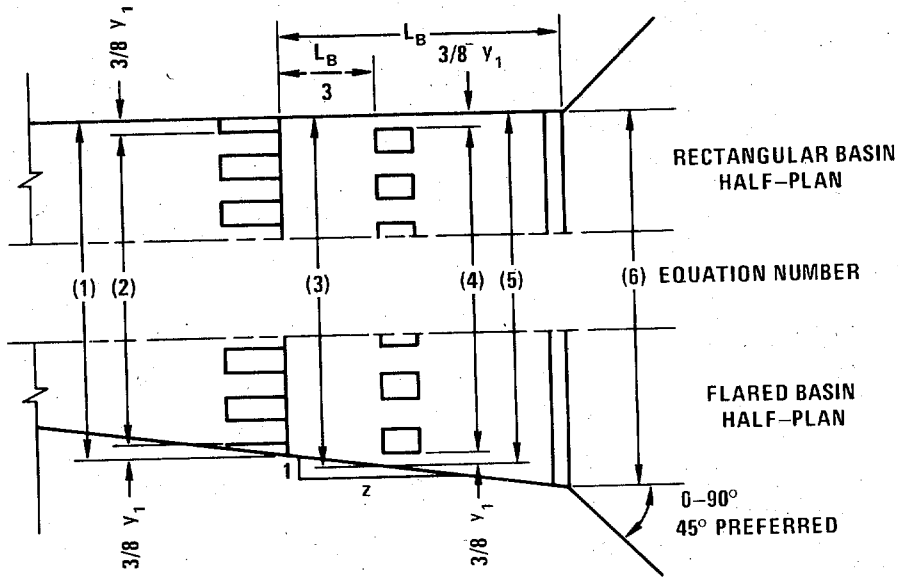
6. Side Wall:

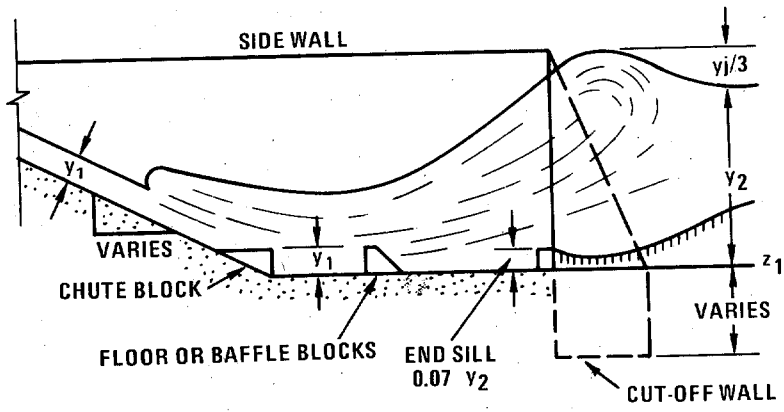
Height$=y_2+y_j/3=7.8+9.2/3=11$ ft.

VII-G-1. Blaisdell, F.W., THE SAF STILLING BASIN, U.S. Government Printing Office, 1959.



EXAMPLE PROBLEM SKETCH (VII-G)





(1) W_B = BASIN WIDTH UPSTREAM

(2) n BLOCKS AT 3/4 y_1 ±

(3) 0.40 W_{B2} ≤ AGGREGATE BLOCK WIDTH ≤ 0.55 W_{B2}

(4) n BLOCKS AT 3/4 y_1 $\frac{W_{B2}}{W_B}$ ±

(5) $W_{B2} = W_B + 2L_B/3z$

(6) $W_{B3} = W_B + 2L_B/z$

FIGURE VII-G-1. SAF STILLING BASIN-FROM REFERENCE VII-G-1

Comparison of baffle block basins:

The three USBR basins, Types II, III, and IV, and the St. Anthony Falls basin (SAF) were all designed using the same flow conditions:

Box Culvert 10x6
Discharge, Q=417 cfs
Velocity, V_o=27.8 fps
Depth, y_o=1.5 feet
Slope, S_o=6.5%
Froude Number, Fr_o=4.0

Type*	y_1	Fr_1	TW Req'd (ft)	L_B (ft)	L (ft)	Basin Elevation (ft)**
USBR						
II	1.0	7.6	11	48	97	84.5
III	1.04	6.9	9.7	26	67	87.5
IV	1.0	7.5	10	63	113	85
SAF	1.1	6.0	7.8	11	40	91.5

*All basins same width, 10 feet, rectangular, with constant cross section.
**The reference elevation of 100 ft. is the culvert outlet invert

VIII-A. CONTRA COSTA ENERGY DISSIPATOR

INTRODUCTION

The Contra Costa energy dissipator (VIII-A-1) was developed at the University of California, Berkeley, in conjunction with Contra Costa County, California. The dissipator was developed to meet the following conditions: (1) to reestablish natural channel flow conditions downstream from the culvert outlet; (2) to have self-cleaning and minimum maintenance properties; (3) to drain by gravity when not in operation; (4) to be easily and economically constructed; and (5) to be applicable for a wide range of culvert sizes and operating conditions.

Field experience with this dissipator has been very limited. Its use should not be extended beyond the range of the model tests.

The dissipator is best suited to small and medium size culverts of any cross section where the depth of flow at the outlet is less than the culvert height. It is applicable for medium and high velocity effluents. The dissipator design is such that the flow leaving the structure will be at minimum energy when operating without tailwater. When tailwater is present, the performance will improve. A sketch of the dissipator arrangement is shown in Figure VIII-A-1.

DESIGN DISCUSSION

The initial step is to determine the equivalent depth of flow (y_e) at the culvert outfall: For box culverts $y_e=y_n$ or y brink. For oval, elliptical, circular, or other shapes, convert the areas of flow at the culvert outfall to an equivalent rectangular cross section with a width equal to twice the depth of flow $y_e= (A/2)^{1/2}$. In so doing, the design information is applicable to oval, elliptical, and circular culverts.

The Froude number is computed by using y_e rather than the actual depth of flow at the culvert outfall, $Fr=V/\sqrt{gy_e}$. By entering Figure VIII-A-2 with Fr^2, and an assumed value of L_2/h_2, a trial height of the second baffle, h_2, can be determined. The equation of the lines in figure VIII-A-2 has been changed slightly from that presented in the original

paper to compensate for replacing the depth of flow in a circular pipe (y_o) by the equivalent rectangular flow depth, y_e. The original equation

$$L_2/h_2Fr^2=1.2(h_2/y_o)^{1.8}$$

has been revised to

$$L_2/h_2Fr^2=1.35(h_2/y_e)^{1.8} \quad \ldots\ldots\ldots \text{VIII-A-1}$$

The remaining two equations, used for determining other dimensions of the dissipator, remain unchanged; these are given below and plotted on figures VIII-A-3 and 4.

$$L_3/L_2=3.75(h_2/L_2)^{0.68} \quad \ldots\ldots\ldots\ldots \text{VIII-A-2}$$

$$y_2/h_2=1.3(L_2/h_2)^{0.36} \quad \ldots\ldots\ldots\ldots \text{VIII-A-3}$$

The three equations may be used for proportioning the dissipator but figures VIII-A-2, 3, and 4 are more convenient and practical to use for design purposes.

The value of L_2/h_2 varied from 2.5 to 7.0 in the experiments and a value of 3.5 is recommended for best performance wherever economically feasible. The value of h_2/y_e should always be greater than unity. After determining values of h_2 and L_2 from figure VIII-A-2, the dimension L_3 can be obtained by entering figure VIII-A-3 with L_2 and the assumed value of L_2/h_2. Should the dimensional proportioning thus obtained be uneconomical or fail to properly fit the site, a second value of L_2/h_2 is assumed and the process repeated.

From figure VIII-A-1, the height h_1 of the first baffle is half the height of the second baffle h_2, and the position of the first baffle is half way between the culvert outlet and the second baffle or $L_2/2$. Side slopes of the trapezoidal basin for all experimental runs were 1:1. The width of basin (W) may vary from one to three times the width of the culvert. The floor of the basin should be essentially level. The height of the end sill may vary from $0.06(y_3)$ to $0.10(y_3)$. After obtaining satisfactory basin dimensions, the approximate maximum water surface depth, y_2, without tailwater, can be obtained from figure VIII-A-4.

DESIGN PROCEDURE

The dissipator design should only be applied within the design limitations:

$2.5<L_2/h_2<7.0$
$D<W<3D$
$y_o<D/2$
and side slopes of 1:1

The following steps outline the procedure for the design of the Contra Costa energy dissipator:

1. Analyze flow conditions that are expected to occur at the outfall of culvert for the design discharge.

   If the depth of flow at the outlet is one half culvert diameter or less, the Contra Costa dissipator is applicable.

2. Compute y_e:
   $y_e = y_o$; for rectangular
   $y_e = (A/2)^{1/2}$; for other shapes

3. Compute the parameter $Fr^2 = V_o^2/gy_e$

4. The width of the basin floor is selected to conform to the natural channel. If there is no defined channel, the width is set at a maximum of three times the culvert width.

5. Assume a value of L_2/h_2 between 2.5 and 7 and with the aid of figures VIII-A-2 and 3, determine h_2, L_2 and L_3: Give due consideration to the optimum value of $L_2/h_2 = 3.5$ as well as to the engineering and economic requirements of the particular situation. Repeat the procedure, if necessary, until a dissipator is defined which optimizes the design requirements. The first baffle height (h_1) is $0.5h_2$.

6. The approximate maximum water surface depth without tailwater can be obtained for the final arrangement from figure VIII-A-4.

7. Riprap may be necessary downstream especially for the low tailwater cases. See chapter II for design recommendations. Freeboard and a cutoff wall also should be considered to prevent overtopping and undermining of the basin.

<u>Example Problem</u>

Given: Diameter of culvert 4.0 feet
$Q = 300$ c.f.s.
$y_o = 2.3$ ft.
$V_o = 40$ f.p.s.
$A = 7.50$ sq. ft.

Find: A Contra Costa energy dissipator dimensions

Solution:

1. y_o=2.3 ft.≈D/2, OK

2. $y_e=(7.5/2)^{1/2}$=1.94 ft.

3. $Fr^2=V_o^2/g(y_e)=40^2/32.2x1.94=25.6$

4. W=2D=8 ft.

5. By assuming L_2/h_2=3.5 and entering figure VIII-A-2 with Fr^2=25.6, a value of 3.50 is obtained for h_2/y_e. Therefore,

   h_2= 3.50(1.94) = 6.8 ft. and

   h_1= .5(6.8) = 3.4 ft.

   L_2= 3.5(6.8) = 23.8 ft.

   Entering figure VIII-A-3 with L_2 = 23.8 feet and L_2/h_2 = 3.5, L_3 is found to be 38 feet. If the maximum rise in water surface, without tailwater, is desired, this can be obtained from figure VIII-A-4. Strictly speaking, the value of y_2 from this chart applies for a bottom width W/D = 2 and 1:1 side slopes. For the problem at hand, these values are essentially correct.

6. Entering figure VIII-A-4 with h_2 = 6.8 ft. and L_2/h_2 = 3.5, gives y_2 = 13.9 feet. If the height of end sill is based on this value,

   h_3 = 0.09(y_2)=0.09(13.9)=1.25 ft.

   If the above proportioning proves compatible with the topography at the site and the dissipator is economically satisfactory, the above dimensions are final; if not, a different value of L_2/h_2 is selected and the design procedure repeated. Assuming the first computation is acceptable, the various dimensions of the dissipator are:

Dimensions in Feet and Inches

	W	First Baffle	Second Baffle	End Sill
Hor. Distance	8-0	12-0	24-0	62-0
Height	-	3-6	7-0	1-3
Length (baffle)	-	8-0	8-0	10-6

VIII-A-1. Keim, R. S., THE CONTRA COSTA ENERGY DISSIPATOR. Journal of Hydraulics Division, A.S.C.E. paper 3077, March 1962, p. 109.

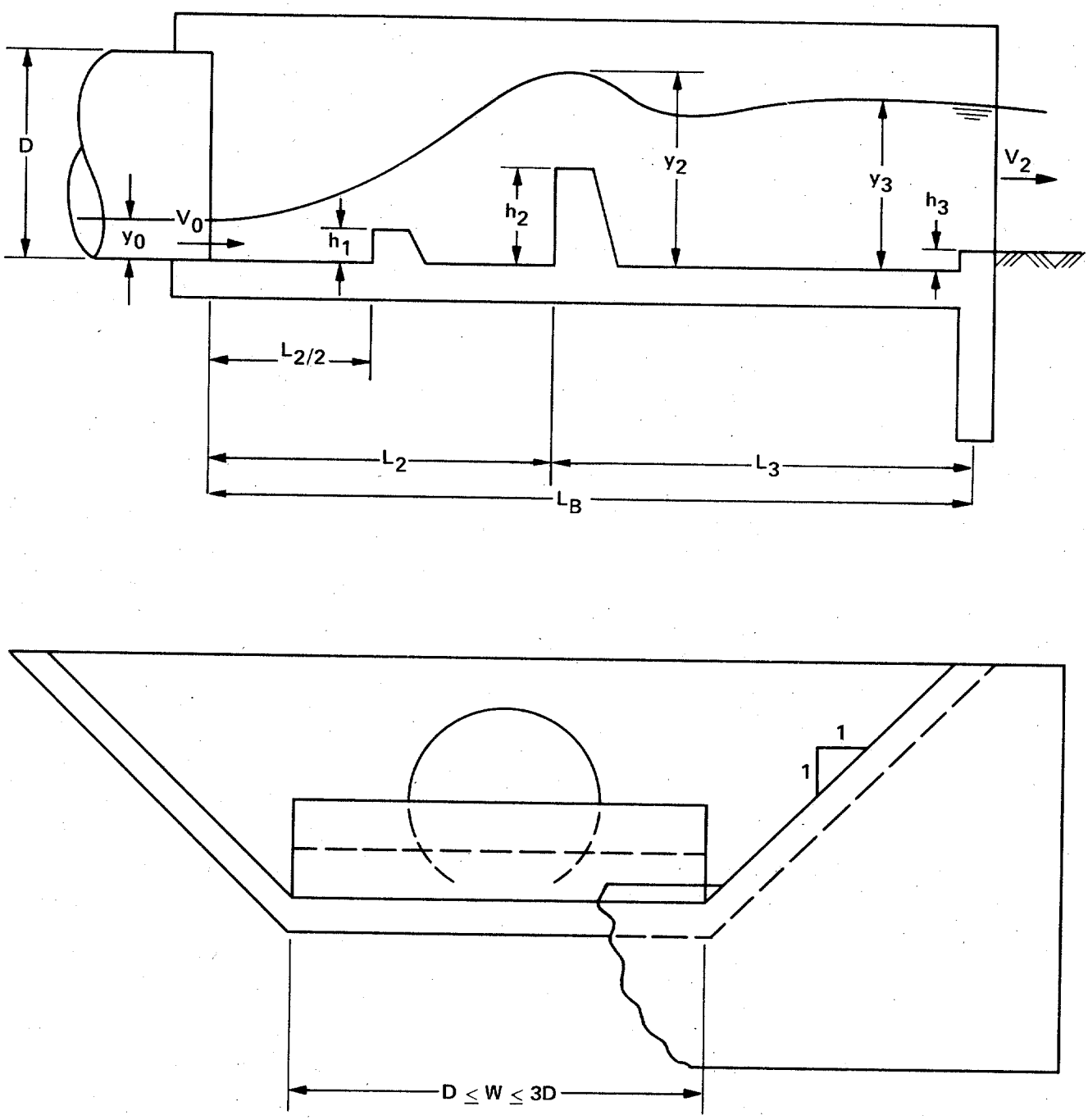


FIGURE VIII–A–1. CONTRA COSTA ENERGY DISSIPATOR

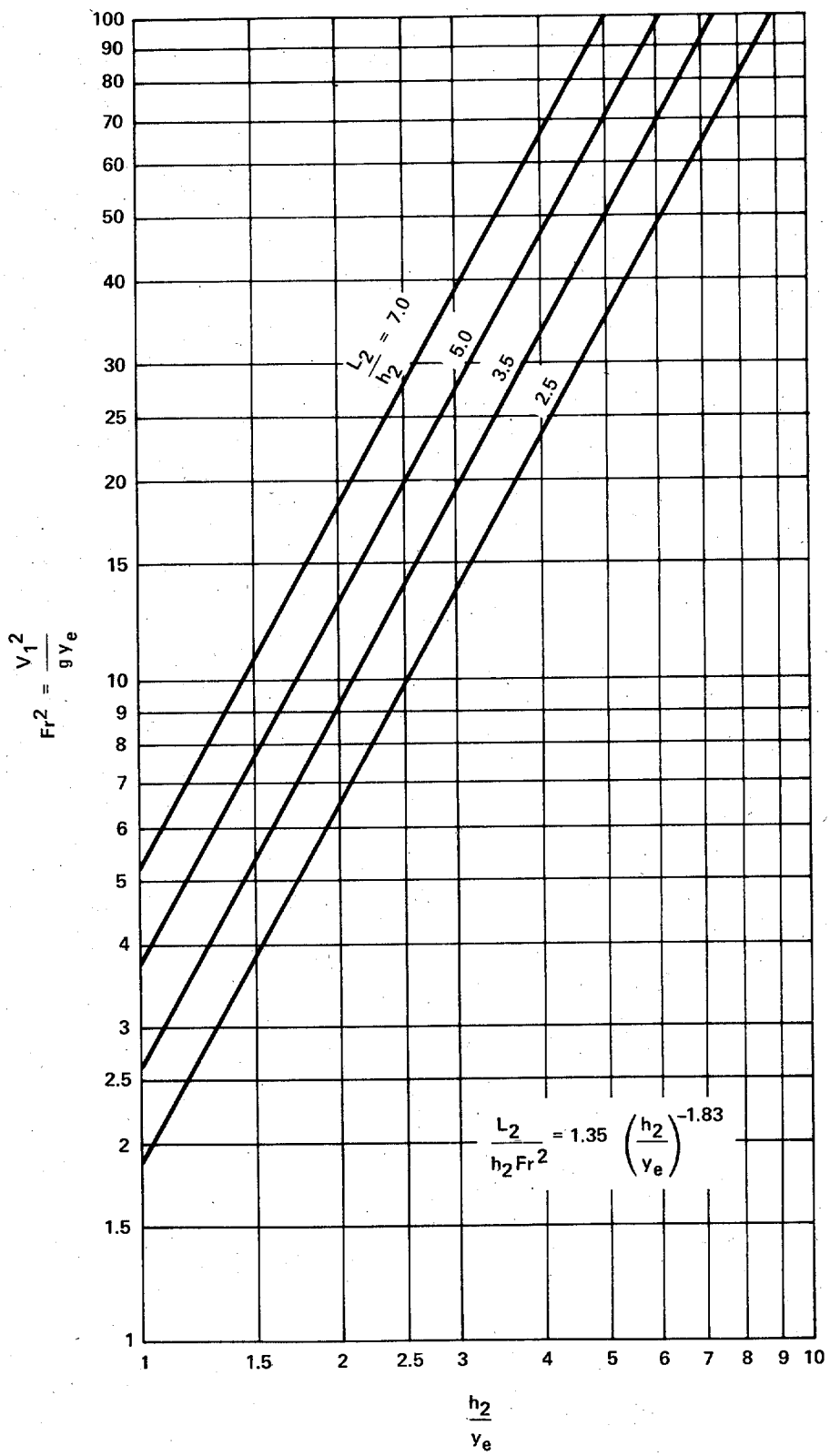


FIGURE VIII–A–2. BAFFLE HEIGHT, CONTRA COSTA ENERGY DISSIPATOR

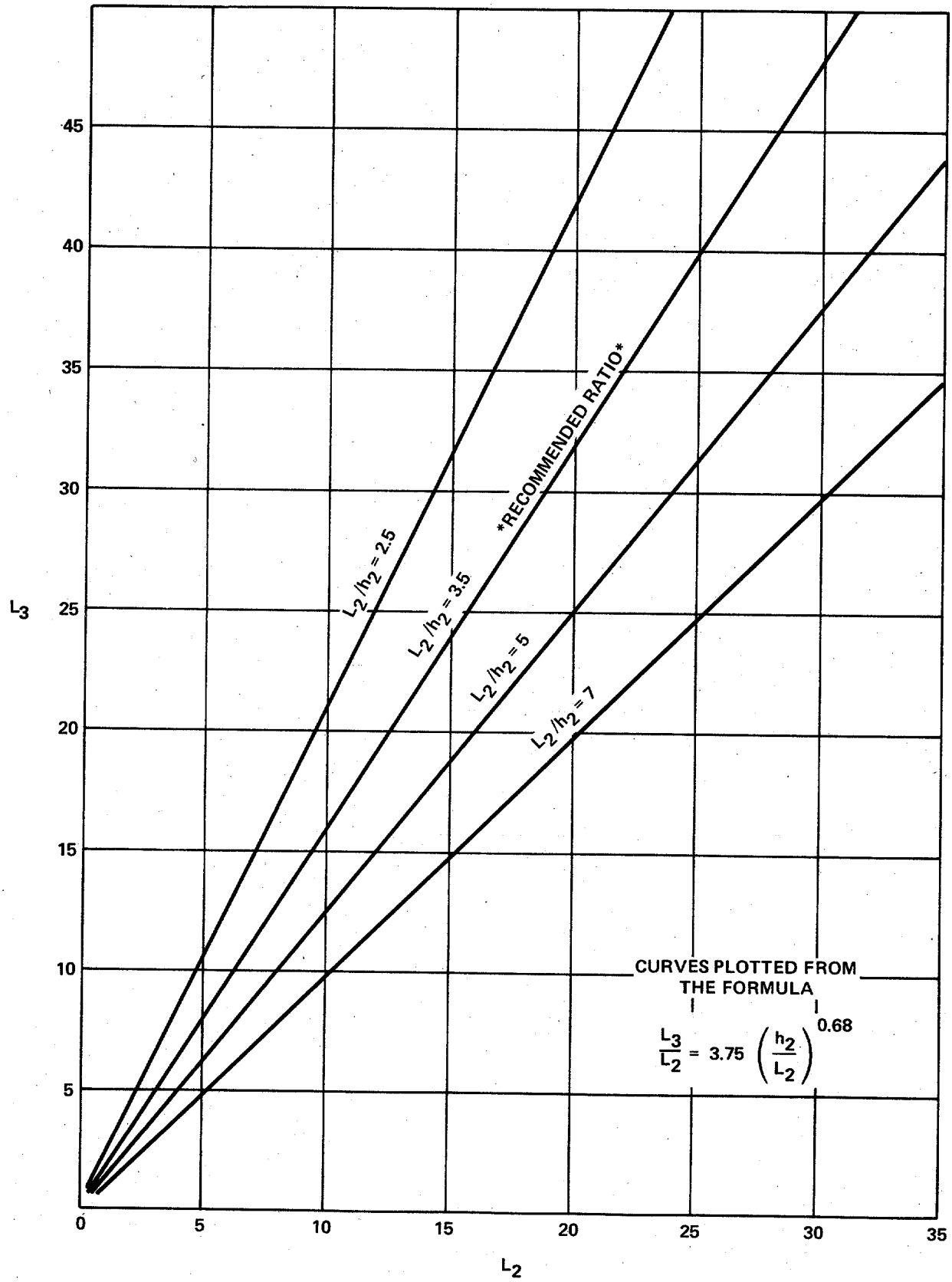


FIGURE VIII--A--3. LENGTH OF CONTRA COSTA ENERGY DISSIPATOR

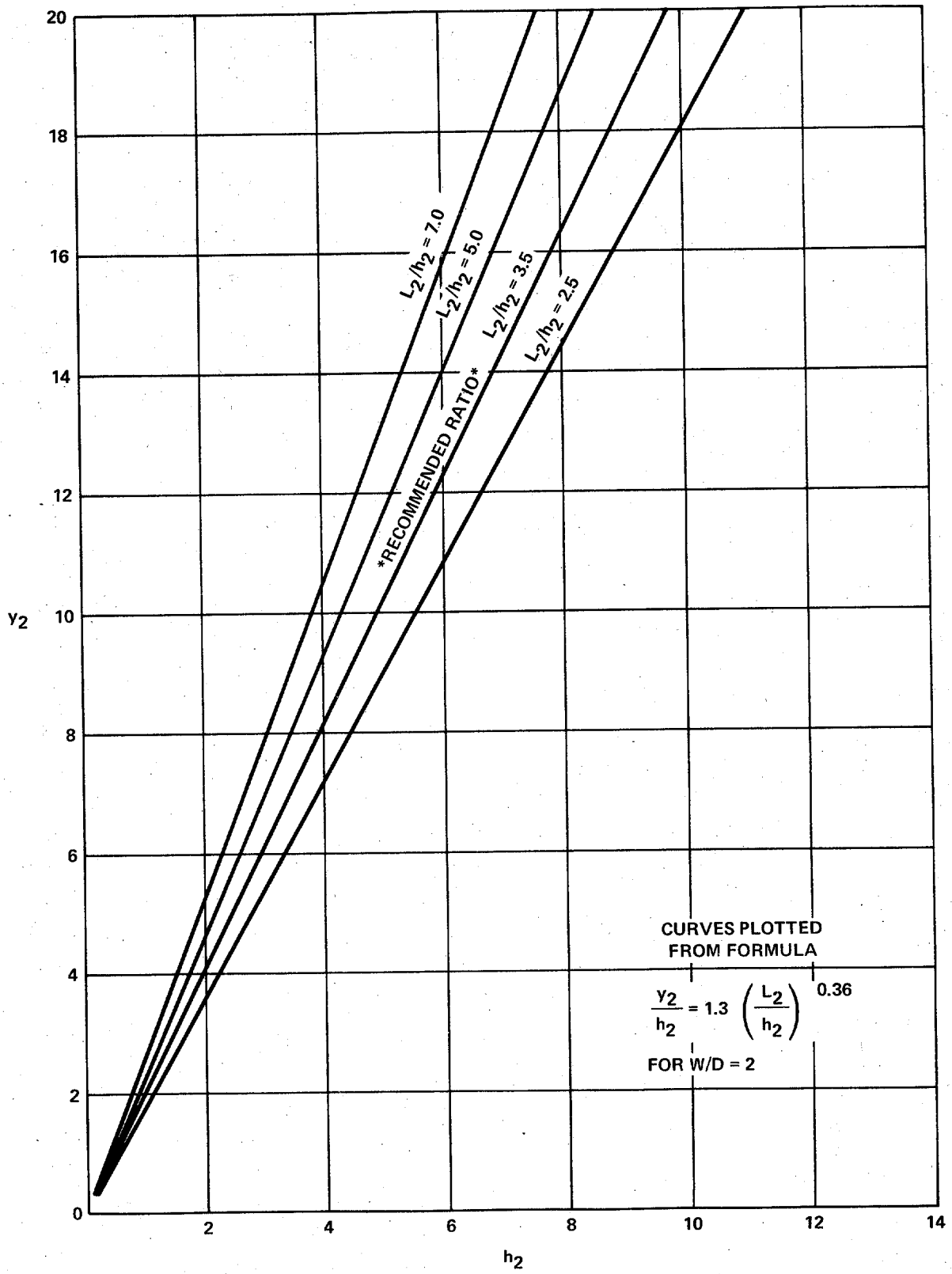


FIGURE VIII–A–4. DEPTH OVER BAFFLE–CONTRA COSTA ENERGY DISSIPATOR

VIII-B. HOOK TYPE ENERGY DISSIPATOR

The Hook or Aero-type energy dissipator was developed at the University of California in cooperation with the California Division of Highways and the Bureau of Public Roads (VIII-B-1). The dissipator was developed primarily for large arch culverts with low tailwater but can be used with box or circular conduits without difficulty. The applicable range of Froude numbers, is from 1.8 to 3.0. Two hydraulic model studies were made in developing the design. The first used a basin with wingwalls warped from vertical at the culvert outlet to side slopes of 1.5:1 at the end sill, and a tapered basin floor as shown in figure VIII-B-1. In the second study, the warped basin was replaced with a trapezoidal channel of constant cross section such as shown in figure VIII-B-5.

WARPED WINGWALLS TYPE BASIN

The higher the velocity ratio, V_o/V_B, the more effective the basin is in dissipating energy and distributing the flow downstream. A flare angle of 5.5 degrees per side (α = 0.10) is the optimum value for Fr>2.45. Increasing the length beyond $L_B = 3W_o$ does not improve basin performance.

The design is a routine operation except for determining the width of the hooks. Judgment is necessary in choosing this dimension to insure that the width is sufficient for effective operation, but not so great that flow passage between the hooks is inadequate. Although a value of W_4/W_o of 0.16 is recommended, each design should be checked to see that the spacing between hooks is 1.5 to 2.5 times the hook width. The effectiveness of the dissipator falls off rapidly with increasing Froude number regardless of hook width, for flare angle exceeding 5.5 degrees. Therefore, the flare angle should be limited to 5.5 degrees per side, if possible.

DESIGN RECOMMENDATIONS - WARPED WINGWALL TYPE

The tangent of the flare angle should equal 0.10 (tan = 0.10) for the highest velocity reduction. At larger flare angles, the velocity ratio remains constant or drops off rapidly.

The best range of L_1/L_B for the A-hooks is 0.75 to 0.80.

The best range of W_2/W_1 values for the above L_1/L_B range is 0.66 to 0.70.

The best range of L_2/L_B for the B-hook is 0.83 to 0.89.

The best width of opening at the end sill ratio, W_5/W_6, is 0.33.

The best height of end sill is approximately two-thirds the flow depth at the culvert outlet, $h_4/y_e = 0.67$, $y_e=(A/2)^{1/2}$.

Test results did not indicate a specific optimum with regards to the height of the side sill. A value of h_5/h_6 of 0.94 may be used.

For wide hooks the velocity reduction will be a maximum, but the apparent maximum flow rate (i.e., the flow rate just before excessive over-topping of the wingwalls) will be reduced. In addition, as the hooks become wider, the spacing between them and the walls decreases and may not be sufficient for the passage of debris. For these reasons a thickness ratio, $W_4/y_e = 0.16$, the minimum value tested, is recommended. The design should be adjusted to obtain the proper spacing.

The basin length cannot be assigned a fixed value since it depends on site conditions. However, the shorter basin lengths give higher velocity reduction over most of the range of Froude numbers tested.

The recommended hook dimensions are shown in figure VIII-B-2.

The height of wingwalls (h_6) should be at least twice the flow depth at the culvert exit or $2(y_e)$. This was the height used in the study to determine the apparent maximum flow rate. This apparent maximum flow rate was the condition used to determine the velocity ratios and Froude numbers. Therefore, the prototype basin should be provided with additional freeboard.

Depending on final velocity and soil conditions, some scour can be expected downstream of the basin. The designer should, where necessary, provide riprap protection in this area. Chapter II contains design guidance for riprap.

Where large debris is expected, armor plating the upstream face of the hooks with steel is recommended.

DESIGN PROCEDURE - WARPED WINGWALL TYPE

See figure VIII-B-1.

1. Compute the culvert outlet conditions.

    a. Velocity, V_o

    b. Depth, y_e

c. Froude number, $Fr=V_o/\sqrt{gy_e}$

d. Check $1.8<Fr<3.0$, if not within this range, select another type of dissipator

2. Compute the flow conditions in the downstream channel.

a. Velocity, V_n

b. Depth, y_n

c. Ratio V_o/V_n

3. Select the tangent of the flare angle, α. The angle α=Arc tan (0.10) is recommended and determine L_B. Assume a valve for W_6 and compute L_B from:

$$L_B = (W_6 - W_o)/2\tan\alpha$$

Where the downstream channel is defined, W_6 should be approximately equal to the channel width.

4. Compute.

a. L_1 using $0.75<L_1/L_B<0.80$ distance to first hooks

b. W_1 from width at first hooks $W_1=2L_1(\tan\alpha)+W_o$ and W_2 using $0.66<W_2/W_1<0.70$ distance between first hooks.

c. L_2 using $0.83<L_2/L_B<0.89$=distance to second hook

d. $W_5=0.33W_6$=width of slot in end sill

e. $h_4=0.67y_e$=height of end sill

f. $h_5=0.94h_6$ where h_6 is equal to twice the incoming flow depth y_e, $h_6=2(y_e)$ minimum

g. $W_4=0.16W_o$=thickness or width of hooks

h. $h_6=2y_e$

i. $W_3=(W_2-W_4)/2$

5. Compute other hook dimension: Figure VII-B-2

a. $h_1=y_e/1.4$

b. $h_2=1.3h_1$

c. $h_3=y_e$

d. $\beta=135°$

e. $r=0.4h_1$

6. Assess scour potential downstream based on soil condition and outlet velocity. Find V_o/V_B from figure VIII-B-3 and compare with V_o/V_n from step 2c. For Riprap projects see chapter II.

7. Where large debris is expected, the upstream face of the hooks should be armored.

SAMPLE DESIGN FOR A BASIN WITH WARPED WINGWALLS

Given: A long concrete arch culvert which is 12 feet wide and 12 feet from the floor to the crown of the semi-circular arch. $S_o= 0.020$, $n = 0.012$, $Q = 2{,}700$ cfs, $y_e = 6.0$ feet and $V_o = 37.5$ fps.

The normal stream channel is trapezoidal with bottom width of 20 feet and side slopes of 1 1/2:1, $S_o = 0.020$, $n = 0.030$, $V_n = 16.1$ fps, and $y_n = 5.5$ feet.

Find: Hook energy dissipator dimensions.

Solution:

1. Calculate the Froude number at the culvert exit $V_o=37.5$ fps, $y_e=6$ ft.

$$Fr_o = V_o/(gxy_e)^{1/2}= 37.5/(32.2x6)^{1/2}= 2.71$$
$$1.8<2.71<3\text{-OK}$$

2. $V_n = 16.1$ fps, $y_n = 5.5$ ft.
$V_o/V_n = 37.5/16.1 = 2.33$

3. For best energy dissipation, the flare angle of the bottom of the transition section should be tan $\alpha= 0.10$ per side. In expanding at this rate from a culvert width of 12 feet to a channel width of 20 feet, the basin must be 42 feet long or about 3.5 times the width of the culvert (see figure VIII-B-4).

4. Following the order of the design recommendations, the position, size, and spacing of the hooks can be determined:

a. For distance to the two A hooks $L_1/L_B = 0.75$, or $L_1 = 0.75(42) = 31.5$ feet.
$W_1 = 2L_1(\tan\alpha)+W_o = 2(31.5)(0.1)+12 = 18.3$ feet

b. The spacing between the A hooks is: $W_2/W_1 = 0.66$, or $W_2 = 0.66(18.3) = 12.1$ feet.

c. The distance to the B hook is $L_2/L_B = 0.84$, or $L_2 = 0.84(42) = 34.9$ feet.

d. The width of opening in the end sill is $W_5/W_6 = 0.33$, or $W_5 = 0.33(20)=6.7$ feet.

e. The best height of end sill is $h_4/y_e = 0.67$, or $h_4 = 0.67(6.0) = 4.0$ feet.

f. The same height of end sill is carried up the sloping sides to: $h_5/h_6 = 0.94$ or $h_5 = 0.94[2(y_e)] = 11.25$ feet.

g. For width of hooks, according to the recommendation, $W_4/W_B = 0.16$ or $W_4 = 0.16(12.0) = 1.88$ feet.

h. $W_5 = (W_2-W_4)/2 = 5.1$ feet

i. $h_6 = 2y_e = 2(6) = 12$ feet

5. The hook dimensions are proportioned according to the sketch on figure VIII-B-2:

a. $h_1 = y_e/1.4 = 4.29$ feet

b. $h_2 = 1.28(h_1) = 5.49$ feet

c. $h_3 = y_e = 6.0$ feet

d. $r = 0.4(h_1) = 1.72$ feet and

e. $\beta = 135°$

The dissipator design is shown on figure VIII-B-4, dimensioned in feet and inches. The spacing between hooks is twice the hook width which is satisfactory.

From figure VIII-B-3, with the Froude number of 2.71 and $L=3.5W_o$, V_o/V_B will be less than 1.9 making $V_B \approx 37.5/1.9=19.7$ fps. This is somewhat higher than the normal velocity in the downstream channel indicating reprap protection may be desirable.

STRAIGHT TRAPEZOIDAL TYPE BASINS

A second set of tests was made with the hooks and end sill placed in a uniform trapezoidal channel as shown on figure VIII-B-5. The width and shape of the cross section in this case closely resembles the natural channel before installation of the culvert. It was found that some of the former values assigned to parameters needed to be changed for the trapezoidal basin of uniform width. For example, the hooks and sill are upstream closer to the outfall of the culvert. The author (VIII-B-1) presents several charts depicting the effect of various variables on the performance of the dissipator. These show that for a given discharge condition widening the basin actually produces some reduction in the velocity downstream, and flattening the side slopes improves the performance for values of the Froude number up to 3.0.

DESIGN RECOMMENDATIONS

The following dimensions are recommended for the trapezoidal hook dissipator (see figures VIII-B-5 or 6 for identification of symbols):

$L_B = 3.0W_o$
$L_1 = 1.25W_o$
$L_2 = 2.1W_o$
$W_2 = 0.65W_o$

$W_5/W_6 = 0.33$
$h_4/y_1 = 0.67$
$h_5/h_6 = 0.70$

Where

$h_6 = 3.33\ y_e$ (for 1 1/2:1 side slopes)
$h_6 = 2.69\ y_e$ (for 2:1 side slopes)
W_o = Width of rectangular culvert (for circular and irregular-shaped culverts $W_o = 2y_e$ where $y_e = (A/2)^{1/2}$
W_6 = Bottom width of trapezoidal channel
W_6/W_o can be as large as 2 without affecting performance--see figure VIII-B-7
$W_4 = 0.16W$
W_3/W_4 = should be unity or greater $W_3/W_4>1$

The height of hook (figure VIII-B-6) has been changed from the warped wingwall basin so that $h_2 = y_e$.

The design recommendations for the warped wingwall basin which concerns riprapping the downstream channel and armoring the hook faces, also apply to the straight trapezoidal design.

DESIGN PROCEDURE FOR STRAIGHT TRAPEZOIDAL TYPE BASINS

See figure VIII-B-5.

1. Compute the culvert outlet condition

   a. Width W_o = width of rectangular culvert, for circular or other shapes, $W_o=2y_e$ and $y_e=(A/2)^{1/2}$

   b. Velocity, V_o

   c. Depth, y_e

   d. Froude number, Fr

   e. Check $1.8<Fr<3.0$
      If outside this range select a different basin.

2. Compute the flow conditions in the downstream channel

   a. Velocity, V_n

   b. Depth, y_n, and

   c. Ratio V_o/V_n

3. Select side slope: 1 1/2:1 or 2:1 and compute L_B and W_6
   W_6/W_o can be as large as 2.0--
   see figure VIII-B-7.

   $$L_B=3.0W_o$$

4. Compute:

   a. $L_1=1.25W_o$; length to first hooks

   b. $W_2=0.65W_o$; width between first hooks

   c. $L_2=2.085W_o$; length to second hook

   d. $W_5=0.33W_6$; where W_6 is the bottom width of trapezoidal channel
      W_5=slot width at dissipator end

   e. $h_4=0.67y_e$; height of end sill

f. $h_5=0.70h_6$; where $h_6=3.33y_e$ for 1 1/2:1 side slopes and $h_6=2.69y_e$ for 2:1 side slopes

g. $W_4=0.16W_o$; thickness of hooks

h. Check $W_3/W_4 \geq 1$; where $W_3=(W_2-W_4)/2$

5. Compute other hook dimension figure VIII-B-6

a. $h_2=y_e$

b. $h_1=0.78y_e$

c. $h_3=1.4h_1$

d. $\beta=135°$

e. $r=0.4h_1$

6. Assess downstream scour potential. Determine V_o/V_B from figure VIII-B-7 and compare with V_o/V_n from step 2c. See chapter II for riprap recommendations.

7. When large debris is expected, consider armoring upstream face of hooks.

SAMPLE DESIGN--STRAIGHT TRAPEZOIDAL TYPE

Values which remain the same as in the previous example are:

1&2. $F_r = 2.71$ | $y_e = 6.0$ feet
$V_o = 37.5$ fps | $W_o = 12.0$ feet
$V_n = 16.1$ fps | $W_6 = 20$ feet
$V_o/V_n = 2.33$ | $y_2 = 5.5$ feet

3. Proportion a hook-type energy dissipator in a uniform trapezoidal channel with bottom width of 20 feet and 1 1/2:1 side slopes for the same inflow conditions as given in the former example.

$L_B = 3.0W_o = 3.0(12) = 36$ ft.

4. The other dimensions recommended for this dissipator are:

a. $L_1 = 1.25W_o = 15.0$ ft.

b. $W_2 = 0.65W_o = 7.8$ ft.

c. $L_2 = 2.1W_o = 25.2$ ft.

d. $W_5/W_6 = 0.33$, $W_5 = 0.33(20) = 6.67$ ft.

e. $h_4/y_e = 0.67$, $h_4 = 0.67(6.0) = 4.0$ ft.

f. $h_6 = 3.33\ y_e = 20.0$ ft.
$h_5/h_6 = 0.70$, $h_5 = 0.70(20) = 14.0$ ft.

g. $W_4 = 0.16W_o = .157(12) = 1.9$ ft.

h. $W_3 = (W_2-W_4)/2 = (7.8-1.9)/2 = 3$ ft.,
$W_3/W_4 = 3/1.9 = 1.6$ OK

5. Compute other hook dimensions

a. $h_2 = y_e = 6.0$ ft.

b. $h_1 = 0.78(y_e) = 4.68$ ft.

c. $h_3 = 1.4(h_1) = 6.55$ ft.

d. $\beta = 135°$

e. $r = 0.4(h_1) = 1.87$ ft.

A sketch of this energy dissipator with dimensions given in feet and inches is shown in figure VIII-B-8. The spacing of the hooks $W_3/W_4 = 1.0$ is permissible for this basin since the longitudinal distance between the A and B hooks is greater than for the warped wingwall type of dissipator.

From figure VIII-B-7, with a Froude number of 2.71 and $W_6/W_o = 20/12 = 1.67$, $V_o/V_B \simeq 2.0$ making $V_B \simeq 37.5/2 = 18.75$ fps. This is slightly higher than the normal channel velocity, V_2, indicating minimum riprap protection may be necessary.

The dimensions of the two basins, each designed for the same inflow and outflow conditions, are presented below.

DIMENSIONS IN FEET AND INCHES

Symbol	Warped Wingwall	Straight Trapezoidal
Y_e	6-0	6-0
W_o	12-0	12-0
L_B	42-0	36-0
L_1	31-6	15-0
L_2	35-0	25-0
W_2	12-0	8-0
h_4	4-0	4-0
W_5	6-8	6-8
W_6	20-0	20-0
h_6	12-0	20-0
h_5	11-4	14-0
W_4	2-0	2-0
W_3	5-0	3-0
h_1	4-3	4-8
h_2	5-6	6-0
h_3	6-0	6-6 1/2
r	1-9	1-10 1/2

VIII-B-1 MacDonald, T. C., MODEL STUDIES OF ENERGY DISSIPATORS FOR LARGE CULVERTS, Hydraulic Engineering Laboratory Study HEL-13-5, University of California, Berkeley, November 1967.

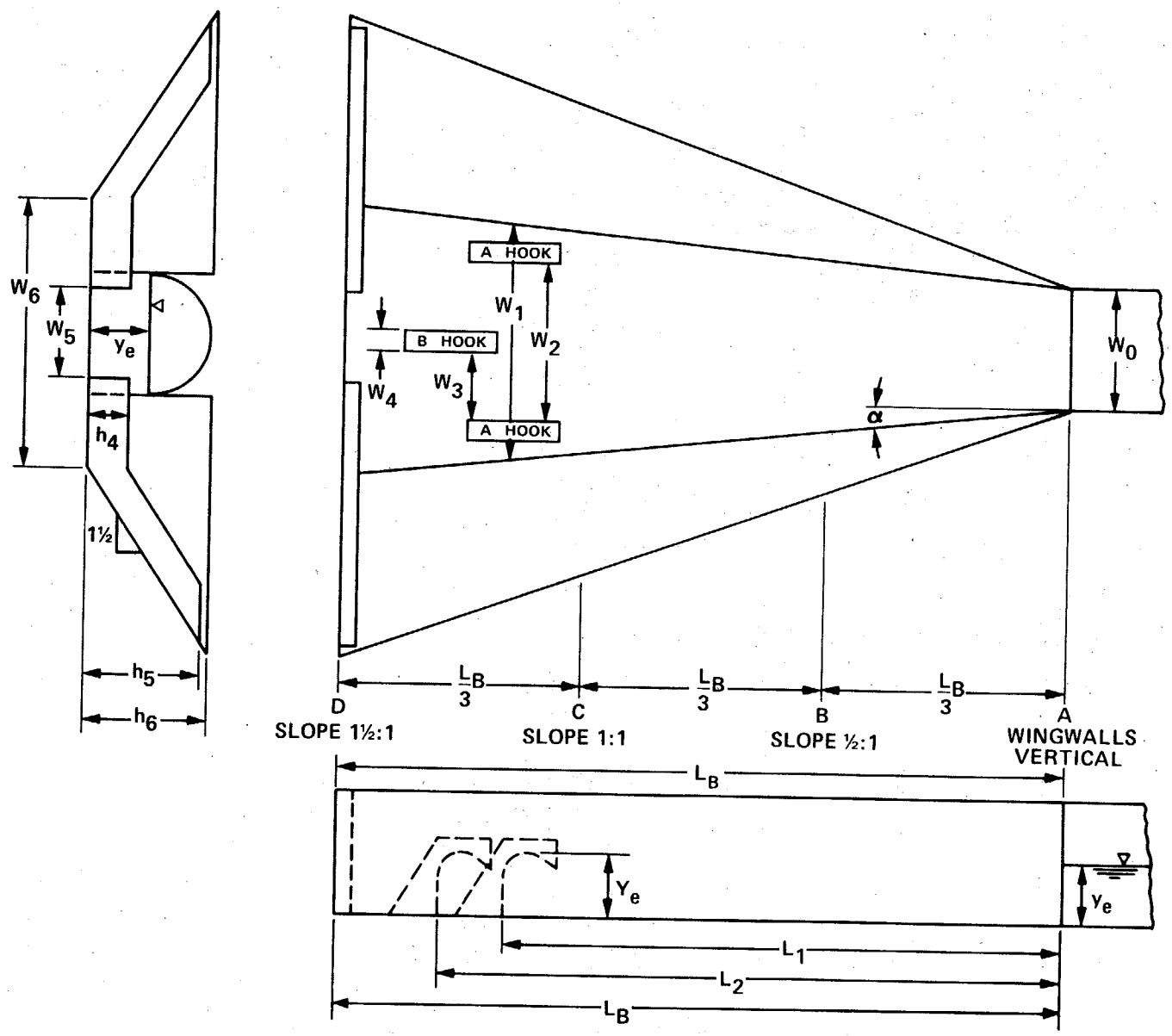
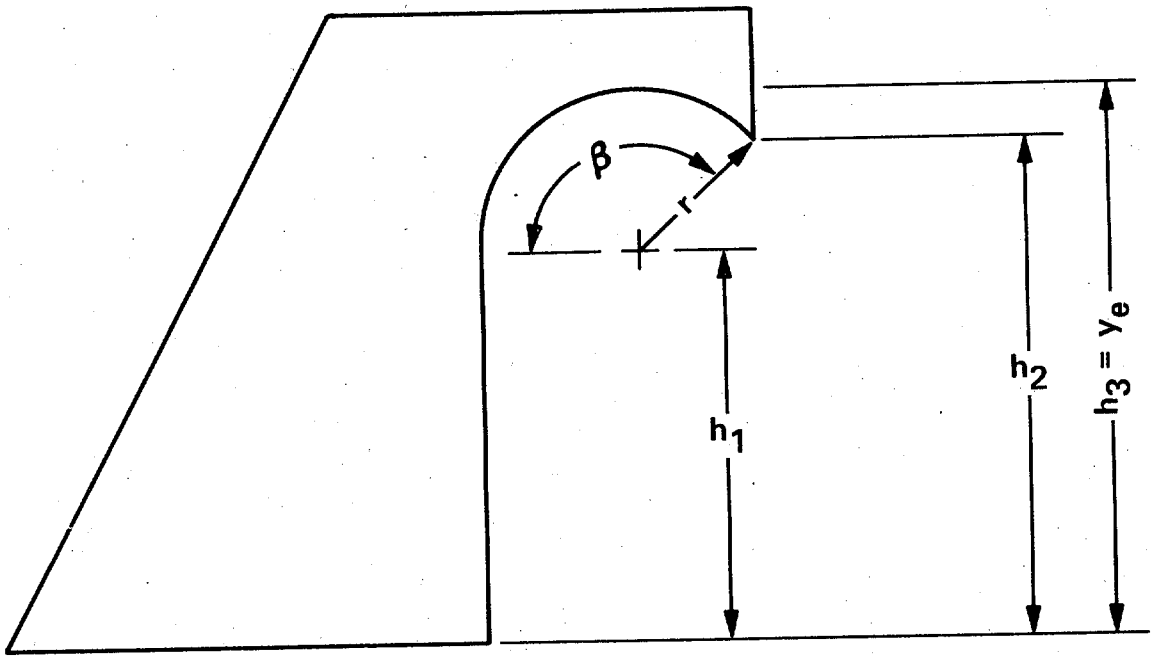


FIGURE VIII–B–1. WARPED WINGWALL TYPE BASIN



$h_3 = y_e$

$h_2 = 1.28\ h_1$

$h_1 = y_e/1.4$

$\beta = 135^o$

$r = 0.4\ h_1$

FIGURE VIII–B–2. HOOK FOR WARPED WINGWALL BASIN

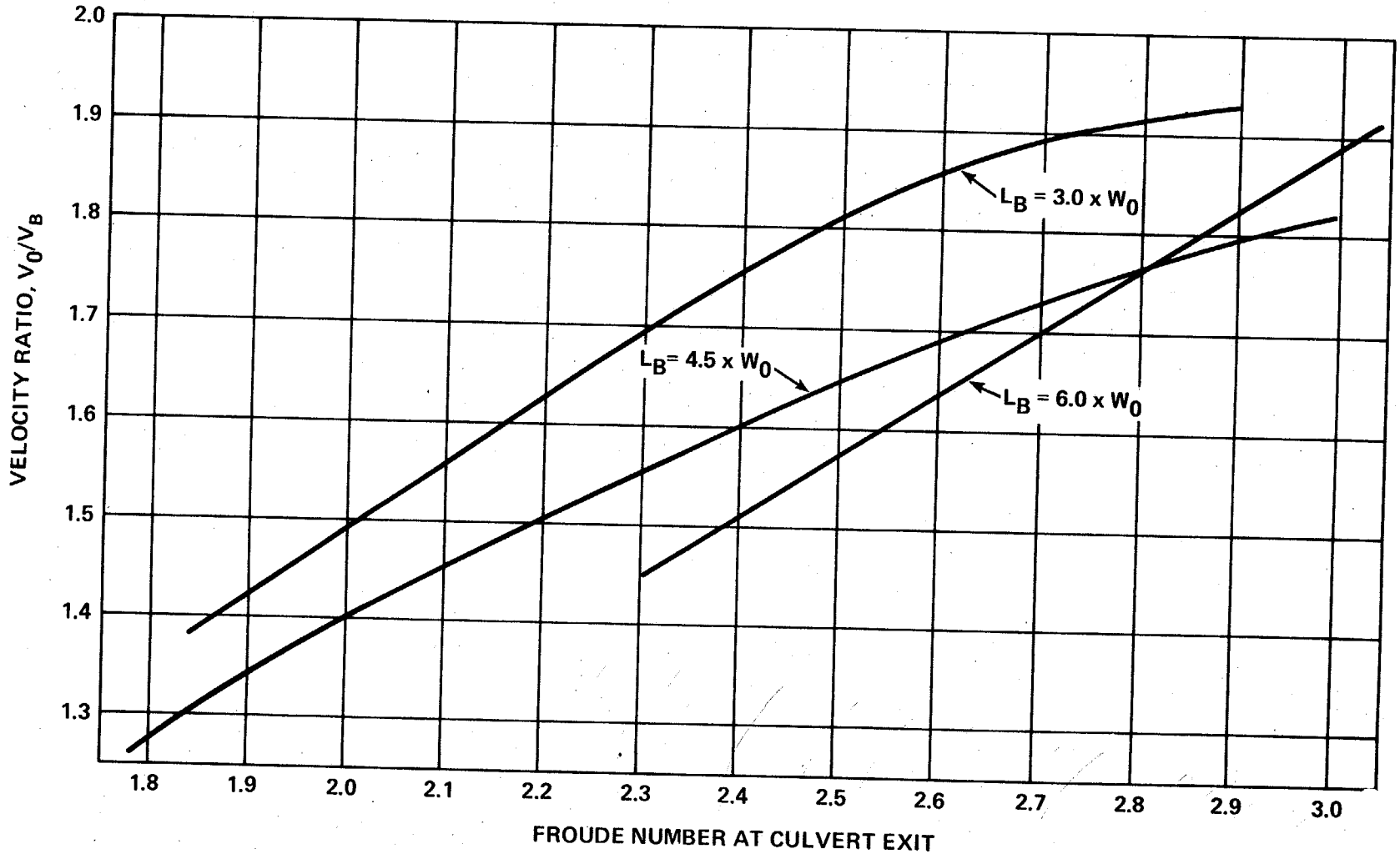


FIGURE VIII–B–3. FROUDE NUMBER VS. VELOCITY RATIO FOR VARIOUS BASIN LENGTHS

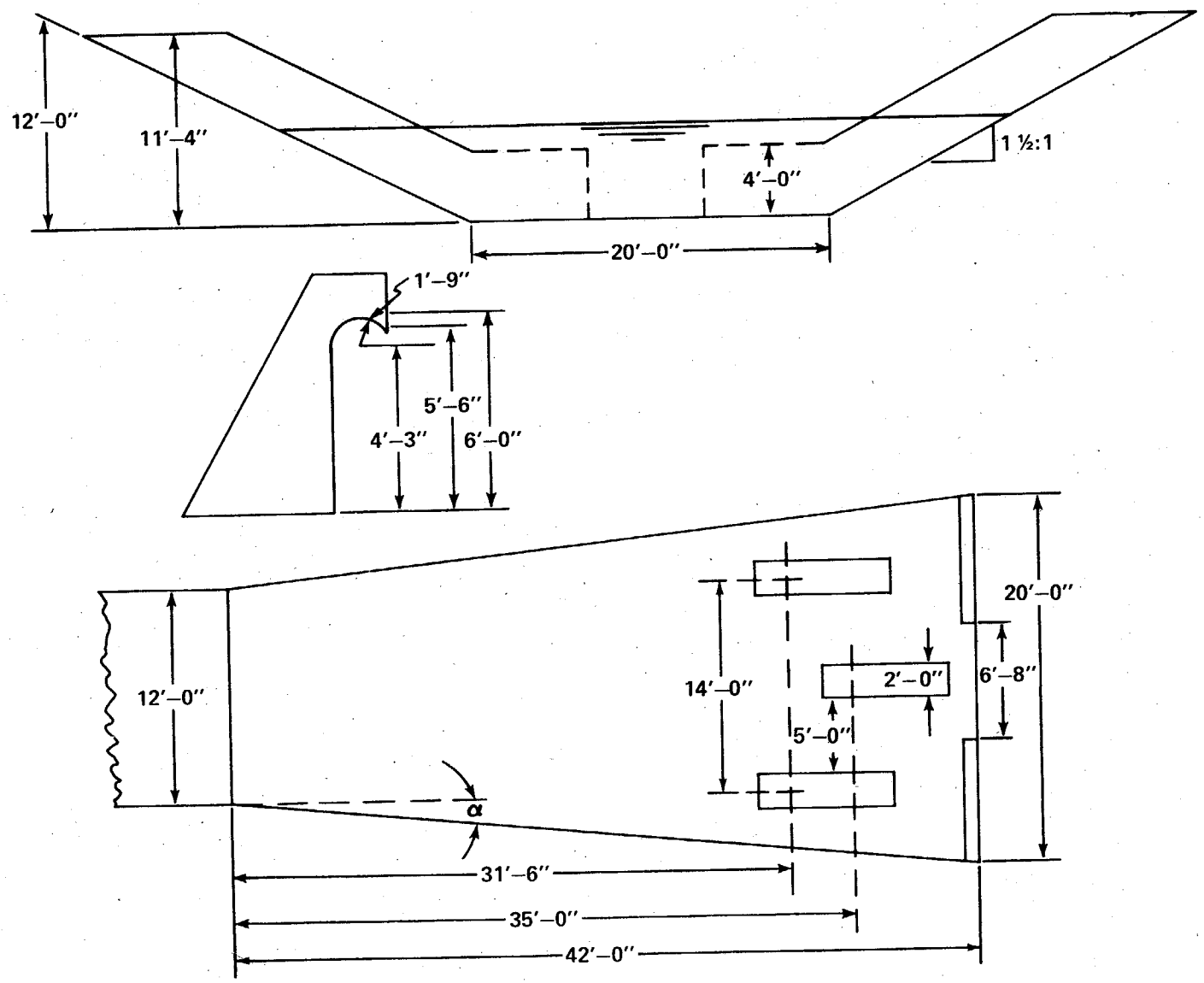


FIGURE VIII–B–4. SAMPLE DESIGN–WARPED WINGWALL BASIN

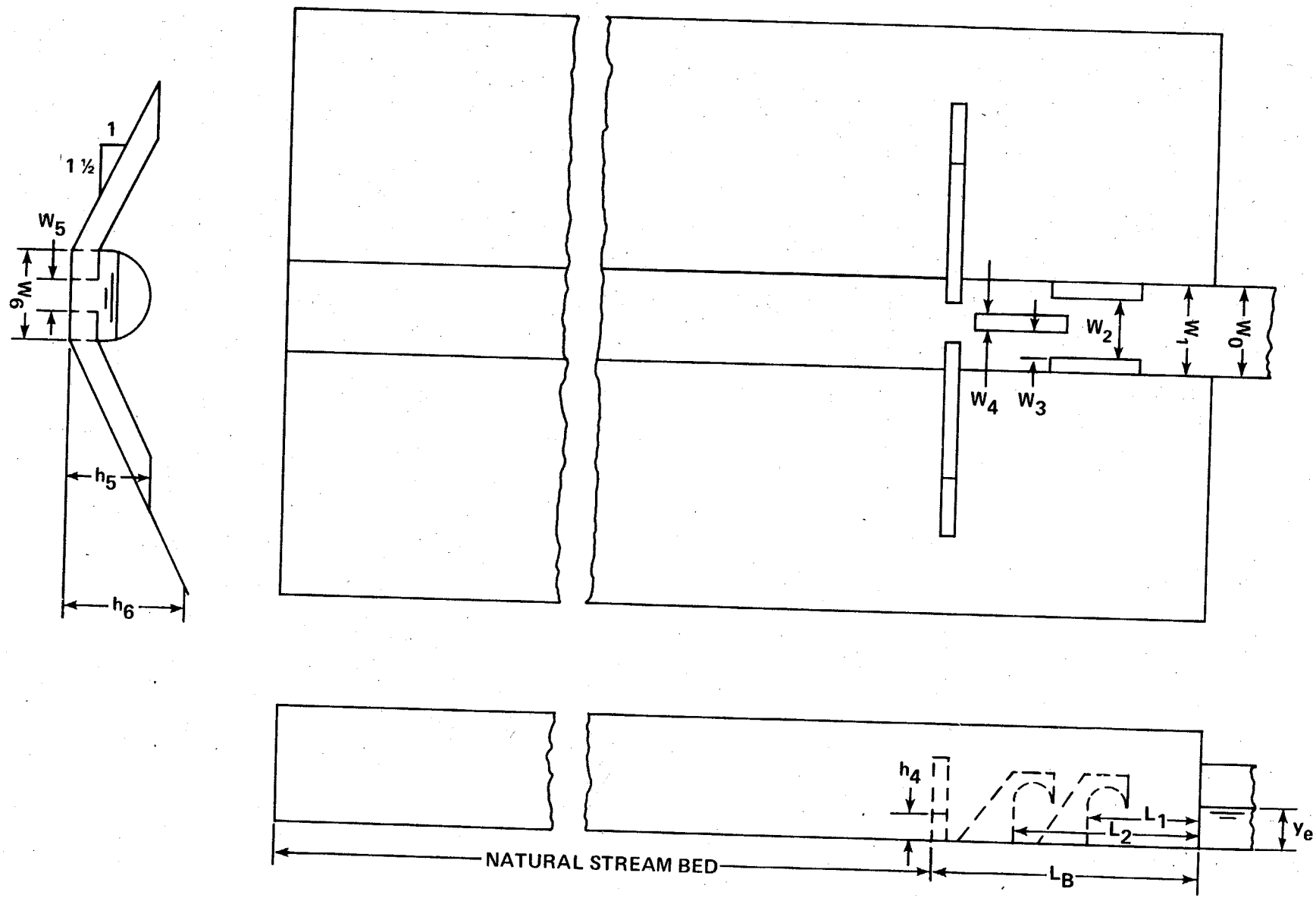
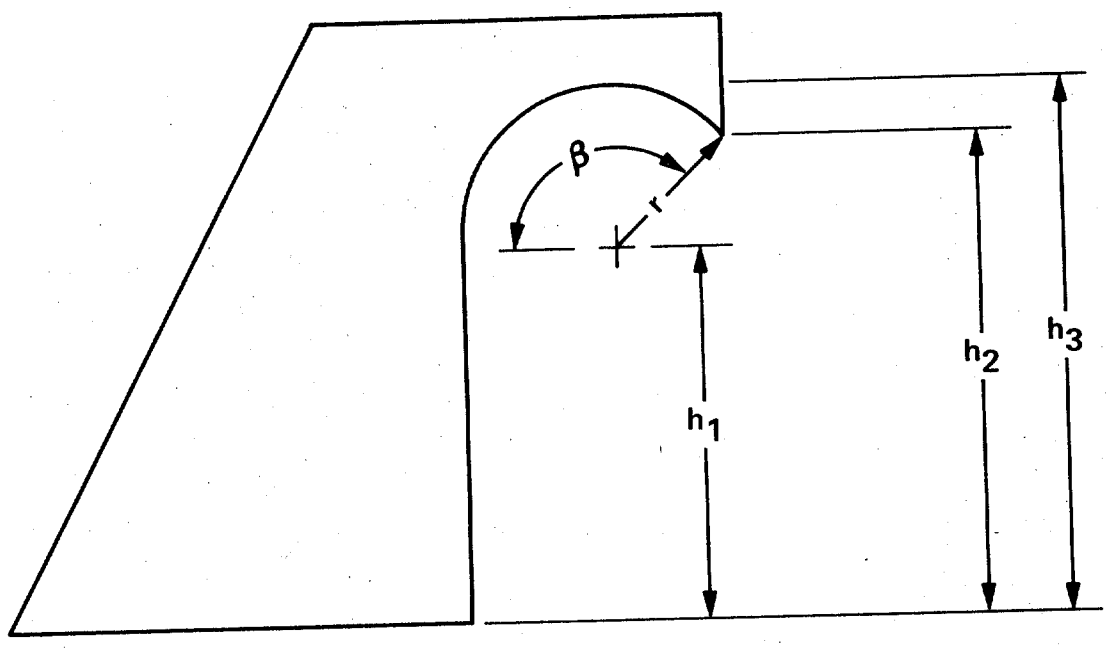


FIGURE VIII–B–5. HOOK TYPE ENERGY DISSIPATOR IN A STRAIGHT TRAPEZOIDAL BASIN



$h_2 = y_e$

$h_1 = 0.78\, y_e$

$h_3 = 1.4\, h_1$

$r = 0.4\, h_1$

$\beta = 135^o$

FIGURE VIII–B-6. TYPICAL SHAPE OF HOOK STRAIGHT TRAPEZOIDAL BASIN

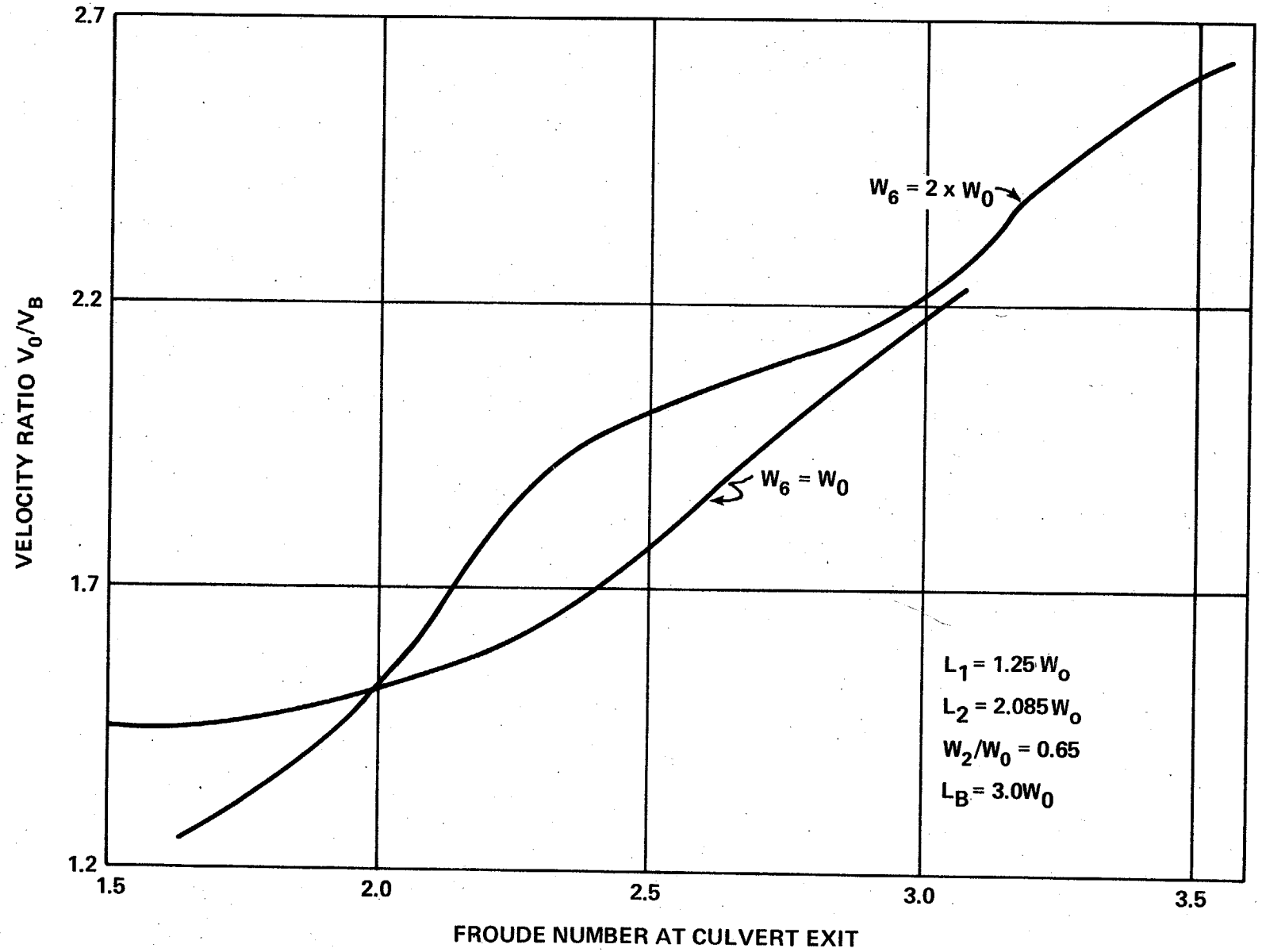


FIGURE VIII–B–7. VELOCITY REDUCTION VS. FROUDE NUMBER FOR VARIOUS BASIN WIDTHS

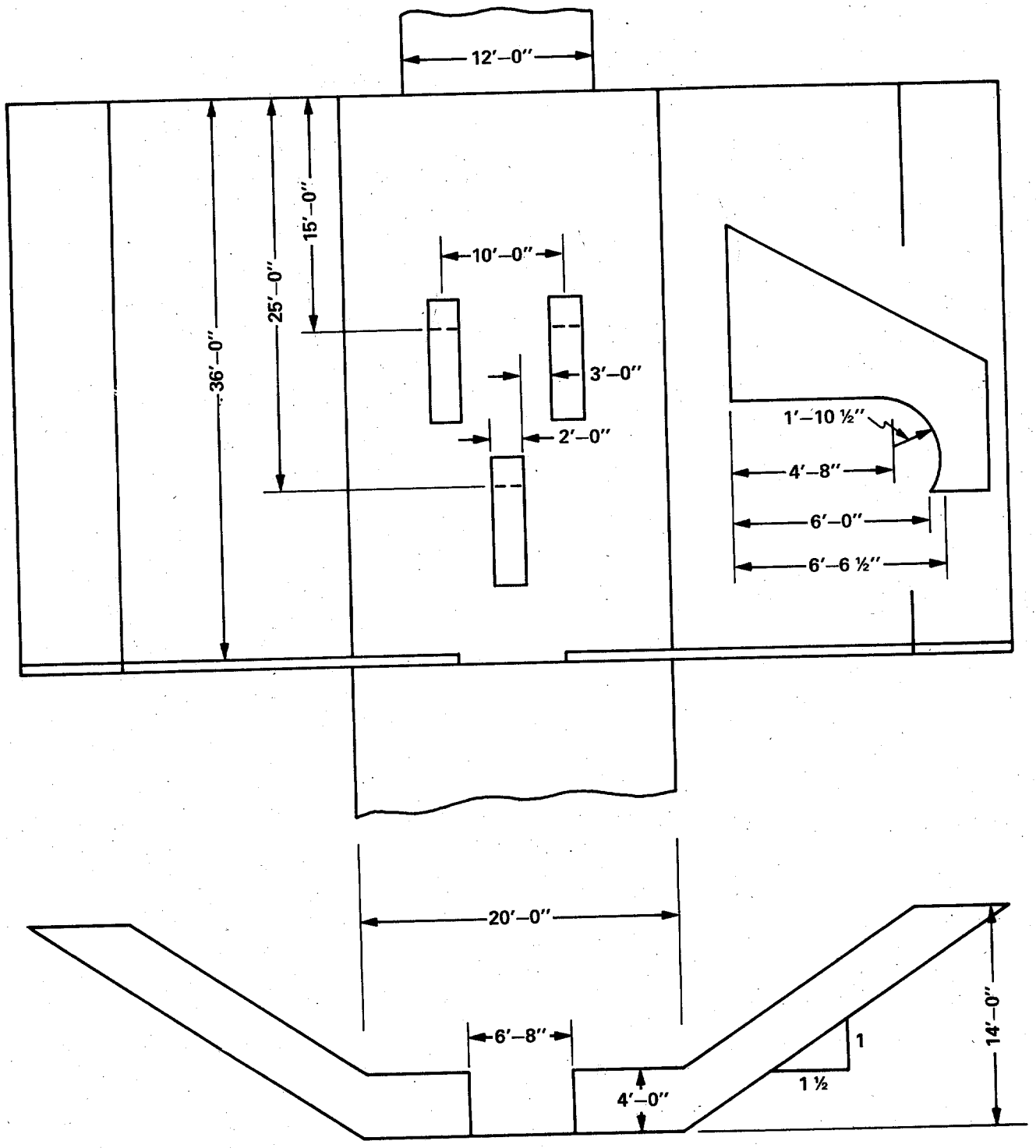


FIGURE VIII–B–8. SAMPLE DESIGN–STRAIGHT TRAPEZOIDAL BASIN

VIII-C. IMPACT-TYPE ENERGY DISSIPATOR

The impact-type energy dissipator developed by the Bureau of Reclamation (VIII-C-1) is contained in a relatively small box-like structure, which requires no tailwater for successful performance. Although the emphasis in this discussion is placed on its use at culvert outlets, the structure may be used in open channels as well.

DEVELOPMENT OF BASIN

The shape of the basin has evolved from extensive tests, but these were limited in range by the practical size of field structures required. With the many combinations of discharge, velocity, and depth possible for the incoming flow, it became apparent that some device was needed which would be equally effective over the entire range. The vertical hanging baffle proved to be this device, figure VIII-C-1. Energy dissipation is initiated by flow striking the vertical hanging baffle and being deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies.

The notches shown in the baffle, figure VIII-C-1, are provided to aid in cleaning the basin after prolonged nonuse of the structure. If the basin is full of sediment, the notches provide concentrated jets of water for cleaning. The basin is designed to carry the full discharge over the top of the baffle if the space beneath the baffle becomes completely clogged. Although this performance is not good, it is acceptable for short periods of time.

DESIGN DISCUSSION

The design information is presented as a simple dimensionless curve, figure VIII-C-2. This curve incorporates the original information contained in reference VIII-C-1, plus the results of additional experimentation performed by the Department of Public Works, City of Los Angeles. It represents the ratio of energy entering the dissipator to the width of dissipator required, plotted with respect to the Froude number. The Los Angles tests indicate that limited extrapolation of this curve is permissible.

In calculating the energy and the Froude number, the equivalent depth of flow entering the dissipator from a pipe or irregular-shaped conduit must be computed on the basis of:

$$y_e = (A/2)^{1/2}$$

In other words, the cross section flow area in the pipe is converted into an equivalent rectangular cross section in which the width is twice the depth of flow. The conduit preceding the dissipator can be open, closed, or of any cross section. The design method is enhanced by ignoring the size and shape of conduit entirely, except for the determination of the depth of flow entering the dissipator.

To take a simple case for illustration: suppose the conduit leading to the dissipator is circular and flowing half full, the water area will be $\pi D^2/8$. The representative depth of flow used in computing the energy entering, H_o, and the Froude number, Fr , will be:

$$y_e = (A/2)^{1/2} \quad \text{or} \quad (\pi D^2/16)^{1/2}$$

The energy $H_o = y_e + V_o^2/2g$ and the Froude number $Fr = V_o/(gy_e)^{1/2}$.

The Los Angeles experiments simulated discharges up to 400 cfs and velocities as high as 50 fps, and some structures already built have been designed to exceed these values. Thus, the only limitations are entrance velocity and size of structure. Velocities up to 50 fps can be used without subjecting the structure to damage from cavitation forces. If needed, two or more basins may be constructed side by side.

The effectiveness of the basin is best illustrated by comparing the energy losses within the structure to those in a natural hydraulic jump, figure VIII-C-3. The energy loss was computed based on depth and velocity measurements made in the approach pipe and also in the downstream channel with no tailwater. Compared with the natural hydraulic jump, the impact basin shows a greater capacity for dissipating energy.

Although tailwater is not necessary for successful operation, a moderate depth of tailwater will improve the performance. For best performance set the basin so that maximum tailwater does not exceed $h_3 + (h_2/2)$.

The basin should be constructed horizontal for all entrance conduits with slopes greater than 15°, a horizontal section of at least four conduit widths long should be provided immediately upstream of the dissipator. Although the basin will operate fairly effectively with entrance pipes on slopes up to 15°, experience has shown that it is more efficient when the recommended horizontal section of pipe is used. In every case, the proper position of the entrance invert, as shown on the drawing, should be maintained.

When a hydraulic jump is expected to form in the downstream end of the pipe and the entrance is submerged, a vent about one-sixth the pipe diameter should be installed at a convenient location upstream from the jump.

For erosion reduction and better basin operation, use the alternative end sill and 45° wingwall design as shown in figure VIII-C-1.

For protection against undermining, a cutoff wall should be added at the end of the basin. Its depth will depend on the type of soil present.

Riprap should be placed downstream of the basin for a length of at least four conduit widths. For riprap size recommendations see chapter II.

The sill should be set as low as possible to prevent degradation downstream. For best performance, the downstream channel should be at the same elevation as the top of the sill. A slot should be placed in the end sill to provide for drainage during periods of low flow.

To provide structural support and aid in priming the device, a short support should be placed under the center of the baffle wall.

Use of the basin is limited to installations where the velocity at the entrance to the stilling basin does not exceed 50 feet per second and discharge is less than 400 cfs. This dissipator is not recommended where debris or ice buildup may cause substantial clogging.

DESIGN PROCEDURES

1. From the maximum discharge and velocity, compute the flow area at the end of the approach pipe. Compute y_e for a rectangular section of equivalent area twice as wide as the depth of flow, $y_e = (A/2)^{1/2}$.

2. Compute the Froude number Fr and the energy at the end of the pipe H_o. Enter the curve on figure VIII-C-2 and determine the required width of basin W.

3. With W known, obtain the remaining dimensions of the dissipator structure from table VIII-C-1.

EXAMPLE PROBLEM

Given:

$D = 48$ inches
$S_o = 0.15$
$Q = 300$ cfs

Find: USBR Impact Basin dimensions for use at the outlet of a concrete pipe.

$n = 0.015$
$V_o = 40$ fps
$y_o = 2.3$ feet

Solution: Since Q is less than 400 cfs and V_o less than 50 fps, the dissipator may be tried at this site.

1. Compute y_e
   $y_e = (A/2)^{1/2}$
   $A = Q/V_o = 300/40 = 7.5$ sq.ft.
   $y_e = (7.5/2)^{1/2} = 1.94$ feet

2. Compute Fr and H_o and find W
   $Fr = V_o/(gy_e)^{1/2} = 40/(32.2 \times 1.94)^{1/2} = 5.05$
   $H_o = y_e + V_o^2/2g = 1.94 + (40)^2/64.4 = 26.8$ feet

   From figure VIII-C-2
   $H_o/W = 1.68$
   $W = 26.8/1.68 = 16.0$ feet

3. From table VIII-C-1 select remaining dimensions

Design a second baffle wall dissipator at the end of a long rectangular concrete channel 4 feet wide using the same depth of flow = 2.3 feet, $S_o = 0.15$ and $n = 0.015$ as in the previous example and compare results.

The discharge for the rectangular channel flowing at a depth of 2.3 feet will be 375 cfs. The computations and comparison with the first example are tabulated below.

Channel			Circular	Rectangular
Depth of flow	y_o	ft.	2.3	2.3
Area of flow	A	sq. ft.	7.50	9.20
Discharge	Q	c.f.s.	300	375
Velocity	V_o	f.p.s.	40	40.9
Flow depth (rect. section)	y_e	ft.	1.9	2.3
Velocity Head	$V_o^2/2g$	ft.	24.9	26.
$H_o = y_e + V_o^2/2g$		ft.	26.8	28.3
$Fr = V_o/(gy_e)^{1/2}$		--	5.05	4.75
From Figure VIII-C-2	H_o/W	--	1.68	1.55
Width of basin	W	ft.	16.0	18.3
H_L/H_o(100)-Low Tailwater			67%	65%

Entering table VIII-C-1 with W = 16 feet and W = 18, the remaining dimensions of the two dissipators can be read directly in feet and inches. The basin width is taken to the nearest half foot, while the other dimensions are read to the nearest inch. This degree of accuracy is sufficient.

VIII-C-1 "Hydraulic Design of Stilling Basins," Journal of the Hydraulics Division, A.S.C.E., paper 1406, October 1957.

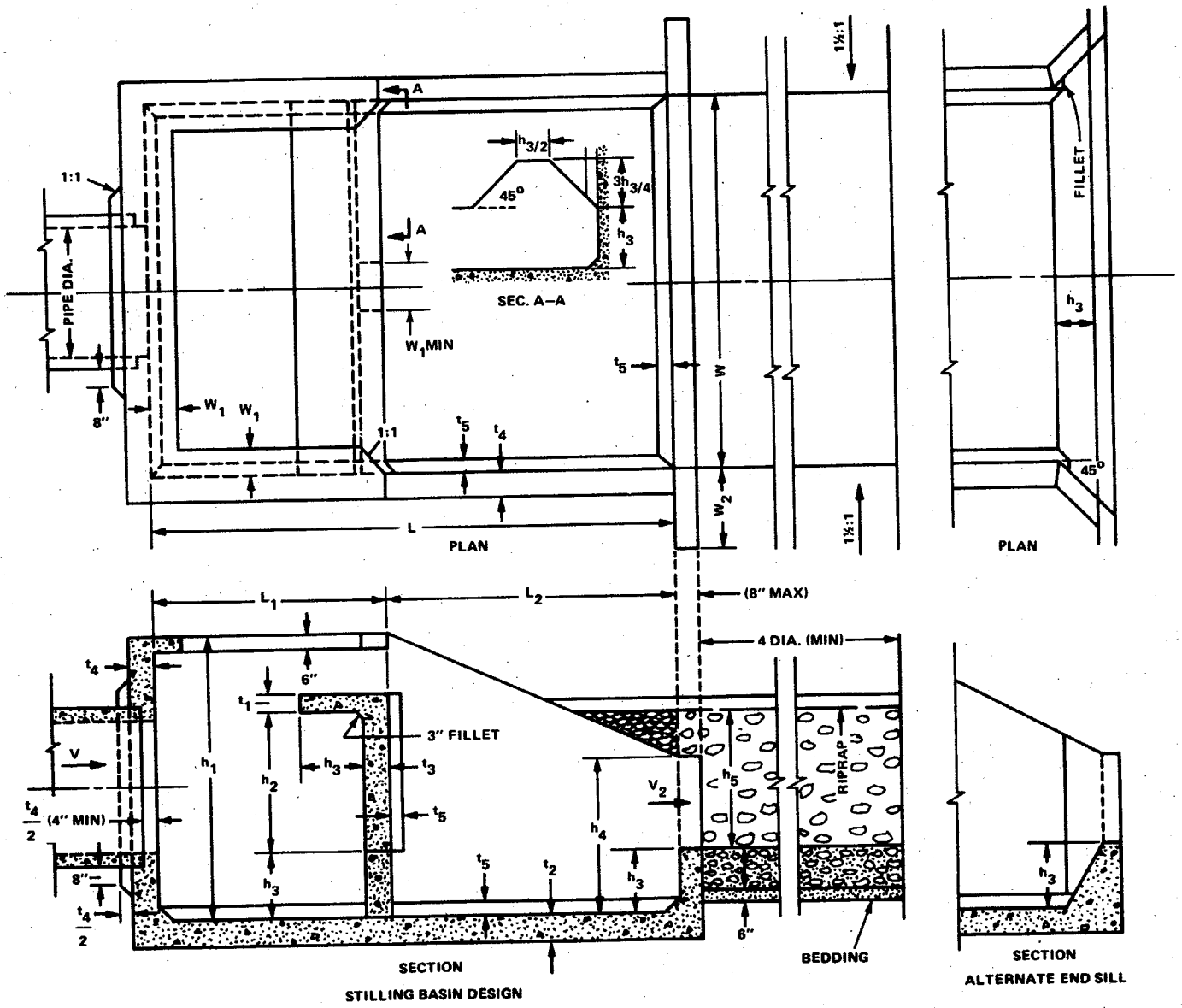


FIGURE VIII–C–1. BAFFLE–WALL ENERGY DISSIPATOR – USBR TYPE VI

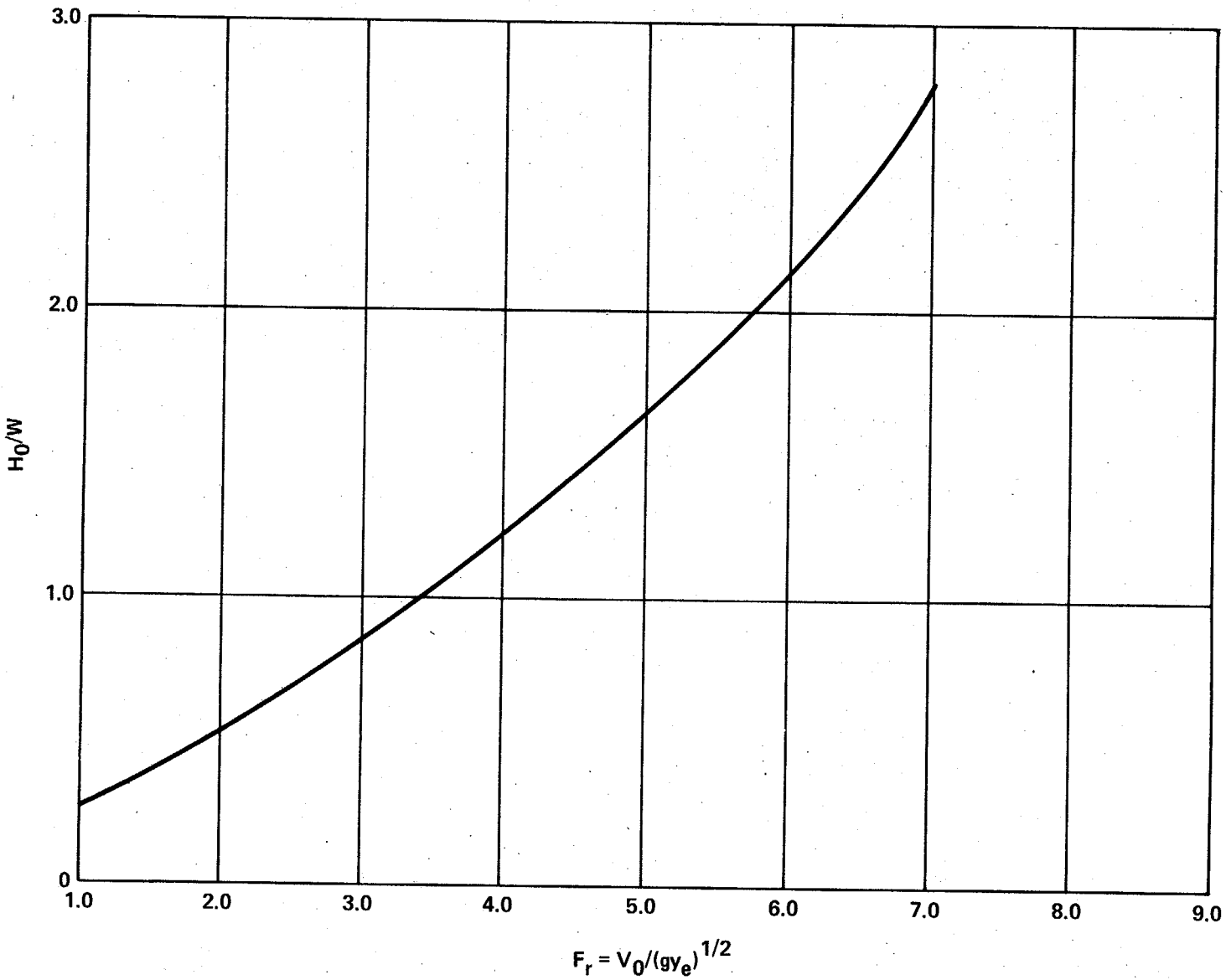


FIGURE VIII–C–2. DESIGN CURVE – BAFFLE WALL DISSIPATOR

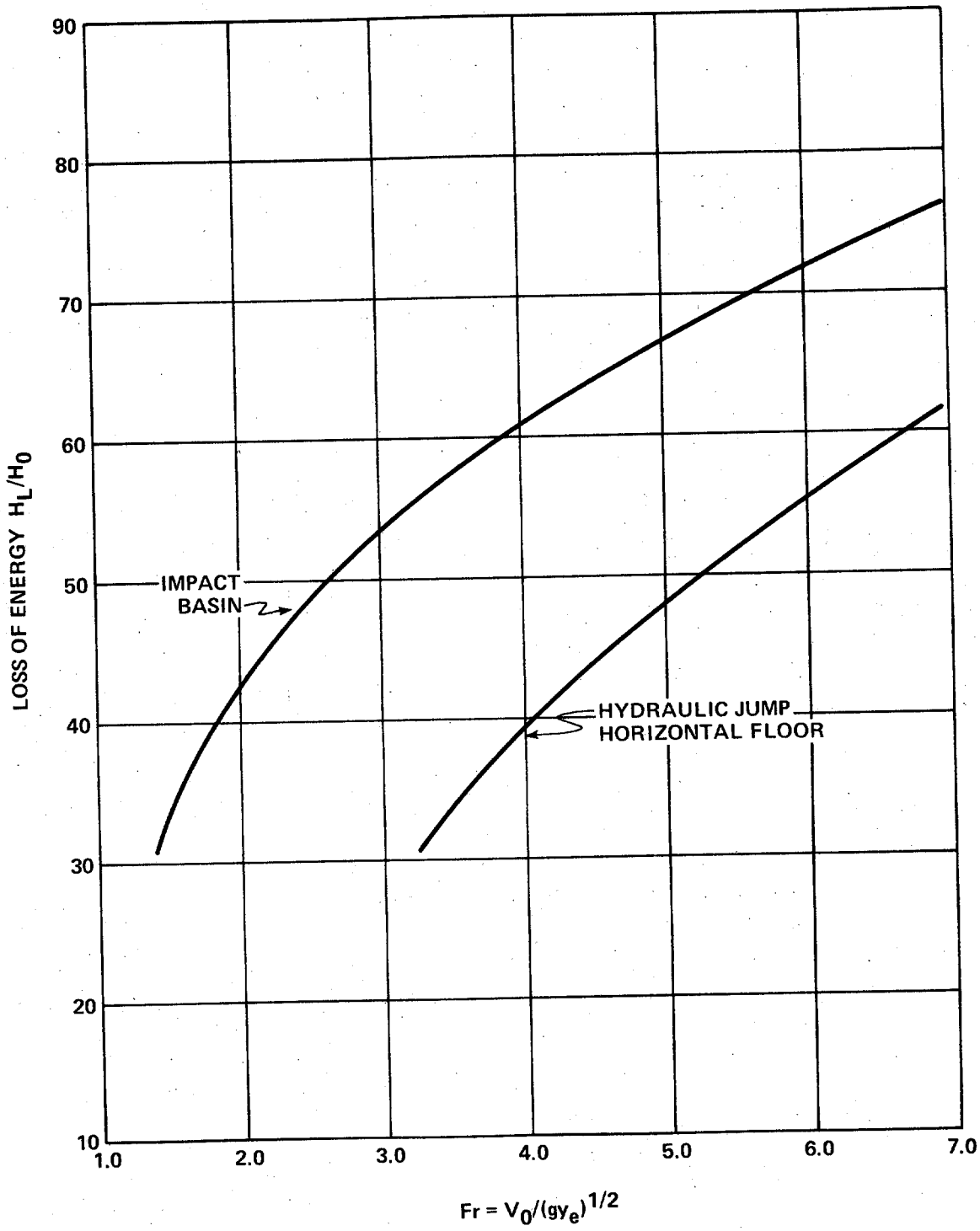


FIGURE VIII–C–3. ENERGY LOSS–IMPACT BASIN–HYDRAULIC JUMP

BAFFLE WALL DISSIPATOR - TABLE VIII-C-1.

Dimensions of basin in feet and inches

W	h_1	L	h_2	h_3	L_1	L_2	h_4	W_1	W_2	t_3	t_2	t_1	t_4	t_5
4-0	3-1	5-5	1-6	0-8	2-4	3-1	1-8	0-4	1-1	0-6	0-6	0-6	0-6	0-3
5-0	3-10	6-8	1-11	0-10	2-11	3-10	2-1	0-5	1-5	0-6	0-6	0-6	0-6	0-3
6-0	4-7	8-0	2-3	1-0	3-5	4-7	2-6	0-6	1-8	0-6	0-6	0-6	0-6	0-3
7-0	5-5	9-5	2-7	1-2	4-0	5-5	2-11	0-6	1-11	0-6	0-6	0-6	0-6	0-3
8-0	6-2	10-8	3-0	1-4	4-7	6-2	3-4	0-7	2-2	0-7	0-7	0-6	0-6	0-3
9-0	6-11	12-0	3-5	1-6	5-2	6-11	3-9	0-8	2-6	0-8	0-7	0-7	0-7	0-3
10-0	7-8	13-5	3-9	1-8	5-9	7-8	4-2	0-9	2-9	0-9	0-8	0-8	0-8	0-3
11-0	8-5	14-7	4-2	1-10	6-4	8-5	4-7	0-10	3-0	0-9	0-9	0-8	0-8	0-4
12-0	9-2	16-0	4-6	2-0	6-10	9-2	5-0	0-11	3-0	0-10	0-10	0-8	0-9	0-4
13-0	10-0	17-4	4-11	2-2	7-5	10-0	5-5	1-0	3-0	0-10	0-11	0-8	0-10	0-4
14-0	10-9	18-8	5-3	2-4	8-0	10-9	5-10	1-1	3-0	0-11	1-0	0-8	0-11	0-5
15-0	11-6	20-0	5-7	2-6	8-6	11-6	6-3	1-2	3-0	1-0	1-0	0-8	1-0	0-5
16-0	12-3	21-4	6-0	2-8	9-1	12-3	6-8	1-3	3-0	1-0	1-0	0-9	1-0	0-6
17-0	13-0	22-6	6-4	2-10	9-8	13-0	7-1	1-4	3-0	1-0	1-1	0-9	1-0	0-6
18-0	13-9	23-11	6-8	3-0	10-3	13-9	7-6	1-4	3-0	1-1	1-1	0-9	1-1	0-7
19-0	14-7	25-4	7-1	3-2	10-10	14-7	7-11	1-5	3-0	1-1	1-2	0-10	1-1	0-7
20-0	15-4	26-7	7-6	3-4	11-5	15-4	8-4	1-6	3-0	1-2	1-2	0-10	1-2	0-8

VIII-D. USFS METAL IMPACT ENERGY DISSIPATOR

The metal impact energy dissipator was developed by the Bureau of Reclamation for the U. S. Forest Service (VIII-D-2). The Forest Service required a dissipator:

1. to use with either helical or corrugated-metal pipe, up to 36 inches in diameter

2. for pipe slopes up to 66 2/3 percent,

3. with self cleaning characteristics,

4. with individual parts readily handled manually and assembled in place,

5. with any parts subject to wear easily replaced.

The dissipator developed for an 18-inch pipe will operate with up to 10 feet of specific energy head, and perform satisfactorily regardless of tailwater elevation.

STUDY RESULTS

The design is somewhat unique in that it provides a "basic" dissipator which is suitable for an 18-inch pipe. The basic design conditions and dimensions are modified to obtain designs for other pipe sizes up to 36 inches.

The dissipator is independent of the incoming pipe, i.e., neither attached to or supported by the pipe, figure VIII-D-2. This leaves an open area between the pipe and the dissipator back plate. To prevent backflow and scour behind the dissipator, a 4-inch wide splash guard is incorporated into the design.

For pipe slopes greater than 40 percent, the addition of two fillets, (Detail X, figure VIII-D-2) is necessary for satisfactory operation. The performance of the dissipator is the same with either helical or annular corrugated-metal pipes. With no tailwater, the flow leaving the dissipator will pass through critical depth at the exit lip. Without high tailwater, some additional channel protection will be needed to protect against scour. Chapter II contains riprap recommendations.

Figure VIII-D-1 provides a family of dimension factor curves for the design of dissipators for an 18-inch corrugated-metal pipe. The dimensions of the 18-inch dissipator are determined

by applying the interpolated dimension factor, or the next higher factor, to all dimensions shown in figure VIII-D-2, and to the L_1 and h_1 values for the location of the outfall end of the corrugated-metal pipe shown in figure VIII-D-3.

For other size pipes, the dimension factor obtained from figure VIII-D-1 is multiplied by the ratio "C" to determine the scale factor. C is D/18, where D = nominal corrugated-metal pipe size in inches.

Parameter	Multiplier
Linear Measurements	C
Energy Head $H_o = V_o^2/2g + y$	C
Discharge, Q	$C^{5/2}$
Pipe Slope, S_o	1

Figures VIII-D-1, 2, and 3 are applicable for energy dissipators to be used with 18-inch corrugated-metal pipes. Revised curves, using the multiplier above, should be limited to corrugated-metal pipes not larger than 36 inches.

In addition to its other uses, the portability of this dissipator makes it attractive as a temporary erosion control device during construction.

DESIGN PROCEDURE

The size and slope of the incoming pipe and the design flow are necessary input values. The design procedure for determining energy dissipator dimensions for all conditions is:

1. 18-inch CMP's: With the slope of the CMP and the flow rate known, obtain a dimension factor from the dimension factor chart, figure VIII-D-1. Multiply all dimensions of basic 18-inch dissipator, figure VIII-D-2, by this dimension factor to obtain the dissipator dimensions for the 18-inch pipe with the given slope and flow conditions.

2. Other sizes of CMP's

   a. Compute C and $C^{5/2}$ from C=D/18, where D is diameter of the CMP in inches

b. Compute Q_{18} from $Q_{18}=Q_D/C^{5/2}$, where Q_D is the discharge for the given diameter pipe, and Q_{18} is the discharge needed to enter figure VIII-D-1.

c. Enter figure VIII-D-1 with Q_{18} and for the given slope, determine the Dimension Factor, DF. Slope is independent of the dimension factor and does not require adjustment.

d. Compute the Scale Factor, SF, from SF=(DF)(C)

e. Apply SF to all dimensions of figure VIII-D-2 to determine dimensions of required dissipator.

3. To install the dissipator at an existing culvert, the L_1 and h_1 dimensions (figure VIII-D-3) must be known in order to properly position the discharge lip.

Calculate L_1 by determining the dimension of the splash guard and multiplying by 1.25. Subtract this from the distance between the baffle and the back plate to obtain L_1. Enter figure VIII-D-3 with L_1 to determine h_1.

These dimensions, L_1 and h_1, can be measured from the outfall of the culvert to locate the required position of the discharge lip (see installation sketch, figure VIII-D-4).

The elevation of the dissipator lip and the L_1, h_1 distances in figure VIII-D-3 are critical. The lip should be placed at the elevation of natural ground and care taken to insure that the dissipator discharge lip is level and the baffle wall vertical.

The distance L_2, figure VIII-D-4, represents the extension of the pipe into the dissipator. It can be calculated by

$$L_2 = \frac{\text{Splash guard dimension } 1.25}{\cos(\text{Pipe Angle})}$$

Riprap should be placed downstream from the lip to protect against undercutting. See chapter II for design recommendations.

DESIGN EXAMPLE NO. 1

The necessary input is:

18-inch cmp
30% slope
Q=5.0 cfs

1. From figure VIII-D-1 with Q and S_o, the dimension factor is 0.56. Apply this to the dimension of figure VIII-D-2.

3. Compute L_1 and h_1, figure VIII-D-3. The distance from the baffle to the back plate is

   33.8(0.56)=18.9 inches

   The splash guard dimension is

   4.0(0.56)=2.25 inches

   The L_1 distance is determined by placing the pipe inside the dissipator:

   L_1=(18.9-(2.25x1.25))=16.1 inches

   Convert this and enter figure VIII-D-3

   16.1/0.56=28.8

   This gives an initial h_1 distance of 24, which is converted to 24(0.56) = 13.5 inches, the final h_1 dimension.

   The remaining dimensions are obtained from figure VIII-D-2 and the final design is given in figure VIII-D-5.

DESIGN EXAMPLE NO. 2

To apply the design to a 36-inch pipe with 35 cfs requires altering figures VIII-D-1 and 2 using the "C" multipliers:

1. Start with step 2 for other CMP.

2.a. C=D/18=36/18=2
   $C^{5/2}$=5.64

   b. Q=35 cfs = Design Discharge for 36-inch pipe

   Q_{18}=35/5.64=6.22 cfs

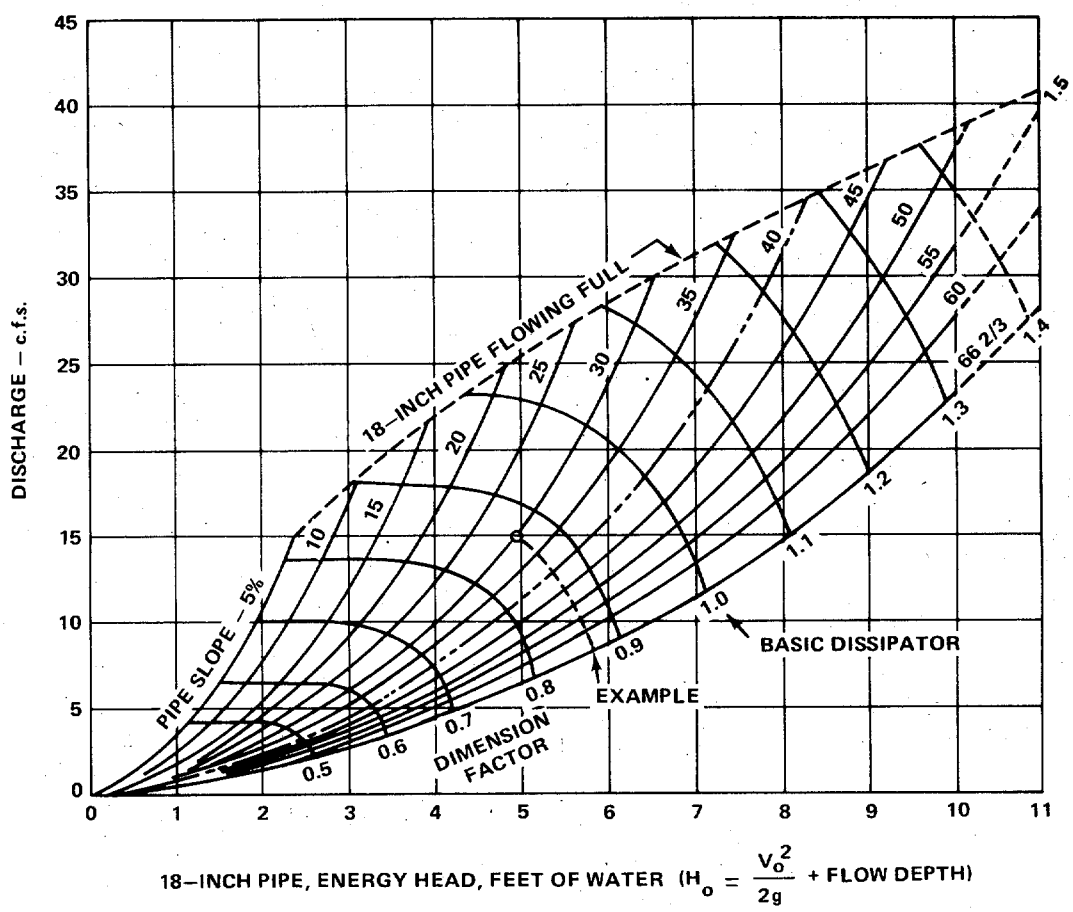
c. Entering figure VIII-D-1 with this discharge (6.22 cfs) gives a dimension factor (DF)=0.62.

d. SF=DF(C)=.62(2)=1.24

e. The dimension in figures VIII-D-2 and 3 are multiplied by 1.24 to obtain dimensions for the 36-inch pipe.

3. L_1 and h_1 can be determined as above.

VIII-D-1. Colgate, D., HYDRAULIC MODEL STUDIES OF CORRUGATED-METAL PIPE UNDERDRAIN ENERGY DISSIPATORS, USBR, REC-ERC-71-10, January 1971.



NOTES

FOR PIPE SLOPES GREATER THAN 40%, DEFLECTOR FILLERS MUST BE USED TO ACHIEVE PROPER FLOW DISTRIBUTION AND DISSIPATOR SELF CLEANING CAPABILITIES. DETAIL FIG. VIII-D-2.

FOR PIPE SLOPES UP TO 40% THE DISSIPATOR OPERATES EQUALLY WELL WITH OR WITHOUT FILLERS.

DIMENSION FACTOR LINE IS THE UPPER LIMIT OF SATISFACTORY OPERATION FOR THE DISCHARGE AND PIPE SLOPE SHOWN.

EXAMPLE: FOR AN 18–INCH CORRUGATED PIPE INSTALLED ON A 30% SLOPE AND DISCHARGING 15 cfs, THE DIMENSION OF THE BASIC ENERGY DISSIPATOR. FIGURE VIII-D-2 SHOULD BE MODIFIED BY THE FACTOR 0.87.

FIGURE VIII–D–1. DIMENSION FACTOR CHART FOR 18–INCH CORRUGATED PIPE FROM REFERENCE VIII-D-1

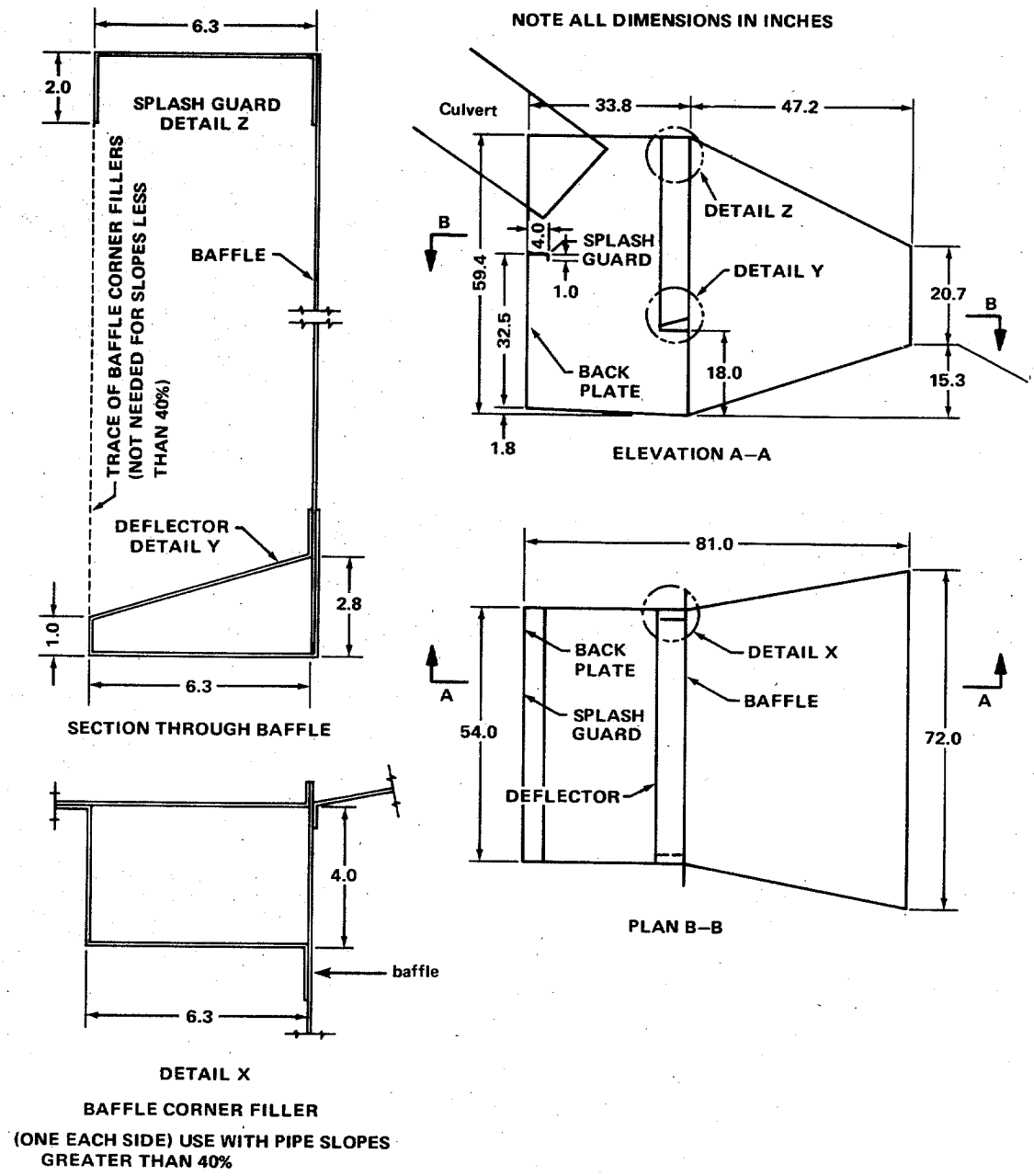
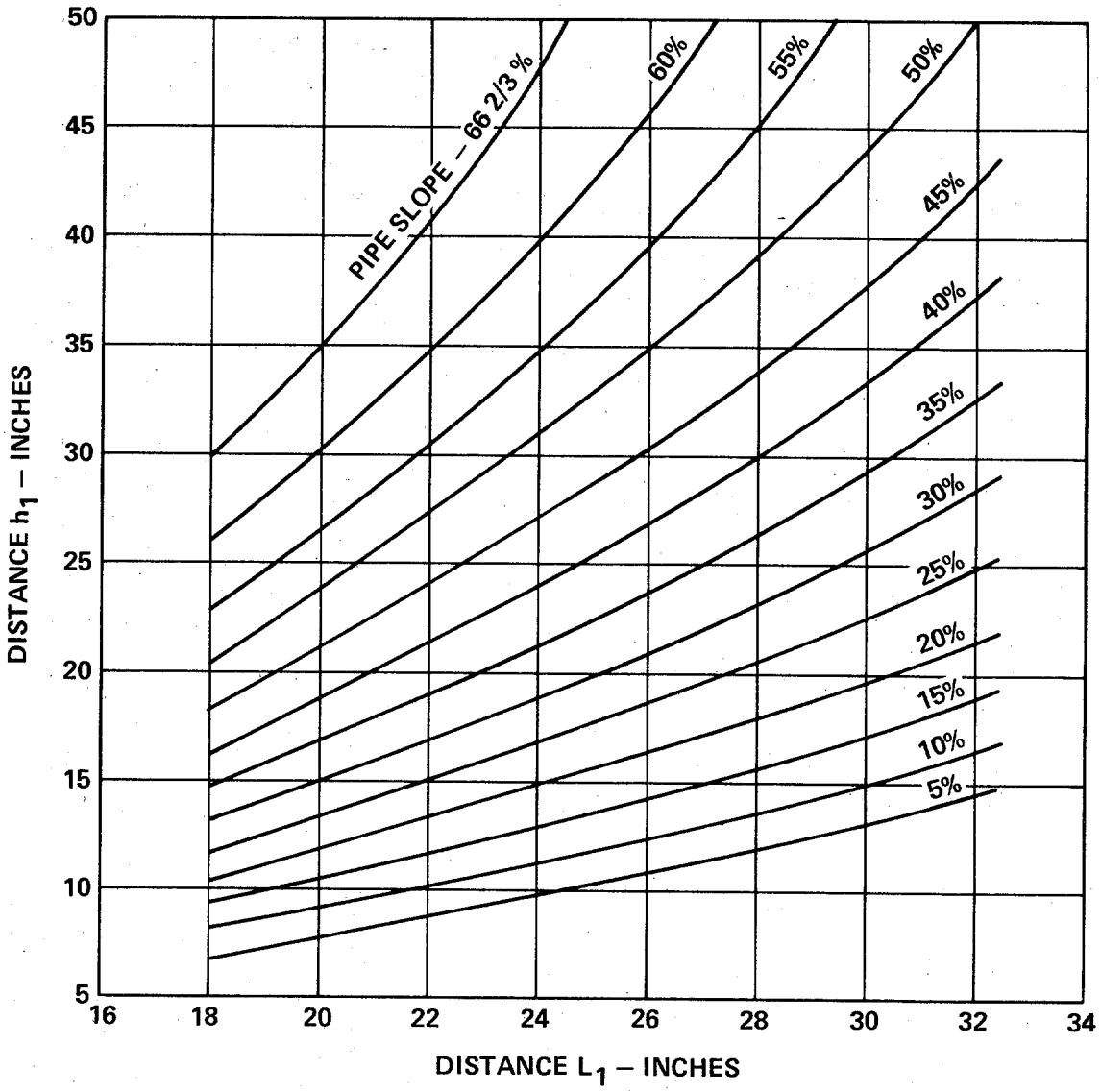


FIGURE VIII - D - 2. BASIC ENERGY DISSIPATOR DIMENSIONS – 18–INCH CORRUGATED PIPE (DIMENSION FACTOR = 1.0) FROM REFERENCE VIII-D-1.



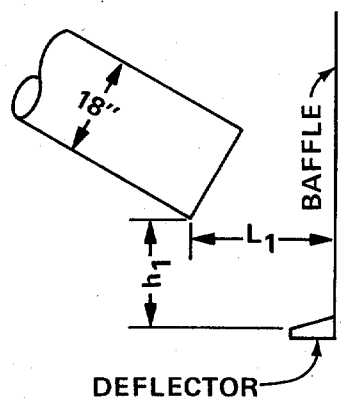


FIGURE VIII–D–3. LOCATION OF INVERT OF DISCHARGE END OF 18–INCH CORRUGATED–METAL PIPE FOR VARIOUS PIPE SLOPES FROM REFERENCE VIII-D-1

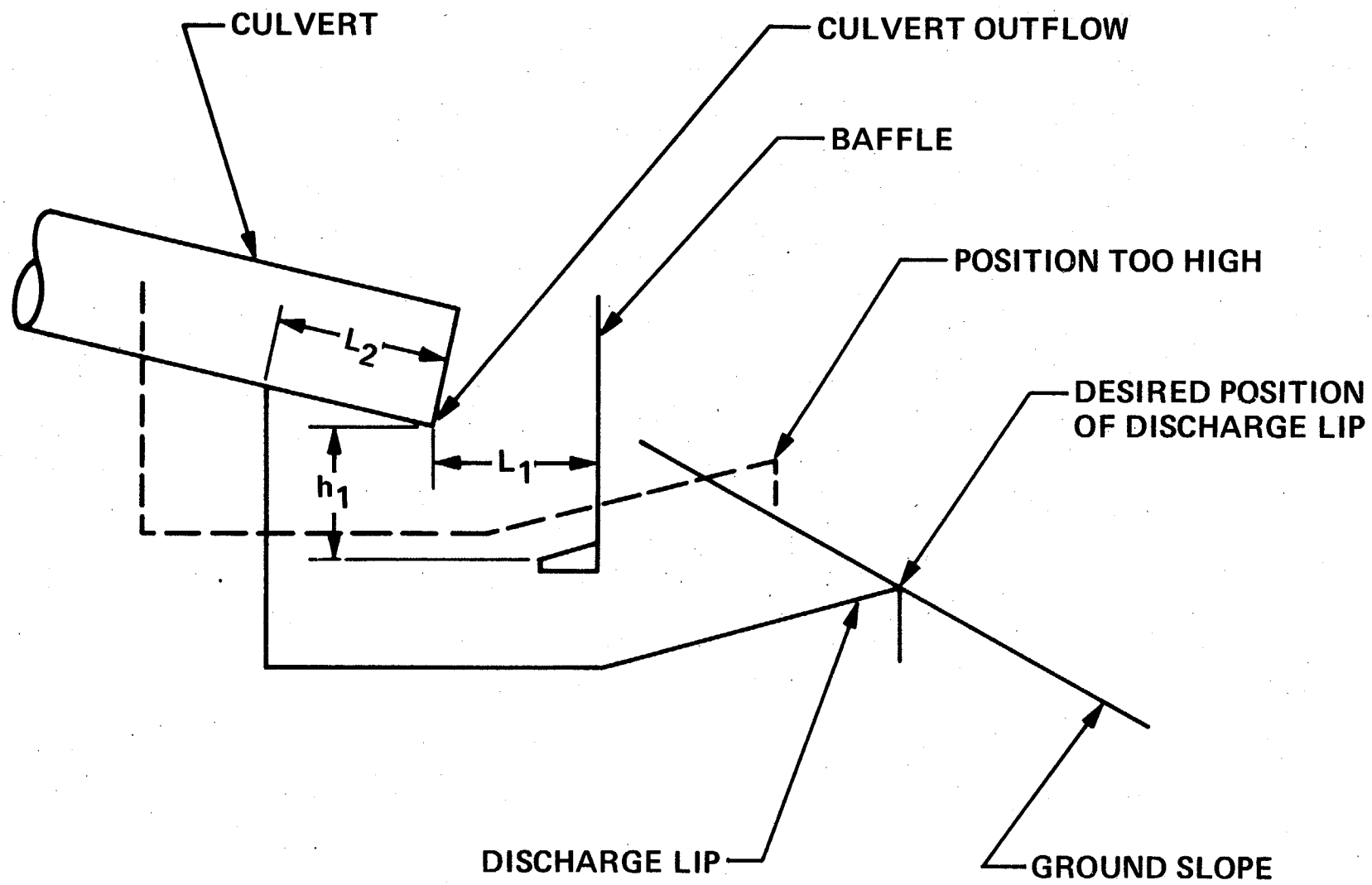


FIGURE VIII–D–4. INSTALLATION SKETCH

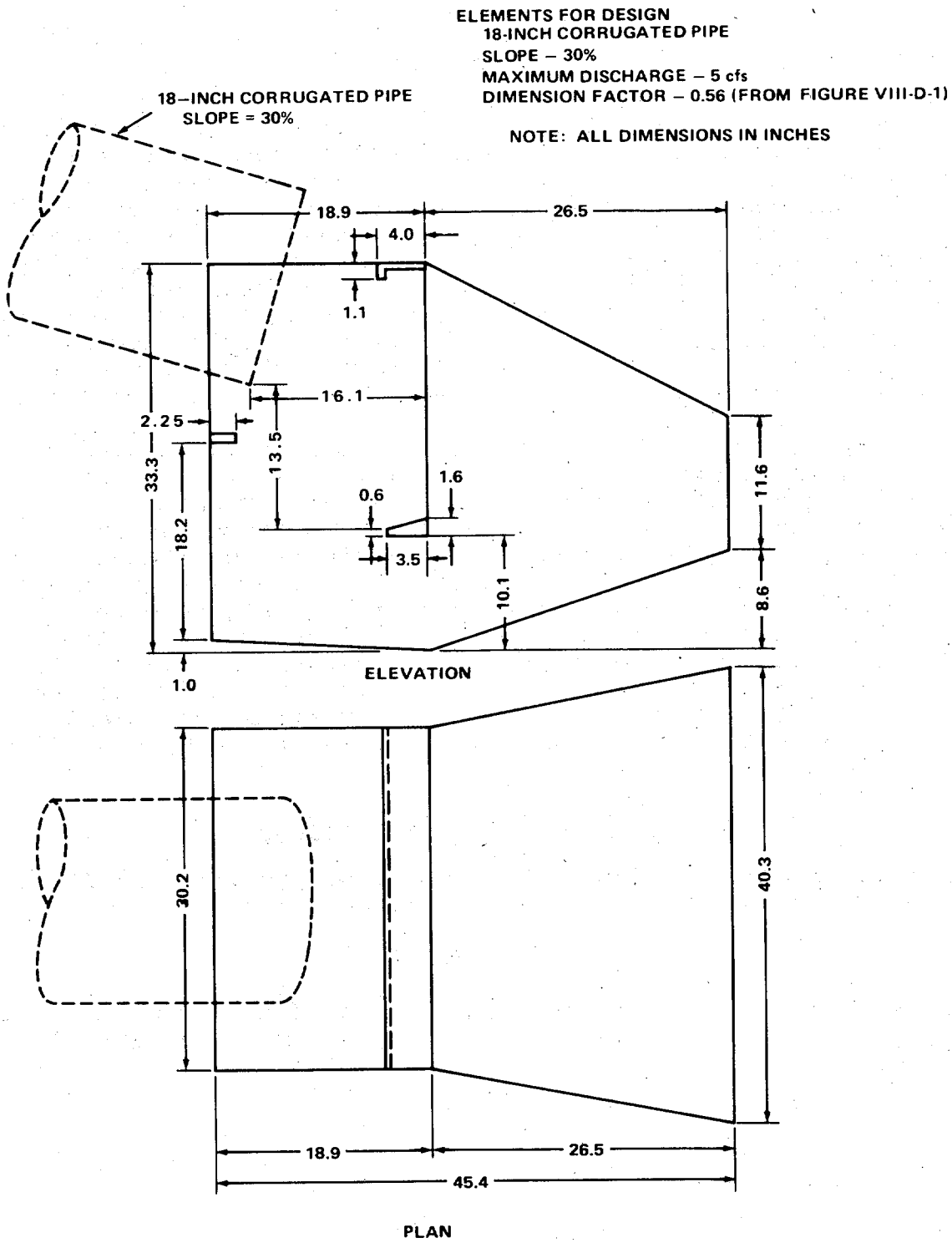


FIGURE VIII–D–5 ENERGY DISSIPATOR FOR A DIMENSION FACTOR OF 0.56

IX. DROP STRUCTURES

Drop structures are commonly used for flow control and energy dissipation. Changing the channel slope from steep to mild, by placing drop structures at intervals along the channel reach, changes a continuous steep slope into a series of gentle slopes and vertical drops. Instead of slowing down and transferring high erosion producing velocities into low non-erosive velocities, drop structures control the slope of the channel in such a way that the high, erosive velocities never develop. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by a specially designed apron or stilling basin.

The drop structures discussed here require aerated nappes and are, in general, for subcritical flow in the upstream as well as downstream channel. The effect of upstream supercritical flow on drop structure design is discussed in a later section. The stilling basin protects the channel against erosion below the drop and dissipates energy. This is accomplished through the impact of the falling water on the floor, redirection of the flow, and turbulence. The stilling basin used to dissipate the excess energy can vary from a simple concrete apron to an apron with flow obstructions such as baffle blocks, sills, or abrupt rises. The length of the concrete apron required can be shortened by the addition of these appurtenances.

FLOW GEOMETRY CONSIDERATIONS

The flow geometry at straight drop structures, figure IX-1, can be described by the drop number, defined:

$$N_d = q^2/gh_o^3$$

Where q is the discharge per unit width of the crest overfall, g is the acceleration of gravity, and h_o is the height of the drop. The functions are:

$$L_1/h_o = 4.30\ N_d^{0.27},$$

$$y_1/h_o = 1.00\ N_d^{0.22},$$

$$y_2/h_o = 0.54\ N_d^{0.425} \text{ and}$$

$$y_3/h_o = 1.66\ N_d^{0.27}.$$

Where L_1, the drop length, is the distance from the drop wall to the position of the depth y_2; y_1 is the pool depth under the nappe; y_2 is the depth of flow at the toe of the nappe or the beginning of the hydraulic jump; and y_3 is the tailwater depth sequent to y_2, figure IX-1.

The free-falling nappe reverses its curvature and turns smoothly into supercritical flow on the apron at the distance L_1 from the drop wall. The mean velocity at the distance L_1 is parallel to the apron; the depth y_2 is the smallest depth in the downstream channel, and the pressure is nearly hydrostatic. The depth of supercritical flow in the downstream direction increases due to channel resistance, and at some point will reach a depth sufficient for the formation of a hydraulic jump.

For a given drop height, h_o, and discharge, q, the sequent depth, y_3, in the downstream channel and the drop length, L_1, may be computed. The length of jump Lj, is discussed in chapter VI. By comparing the channel tailwater depth, TW, with the computed, y_3, the flow type (TW less than y_3, TW $=y_3$, or TW greater than y_3) can be determined. The flow type determines the design of the stilling basin for the drop structure.

If the tailwater depth, TW, is less than y_3, the hydraulic jump will recede downstream. If the tailwater depth is greater than y_3, the hydraulic jump will be submerged. If TW is equal to y_3, the hydraulic jump begins at depth y_2 (figure IX-1), no supercritical flow exists on the apron, and the distance L_1 is a minimum.

When the tailwater depth, TW, is less than y_3, it is necessary to provide:

1. an apron at the bed level and an end sill or baffles

2. an apron below the downstream bed level, and an end sill.

The choice of design type and the design dimensions will depend, for a given unit discharge (q), on the drop height (h_o) and on the downstream depth (TW).

The apron may be designed to extend to the end of the hydraulic jump. However, including an end sill allows the use of a shorter and more economical stilling basin.

The geometry of the undisturbed flow should be taken into consideration in the design of a straight drop stilling basin. If the overfall crest length is less than the width of the approach channel, it is important that a transition be properly designed by shaping the approach channel to reduce the effect of end contractions. Otherwise the contraction at the ends of the spillway notch may be so pronounced that the jet will land beyond the stilling-basin and the concentration of high velocities at the center of the outlet may cause additional scour in the downstream channel (see Chapter IV, Flow Transitions).

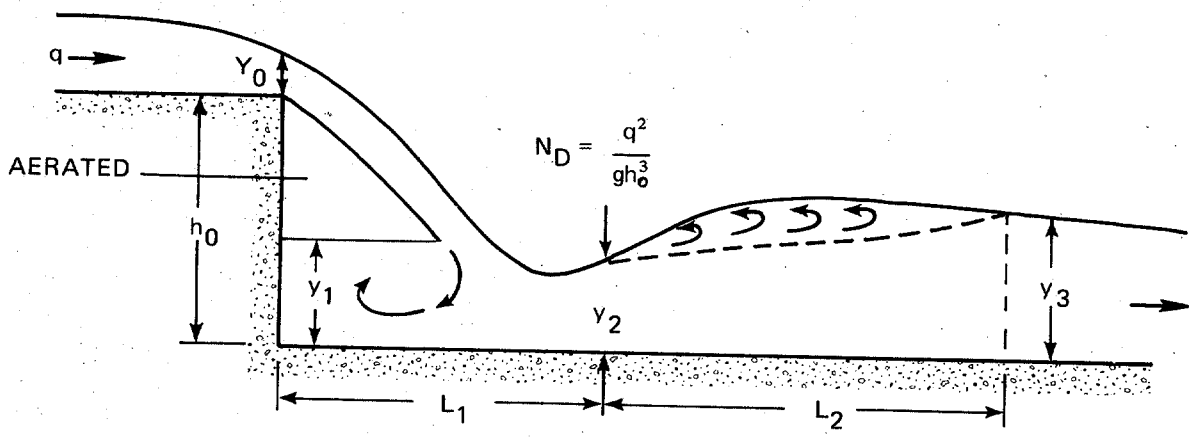


FIGURE IX - 1 FLOW GEOMETRY OF A STRAIGHT DROP SPILLWAY

GRATE DESIGN

A grate or series of rails forming a "grizzly" may be used in conjunction with drop structures, figure IX-2. The incoming flow is divided into a number of jets as it passes through the grate. These fall almost vertically to the downstream channel resulting in good energy dissipator action.

This type of design is also utilized as a debris ejector where the debris rides over the grate and falls into a holding area for later removal and the water passes through the grate.

The Bureau of Reclamation has published design recommendations for grates (VI-1) for use where the incoming flow is subcritical:

1. Select a slot width. Provide a full slot width at each wall.

2. Compute:

a. the beam length L_G from,

$$L_G = (Q)/C(W)N(2gy_o)^{1/2}$$

where C is an experimental coefficient equal to 0.245, W is the width of the slots in feet, and N is the number of slots or spaces between beams.

b. the beam width = 1.5W. The designer can adjust the quantity "WN" until an acceptable beam length (L_G) is obtained.

3. Tilt the grate about 3° downstream to be self-cleaning.

HIGH APPROACH VELOCITY

Examination of the beam length equation in the previous section, indicates the relative effect of higher approach velocities on the design of drop structures.

Assuming the slot width, W, approaches the channel width making N equal to 1, then

$$L_G \simeq Q/(y_o)^{1/2}$$

As the approach velocity increases, for a constant Q, the approach depth decreases and the length L increases inversely with the square root of the depth. Therefore, for high velocity flow, above critical velocity, the length of drop structure required, to contain the jet, may very rapidly exceed practical limits.

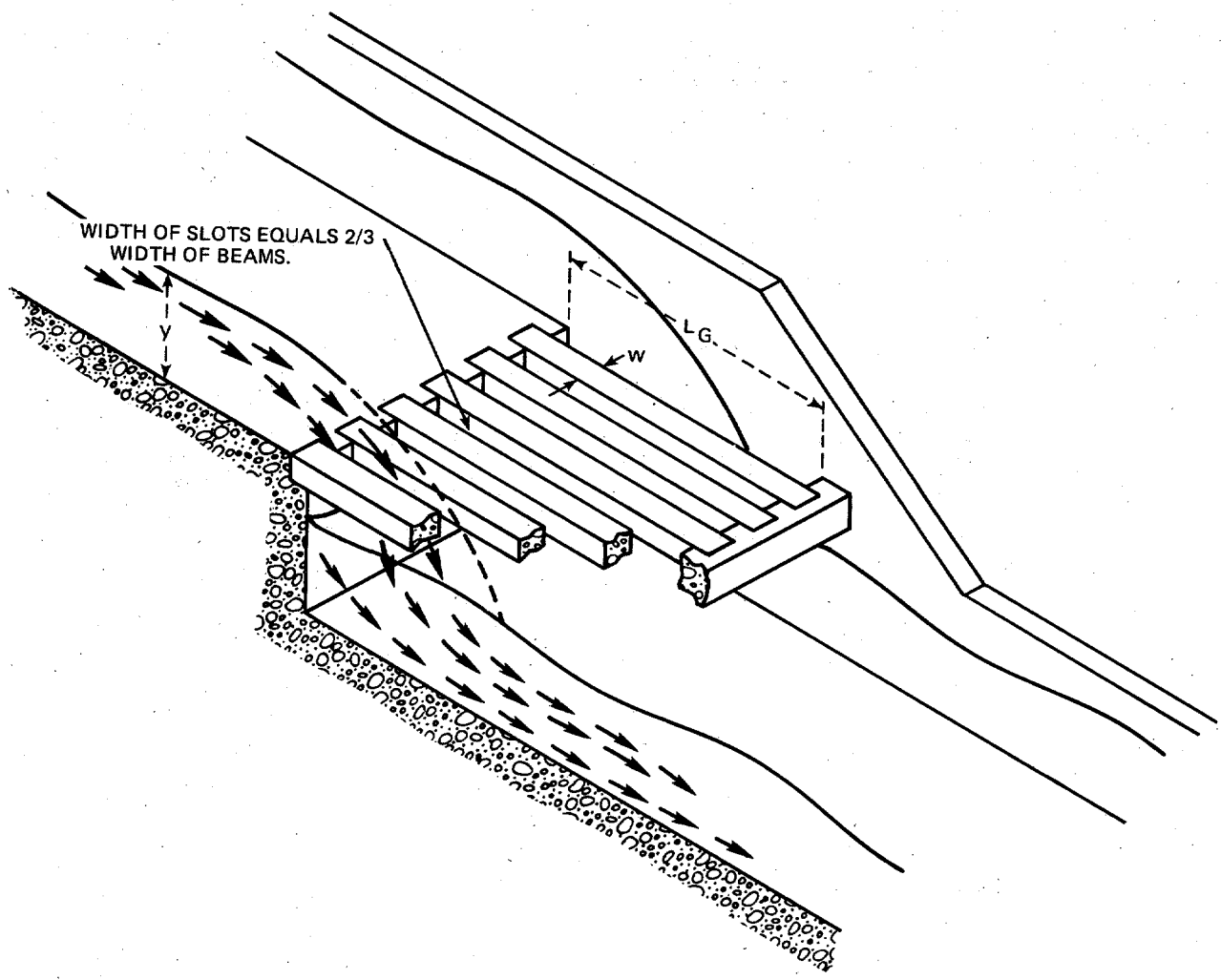


FIGURE IX – 2 ENERGY DISSIPATOR WITH GRATE

IX-A. STRAIGHT DROP STRUCTURE

A general design for a stilling basin at the toe of a drop structure was developed by the Agricultral Research Service, St. Anthony Falls Hydraulic Laboratory, University of Minnesota (IX-A-1). The basin consists of a horizontal apron with blocks and sills to dissipate energy. Tailwater also influences the amount of energy dissipated. The stilling basin length computed for the minimum tailwater level required for good performance may be inadequate at high tailwater levels. Dangerous scour of the downstream channel may occur if the nappe is supported sufficiently by high tailwater so that it lands beyond the end of the stilling basin. A method for computing the stilling basin length for all tailwater levels is presented.

The design is applicable to relative heights of fall ranging from $1.0(h_o/y_c)$ to $15(h_o/y_c)$ and to crest lengths greater than $1.5y_c$. Here h_o is the vertical distance between the crest and the stilling basin floor, and y_c is the critical depth of flow at the crest. The straight drop structure is effective if the drop does not exceed 15 feet and if there is sufficient tailwater.

There are several elements which must be considered in the design of this stilling basin. These include the length of basin, the position and size of floor blocks, the position and height of end sill, the position of the wingwalls, and the approach channel geometry. Figure IX-A-1 illustrates a straight drop strucutre which provides adequate protection from scour in the downstream channel.

DESIGN PROCEDURE

1. Calculate the specific head in approach channel.

$$H = y_o + \frac{V_o^2}{2g} \quad \text{.....IX-A-1}$$

2. Calculate critical depth.

$$y_c = \frac{2}{3} H \quad \text{.....IX-A-2}$$

3. Calculate the minimum height for tailwater surface above the floor of the basin.

$$y_3 = 2.15\ y_c \quad \text{.....IX-A-3}$$

4. Calculate the vertical distance of tailwater below the crest. This will generally be a negative value since the crest is used as a reference point.

$$h_2 = -\ (h - y_o) \quad \text{.....IX-A-4}$$

5. Determine the location of the stilling basin floor relative to the crest.

$$h_o = h_2 - y_3 \quad \text{IX-A-5}$$

6. Determine the minimum length of the stilling basin, L_B, using:

$$L_B = L_1 + L_2 + L_3 = L_1 + 2.55y_c \quad \text{IX-A-6}$$

where L_1 is the distance from the headwall to the point where the surface of the upper nappe strikes the stilling basin floor. This is given by:

$$L_1 = (L_f + L_s)/2 \quad \text{IX-A-7}$$

where

$$L_f = (-0.406 + \sqrt{3.195-4.368h_o/y_c})y_c$$

$$L_s = [0.691 + 0.228\ (L_t/y_c)^2 - (h_o/y_c)]y_c/[0.185 + 0.456(L_t/y_c)]$$

$$L_t = (-0.406 + \sqrt{3.195 - 4.368\ h_2/y_c})y_c$$

or L_1 can be found graphically from figure IX-A-2.

L_2 is the distance from the point at which the surface of the upper nappe strikes the stilling basin floor to the upstream face of the floor blocks, figure IX-A-1. This distance can be determined by:

$$L_2 = 0.8y_c \quad \text{IX-A-8}$$

L_3 is the distance between the upstream face of the floor blocks and the end of the stilling basin. This distance can be determined from:

$$L_3 > 1.75y_c \quad \text{IX-A-9}$$

7. Proportion the floor blocks as follows:

   a. height is 0.8 y_c,

   b. width and spacing should be 0.4 y_c with a variation of $\pm$ 0.15y_c permitted,

   c. blocks should be square in plan, and

   d. blocks should occupy between 50 percent and 60 percent of the stilling basin width.

8. Calculate the end sill height, ($0.4y_c$).

9. Longitudinal sills, if used, should pass through, not between, the floor blocks. These sills are for structual purposes and are neither beneficial nor harmful hydraulically.

10. Calculate the sidewall height above the tailwater level, ($0.85y_c$).

11. Wingwalls should be located at an angle of 45° with the outlet centerline and have a top slope of 1 to 1.

12. Modify the approach channel as follows:

    a. crest of spillway should be at same elevation as approach channel,

    b. bottom width should be equal to the spillway notch length, W_o at the headwall, and

    c. protect with riprap or paving for a distance upstream from the headwall equal to three times the critical depth, y_c. See chapter II for recommendations on riprap design.

13. No special provision of aeration of the space beneath the nappe is required if the approach channel geometry is as recommended in 12.

DESIGN EXAMPLE

Given: $Q = 250$ cfs

$S_o = 0.002$ ft/ft, and the downstream channel has 3:1 side slopes with a 10 foot bottom.

$n = 0.03$

Normal depth of flow, $y_o = 3.36$ ft.

Normal velocity, $V_o = 3.7$ fps.

Vertical drop, $h = 6$ ft.

Find: Straight drop structure dimensions

Solution:

1. $$H = 3.36 \text{ ft} + \frac{(3.7 \text{ fps})^2}{2 \times 32.2 \text{ ft/sec}^2} = 3.57 \text{ ft.}$$

2. $y_c = (2/3)\ (3.57\ ft.) = 2.38\ ft.$

3. $y_3 = 2.15\ (2.38\ ft.) = 5.12\ ft.$

4. $h_2 = -\ (6.0 - 3.36) = -2.64\ ft.$

5. $h_o = -2.64 - 5.12 = -7.76\ ft.$

   The floor of the stilling basin is, therefore, 1.76 ft. below the grade line of the downstream channel.

6. $h_o/y_c = -7.76/2.38 = -3.26$
   $h_2/y_c = -2.64/2.38 = -1.11$

   From Figure IX-A-2

   $L_1/y_c = 3.95$
   $L_1 = 9.40\ ft.$
   $L_2 = 0.8\ y_c = (0.8)(2.38) = 1.90\ ft.$
   $L_3 \geq 1.75\ y_c = 1.75(2.38) = 4.17\ ft.$ or $4.20\ ft.$
   $L_B = 9.40 + 1.90 + 4.20 = 15.50\ ft.$

7. Proportion floor blocks

   a. Height $= 0.8\ y_c = 0.8(2.38) = 1.90\ ft.$

   b. Width $= 0.4\ y_c = 0.4(2.38) = 0.95\ ft.$
      Spacing $= 0.4\ y_c = 0.4(2.38) = 0.95\ ft.$

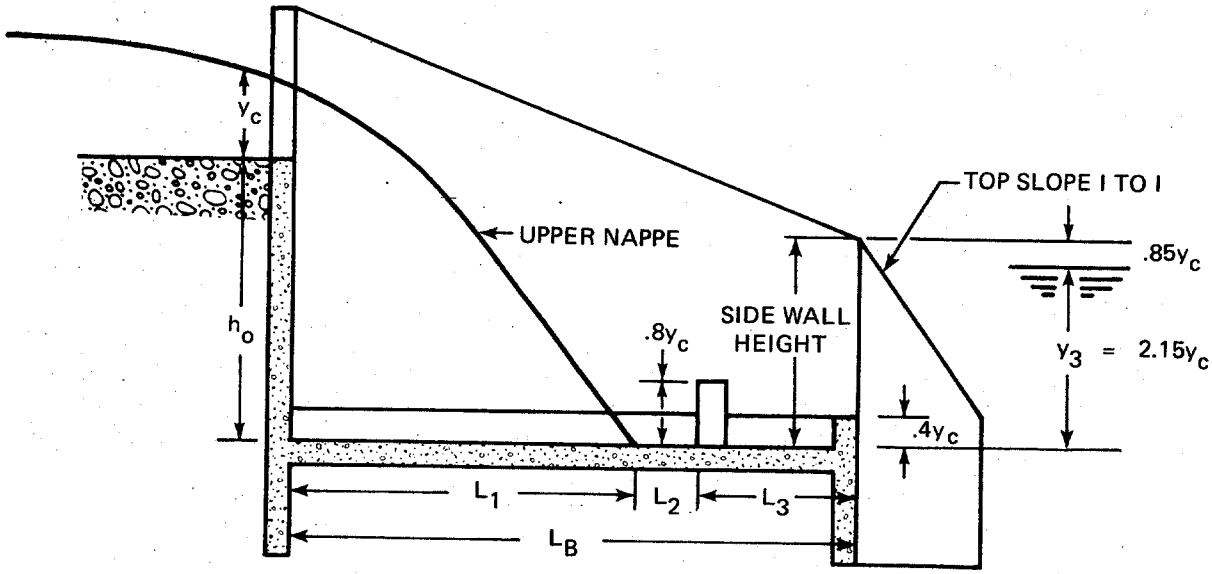
8. Calculate end sill height $= 0.4\ y_c = 0.4(2.38) = 0.95\ ft.$

9. Use longitudinal sills passing through the floor blocks.

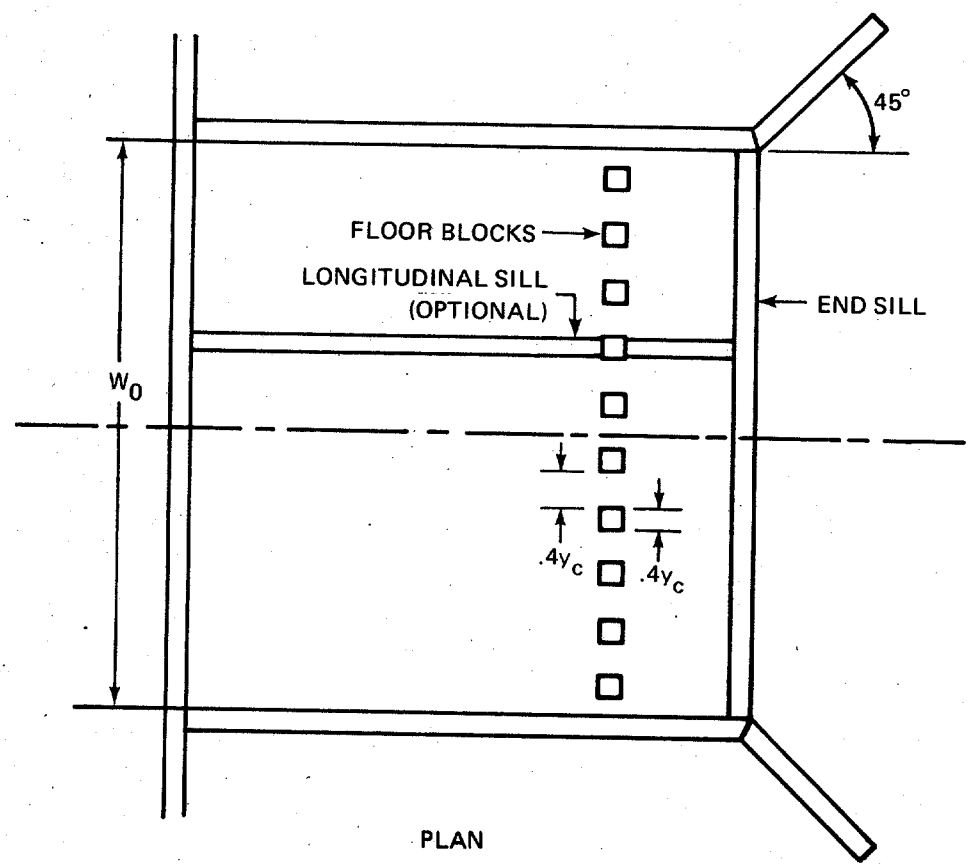
10. Calculate sidewall height above tailwater
    $= 0.85\ y_c = 0.85(2.38) = 1.90\ ft.$

11. Locate wingwalls at 45° angle with outlet centerline.

12. Protect approach channel with riprap or paving for 7.14 ft. (3 X 2.38) upstream of the headwall.



SECTION AT CENTER LINE



PLAN

FIGURE IX - A - 1 STRAIGHT DROP SPILLWAY STILLING BASIN FROM REFERENCE IX-I

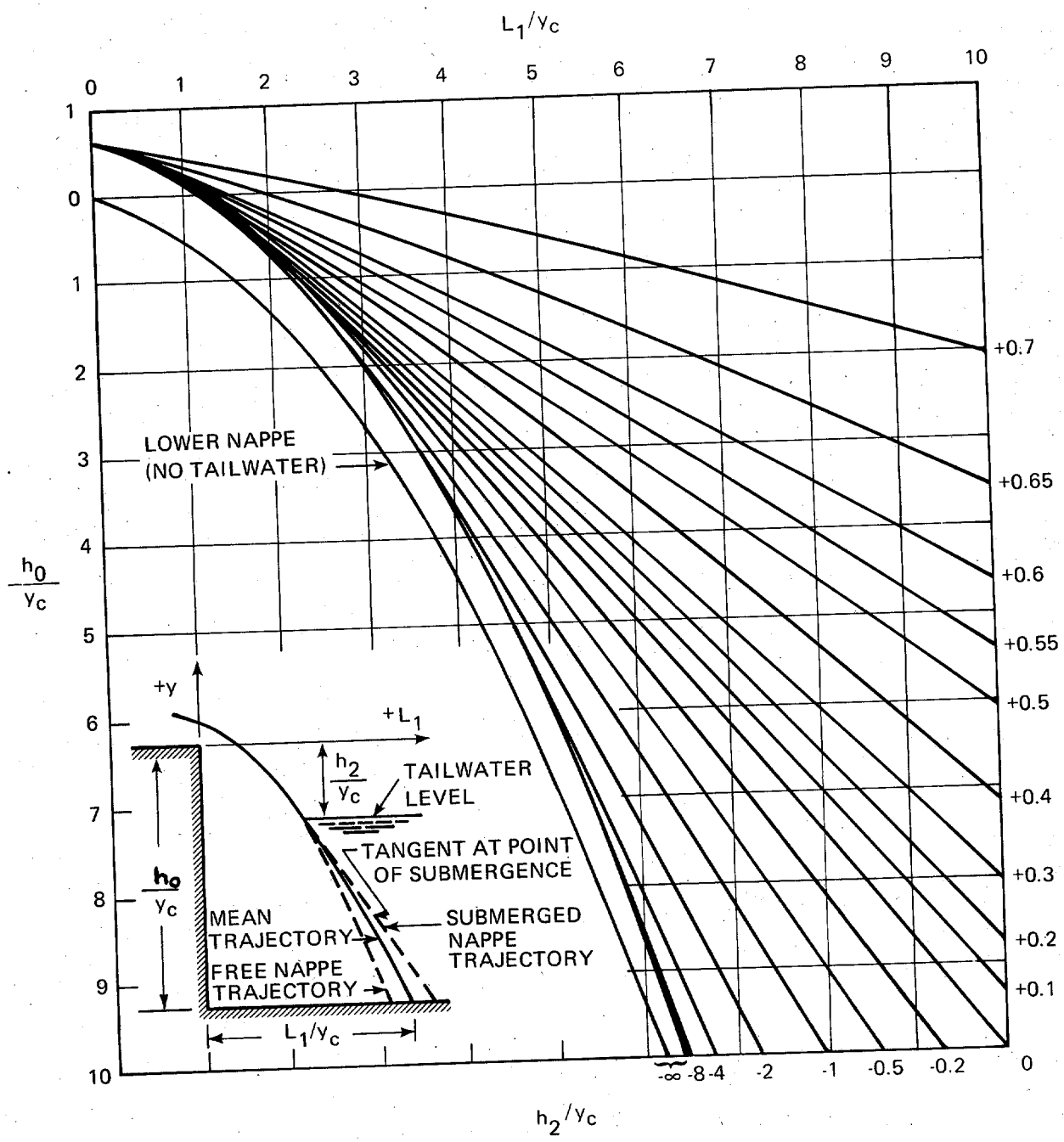


FIGURE IX - A - 2 DESIGN CHART FOR DETERMINATION OF L_1

IX-B. BOX INLET DROP STRUCTURE

The box inlet drop structure may be described as a rectangular box open at the top and downstream end (figure IX-B-1). Water is directed to the crest of the box inlet by earth dikes and a headwall . Flow enters over the upstream end and two sides. The long crest of the box inlet permits large flows to pass at relatively low heads. The width of the structure need be no greater than the downstream channel. It is applicable for drops from 2 feet to 12 feet.

The outlet structure can be adjusted to fit a wide variety of field conditions. It is possible to lengthen the straight section and cover it to form a highway culvert. The sidewalls of the stilling basin section can be flared if desired, thus permitting use with narrow channels or wide flood plains. Flaring the sidewalls also makes it possible to adjust the outlet depth to that in the natural channel.

The design information is based on an extensive experimental program performed by the Soil Conservation Service, St. Anthony Falls Hydraulic Laboratory, Minneapolis, (IX-B-1).

Two different sections are effective in controlling the flow: the crest of the box inlet and the opening in the headwall. The flow at which the control changes from one point to the other is dependent upon a number of factors, the principal factors being the box-inlet depth and its length.

DESIGN DISCUSSION

The design of the box inlet drop structure involves determining which section (crest or headwall opening) controls at the design flow.

The initial step is to choose a drop height, h_o, which will reduce the channel slope to a mild slope. Assume crest control and calculate the head, y_o, at the crest of the box inlet drop structure for the design discharge. The general equation relating discharge to head for a rectangular weir is

$$Q=3.43L_c y_o^{3/2}$$

Solving for head

$$y_o=(Q/3.43L_c)^{2/3} \quad \text{IX-B-1}$$

where L_c= length of box inlet crest = W_2+2L_1
W_2= width of box inlet
L_1= length of box inlet

Various lengths of crest, L_c, are chosen and equation IX-B-1 solved to obtain a head acceptable for the existing conditions.

The discharge coefficient (3.43) in equation IX-B-1 must be multiplied by:

a. The correction for head given in figure IX-B-2.

b. The correction for box inlet shape given in figure IX-B-3.

c. The correction for approach channel width given in figure IX-B-4.

d. The correction for dike proximity to the box inlet crest given in table IX-B-1--these values have a low precision.

It is not necessary to make these correction until after it is determined which section controls the design flow. The design assumes that the approach channel is level with the crest of the inlet.

The precision of the design curves is within ±7 percent when there is no dike effect and ±15 percent when dikes are used.

Next, assume control at the headwall opening and calculate the head, y_o, to determine if this head is greater than that obtained for the box-inlet crest. The general equation relating discharge and head for a rectangular weir is

$$Q=C_2W_2(2g)^{1/2}(y_o+C_H)^{3/2}$$

Solving for head

$$y_o=[Q/C_2W_2(2g)^{1/2}]^{2/3}-C_H \quad . . . . . \text{IX-B-2}$$

The discharge coefficient, C_2, is obtained from figure IX-B-5.

The head correction C_h is given in figure IX-B-6. If h_o/W_2 is between 1/4 and 1, C_h may be more readily determined from figure IX-B-6.

The precision of the design curves for headwall control is probably within ±10 percent.

When the box inlet drop structure operates under submerged conditions, reference should be made to the reports entitled "Hydraulic Design of the Box Inlet Drop Spillway" (IX-B-1) to determine the submerged design. However, this is not a desirable design condition.

The outlet for a box inlet drop structure should be designed as follows. Critical depth in the straight section is

$$y_c=[Q/W_2)^2/g)]^{1/3} \quad \text{IX-B-3}$$

Critical depth at the exit of the stilling basin is

$$y_{c3}=[(Q/W_3)^2/g]^{1/3} \quad \text{IX-B-4}$$

The minimum length of the straight section is

$$L_2=y_c(.2/(L_1/W_2)+1) \quad \text{IX-B-5}$$

for values of L_1/W_2 equal to or greater than 0.25.

The sidewalls of the stilling basin may flare from 1 to infinity (parallel extensions of the section walls) to 1 transverse in 2 longitudinal.

The minimum length of the stilling basin is

$$L_3=L_c/(2L_1/W_2) \quad \text{IX-B-6a}$$

or

$$L_3=z(W_3-W_2)/2 \quad \text{IX-B-6b}$$

which ever is larger. However, equation IX-B-6a is valid for L_1/W_2 values equal to or greater than 0.25, only.

When the stilling basin is less than $11.5y_{c3}$ wide at the exit, the minimum tailwater depth over the basin floor is

$$y_3=1.6y_{c3} \quad \text{IX-B-7}$$

When the stilling basin is more than $11.5y_{c3}$ wide at the exit, the minimum tailwater depth over the basin floor is

$$y_3=y_{c3}+0.052W_3 \quad \text{IX-B-8}$$

However, a stilling basin as wide as $11.5y_{c3}$ may make inefficient use of the outlet.

The height of the end sill is

$$h_4=y_3/6 \quad \ldots\ldots\ldots\ldots \text{IX-B-9}$$

Longitudinal sills will improve the flow distribution in the outlet. Considerations for their use are:

a. When the stilling basin sidewalls are parallel, the longitudinal sills may be omitted.

b. The center pair of longitudinal sills should start at the exit of the box inlet and extend through the straight section and stilling basin to the end sill.

c. When W_3 is less than $2.5W_2$, only two sills are needed. These sills should be located at a distance W_5, each side of the centerline.

d. When W_3 exceeds $2.5W_2$ two additional sills are required. These sills should be located parallel to the outlet centerline and midway between the center sills and the sidewalls at the exit of the stilling basin.

e. The height of the longitudinal sills should be the same as the height of the end sill.

The minimum height of the sidewalls above the water surface at the exit of the stilling basin should be

$$h_3=y_3/3 \quad \ldots\ldots\ldots\ldots \text{IX-B-10}$$

The sidewalls should extend above the tailwater surface under all conditions.

The wingwalls should be triangular in elevation and have a top slope of 45° with the horizontal. Top slopes as flat as 30° are permissible.

The wingwalls should flare in plan at an angle of 60° with the outlet centerline. Flare angles as small as 45° are permissible; however, wingwalls parallel to the outlet centerline are not recommended.

TABLE IX-B-1. CORRECTION FOR DIKE EFFECT, C_E
(Control at Box-Inlet Crest)

$\frac{L_1}{W_2}$	W_4/W_2						
	0.0	0.1	0.2	0.3	0.4	0.5	0.6
0.5	0.90	0.96	1.00	1.02	1.04	1.05	1.05
1.0	.80	.88	.93	.96	.98	1.00	1.01
1.5	.76	.83	.88	.92	.94	.96	.97
2.0	.76	.83	.88	.92	.94	.96	.97

DESIGN PROCEDURE

1. Select h_o.

2. Select L_1, W_2, and L_c.

3. Calculate y_o for the crest, equation IX-B-1

4. Calculate h_o/W_2 and determine coefficient of discharge, C_2; figure IX-B-5.

5. Calculate L_1/h_o and determine relative head correction, C_H; figure IX-B-6.

6. Calculate y_o for the headwall opening; equation IX-B-2.

7. Compare the values of y_o obtained from steps 3 and 6. The larger value controls. If crest controls, adjust y_o from step 3:

   a. Calculate y_o/W_2 and determine correction for head, C_1, figure IX-B-2

   b. Calculate L_1/W_2 and determine correction for box-inlet shape, C_S, figure IX-B-3.

   c. Calculate W_1/L_c and determine correction for approach channel width, C_A, figure IX-B-4.

   d. Calculate W_4/W_2 and determine correction for dike effect, C_E, table IX-B-1.

   e. Determine adjusted y_o for crest from corrections found in steps a through e.

8. Calculate y_c; equation IX-B-3

9. Calculate y_{c3}; equation IX-B-4

10. Calculate L_2; equation IX-B-5

11. Calculate L_3; equation IX-B-6a or b

12. Calculate y_3
    If $W_3<11.5y_{c3}$, use equation IX-B-7
    If $W_3>11.5y_{c3}$, use equation IX-B-8

13. Calculate h_4; equation IX-B-9

14. Determine number of longitudinal sills
    If $W_3<2.5W_2$, use two sills
    If $W_3>2.5W_2$, use four sills

15. Calculate h_3; equation IX-B-10

Example Problem

Given: $Q=250$ cfs,
$TW=2.8$ feet,
$S_o=0.002$ ft/ft, and the downstream channel has 2:1 side slopes with a 20 foot bottom.

Find: A box inlet drop structure dimensions

Solution:

1. Select $h_o=4$ feet

2. Select $L_c=12$ feet
   $L_1=4$ feet
   $W_2=4$ feet

3. Compute y_o for crest control

   $y_o=(Q/3.43L_c)^{2/3}=(250/3.43x12)^{2/3}=3.33$ feet

4. Calculate $h_o/W_2=4.0/4=1.0$

   $C_2\sqrt{2g}=3.43$ from figure IX-B-5
   $C_2=0.43$

5. Calculate $L_1/h_o=4/4=1.0$
   $C_H/h_o=0.49$, figure IX-B-6
   $C_H=1.96$

6. Compute y_o for headwall control

   $y_o=[Q/c_2W_2(2g)^{1/2}]^{2/3}-C_H=[250/0.43(4)(64.4)^{.5}]^{2/3}-1.96$
   $=4.94$ feet

7. The head for the headwall controls, 4.94>3.33, so steps 7a through 7e are omitted.

8. $Y_c=[(Q/W_2)^2/g]^{1/3}=[(250/4)^2/32.2]^{1/3}=4.95$ feet

9. $Y_{c3}=[(Q/W_3)^2/g]^{1/3}$
where $W_2=20$ feet = downstream channel bottom width
$Y_{c3}=[(250/20)^2/g]^{1/3}$
$Y_{c3}=1.69$ feet

10. Calculate L_2

$L_2=Y_c(0.2/(L_1/W_2)+1)=4.94(0.2/(4/4)+1)=5.93$ feet

11. Calculate L_3
$L_3=L_c/(2L_1/W_2)$ or
$L_3=z(W_3-W_2)/2$ which ever is larger
$Z=2.0$
Since $L_1/W=4/4=1$ the first equation is valid
$L_3=12[2(4/4)]=24$ feet
$L_3=2.0(20-4)/2=16$ feet
use $L_3=24$ feet

12. $11.5(Y_{c3})=11.5(1.69)=19.44<W_3$
so $Y_3=Y_{c3}+0.052W_3$

$Y_3=1.69+0.052(20.0)=2.73$ feet

13. $h_4=Y_3/6=2.73/6=0.46$ feet

14. Longitudinal sills
$2.5W_2=2.5(4)=10$
$W_3>10$ so use four sills

15. $h_3=Y_3/3=2.73/3=0.91$ feet

IX-1. Rand, Walter, FLOW GEOMETRY AT STRAIGHT DROP SPILLWAYS, Paper 791, Proceedings, ASCE, Volume 81, pp. 1-13, September 1955.

IX-2. Chow, Ven Te, OPEN-CHANNEL HYDRAULICS, McGraw-Hill Book Company, Inc., New York, 1959.

IX-A-1. Donnelly, Charles A., and Blaisdell, Fred W., STRAIGHT DROP SPILLWAY STILLING BASIN, University of Minnesota, St. Anthony Falls Hydraulic Laboratory, Technical Paper 15, Series B, November 1954.

IX-B-1. Blaisdell, Fred W., and Donnelly, Charles A., THE BOX INLET DROP SPILLWAY AND ITS OUTLET, Transactions, ASCE, Vol. 121, pp. 955-986, 1956.

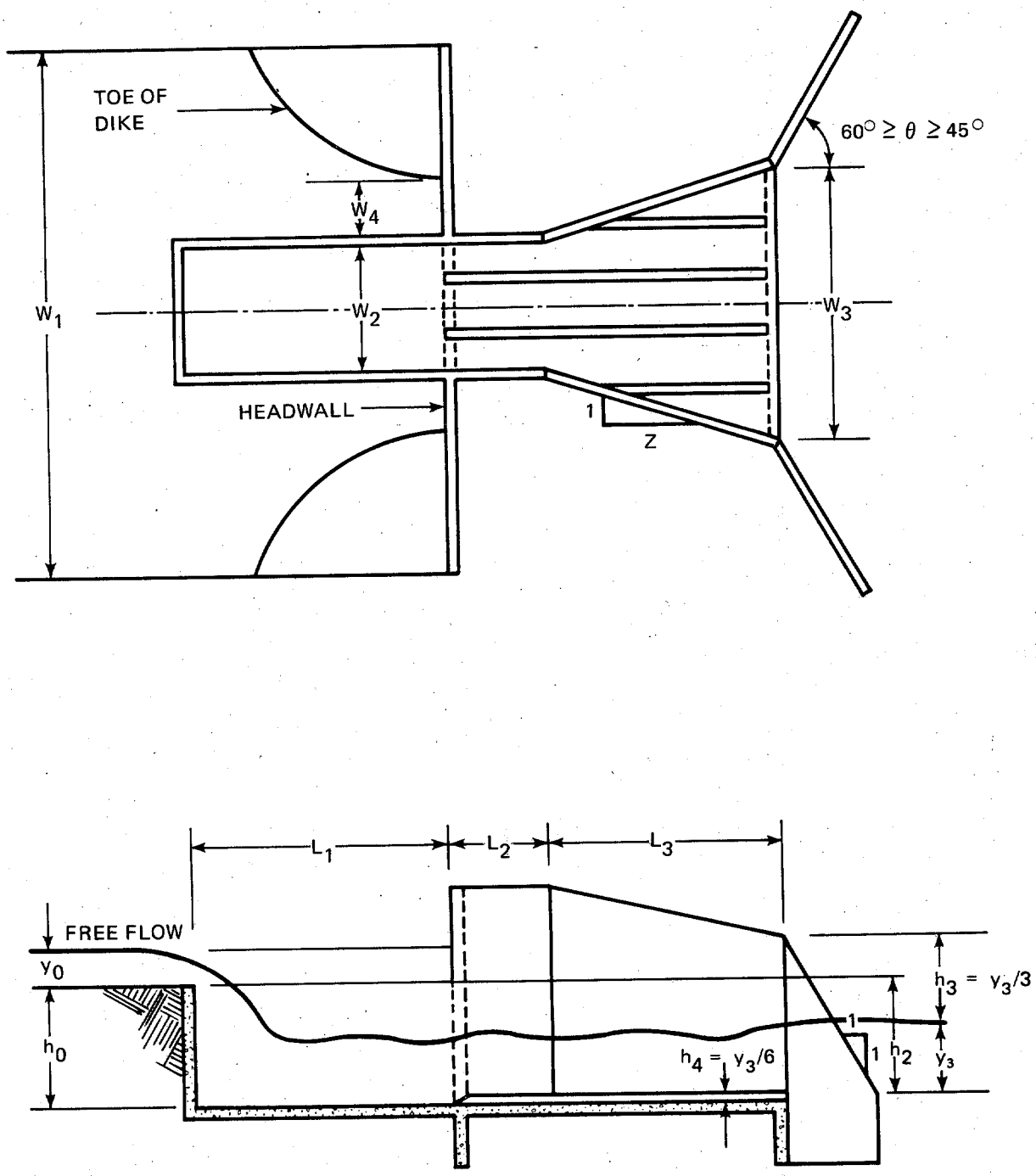


FIGURE IX - B - 1 BOX - INLET DROP STRUCTURE

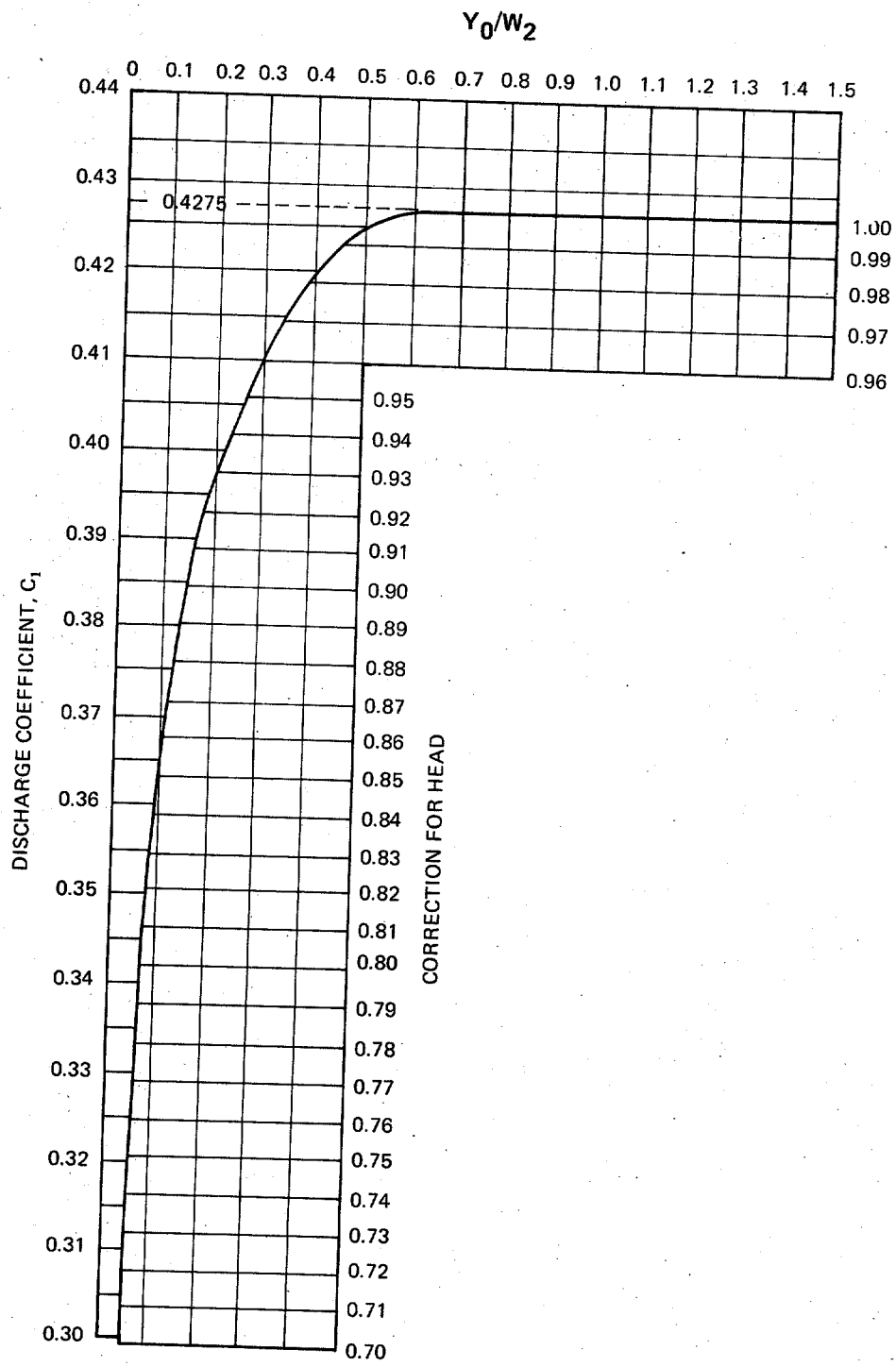


FIGURE IX - B - 2. DISCHARGE COEFFICIENT AND CORRECTION FOR HEAD, WITH CONTROL AT BOX - INLET CREST

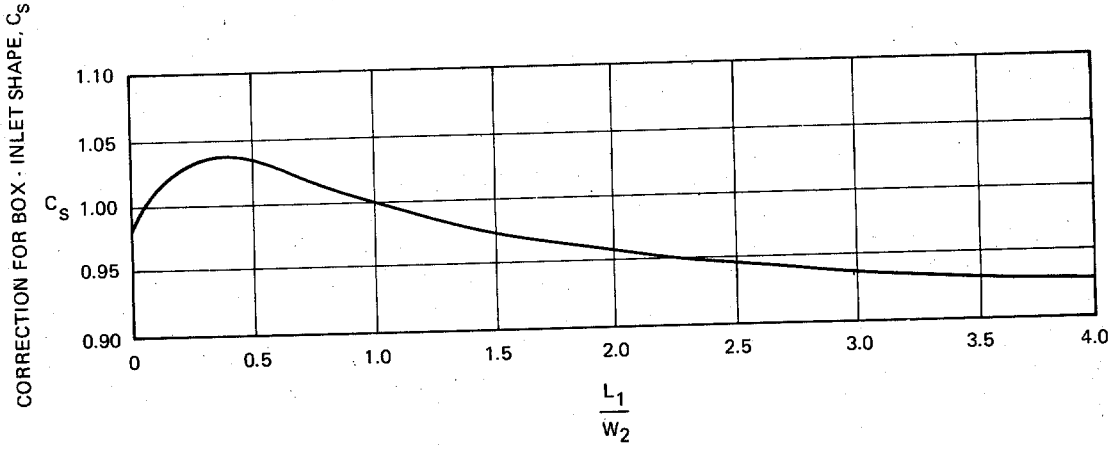


FIGURE IX - B - 3 CORRECTION FOR BOX - INLET SHAPE, WITH CONTROL AT BOX - INLET CREST ($\frac{W_1}{L_c} \geq 3$).

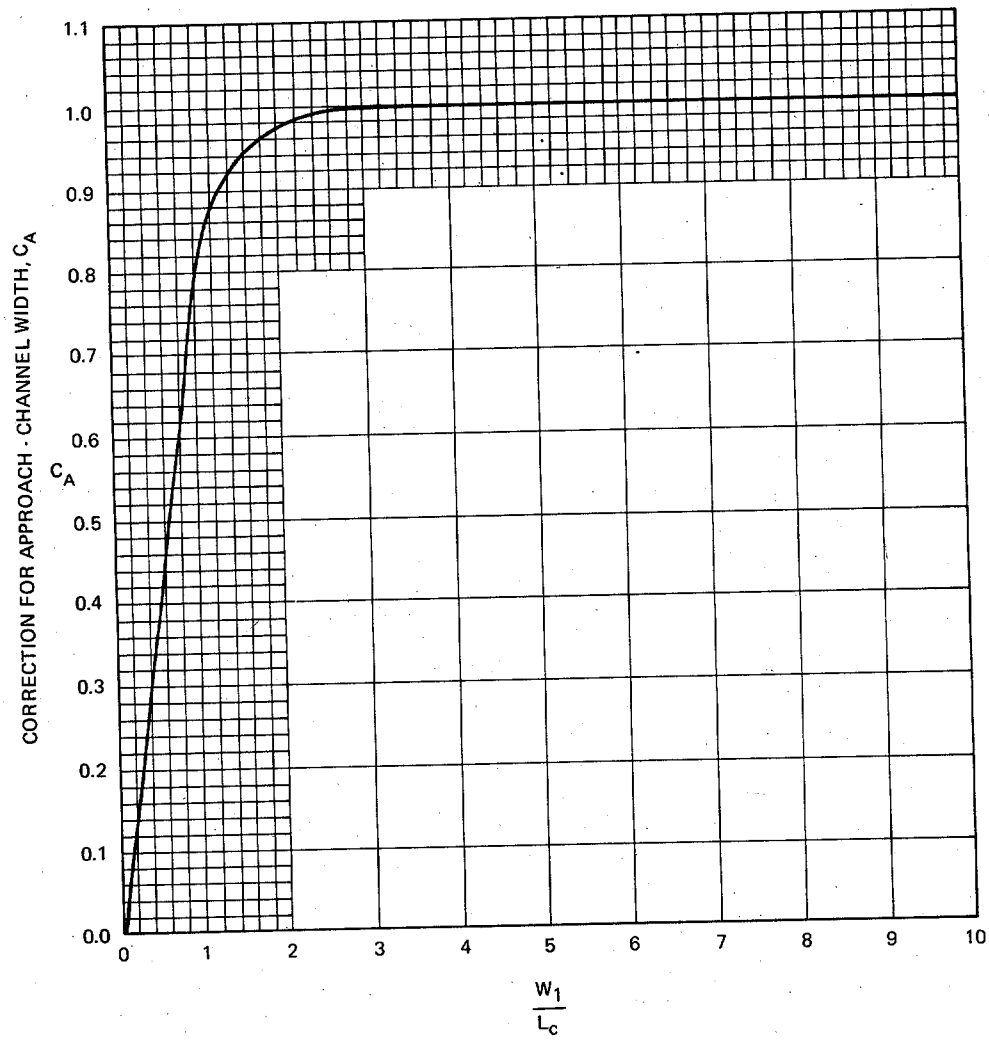


FIGURE IX - B - 4 CORRECTION FOR APPROACH - CHANNEL WIDTH, WITH CONTROL AT BOX - INLET CREST

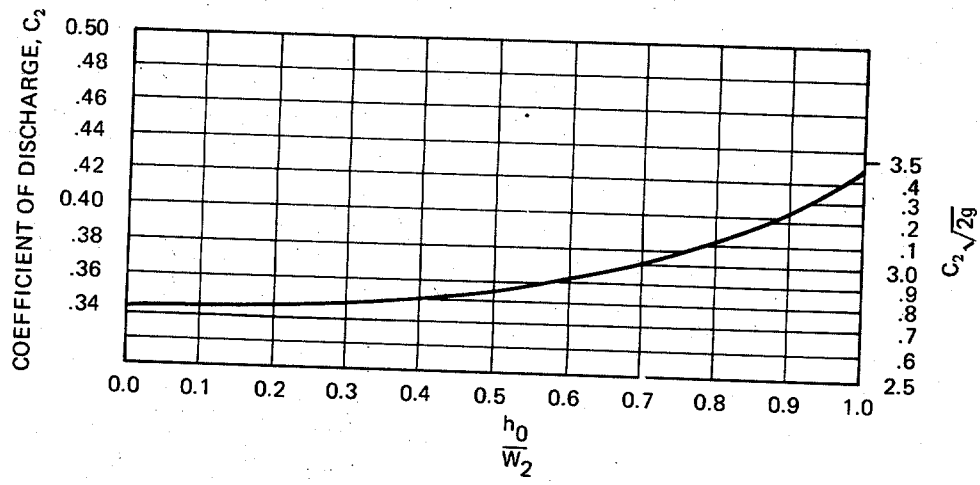


FIGURE IX - B - 5 **COEFFICIENT OF DISCHARGE, WITH CONTROL AT HEADWALL OPENING**

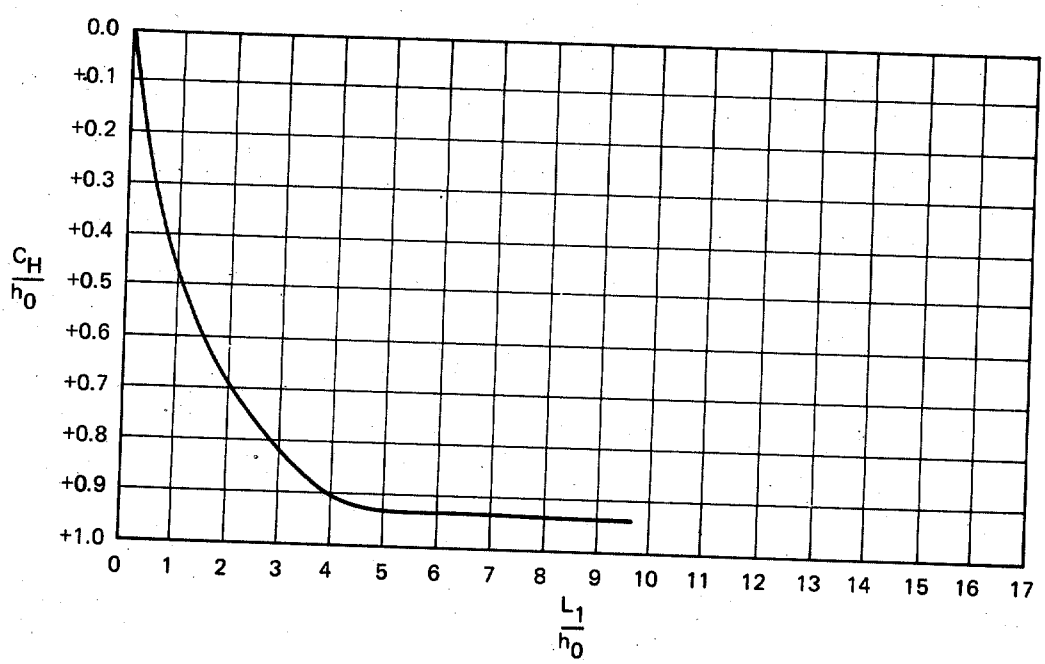


FIGURE IX - B - 6 **RELATIVE HEAD CORRECTION FOR $h_0/W_2 \geq ¼$, WITH CONTROL AT HEADWALL OPENING**

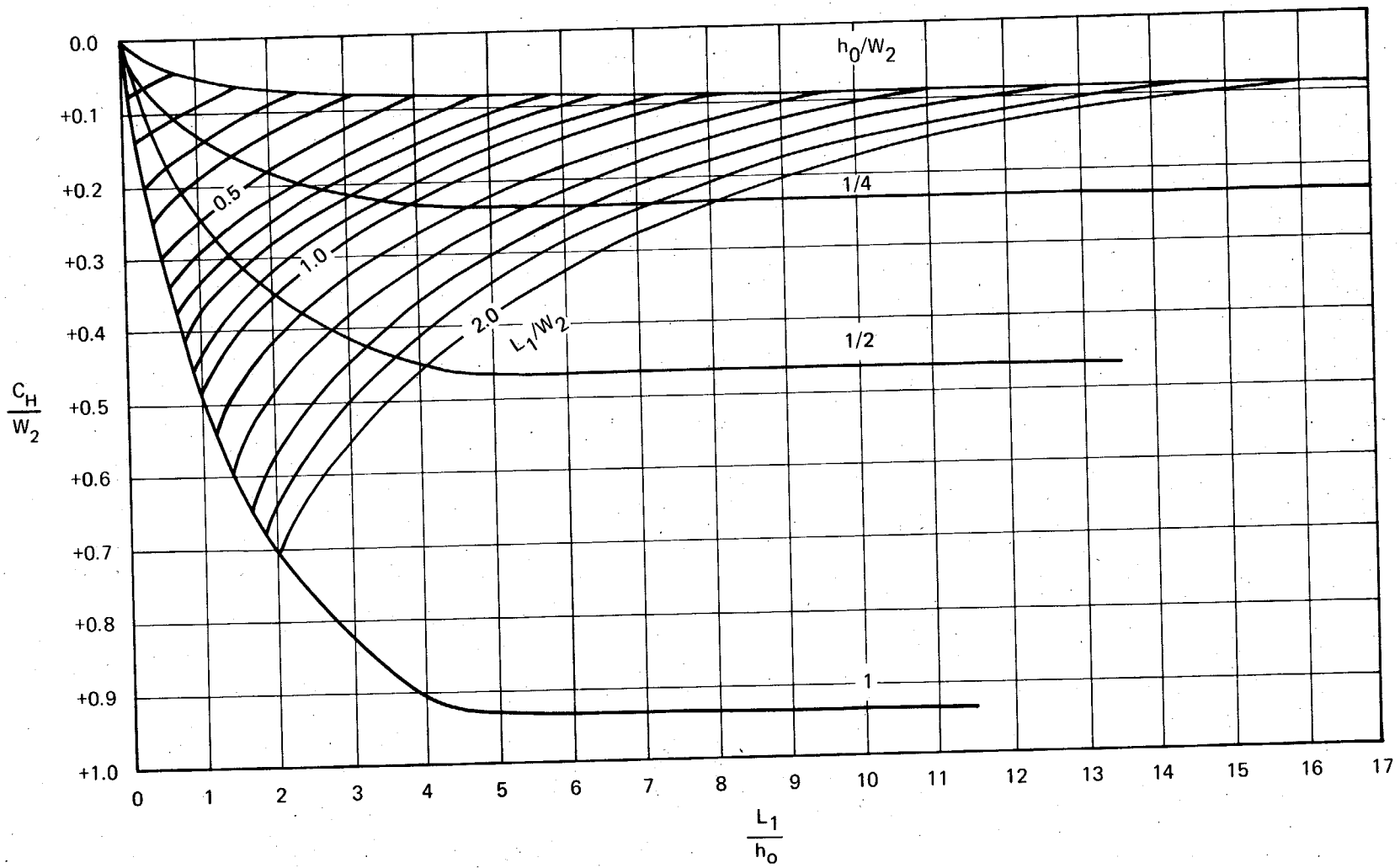


FIGURE IX-B-7 RELATIVE HEAD CORRECTION, WITH CONTROL AT HEADWALL OPENING

X. STILLING WELLS

Stilling wells dissipate kinetic energy by forcing the flow to travel vertically upward to reach the downstream channel. Two types of stilling wells are of interest to the highway engineer--the Manifold Stilling Basin and the Corps of Engineers Stilling Well.

X-A. MANIFOLD STILLING BASIN

The manifold stilling basin (see figure X-A-1) is unique since it dissipates the excess kinetic energy in a vertical upward direction instead of the conventional horizontal or vertical downward directions. As shown in figure X-A-1, it employs jets issuing upward into an overlying tailwater. Energy dissipation is accomplished in two ways: first, the jet entrains a part of the surrounding fluid and distributes the excess energy throughout a greater quantity of fluid. Much of this kinetic energy is converted into heat from the resulting shear, either directly or indirectly, by the creation of relatively fine-grained turbulence; second, the remaining kinetic energy creates a boil (a rise in the water surface as shown in figure X-A-2) and is rapidly reduced and dispersed as the boil spreads radially.

Advantages and Disadvantages

There are advantages and disadvantages in using the manifold as an energy dissipator in highway work. Some of the advantages are: the manifold is an efficient energy dissipator, it can be used where the total drop in elevation of the water surface is several tens of feet, it has no high outlet velocity or concentrations of flow which can be directed against the bed or banks, it can be used in either open channels or closed conduits, and it is an economical structure to build. On the other hand, its disadvantages are; it needs a fairly deep tailwater to function efficiently; it may be subject to clogging if debris is a problem and would require a debris-control structure upstream of its entrance, and it could become a rather long structure for large discharges.

An ideal site for a manifold dissipator is the relocation of hydroelectric or irrigation canals, where a drop in elevation is involved, discharge is controlled and debris is minimal. Another potential site is at outlet conduits where highway embankments are constructed as earth-fill dams.

Design Recommendations

The successful manifold design is based on two requirements. The manifold must be such that the velocity and discharge per foot of length are approximately the same at all points; and the design must be practical both in shape and dimensions to meet economic and construction requirements. The following design considerations are recommended to meet these criteria:

(1) Uniform distribution of the velocity is assumed at the entrance of the manifold which is rectangular in shape. When circular conduits are used, a transition from circular to rectangular, two to three diameters in length, is recommended. This provides a reasonably uniform velocity distribution.

(2) The length of the basin (L_B) may vary to fit particular locations, but the ratio of L_B, to the square root of approach culvert area (A), must be less than or equal to 10.

$$L_B/\sqrt{A} < 10 \quad \text{X-A-1}$$

(3) The bars at the top of the manifold should be square in cross section. The space between the bars (L_2) can vary but L_2/L_1 should be 0.5, 1.0, 1.5, or 2.0. Number of slots (N) = $(L_B)/(L_2+L_1)$.

(4) The jet velocity (V_1) can be found from table X-A-1.

TABLE NO. X-A-1

For ranges of y/L_1 from 5 to 20

L_2/L_1	V_1/V_o
0.5	1.38
1.0	1.24
1.5	1.19
2.0	1.14

(5) The effective jet width (L_3) is calculated by dividing the discharge per slot (Q_1) = Q/N by V_1 to obtain the jet area A_1; then dividing A_1 by the basin width (W_B).

$$L_3 = Q/NV_1W_B \quad \text{X-A-2}$$

(6) Waves caused by the boil of the vertical jet may result in some erosion of the channel banks and bed. The degree of erosion will be dependent on the wave height (h_W), which is a function of the boil height (h_B), and may be determined by the use of figure X-A-3. This figure is a plot of $h_W/(V_1{}^2/2g)$ against y/L_3 in terms of L_2/L_1. After determining the wave height, and considering the erodibility of the banks and bed, the designer may choose the type of protection needed for each installation.

(7) The boil height may also be computed if one desires. Figure X-A-4 gives the boil height (h_B) in the same manner that figure X-A-3 gives the wave height.

Summary

(1) The entrance cross section is either square or rectangular with the width of the manifold (W_B) equal to the width of the incoming conduit.

(2) The length of the manifold may vary, but must conform to equation X-A-1.

(3) Choose (L_2) and (L_1) and calculate N and L_2/L_1. L_2/L_1 should be either 0.5, 1.0, 1.5, or 2.0.

(4) Calculate V_1 using table X-A-1.

(5) Use equation X-A-2 to find L_3.

(6) With the aid of figure X-A-3 find h_W. The amount of erosion protection required will depend on site conditions.

(7) Figure X-A-4 is used to find h_B.

Example Problem

The following example problem was taken from the paper by Fiala and Albertson (X-A-1).

Given: Q = 300 cfs, D of pipe = 72", and tailwater depth (y) = 8 ft.

Find: Design a manifold stilling basin for the outlet of the pipe so that the wave height run-up on the 2:1 side slope of the channel downstream does not exceed 1.5 feet.

Solution:

(1) To help provide the assumed uniform velocity distribution at the manifold entrance, use a transition of length 2D (2 x 6' = 12') from the circular pipe to the square entrance of the 6' x 6' entrance dimension of the manifold.

(2) Assume an $L_B/\sqrt{A}$ ratio of 4. Then $L_B=4\sqrt{36}=4(6)=24$ ft.

(3) Also assume, for ease of construction that $L_1=L_2=12"$ which makes the number of slots $(N)=L_B/(L_2+L_1)=24/2=12$ and L_2/L_1 ratio = 1.0, a ratio of one is satisfactory.

(4) In order to be conservative in estimating the velocity at the manifold entrance, use the pipe area to determine V_o. $V_o=Q/A=300/\pi 6^2/4=10.6$ fps. The y/L_1 ratio = 8 /1 =8. This lies within the experimental range of 5 to 20 and from table X-A-1 with a L_2/L_1 ratio of 1.0 the V_1/V_o ratio = 1.24 where V_1 is the jet velocity. $V_1=1.24V_o=1.24(10.6)=13.2$ fps.

(5) The effective jet width $(L_3)=Q/NV_1W_B$ or $L_3=300/12(13.2)6=.316$ ft.

(6) Obtain wave height (h_W) from figure X-A-3. For $L_2/L_1=1.0$ and $y/L_3=8/.316=25.3, h_W/V_1^2/2g=0.42$ $h_W=0.42(13.2)^2/64.4=1.14$ ft. which is less than the 1.5 ft. set up as a design condition.

(7) If the boil height (h_B) is desired, see figure X-A-4. For y/L_3 of 25.3 and $L_2/L_1=1.0$, $h_B/V_1^2/2g)=0.3$ $h_B=0.3(13.2)^2/64.4=.81$ ft.

Design Summary

$L_B=24'$	$A=36'$	$L_1=L_2=12"$	$N=12$
$V_o=10.6$ fps	$V_1=13.2$ fps	$y=8'$	
$L_3=0.316$ ft.	$h_W=1.4'$	$h_B=.81'$	

Prototype Installations

In the discussion and closure of the paper by Fiala and Albertson (X-A-1), three prototype manifold stilling basins were described. Each of these installations performed very satisfactorily and are described briefly as follows:

Site No. 1 - Located at end of pipe drop from reservoir to a canal. Longitudinal axis perpendicular to canal flow. Q = 225 cfs, vertical drop 36 ft. When manifold is not in operation it becomes completely filled with canal-carried sediment. This sediment is not a problem since it is carried away rapidly when flow starts through the manifold.

Site No. 2 - Located at end of pipe drop in a canal. Longitudinal axis aligned with canal flow, Q = 200 cfs, vertical drop 34 ft., minimum tailwater depth 6 ft.

Site No. 3 - Triple manifold installation for conduits through earth-storage dam. Longitudinal axis aligned with flow. Q = 500 cfs.

X-A-1. Fiala, G. R., and Albertson, M. L., MANIFOLD STILLING BASIN, Journal of Hydraulics Division, ASCE, HY-4, July 1961, pp. 55-81.

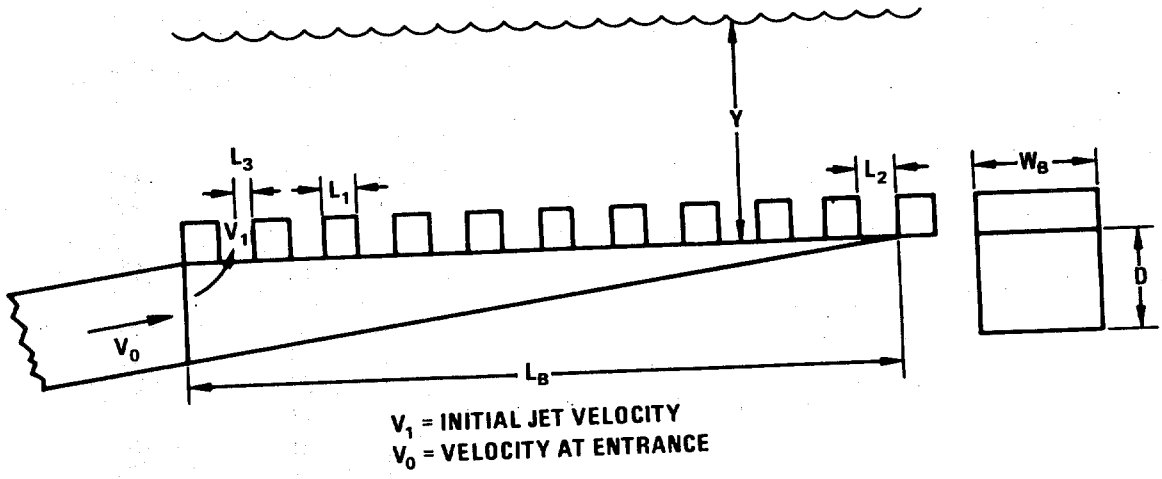


FIGURE X-A-1 MANIFOLD STILLING BASIN SKETCH

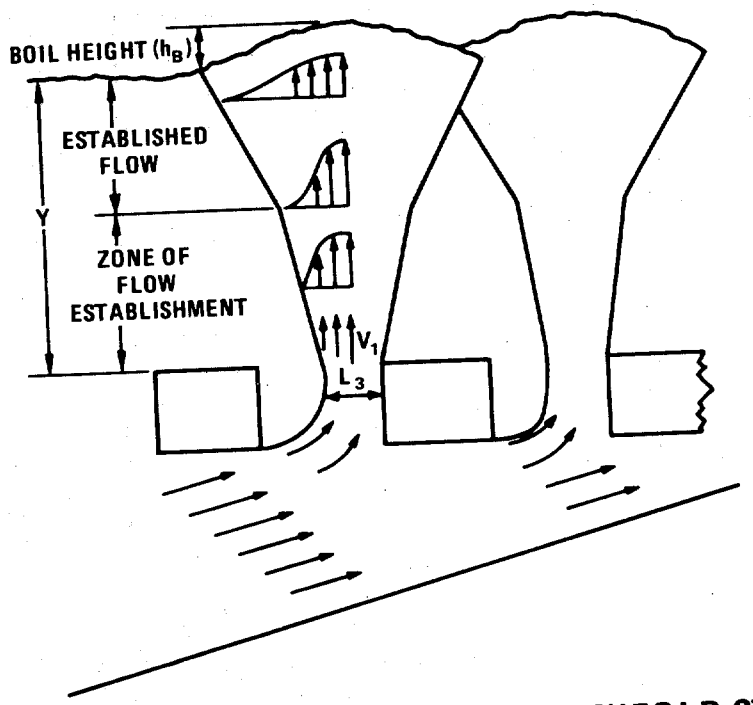


FIGURE X-A-2 JET DIAGRAM FOR MANIFOLD STILLING BASIN FROM REFERENCE X-A-1

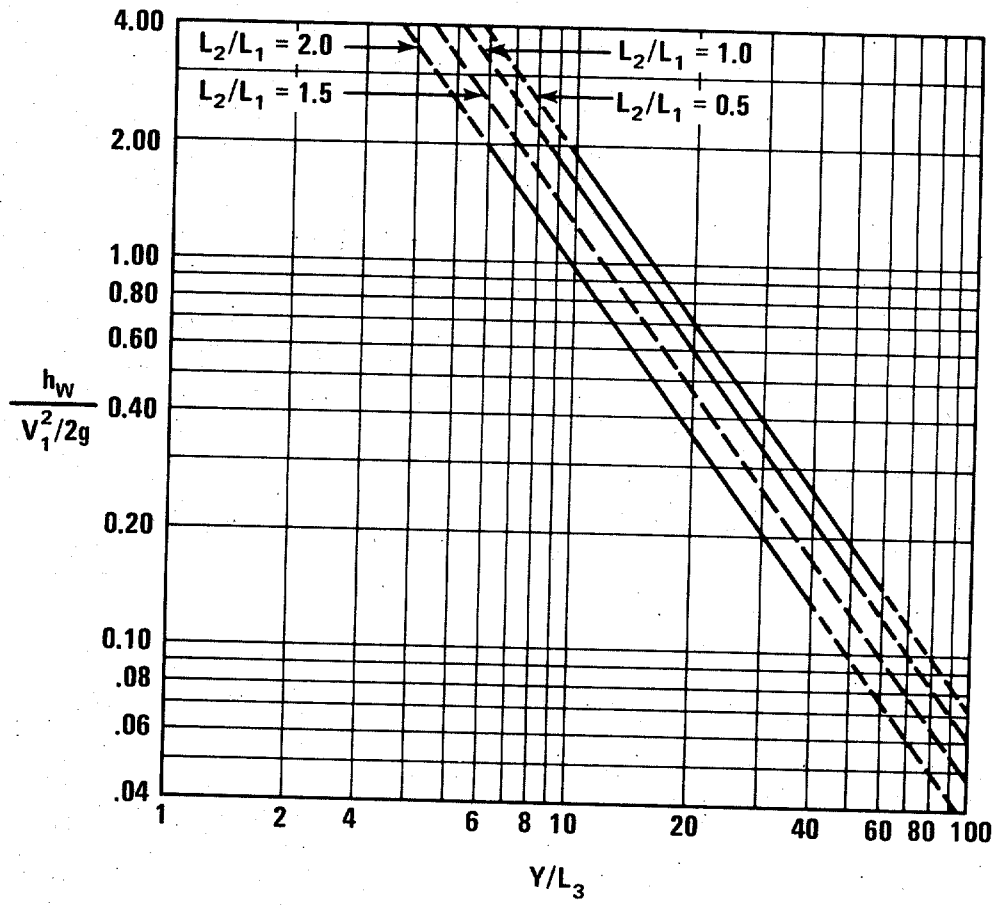


FIGURE X-A-3 WAVE HEIGHT (h_W) FOR MANIFOLD STILLING BASIN FROM REFERENCE X-A-1

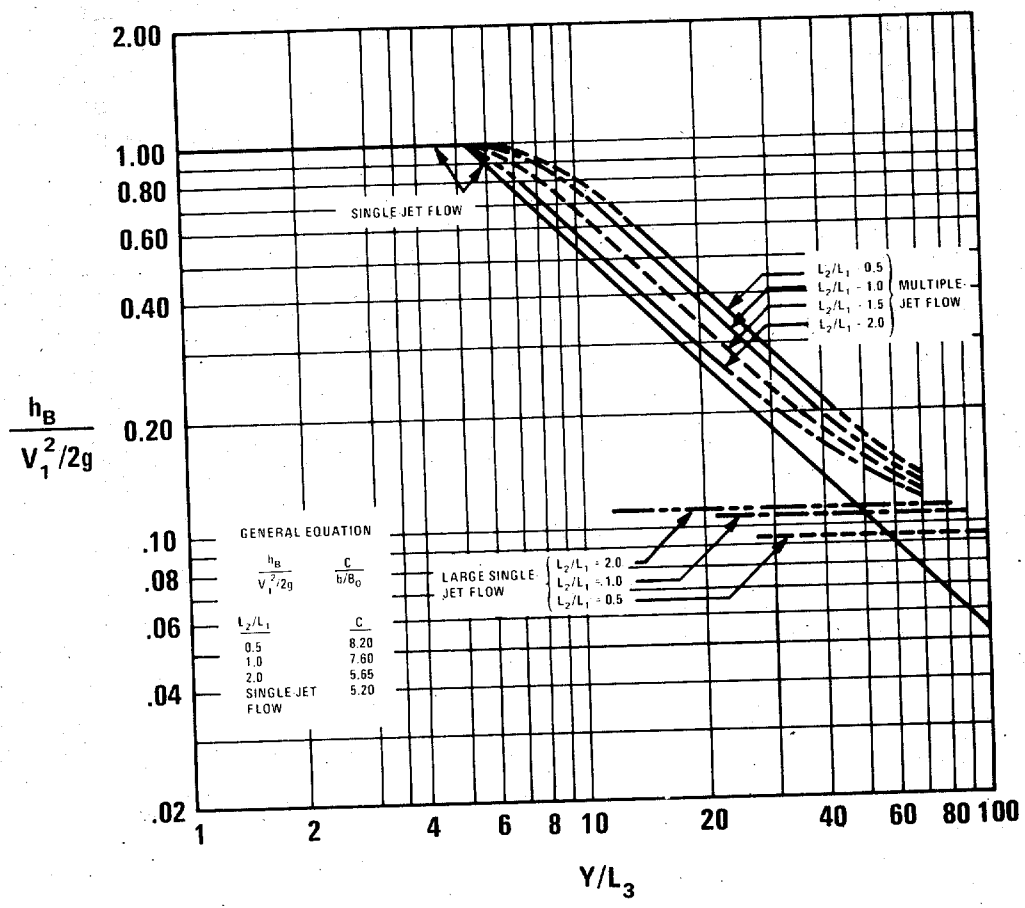


FIGURE X-A-4 BOIL HEIGHT (h_B) FOR MANIFOLD STILLING BASIN FROM REFERENCE X-A-1

X-B. CORPS OF ENGINEERS STILLING WELL

The design of this type of stilling well energy dissipator is based on model tests conducted by the Corps of Engineers. (X-B-1 and 2)

The dissipator has application where debris is not a serious problem. It will operate with moderate to high concentrations of sand and silt but is not recommended for areas where quantities of large floating or rolling debris is expected unless suitable debris-control structures are utilized. Its greatest potential, as far as highways are concerned, is at the outfalls of storm drains, median, and pipe down drains where little debris is expected. It may also be useful as a temporary erosion control device during construction.

Design Recommendations

The design is straightforward. Once the size and discharge of the incoming pipe are determined, figure X-B-1 is used to select the stilling well diameter (D_W). The model tests indicated that satisfactory performance can be maintained for $Q/D^{5/2}$ ratios as large as 10.0, with stilling well diameters of one, two, three, and five times that of the incoming conduits. These ratios were used to define the curves shown in figure X-B-1.

The tests also indicated that there is an optimum depth of stilling well below the invert of the incoming pipe. This depth is determined by entering figure X-B-2 with the slope of the incoming pipe and using the stilling well diameter (D_W) previously obtained from figure X-B-1.

The height of the stilling well above the invert is fixed at twice the diameter of the incoming pipe (2D). This dimension results in satisfactory operation and is practical from a cost standpoint; however, if increased, greater efficiency will result.

Tailwater also increases the efficiency of the stilling well. Whenever possible, it should be located in a sump or depressed area.

It is recommended that riprap or other types of channel protection be provided around the stilling well outlet and for a distance of at least $3D_W$ downstream.

The outlet may also be covered with a screen or grate for safety. However, the screen or grate should have a clear opening area of at least 75 percent of the total stilling well area and be capable of passing small floating debris such as cans and bottles.

Design Procedures

(1) Select approach pipe diameter (D) and discharge (Q).

(2) Obtain well diameter (D_W) from figure X-B-1.

(3) Calculate the culvert slope = (Vertical/horizontal distance). The depth of the well below the culvert invert (h_1) is determined from figure X-B-2.

(4) The depth of the well above the culvert invert (h_2) is equal to 2(D) as a minimum but may be greater if the site permits.

(5) The total height of the well (h_W) = h_1+h_2.

Example Problem

Given: 24" CMP downdrain on a 2:1 slope carrying a Q = 15 cfs

Find: Stilling well dimensions

Solution:

(1) D=2 ft., Q=15 cfs
(2) From figure X-B-1 D_W=1.5D=3 ft.
(3) Slope=1/2=.5, h_1/D_W=.42 from figure X-B-2
h_1=.42(3.0)=1.26 ft., Use h_1=1.3 ft.
(4) h_2=2(D)=2(2)=4 ft.
(5) h_W= h_1+h_2=1.3+4=5.3 ft.

X-B-1. IMPACT-TYPE ENERGY DISSIPATOR FOR STORM-DRAINAGE OUTFALLS STILLING WELL DESIGN, U. S. Army Corps of Engineers, Technical Report No. 2-620 March 1963, WES, Vicksburg, Mississippi.

X-B-2. Grace, J. L., Pickering, G. A., EVALUATION OF THREE ENERGY DISSIPATORS FOR STORM DRAIN OUTLETS, U.S. Army WES, HRB 1971, Washington, D.C.

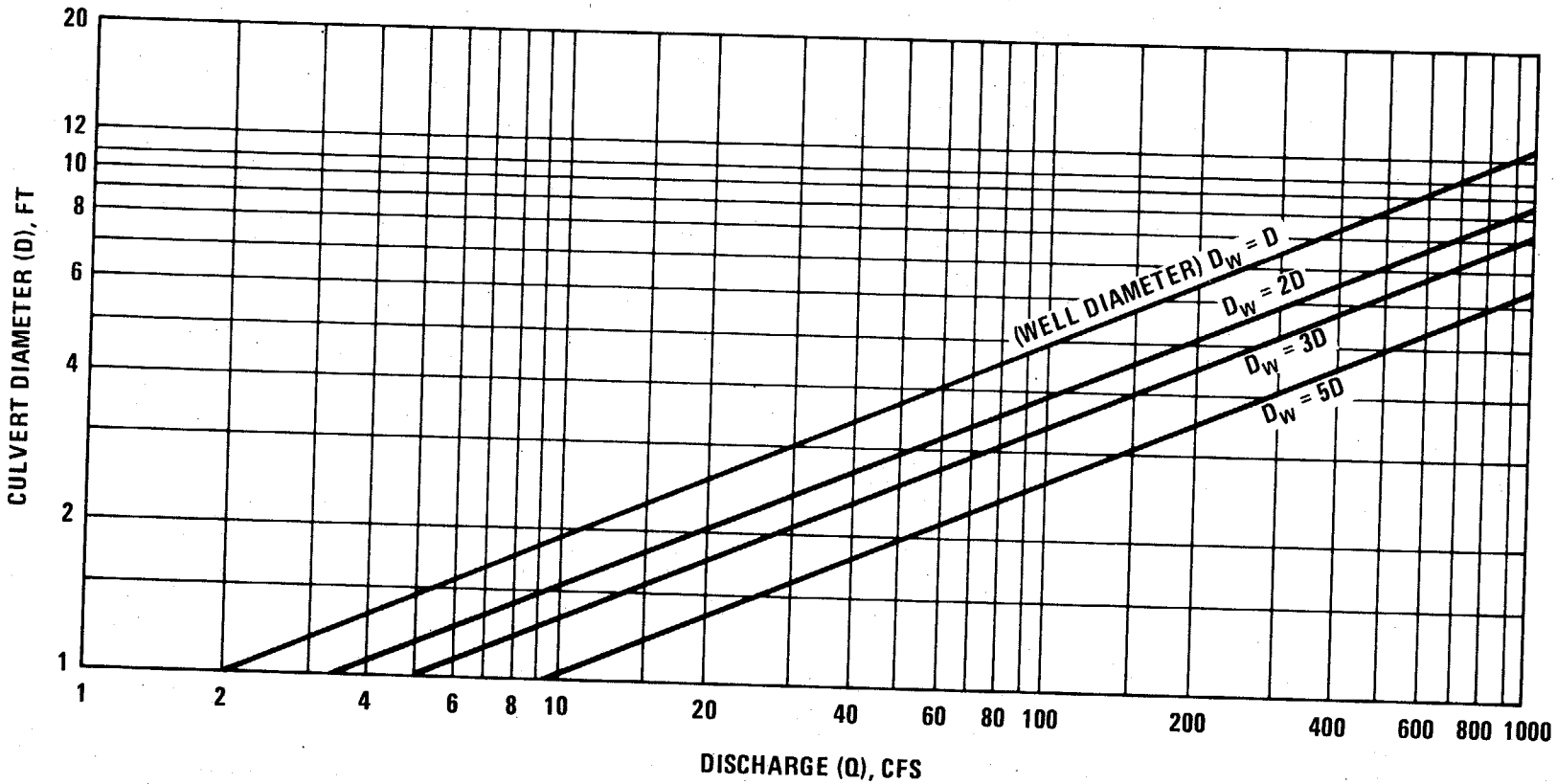


FIGURE X-B-1. STILLING WELL DIAMETER (D_W) FROM REFERENCE X-B-1

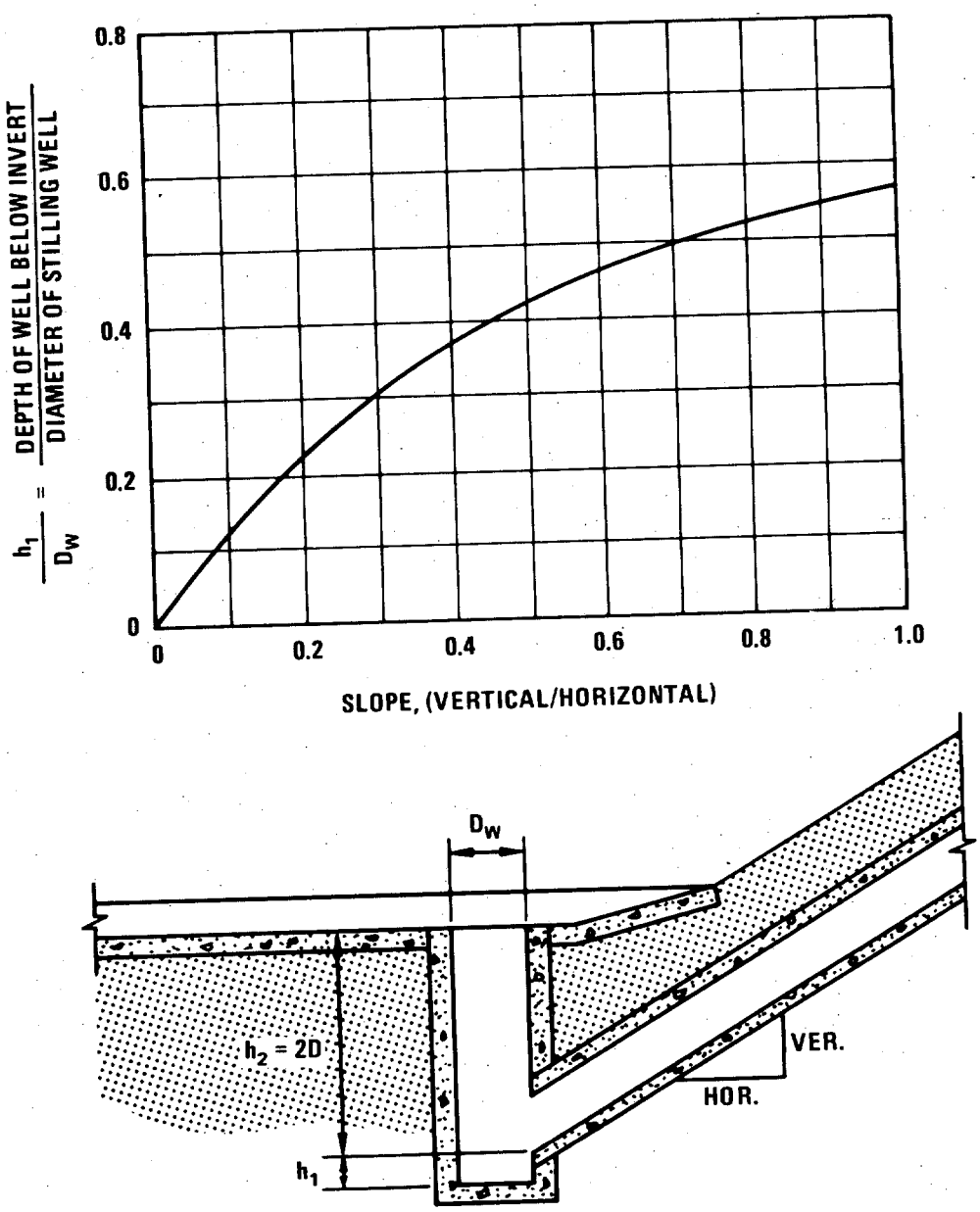


FIGURE X-B-2. STILLING WELL HEIGHT FROM REFERENCE X-B-1

XI. RIPRAP BASINS

The design procedure for riprap energy dissipators is based on data obtained during a study "Flood Protection at Culvert Outfalls" (XI-1, XI-2) sponsored by the Wyoming Highway Department and conducted at Colorado State University. The purpose of the experimental program was to establish relationships between flow properties and the dimensions of riprapped basins at culvert outfalls.

Tests were conducted with 6-inch, 12-inch, 18-inch, and 36-inch pipes, and 6 by 12-inch, 6 by 18-inch, and 6 by 24-inch model box culverts with discharges ranging from 0.1 to 100 cfs. Both angular and rounded rock with an average size (d_{50}) ranging from 1.05 to 4.29 were tested. Two pipe slopes were considered, 0 and 3.75%. In all, 459 model basins were studied.

The following conclusions were drawn from an analysis of the experimental data and observed operating characteristics.

The depth (h_s), length (L_s), and width (W_s) of the scour hole were related to the characteristic size of riprap (d_{50}), discharge (Q), brink depth (y_o), and tailwater depth (TW).

The dimensions of a scour hole in a basin constructed with angular rock were approximately the same as the dimensions of a scour hole in a basin constructed of rounded material when rock size and other variables were similar.

When the ratio of tailwater depth to brink depth (TW/y_o) was less than 0.75 and the ratio of scour depth to size of riprap (h_s/d_{50}) was greater than 2.0, and the scour hole functioned very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunged into the hole, a jump formed against the downstream extremity of the scour hole, and flow was generally well dispersed as it left the basin.

The mound of material which formed on the bed downstream of the scour hole contributed to the dissipation of energy and reduced the size of the scour hole; i.e., if the mound from a stable scoured basin was removed and the basin was again subjected to design flow, the scour hole enlarged somewhat.

For high tailwater basins (TW/y_o greater than 0.75) the high velocity core of water emerging from the culvert retained its jetlike character as it passed through the basin, and diffused in a manner very similar to that of a concentrated jet diffusing in a large body of water. As a result, the scour hole was much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

General details of the basin recommended in this report are shown on figure XI-1. Principal features of the basin are:

1. The basin is preshaped and lined with riprap.

2. The surface of the riprapped floor of the energy dissipating pool is constructed at an elevation h_s below the culvert invert. h_s is the approximate depth of scour that would occur in a thick pad of riprap, constructed at the outfall of the culvert, if subjected to the design discharge. The ratio of h_s to d_{50} of the material should be greater than 2 and less than 4.

3. The length of the energy dissipating pool is $10(h_s)$ or $3W_o$ which ever is larger. The overall length of the basin is $15(h_s)$ or $4W_o$ which ever is larger.

DESIGN PROCEDURE

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that $TW/y_o \leq 0.75$ for design discharge.

2. For subcritical flow conditions (culvert set on mild or horizontal slope) utilize figures III-9 or III-10 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

3. From site inspection and from field experience in the area, determine whether or not channel protection is required at the culvert outlet.

4. If channel protection is required, compute the Froude number for brink conditions ($y_e=(A/2)^{1/2}$). Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25<d_{50}/y_e<0.45$). Obtain h_s/y_e from figure XI-2, and check to see that $2<h_s/d_{50}<4$. Recycle computations if h_s/d_{50} falls out of this range.

5. Size basin as shown in figure XI-1.

6. Design procedures where allowable dissipator exit velocity is specified:

   a. Determine the average normal flow depth in the natural channel for the design discharge.

   b. Extended the length of the energy basin (if necessary) so that the width of the energy basin at section A-A, figures XI-1, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

7. In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

8. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

   Design a conventional basin for low tailwater conditions in accordance with the instructions above. Estimate centerline velocity at a series of downstream cross sections using the information shown in figure XI-3. Shape downstream channel and size riprap using figure II-C-1 and the stream velocities obtained above. Material, construction techniques, and design details for riprap should be in accordance with specifications in HEC No. 11 (XI-3) or similar highway department specifications.

<u>Design Example No. 1</u>

Given: 8 ft. by 6 ft. box culvert, Q=800 cfs, supercritical flow in culvert, normal flow depth = brink depth y_o=4 ft., Tailwater depth TW=2.8 ft.

Find: Riprap basin dimensions for these conditions:

Solution:

1. $y_o=y_e$ for rectangular section, $y_e=4$ ft.

2. $V_o=Q/A=800/(4)(8)=25$ fps

3. $Fr=V_o/[(32.2)(y_e)]^{1/2}=25/[(32.2)(4)]^{1/2}=2.20$

4. $TW/y_e=2.8/4.0=0.7$ $TW/y_e<0.75$ O.K.

5. Try $d_{50}/y_e=0.45$, $d_{50}=(0.45)(4)=1.80$ ft.

   From figure XI-2, $h_s/y_e=1.6$

   $h_s=(4)(1.6)=6.4$ ft.
   $h_s/d_{50}=6.4/1.8=3.6$ ft. $2<h_s/d_{50}<4$ O.K.

6. $L_s=(10)(6.4)=64$ ft.
   L_s min$=(3)(W_o)=(3)(8)=24$ ft., use $L_s=64$ ft.
   $L_B=(15)(6.4)=96$ ft.
   L_B min$=(4)(W_o)=(4)(8)=32$ ft., use $L_B=96$ ft.

Other basin dimensions designed in accordance with details shown in figure XI-1.

Design Example No. 2

Given: 8 ft. by 6 ft. box culvert, Q=800 cfs, supercritical flow in culvert, normal flow depth = brink depth, $y_o=4$ ft., tailwater depth, TW =4.2 ft., downstream channel can tolerate 7 fps for design discharge.

Find: Riprap basin dimensions for these conditions.

Solution:

Note--High tailwater depth, $TW/y_o=1.05>0.75$

1. Design riprap basin (Design Example 1) use steps 1-7
   $d_{50}=1.8$ ft., $h_s=6.4$ ft., $L_s=64$ ft., $L_B=96$ ft.

8. Design riprap for downstream channel. Utilize figure XI-3 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

   $A=\pi D_e^2/4=(y_o)(W_o)=(4)(8)=32$ ft.^{2}
   $D_e=[32(4)/\pi]^{1/2}$
   $D_e=6.4$ ft.
   $V_o=25$ fps (Design Example 1)

L/D_e	L (Compute) ft.	V_L/V_o (Figure XI-3)	V_L ft./ sec	Rock size d_{50} Figure II-C-1 ft.
10	64	0.59	14.7	1.4
15	96	0.36	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

Riprap should be at least the size shown. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 feet downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in HEC No. 11.

Design Example No. 3

Given: 6 feet diameter cmp, Q=135 cfs, S_o=0.004, Mannings n=0.024 normal depth in pipe for Q=135 cfs is 4.5 feet, normal velocity is 5.9 fps, flow is subcritical, tailwater depth (TW) is 2.0 feet.

Find: Riprap basin dimensions for these conditions:

Solution:

1. Determine y_o and V_o :

   $Q/D^{2.5}=135/(6)^{2.5}=1.53$
   $TW/D=2.0/6=0.33$

   From figure III-9, $y_o/D=0.45$

   $y_o=(0.45)(6)=2.7$ ft.
   $TW/y_o=2.0/2.70=0.74$ $TW/y_o<0.75$ O.K.
   Brink Area (A) for $y_o/D=0.45$ is
   $A=(0.343)(36)=12.3$ sq. ft.
   (0.343 is from table III-2)
   $V_o=Q/A=135/12.3=11.0$ fps

2. $y_e=(A/2)^{1/2}=(12.3/2)^{1/2}=2.48$ ft.

3. $Fr=V_o/[(32.2)(y_e)]^{1/2}=11/[(32.2)(2.48)]^{1/2}=1.23$

4. Try $d_{50}/y_e=0.25$, $d_{50}=(0.25)(2.48)=0.62$ ft.
   From figure XI-2, $h_s/y_e=0.75$, $h_s=(0.75)(2.48)=1.86$ ft.
   check: $h_s/d_{50}=1.86/0.62=3$, $2<h_s/d_{50}<4$ O.K.

5. $L_S=(10)(h_S)=(10)(1.86)=18.6$ ft.
or $L_S=(3)(W_O)=(3)(6)=18$ ft., Use $L_S=18.6$ ft.

$L_B=(15)(h_S)=(15)(1.86)=27.9$ ft.
or $L_B=(4)(W_O)=(4)(6)=24$ ft., Use $L_B=27.9$ ft.

$d_{50}=0.62$ ft. use $d_{50}=8$ in.

Other basin dimensions are designed in accordance with details shown on figure XI-1.

The design procedure recommended in this chapter is a compromise between the design procedure utilizing the CSU experimentally derived functional relationships and traditional design methods for riprapped basins. It is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap by the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, the damage should be superficial.

Concerning the use of filter material, several factors should be considered. Bank material adjacent to a culvert is not subjected to flow for long continuous periods. Also, the streambed material may be sufficiently well graded and not require a filter. If some siltation of the basin accompanied by plant growth is anticipated, it may be that a filter will not be required. If required, a filter cloth or filter material designed in accordance with instructions in reference XI-3 should be specified.

DISCUSSION

CSU Design Procedure

Design criteria were developed for three types of rock riprapped basins, the "standard non-scouring basin," the "hybrid basin," and the "standard scoured basin" (XI-2). Experimental data were used to establish empirical relationships between the following dimensionless parameters:

1. Froude number at the culvert outfall
2. relative grain size of riprap
3. relative tailwater depth
4. relative depth of scour hole

5. relative width of scour hole

6. relative length of scour hole

An excellent correlation of data was achieved by utilizing a weighted particle size (d_m) for scaling the grain size of riprap. However, the weighted particle size, d_m, is an unfamiliar parameter to most hydraulic engineers and is also difficult to obtain for quarry-run rock. For these reasons, the more familiar d_{50} (the median size of rock by weight) is used to characterize rock size in the design procedures presented in this manual.

The design procedure suggested by the CSU study is also very sensitive to tailwater depth and is somewhat cumbersome to use. The method requires a conversion of Froude numbers when a culvert operates with a free surface (the usual case) and requires a conversion of Froude numbers from circular pipe flow to rectangular pipe flow (or vice versa) if the designer wishes to use the complete range of design data for both pipe and box culvert basins. Because of these complexities, a simplified design procedure was developed. The information and design procedures from the CSU studies suggested the design parameters and the CSU data (XI-1) were utilized for developing design relationships.

Design Procedure Development

The design criteria specified in this report were developed in the following manner:

1. All CSU scour data for circular and rectangular pipes, angular and rounded material, sloping and horizontal pipes for plain outfall conditions were used for development of design aids. (Note, data for scour holes formed below model culverts constructed with standard or modified end sections were not used and data for $d_{50}/y_e<0.1$ were not used.)

   In all, data from 347 runs were used for this development. This included data for 6-inch, 12-inch, 18-inch, and 36-inch pipes and 6 by 12-inch, 6 by 18-inch, and 6 by 24-inch model box culverts. Two pipe slopes were considered, 0 and 3.75%. Discharges ranged from 0.1 to 100 cfs for basins constructed with angular and rounded rock with median rock size (d_{50}) ranging from 0.02 to 0.58 ft. Only data from runs with gradation coefficients ranging from 1.05 to 2.66 were used.

The gradation coefficient σ is defined as

$$\sigma = 1/2(d_{84}/d_{50} + d_{50}/d_{16})$$

and is a means of describing whether the rock mixture is predominantly one size or a range of sizes. When σ is near one, all material is about the same size. Where rock sizes extend over a large range, σ takes on larger values. The gradation coefficient σ of riprap will be satisfactory for design purposes if the gradation curve for the riprap is similar to curves for rock A(24) or B(16) shown in figure 8 of HEC No. 11, (XI-3).

2. Based upon an examination of CSU plots and data, the following significant dimensionless parameters were selected:

   a. d_{50}/y_e--the relative size of riprap defined as the ratio of the median size by weight of the rock mixture to the equivalent depth of water at the brink of the culvert. The equivalent depth y_e is defined as the brink depth for box culverts, and $(A/2)^{1/2}$ for non-rectangular sections, where A is the wetted area at the brink of the culvert. y_e computed in this manner is the height of a rectangle twice as wide as it is high with an area equal to the wetted area of the non-rectangular section.

   b. h_s/y_e--the relative depth of scour. h_s is the depth from the invert of the culvert at the brink to the lowest point in the scour hole (figure XI-1).

   c. $Fr = V_o/[(g)(y_e)]^{1/2}$--the Froude number at the brink of the culvert. V_o is the average velocity of flow at the brink of the culvert (discharge Q divided by wetted area, A) and g is the acceleration of gravity. This definition of the Froude number eliminates the intervening steps of converting non-full pipe flow to full-pipe flow and Froude numbers for circular pipe flow to Froude numbers for rectangular pipe flow (or vice versa) as is required by the CSU procedure.

   d. TW/y_e--the relative depth of tailwater. TW is the depth of tailwater referenced to the culvert invert.

3. The dimensionless parameters cited above were computed for each set of data and were segregated into the following categories:

$$0.10 < d_{50}/y_e < 0.2$$

$$0.21 < d_{50}/y_e < 0.3$$

$$0.31 < d_{50}/y_e < 0.4$$

$$0.41 < d_{50}/y_e < 0.5$$

$$0.51 < d_{50}/y_e < 0.6$$

$$0.61 < d_{50}/y_e < 0.7$$

4. For each subset of data (as an example, all data for the condition that $0.21 < d_{50}/y_e < 0.30$) the data were further subdivided into the following categories:

   a. model pipe culverts with a diameter equal to or less than 1 ft., basin constructed with rounded material,

   b. model pipe culvert with a diameter equal to or less than 1 ft., basin constructed with angular material,

   c. model pipe culvert with a diameter greater than 1 ft., basin constructed with rounded material,

   d. model pipe culvert with a diameter greater than 1 ft., basin constructed with angular material, and

   e. model box culverts with a height of 0.5 ft., basin constructed with rounded material (the only material tested for box culverts).

5. A symbol associated with the magnitude of the relative tailwater depth (in increments of 0.2) was assigned to each data point for each of the categories designated in 4) above (i.e., if $TW/y_e = 0.51$ for a particular run, the symbol associated with $0.41 < TW/y_e < 0.6$ was assigned to the data point).

6. A series of plots of relative depth of scour h_s/y_e versus the Froude number $V_o/[(g)(y_e)]^{1/2}$ were constructed with the relative size of riprap d_{50}/y_e as a third variable. By symbol and color code the various categories described in 4) and 5) above could be identified on the plots. Additional supplemental plots were also constructed to help identify significant parameters.

The parameters selected grouped the data in a systematic way though there was significant scatter. Scour depth did not appear to be a function of angular or rounded riprap. Data from pipe runs could not be segregated from data for box culvert. The only scale effect that could be detected were associated with high Froude number (>3.0) runs where model flow depths were on the order of one-half foot with tailwater depths of 0 to 2 inches. The scaled riprap was approximately one-half inch in size.

Because of the difficulty in obtaining precise measurements in models of this size and also because of the possibility of scale effects with small scale models these data points were not given quite as much consideration when design curves were developed. However, these data were used to establish the approximate slope of design curves.

Relative tailwater depth was a significant variable. For conditions where all other variables were similar, maximum scour depths were associated with low tailwater depths. Maximum length of scour holes were associated with high tailwater depths.

When considering a culvert installation, several points are obvious. For ephemeral, flashy streams usually associated with culvert crossings, the tailwater depth will lag significantly when the stream is rising. Thus, low tailwater will always exist at least up to the point when and if equilibrium conditions occur. Also, because of seasonal changes in vegetation, changes in downstream channel cross section as a result of flood events or man's activity, and the general difficulty in obtaining channel properties in poorly defined ephemeral streams, the computed tailwater depth will be at best an estimate. For these reasons it was decided to base design criteria on the assumption that the worst possible tailwater conditions exist, i.e., low tailwater conditions.

On the basis of the above assumptions, envelope curves were constructed for each plot of h_s/y_e versus $V_o/[(g)y_e]^{1/2}$ with d_{50}/y_e as the third variable.

On each plot, the curve was drawn to the left of approximately 98% of the data points and thus the predicted scour based on the use of a curve will be as deep or deeper than was actually measured in the model basin for the worst possible tailwater condition. These envelope curves were then transposed to one plot (figure XI-2) and are the basic design curves for determining h_s as a function of exit velocity, equivalent brink depth, and d_{50}.

Additional information necessary to design a basin includes geometric dimensions, minimum thickness of riprap, approximate shape of gradation curve of riprap, sideslopes of basin, and a determination of whether or not a filter blanket is required.

Length of Basin

Frequency tables for both box culvert data and pipe culvert data of relative length of scour hole ($L_s/h_s<6$, $6<L_s/h_s<7$, $7<L_s/h_s<8$ . . . $25<L_s/h_s<30$), with relative tailwater depth TW/y_e in increments of 0.1 feet as a third variable were constructed utilizing all data from 347 experimental runs. For box culvert runs L_s/h_s was less than 10 for 78% of the data and L_s/h_s was less than 15 for 98% of the data. For pipe culverts, L_s/h_s was less than 10 for 91% of the data and, L_s/h_s was less than 15 for all data.

For all cases the data considered were restricted to relative tailwater depths of less than 0.75. Large values of relative length of scour L_s/h_s were always associated with high tailwater conditions. The curves to be used to predict h_s (the value of h_s to be used for computing L_s the length of energy dissipating pool, in the design procedure) are based on maximum observed scour depths which in turn were always associated with minimum tailwater depths. Based on this argument, a conservative prediction ratio for obtaining a design estimate of relative length of preshaped scour hole is $L_s/h_s>10$ and relative overall length of basin is $L_B/h_s>15$. From a practical viewpoint, the length of the pool L_s should be at least $3W_o$ and L_B should be at least $4W_o$. W_o is the width of the culvert outlet. The dimension L_B is the out-to-out length of the riprap basin measured from the culvert brink to the end of the basin.

Other Basin Details

The $2(d_{50})$ or $1.50(d_{max})$, d_{max} is the maximum size of rock in the riprap mixture, thickness of riprap for the floor and sides of basin are based on experience with conventional riprap design. Thickening of the riprap layer to $3(d_{50})$ or $2(d_{max})$ on the foreslope of the roadway culvert outlet is warranted because of the severity of attack in the area and the necessity for preventing significant undermining and consequent collapse of the culvert.

A 3:1 flare angle is recommended for the basins walls. This angle will provide a sufficiently wide energy dissipating pool for good basin operation.

The mixture of stone used for riprap should meet the specifications (material, gradation, etc.) described in HEC No. 11, (XI-3).

XI-1. Stevens. M. A., and Simons, D. B., EXPERIMENTAL PROGRAMS AND BASIC DATA FOR STUDIES OF SCOUR IN RIPRAP AT CULVERT OUTFALLS, Colorado State University, Fort Collins, Colorado, CER 70-71-MAS-DBS-57, 1971.

XI-2. Simons, D. B., Stevens, M. A., Watts, F. J., FLOOD PROTECTION AT CULVERT OUTLETS, Colorado State University, Fort Collins, Colorado, CER 69-70 DBS-MAS-FJW4, 1970.

XI-3. U. S. Department of Transportation, USE OF RIPRAP FOR BANK PROTECTION, Hydraulic Engineering Circular No. 11, Government Printing Office, Washington, D.C., 1967.

XI-4. U.S. Army, Corps of Engineers, RIPRAP STABILITY ON EARTH EMBANKMENTS TESTED IN LARGE- AND SMALL-SCALE WAVE TANKS, Technical Memorandum No. 37, Coastal Engineering Research Center, Washington, D.C., 1972.

NOTE A – IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS–SECTIONAL AREA AT SECTION A–A SUCH THAT Q_{des}/(CROSS SECTION AREA AT SEC. A–A) = SPECIFIED EXIT VELOCITY.

NOTE B – WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A–A.

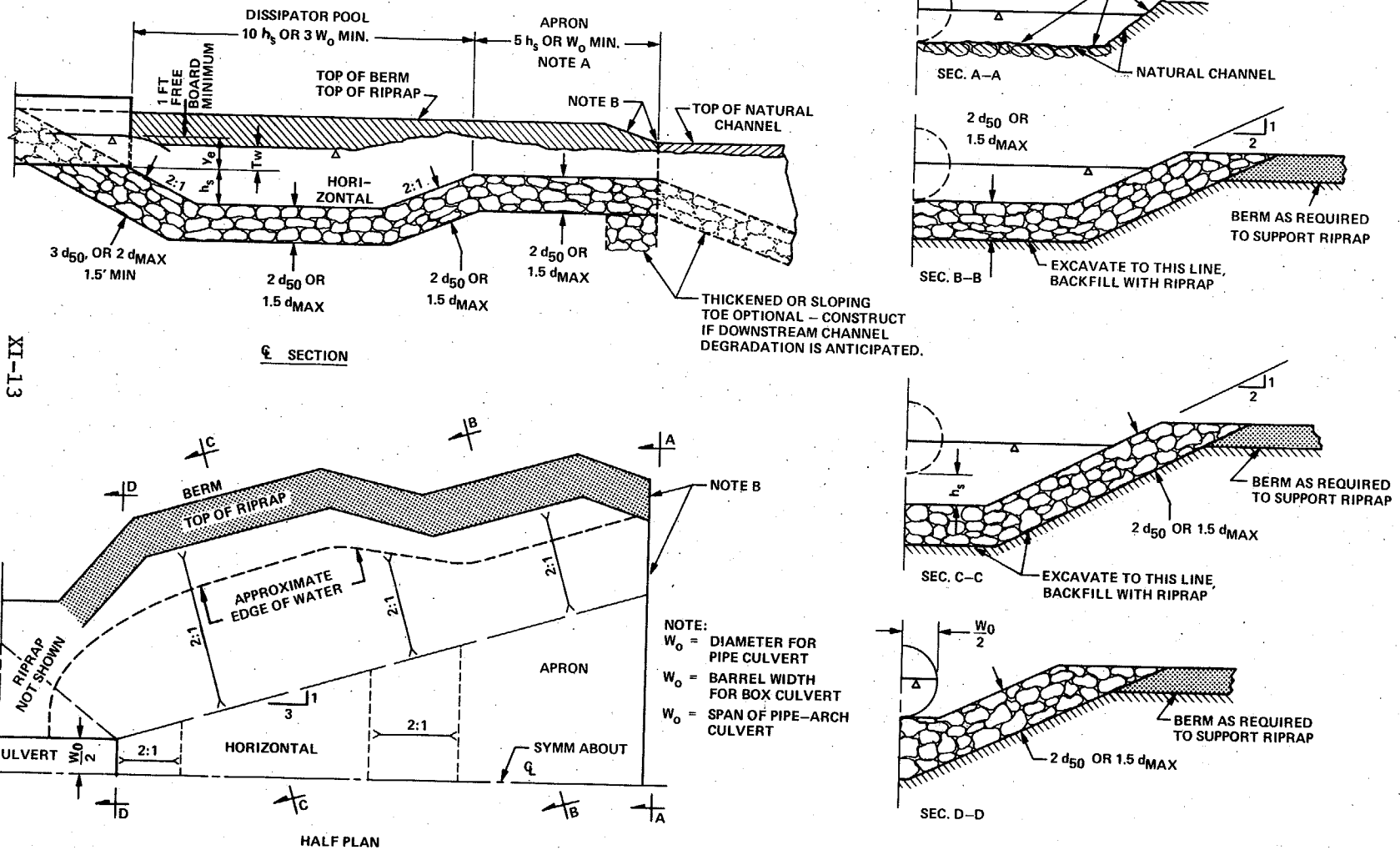


FIGURE X1–1. DETAILS OF RIPRAPPED CULVERT ENERGY BASIN

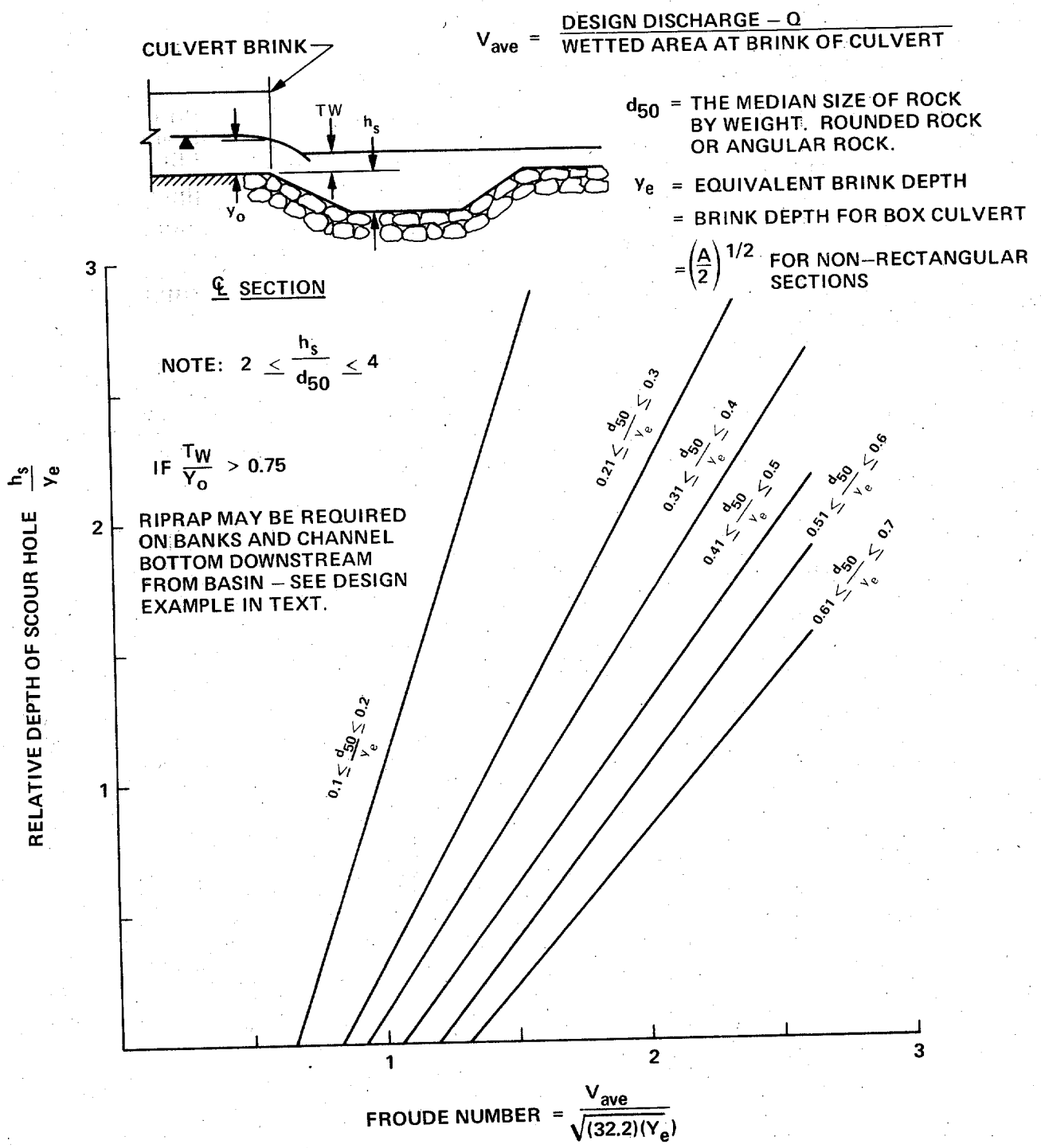


FIGURE X1–2. RELATIVE DEPTH OF SCOUR HOLE VERSUS FROUDE NUMBER AT BRINK OF CULVERT WITH RELATIVE SIZE OF RIPRAP AS A THIRD VARIABLE

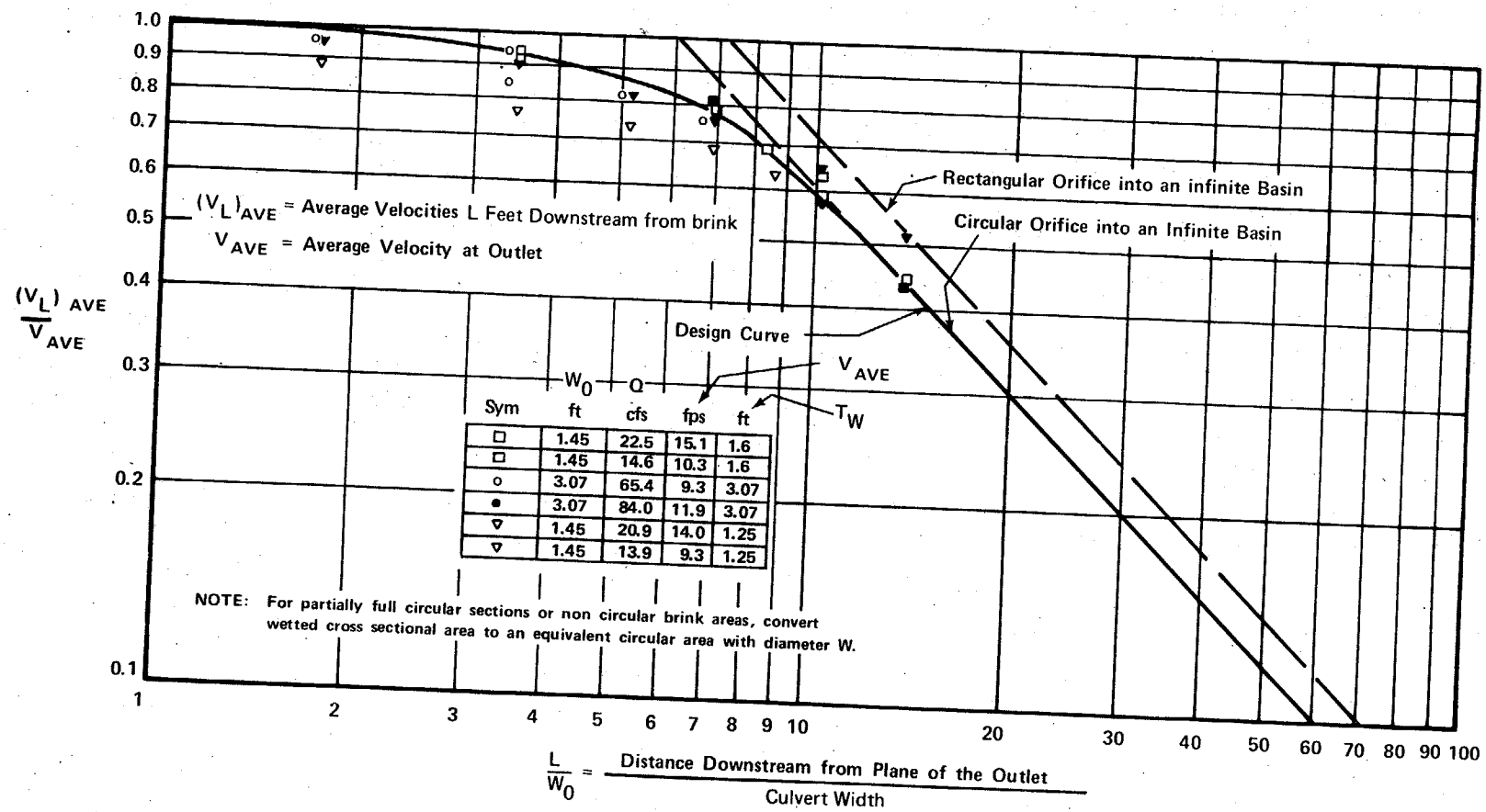


Figure XI - 3 Distribution of Centerline Velocity for Flow from Submerged Outlets from Reference XI - 2. to be used for Predicting Channel Velocities Downstream from Culvert Outlet where High Tailwater prevails. Velocities obtained from the use of this Chart can be used with Figure 2 of HEC No. 11 for sizing riprap (DO not use Figure 1 HEC No. 11, use Mean Velocity Values)

XII. DESIGN SELECTION

The energy dissipator selection process is best illustrated by applying the material presented in the preceding chapters to a series of design problems. A general design procedure outline is shown in figure I-1, the conceptual model. This model can be summarized by the following design steps.

Design Procedure

1. Input Data

   a. Culvert--Types of control, Q, S_o, Y_o, V_o, L, TW, Fr_o

   b. Standard Outlet--y and V at culvert brink, Chapter III
      y and V at end of apron, Chapter IV

   c. Channel--Q, S, geometry, z, Y_n, V_n, soil type, debris, bedload

   d. Allowable scour estimate--h_s, W_s, L_s based on location

2. Natural Scour Computation--h_s, W_s, L_s, V_s from Chapter V if these values exceed allowable values in step 1D, protection is required.

3. Velocity Modification in Culvert

   a. For small velocity adjustment increase culvert resistance, Chapter VII-C

   b. For minimum velocity, V_c, design for tumbling flow, Chapter VII-B

4. Energy Dissipator Design

   a. The dissipator types fall into two general groups based on Fr, see table XII-1:

      1. Fr<3, most designs are in this group

      B. Fr>3, tumbling flow, increased resistance, USBR III, SAF and USBR VI

   Within the two dissipator groups, the designs to be considered can be determined by testing the limitations of debris, TW, and other special considerations against site conditions.

b. Design each of the selected types using the appropriate design chapter.

5. Selection Criteria--Dissipator selection should be governed by comparing the efficiency, cost, natural channel compatibility, and anticipated scour for all the alternatives.

EXAMPLE PROBLEM NO. 1

1. Input Data:

   a. Culvert: 10X6 RCB, $Q = 417$ cfs, $S_o = 6.5\%$
   $y_n = 1.5$ ft., $V_n = 27.8$ fps, inlet control,
   $L = 300$ ft., $TW = 1.9$ ft., $Fr_o = 4$

   b. Standard Outlet: 45° wingwall--abrupt transition since culvert is in supercritical flow
   $y_o = y_n = 1.5$ ft. and $V_o = V_n = 27.8$ fps

   c. Channel: $Q = 417$ cfs, $S = 6.5\%$, trapezoidal, 2:1
   $y_n = 1.9$ ft., $V_n = 15.9$ fps, graded gravel bed
   debris (no boulders, little floating)

   d. Allowable Scour: Scour hole should be contained within channel $W_s = L_s = 10$ ft. and should be no deeper than 5 ft. This allowable estimate can be obtained by observing scour holes in the vicinity.

2. Scour Estimate: Table V-I

   $Y_e = 2.74$ ft.
   $h_s = 8.3$ ft.
   $W_s = 52$ ft.
   $L_s = 71$ ft.
   $V_s = 963$ cubic yards

   Scour appears to be a problem and consideration should be given to reducing the $V_o = 27.8$ fps to the 16 fps in the channel.

3. Velocity Modification in Culvert:

   a. Increase resistance: Chapter VII-C

   For $h = .4$ ft., $n_r = .053$ for velocity check
   $n_r = .076$ for Q check

   The velocity at the outlet is 11.6 fps. The elements are 3.3 feet apart for 9 rows or 97.7 feet of the culvert barrel is needed for elements.

b. Tumbling Flow: Chapter VII-B

Five elements 1.9 ft. in height spaced 15 feet apart are required to reduce the velocity to V_c=11 fps and y_c=3.8 ft. In order to accomplish this reduction, the last 100 feet of culvert must be increased in height to 6.7 ft.

4. Energy Dissipator Design:

Since Fr>3 try USBR III, SAF, and USBR VI. The USBR III and SAF which were designed in Chapter VII-E and G are summarized below:

USBR Type III: L_B=26 ft., total length including transitions=67 ft., Elevation=87.5, V_2=16 fps

SAF: L_B=11 ft., total length including transitions=40 ft., elevation=91.5 ft., V_2=16 fps

USBR Type VI (Impact): W=11 ft., h_1=8.4 ft., L=14.6 ft. Exit Velocity= 11 fps

5. Selection Criteria:

Comparing the above designs cost is the deciding factor in choosing the USBR Type VI or impact dissipator. This Structure will fit the channel, meets the velocity criteria, and produces 60% energy loss.

EXAMPLE PROBLEM NO. 2

1. Input Data:

   a. Culvert

      Inlet Control
      B = 5
      D = 5
      Elevation of outlet invert = 100.0
      Q_{Design} = 200 cfs
      S_o = 0.03 ft./ft.
      y_o = 2.15 ft.
      V_o = 19 fps
      L = 213 ft.
      TW = Essentially zero
      Fr_o = 2.28

   b. Standard Outlet: A 90° headwall is standard. The culvert outlet flow conditions are the normal flow conditions in the culvert:

      y_o = 2.15 ft
      V_o = 19 fps

   c. Channel: The downstream channel is undefined. The water will spread and decrease in depth as it leaves the culvert making tailwater essentially zero. The channel is a graded sand.

   d. Allowable scour estimate: A scour basin not more than 3.0 feet deep is allowable at this site. Allowable outlet velocity should be about 10 fps.

2. Scour Computation: Chapter V

   Convert to equivalent depth

   y_e = 2.32 ft.
   h_s = 5.6 ft.
   W_s = 31 ft
   L_s = 49 ft.
   V_s = 81 cubic yards

   Since 5.6 feet is greater than the 3.0 feet allowable, an energy dissipator will be necessary.

3. Velocity Modification:

a. Try increased roughness, Chapter VII-C, with elements on bottom and sides of culvert.

For h = 0.26 feet
n_r= 0.03 for velocity check
n_r= 0.05 for Q check

The discharge check indicates that the culvert height has to be increased to 6.0 feet. The length of roughness field and increased culvert diameter requested is 75 feet.

This represents 25 rows spaced at 3.0 feet/row.

b. Tumbling flow; Chapter VII-B.

Five elements. 1.8 feet in height spaced 18 feet apart are required to reduce the velocity to 10.9 fps. The last 108 feet of the culvert must be increased in height to 6.5 feet.

4. Energy Dissipator Designs:

Since F_r<3 try the CSU rigid boundary basin, the USBR type VI, the Hook type and a Riprap basin.

CSU Basin, Chapter VII
W_B= width of basin = 30 feet
L_B= length of basin = 28 feet
N_r= number of roughness rows = 4
N = number of elements = 17
U_e= divergence 1.9:1
W_1= width of elements = 3.0 feet
h = height of elements = 0.75 feet
V_B= velocity at basin outlet 9.5 fps
y_B= depth at basis outlet = 0.70 feet.

USBR type VI, Chapter VII-C
W = width = 12 feet
L = length = 16 feet
h_1= height = 9.2 feet
Exit velocity = 8.13 feet/second.

Hook Type Basin

Assuming the downstream velocity, V_n, equals the allowable, 10 fps, V_o/V_n= 1.9

The dimensions for a straight trapezoidal basin are:

L_B= length = 15 feet
$W_6=2W_o$
Side Sloper = 2:1
L_1= length to first hook = 6.25 feet.
L_2= length to sand hooks = 10. 43 feet
h_3= height of hook = 2.35 feet
V_2= 10 fps

From figure VIII-B-7, $V_o/V_B=2.0$; $V_B=19/2.0=9.5$ fps OK

Riprap Basis:

d_{50}= diameter of rock = 1.31 feet
h_s = depth of pool (scour) = 2.69 feet

Length of pool = 26.9 feet

Length of apron = 13.45 feet

Length of basin = 40.35 feet

Thickness of riprap
on approach 3 d_{50}= 3.93 feet.
remainder of basin $2d_{50}$=2.62 feet.

5. <u>Selecting Criteria</u>

Right-of-way, debris, and dissipator cost are all constraints at this site. ROW is expensive making the longer dissipators more costly. Debris will effect the operation of the impact basin and may be a problem with the CSU roughness elements and tumbling flow designs. In the final analysis, the riprap basin was selected based on cost and anticipated maintenance.

TABLE XII-I DISSIPATOR LIMITATIONS

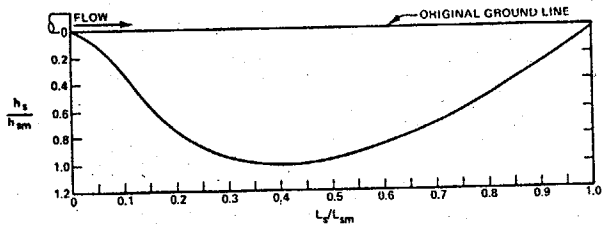
DISSIPATOR TYPE	Froude number Fr	Allowable Debris			Tailwater TW	Special Consideration
		Silt Sand	Boulders	Floating		
Free Hydraulic Jump	>1	H	H	H	Required	
CSU Rigid Boundary	<3	M	L	M	-	
Tumbling Flow	>1	M	L	L	-	$4<S_o<25$
Increased Resistance	-	M	L	L	-	Check Outlet Control HW
USBR Type II	4 to 14	M	L	M	Required	
USBR Type III	4.5 to 17	M	L	M	Required	
USBR Type IV	2.5 to 4.5	M	L	M	Required	
SAF	1.7 to 17	M	L	M	Required	
Contra Costa	<3	H	M	M	<0.5D	
Hook	1.8 to 3	H	M	M	-	
USBR Type VI	-	M	L	L	Desirable	Q<400 cfs, V<50 fps
Forest Service	-	M	L	L	Desirable	D<36 inch
Drop Structure	<1	H	L	M	Required	Drop< 15ft.
Manifold	-	M	N	N	Desirable	
Corps Stilling Well	-	M	L	N	Desirable	
Riprap	<3	H	H	H	-	

Note:
N=none
L=low
M=moderate
H=heavy

Appendix A

Computation Forms

CULVERT, CHANNEL, SCOUR AND OTHER SITE DATA



DIMENSIONLESS CENTER-LINE PROFILE

(1) $y_e = (A/2)^{1/2}$

Project _____
Station _____
Designer _____
Date _____

	TYPE	SIZE	"n"	LENGTH	SLOPE S_o	DISCHARGE	DEPTH y_o	VELOCITY V_o	FLOW AREA	EQUI(1) DEPTH y_e	Fr	END TREATMENT Entrance	END TREATMENT Exit
CULVERT	CONTROL												

	TYPE	SIDE SLOPE	"n"	BOTTOM WIDTH	SLOPE	DISCHARGE Q	VELOCITY	FLOW AREA	TW	FREE BOARD	FR	TYPE OF MATERIAL
CHANNEL												

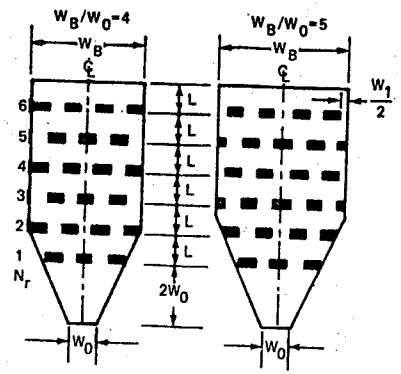
	EQUIVALENT DEPTH y_e	SCOUR DEPTH h_s	SCOUR WIDTH W_s	SCOUR LENGTH L_s	SCOUR VOLUME $\forall_s$
SCOUR COMPUTATIONS					

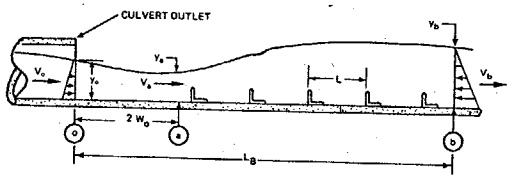
	VELOCITY	SCOUR DEPTH	SCOUR WIDTH	SCOUR LENGTH	OTHER RESTRICTIONS
ALLOWABLE CONDITIONS					

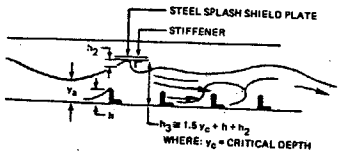
OTHER SITE CONSTRAINTS	

CSU RIGID BOUNDARY BASIN

7-A







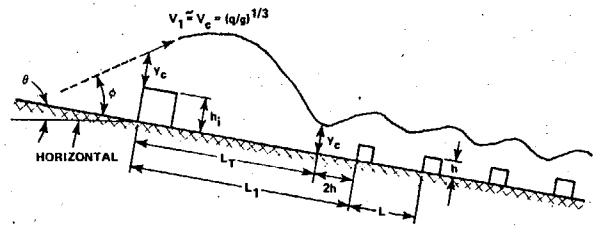
TRIAL DESIGN	W_B/W_o	V_A	y_A	W_1/W_o	N_R	N	h/y_A	L/h	
(1)									
(2)									
(3)									
(4)									
(5)									

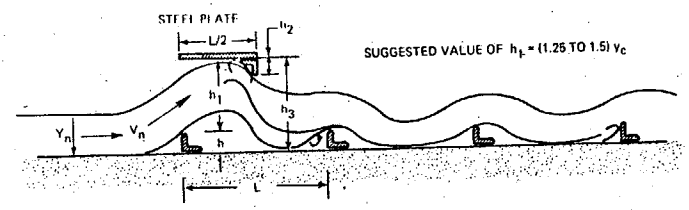
TRIAL DESIGN cont'd	h	L	W_B	W_1	U_e	C_B	A_f	C_p	L_B	$C_BA_fN_{(1)}$	$C_BA_fN_{(2)}$	$W_1/h_{(3)}$	
(1)													
(2)													
(3)													
(4)													
(5)													

	FREEBOARD	y_c	h_2	h_3	L/2
SPLASH SHIELD					

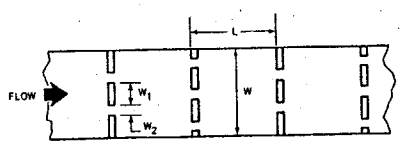
(1) From values in this table
(2) From Equation VII-A-1 or 2
(3) $2 < W_1/h < 8$

TUMBLING FLOW - RECTANGULAR SECTION





	q = Q/W	y_c	V_c+	V_n							
INITIAL CONDITIONS											
	h	L/h	L	Y_2*	h_i*	W_2	W_1				
							1st	2nd	3rd	4th	5th
DISSIPATOR DESIGN											
	h_1	h_2	h_3	Length							
SPLASH SHIELD											

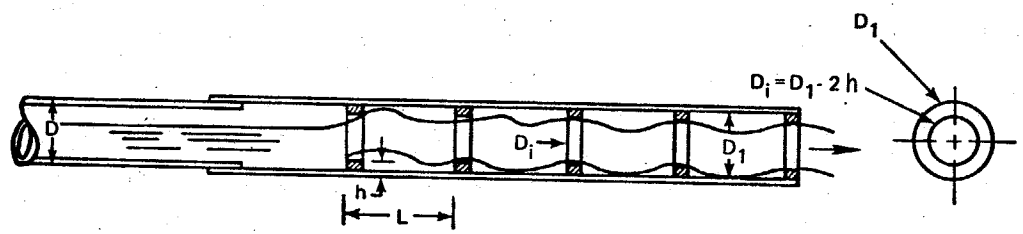


* For design using initial large element - Alternate design

\+ V_c equals outlet velocity for dissipator

TUMBLING FLOW - CIRCULAR SECTION

7-B

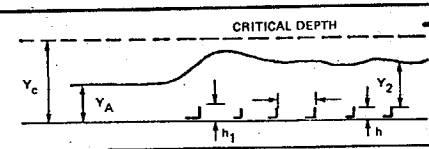


	$1.6(Q^2/g)^{1/5}$	$1.9(Q^2/g)^{1/5}$	D_1	
INITIAL CONDITIONS				

	h	L	D_i	y_c	y_c/D_i	A_c/D_i^2	A_c	V_c	
DISSIPATOR CONDITION									

7-C

INCREASED RESISTANCE - BOX CULVERTS



L/h	h/R_i	L_r	P	L_r/p	$S_o^{1/2}$	R_i - full	
10							

VELOCITY CHECK	n_r/n	n_r	h	y_i	$K = \frac{1.49}{n_r} S_o^{1/2}$	A_i	R_i	$R_i^{2/3}$	$Q=KA_iR_i^{2/3}$	$V = \frac{Q}{A_i}$	No. of Rows (1)

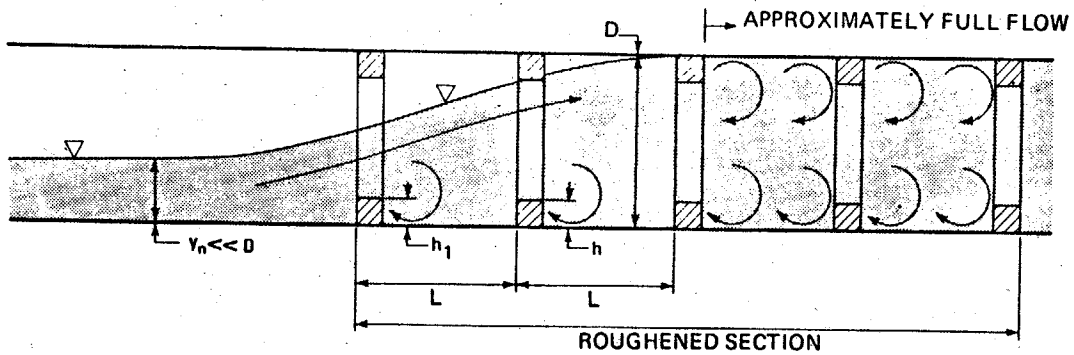
CAPACITY CHECK	n_r/n	n_r	h	y_i	$K = \frac{1.49}{n_r} S_o^{1/2}$	A_i	R_i	$R_i^{2/3}$	$Q=KA\ R^{2/3}$	D_i (4)	

	h (2)	L = 10h	h_i = 2h	V (3)	D_i
FINAL DESIGN					

(1) From Equation VII-C-10
(2) From Capacity Check
(3) From Velocity Check
(4) New Diameter - if necessary

ROUGHNESS ELEMENTS - CIRCULAR CULVERTS

7-C



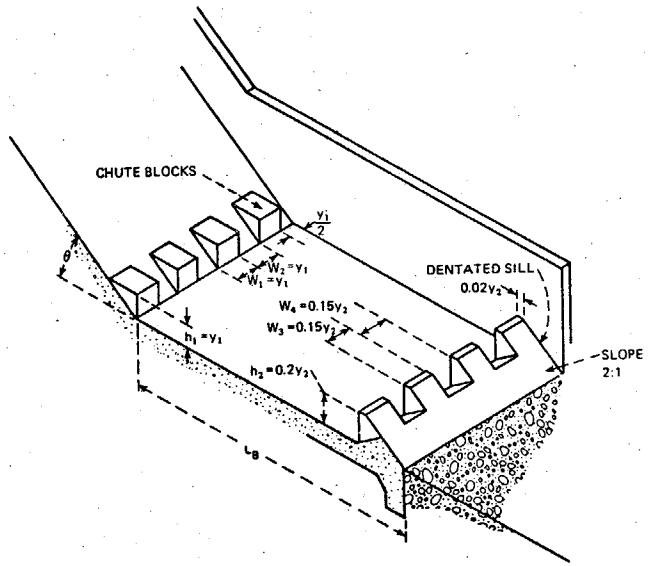
	Q (FULL) (n)	V (FULL) (n)	Q(DESIGN) / Q(FULL)	V(DESIGN) / V(FULL)	n	Type of Flow Control
INITIAL CONDITIONS						

$n/D^{1/6}$	L/D_i	h/D_i	n_r/n	n_r	D_i	Q(FULL) (n_r)	V (FULL) (n_r)	V(OUTLET) / V(FULL)	V (OUTLET)

h	L	h_1

USBR TYPE II

7-D



C_1 = _____

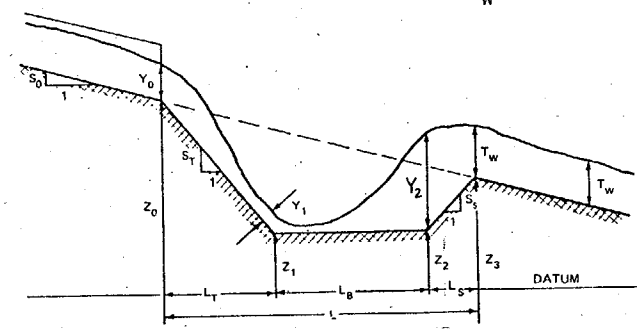
S_S = _____

S_T = _____

W_B = _____

y_2 = _____

T_W = _____



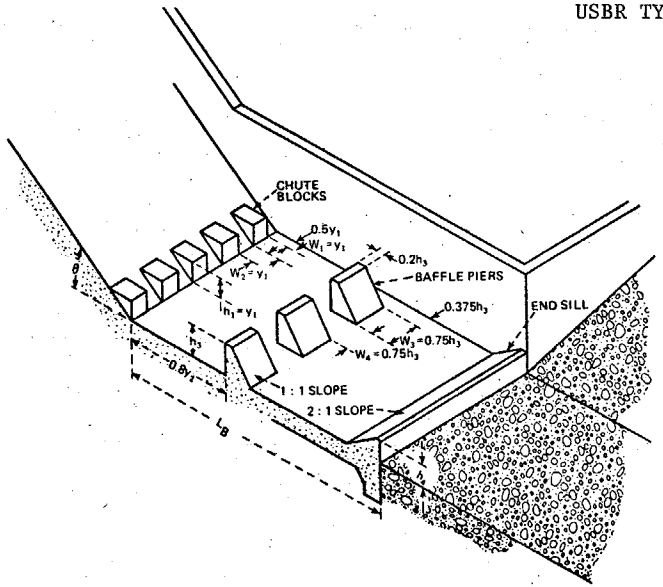
If $y_2 > T_W$, Depress Basin

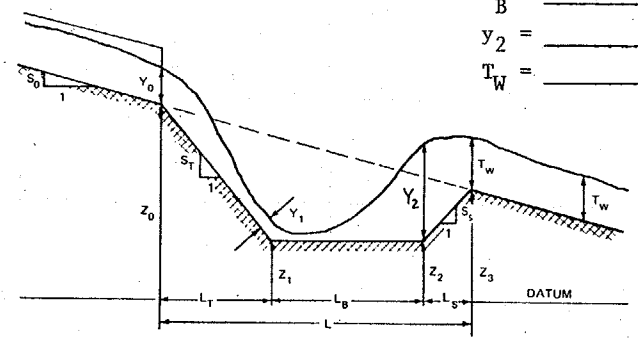
$Z_1 = Z_2$	y_1	V_1	Fr_1	y_2	L_B	L_T	Z_3	$y_2 + Z_2$ (1)	$Z_3 + T_W$ (1)
L_S	L	W_1	h_1	N_c	h_2	W_3	W_4	N_s	

(1) $Z_3 + T_W \geq y_2 + Z_2$

USBR TYPE III

7-E





C_1 = _____

S_s = _____

S_T = _____

W_B = _____

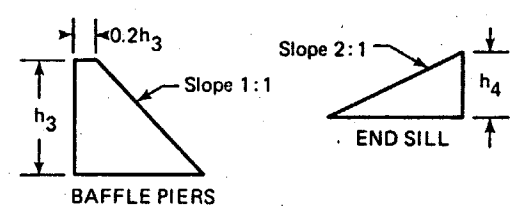
y_2 = _____

T_W = _____

$Z_1 = Z_2$	y_1	V_1	Fr_1	y_2	L_B	L_T	Z_3	$y_2 + Z_2$ (1)	$Z_3 + T_W$ (1)

L_s	L	h_1	W_1	W_2	N_c	h_3	W_3	W_4	N_B

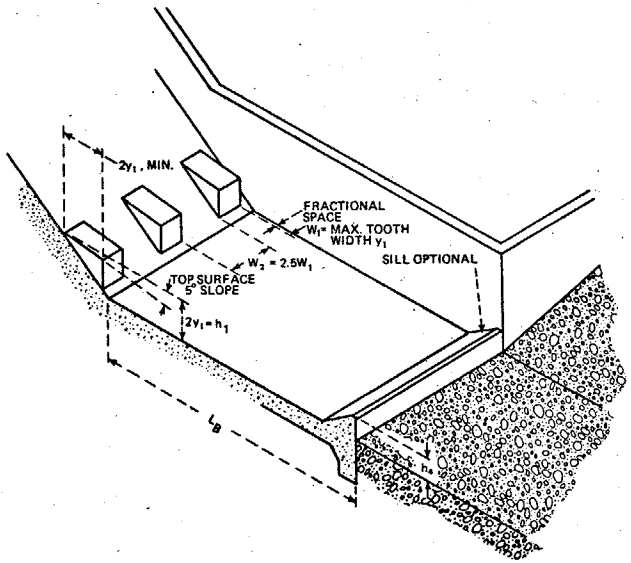
Space at Walls	Length from Chute Blocks	h_4

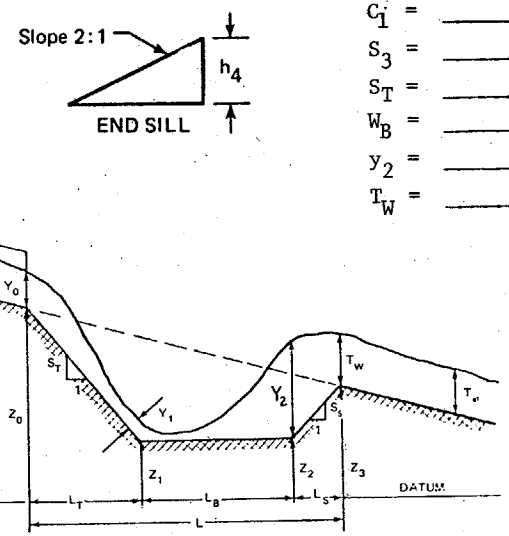


(1) $Z_3 + T_W \geq y_2 + Z_2$

USBR TYPE IV

7-F





C_I = _____

S_3 = _____

S_T = _____

W_B = _____

y_2 = _____

T_W = _____

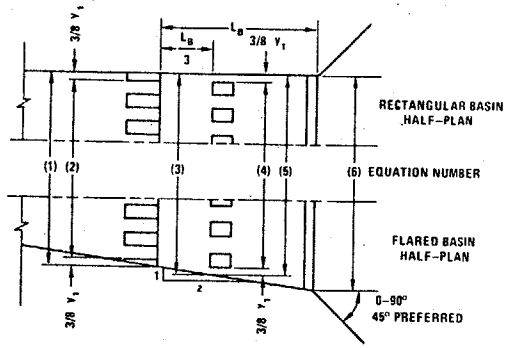
If $y_2 > T_W$, Depress Basin

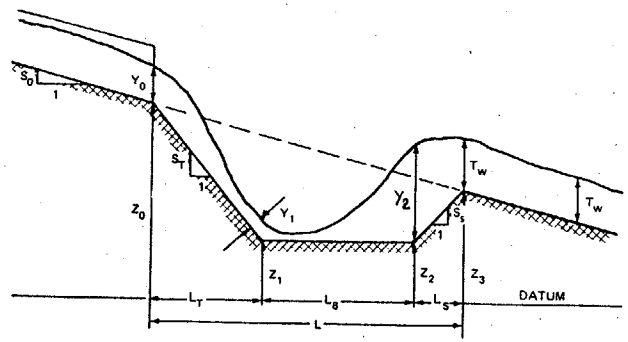
$Z_1 = Z_2$	y_1	V_1	Fr_1	y_2	L_B	L_T	Z_3	$y_2 + Z_2$ (1)	$Z_3 + T_W$ (1)
L_s	L	h_1	W_1	W_2	N_c	Wall Spaces	h_4		

(1) $Z_3 + T_W \geq y_2 + Z_2$

SAF STILLING BASIN

7-G





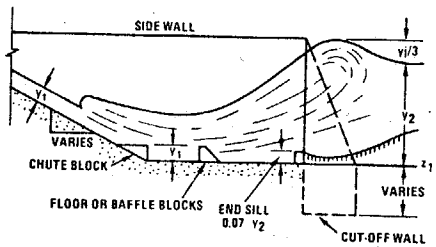
S_s =
S_t =
y_2 =
T_W =
Basin Type
Flared _____
Rect. _____

If $y_2 > T_W$ Depress Basin

$Z_1 = Z_2$	y_1	V_1	Fr_1	y_j	y_2	L_B	L_T	Z_3	$y_2 + Z_3$	$Z_3 + T_W$

h_1	W_1	N_c	W_2	h_3	W_3	W_4	W_{B2}	N_B	

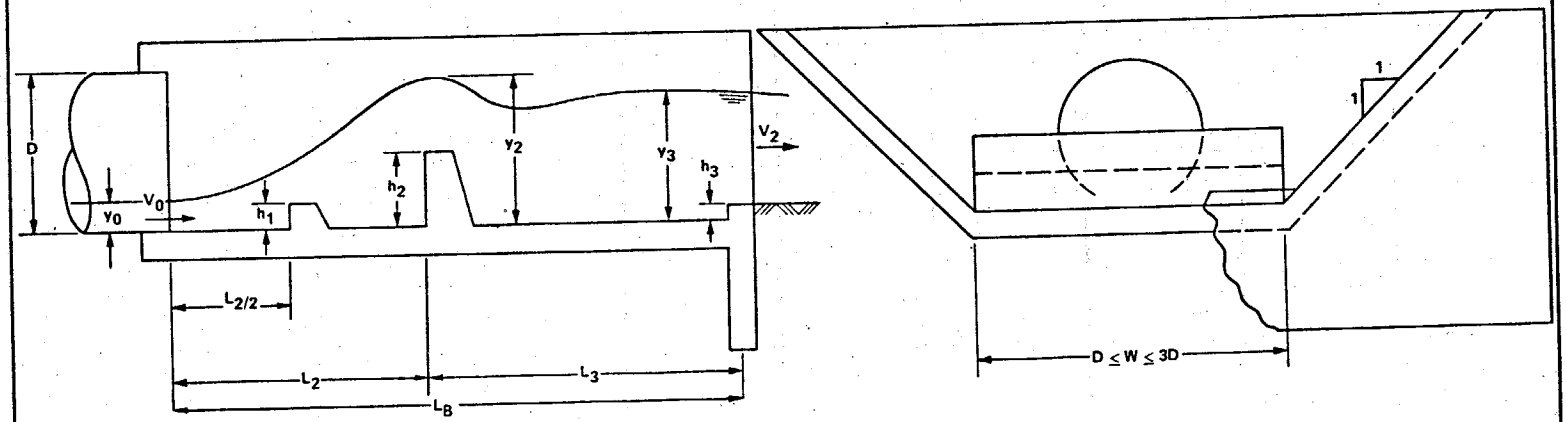
Dist. of Baffle Blocks	h_4	Sidewall height



(1) W_B = BASIN WIDTH UPSTREAM
(2) n BLOCKS AT 3/4 Y_1 ±
(3) 0.40 W_{B2} ≤ AGGREGATE BLOCK WIDTH ≤ 0.55 W_{B2}
(4) n BLOCKS AT 3/4 Y_1 $\frac{W_{B2}}{W_B}$ ±
(5) $W_{B2} = W_B + 2L_B/3z$
(6) $W_{B3} = W_B + 2L_B/z$

CONTRA COSTA

8-A



y_o/D(1)	y_e	F_r^2	W(2)	L_2/h_2	h_2/y_e	h_2	h_1	L_2	L_3

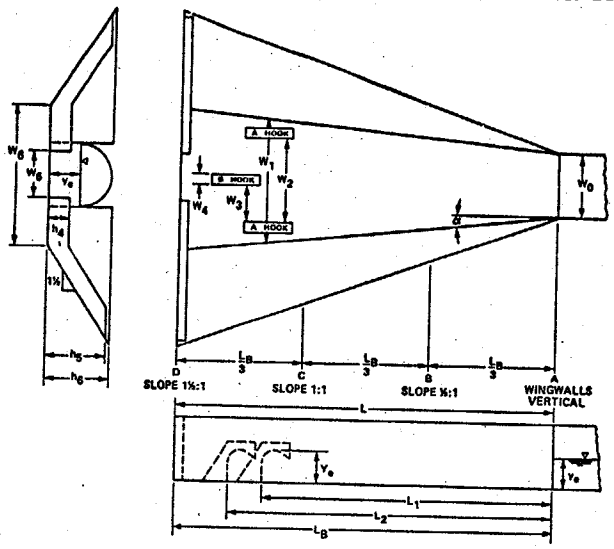
L_B	y_2	h_3

(1) y_o/D should be approx. 0.5

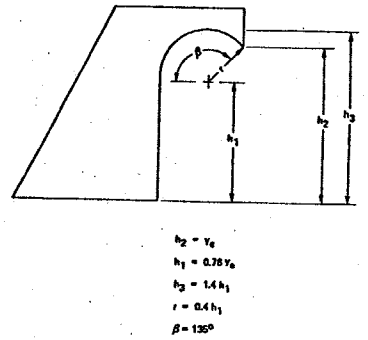
(2) $D<W \leq 3D$

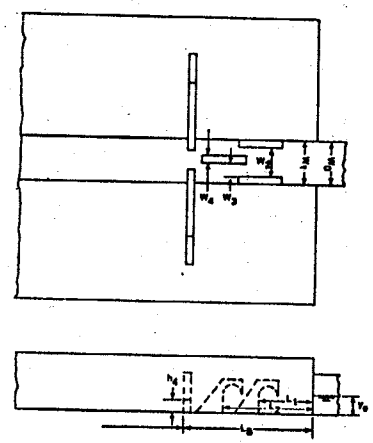
HOOK TYPE ENERGY DISSIPATOR

8-B



WARPED WINGWALL TYPE BASIN

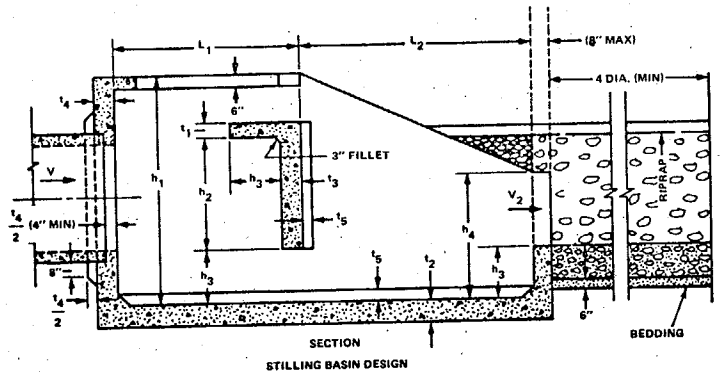


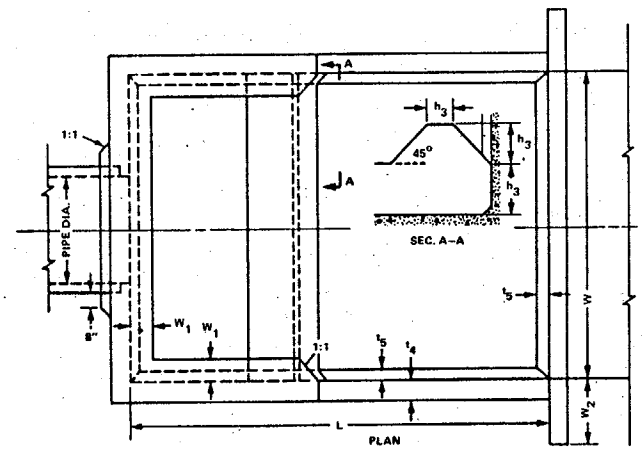


STRAIGHT TRAPEZOIDAL BASIN

TYPE	V_o/V_n	W_6	L_B	L_1	W_1	W_2	L_2	W_5	h_4
WARPED α = _____									
STRAIGHT Z = _____									
	h_5	W_4	h_6	W_3	h_1	h_2	h_3	r	
WARPED									
STRAIGHT									

HANGING BAFFLE TYPE ENERGY DISSIPATOR
USBR VI

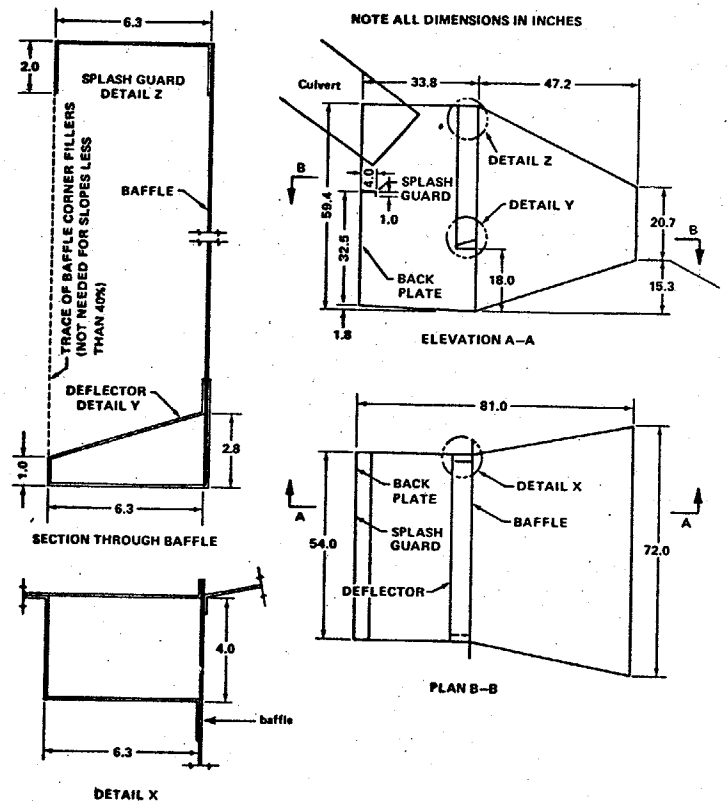




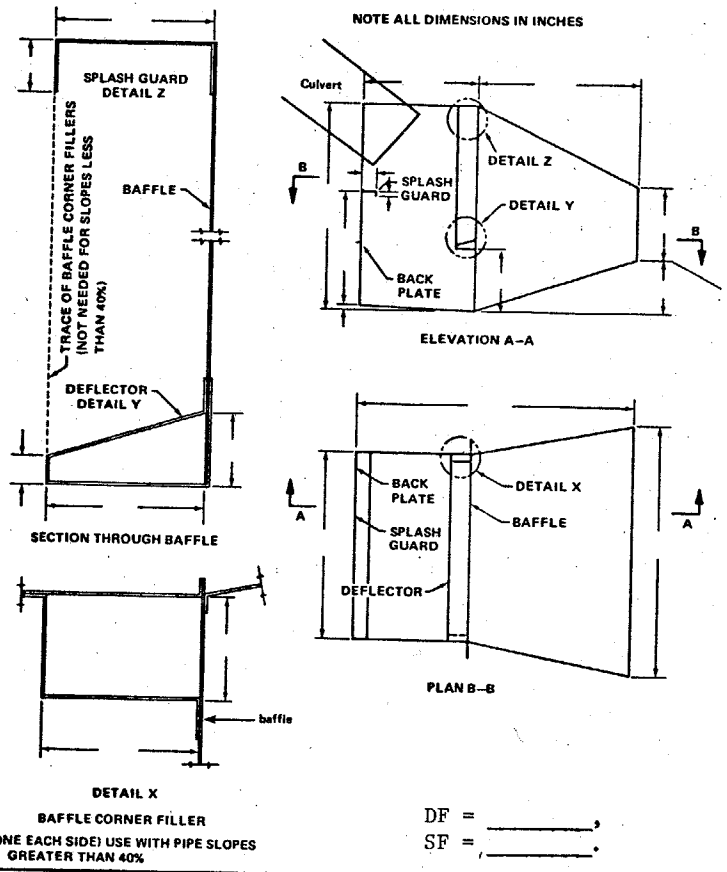
	H_o	H_o/W	W	h_1	L	h_2	h_3	L_1	L_2
	h_4	W_1	W_2	t_3	t_2	t_1	t_4	t_5	

USFS BAFFLE WALL DISSIPATOR

8-D



DF = 1.0

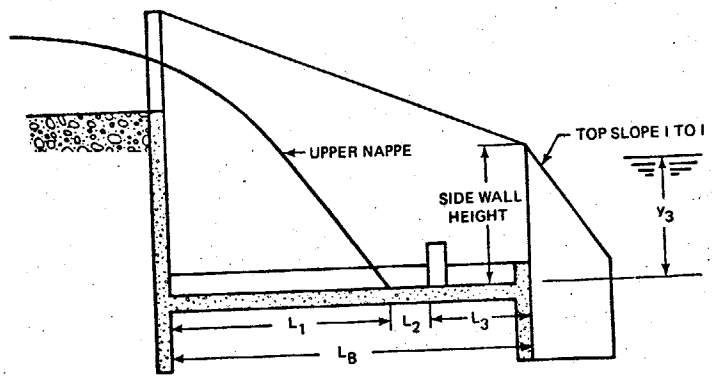


DF = _____,
SF = _____.

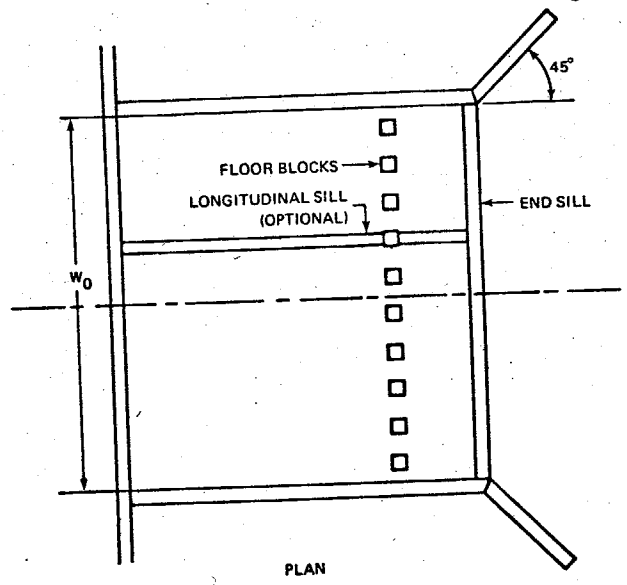
Pipe Size	C	$C^{5/2}$	Q_{18}	DF	SF	L_1	h_1	
18- inch	1.0	1.0						
Other								

9-A

STRAIGHT DROP STRUCTURE



SECTION AT CENTER LINE



PLAN

	Natural S_o	New S_o	Normal Depth Upstream	Normal Depth Downstream	y_c	W_o
INITIAL DATA						

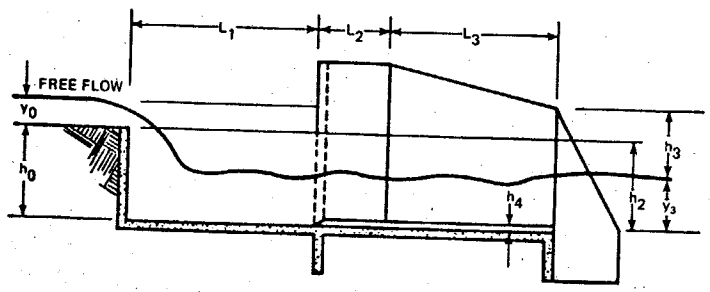
	L_1	L_2	L_3	L_B	Floor Blocks Height	Floor Blocks Width	Spacing (1)	End Sill Height	Sidewall Height
DISSIPATOR DIMENSIONS									

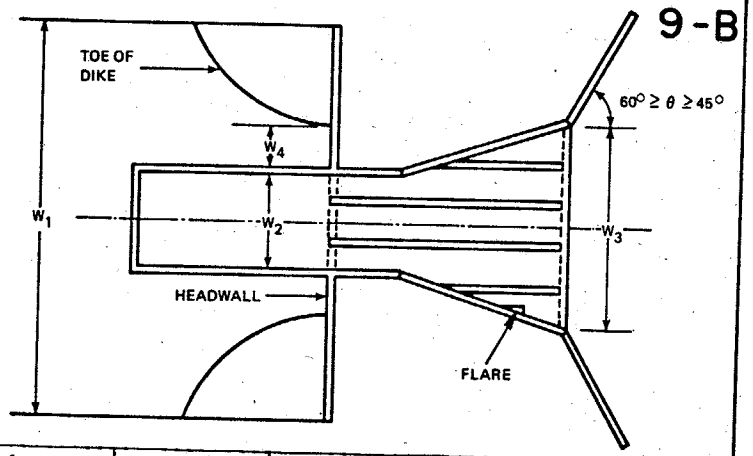
	y_3 min

(1) Block should occupy 50 to 60 percent of Basin Width.

BOX INLET DROP STRUCTURE

9-B

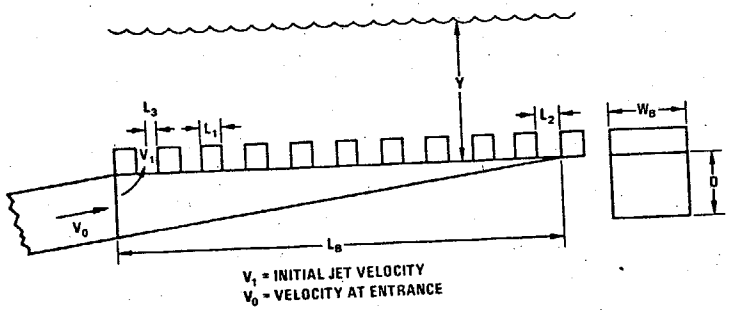


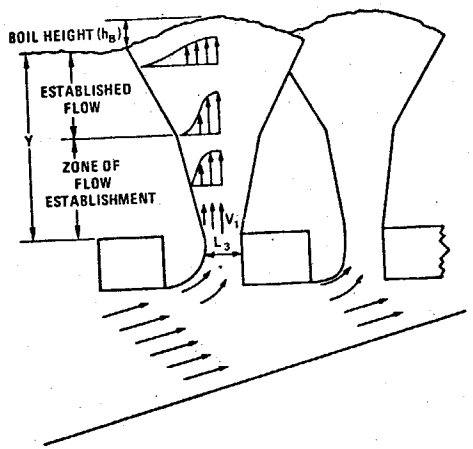


S_o	W_3	W_4	NORMAL DEPTH Upstream	NORMAL DEPTH Downstream	h_o	L_1	W_2	L_c	
Crest y_o	h_o/W_2	$C_2\sqrt{2g}$	C_2	L_1/h_o	C_H/h_o	C_H	Headwall y_o	Controlling y_o	
y_o/W_2	C_1	L_1/W_2	C_s	W_1/L_c	C_a	W_4/W_2	C_e	Adjusted Crest y_o	
y_c	y_{c3}	L_2	L_3	$11.5y_{c3}$	y_3	h_4	No. Sills		

STILLING WELL - MANIFOLD

10-A



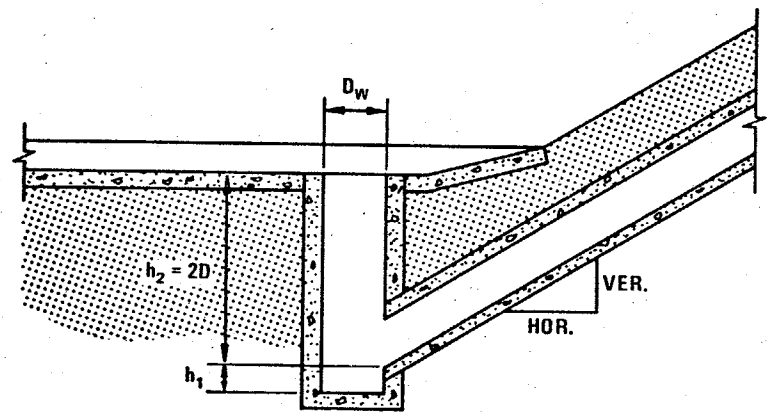


W_B	D	V_o	y	L_B	L_2/L_1	L_2	L_1	N	y/L_1 (1)
V_1/V_o	V_1	L_3	y/L_3	$V_1^2/2g$	h_W	h_B			

(1) $5 < y/L_1 < 20$

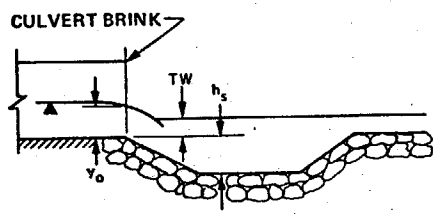
STILLING WELL - Corps of Engineers

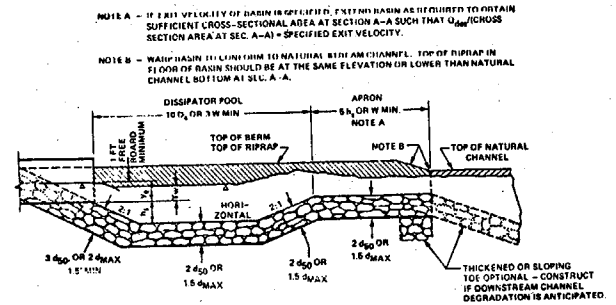
10-B



D	Q	Slope	DW	h_1/D_W	h_1	h_2	h_W		

RIPRAP BASINS





	TW	y_e	(1) TW/y_e	d_{50}/y_e	d_{50}	h_s/y_e	h_s	(2) h_s/d_{50}	
LOW TW $TW/y_e \leq 0.75$									
	V ALLOWABLE	L/D_e (3)	L	V_{ave}/V_L	V_L				
HIGH TW $TW/y_e > 0.75$									

	Larger of		
Length of Pool	$= 10\ h_s$ or $3W_o$	= _____	ft.
Length of Apron	$= 5\ h_s$ or W_o	= _____	ft.
Thickness of Approach	$= 3d_{50}$ or $2d_{max}$	= _____	ft.
Thickness of Remainder of Basin	$= 2d_{50}$ or $1.5d_{max}$	= _____	ft.

(1) $TW/y_e \leq 0.75$ for Low TW Design

(2) $2 < hs/d_{50} < 4$

(3) $D_e = [4A/\pi]^{1/2}$

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