Hydraulic Design of Energy Dissipators for Culverts and Channels

HEC 14
September 1983
Metric Version

Welcome to HEC 14 - Hydraulic Design of Energy Dissipators for Culverts and Channels

NOTES ON THE CONVERSION OF EXAMPLE PROBLEMS AND EQUATIONS TO SI UNITS

1. In most cases, the depth, velocities, and flow rates are given to thousandths to retain original example problem's solution. For actual designs, these values can be rounded to more convenient increments.
2. English dimensions are provided in parentheses if the information provides insight into the selection of estimated or trial values during the design process.
3. Lengths are in meters or millimeters, velocity is in m/s, and flow rate is in m^3/s unless noted in the text.

DISCLAIMER: During the editing of this manual for conversion to an electronic format, the intent has been to convert the publication to the metric system while keeping the document as close to the original as possible. The document has undergone editorial update during the conversion process.

Table of Contents for HEC 14-Hydraulic Design of Energy Dissipators for Culverts and Channels (Metric)

List of Figures for HEC 14-Hydraulic Design of Energy Dissipators for Culverts and Channels (Metric)

Back to Table of Contents

Chart Can Be Used with HEC 11 for Sizing Riprap

Back to Table of Contents

Chapter 1 : HEC 14

Design Concept

Go to Chapter 2

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 result in 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 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 and performance curve
- Standard culvert outlet design utilized: projecting, wingwalls, headwall, and aprons

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, bank and bed 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, design 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 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 and design.

The designer might also consider energy dissipator design as a mini-system involving numerous energy dissipation schemes with overlapping selection criteria. A combination of dissipator and channel protection might be used to solve specific problems. Figure 1-1, "Conceptual Model--Energy Dissipator Design," indicates the input, output, and the various steps in the energy 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 Chapter 2, 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 run-off event is needed; or that energy dissipator or combination energy dissipator and channel protection is necessary.

CONCEPTUAL MODEL
ENERGY DISSIPATOR DESIGN

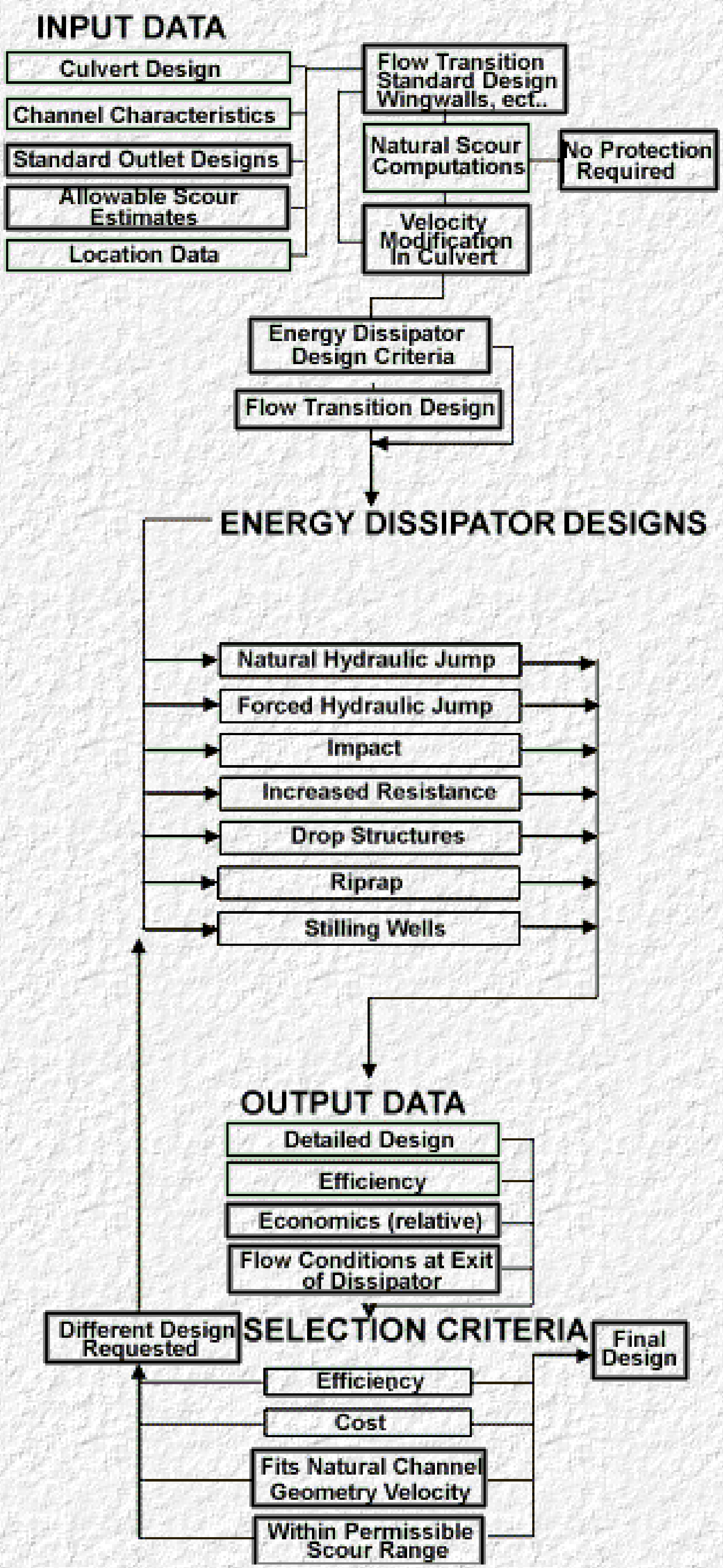


Figure 1-1. Conceptual Model of Energy Dissipator Design

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 to the flow of the downstream channel in a condition which approximates the natural flow regime. This also implies guarding against over-design 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 Chapter 7 "Outlet Velocity and Design." 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 (Chapter 5) provides guidance for the selections 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, detailed design problems, and procedures are discussed in Chapter 12 of this 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 sand bed channels. The design of free hydraulic jumps for various channel shapes and slopes are included along with energy dissipator designs which utilize forced hydraulic jumps. The design of several types of impacts basin, drop structures, and stilling well or vertical flow devices are included. The last design chapter deals with the riprap basin.

Although it is 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 this 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 a dynamic framework within which the material will be added and deleted as new information becomes available.

Go to Chapter 2

Chapter 2 : HEC 14

Erosion Hazards

Go to Chapter 3

2-A Erosion Hazards at Culvert Inlets

Erosion from vortexes, flow over wingwalls, and fill sloughing at culvert inlets are generally not major problems. However, there are some exceptions. For example, some degree of protection may be required if a confined approach channel is not aligned with the culvert axis. The area with the greatest potential for 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 at a distance 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 materials or channel protection is provided. Hydraulic Engineering Circular No. 13 (2A3) discusses the advantages of providing a depression or fall at the culvert entrance to increase culvert capacity.

Culvert invert depressions of 0.30 or 0.61 meters 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. 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 the 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 No. 5, HEC No. 10, and HEC No. 13 (2A1, 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 recommendations of HEC No. 13.

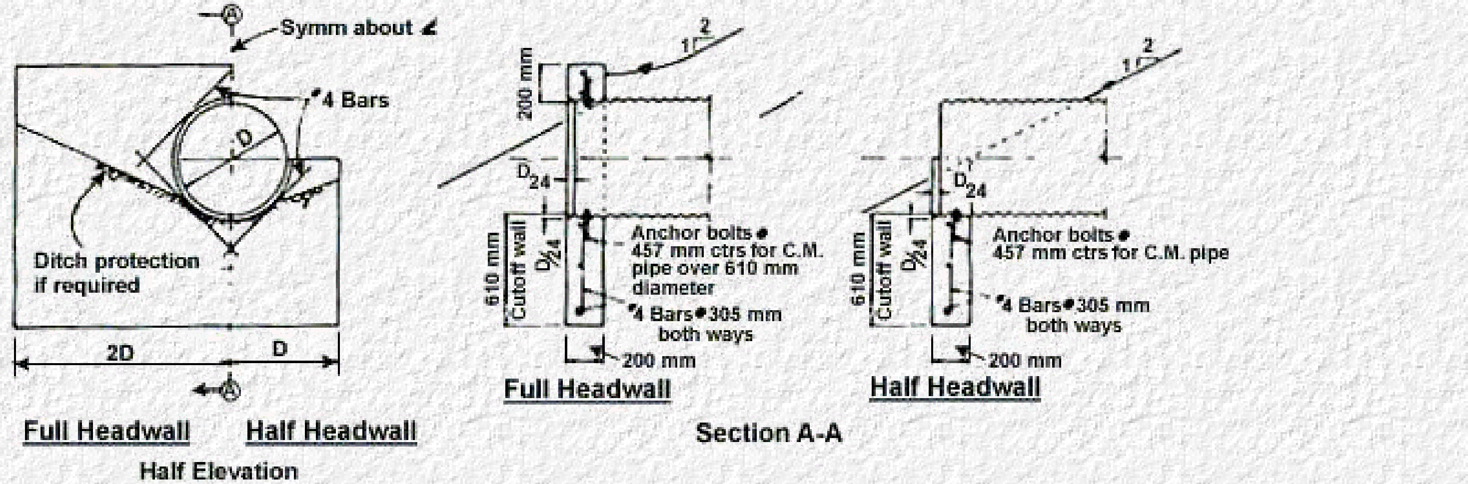
If the flow velocity near the inlet indicates a possibility of scour threatening the stability of wingwall footings, erosion protection should be provided. A concrete apron between wingwalls is the most satisfactory means for providing this protection. The slab has the further advantage that it may be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

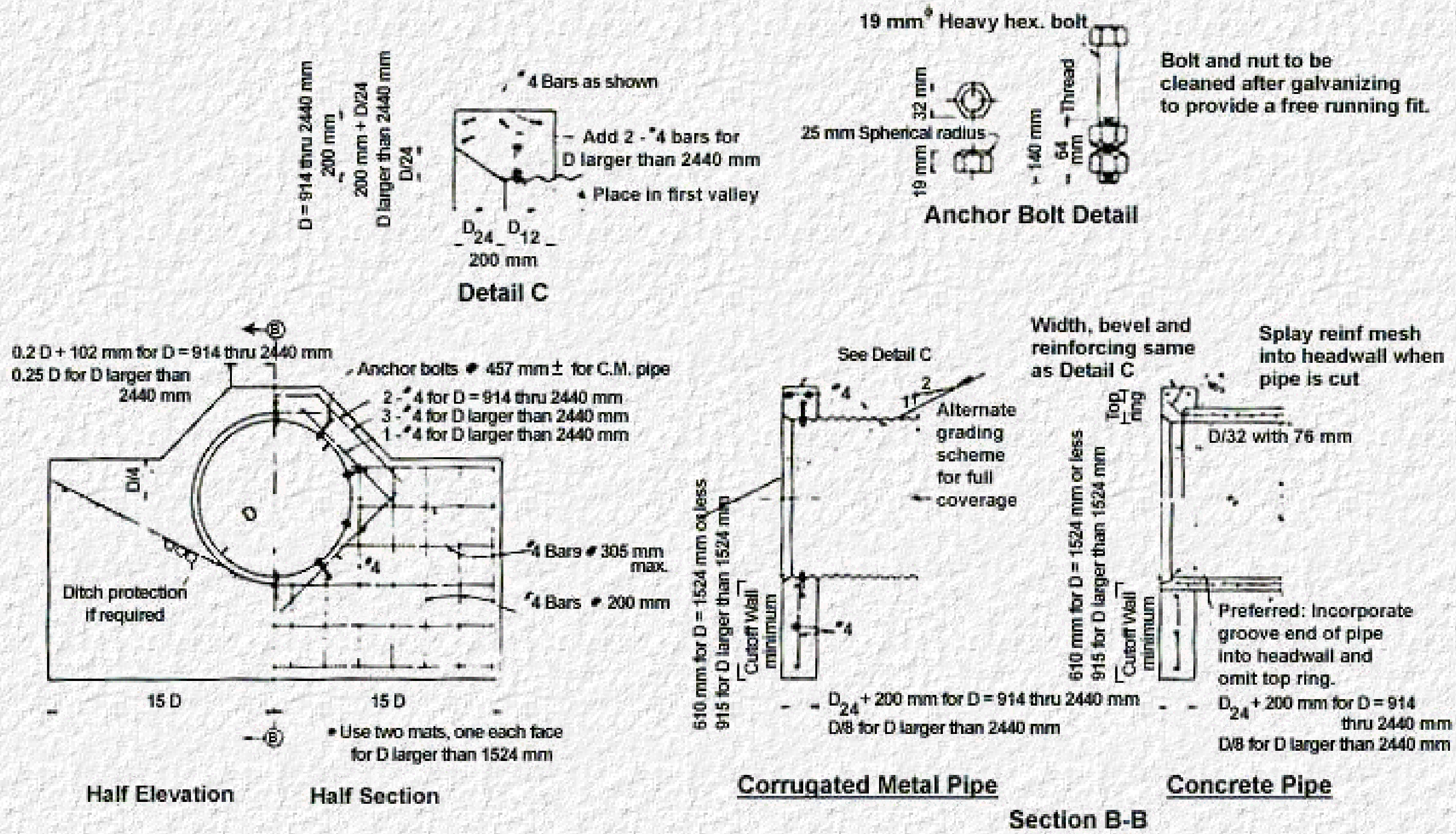
Inlet Failures

Most 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. See Figure 2-A-1 and Figure 2-C-1. Riprap can serve as protection in some instances, but concrete inlet structures anchored to the pipe are safer. Performed concrete or metal sections may be used in lieu of the inlet structures shown. Metal end sections for culvert pipes larger than 1350 mm 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 4800 mm in height.



Inlet Structure for Culvert Sizes 457 mm to 914 mm Diameter



Inlet Structure for Culvert Sizes 914 mm to 4570 Diameter (Not to Scale)

SUMMARY OF QUANTITIES				
Culvert Dia	Full Headwall		Half Headwall	
	concrete	reinf. steel	concrete	reinf. steel
mm	m^3	kg	m^3	kg
457	0.436	20.41	0.138	6.80
610	0.650	29.48	0.199	9.07
762	87.924	38.56	0.260	11.34
914	114.683	52.16	0.329	15.88

SUMMARY OF QUANTITIES		
culvert dia mm	concrete m^3	reinf. steel kg
914	0.765	34.02
1067	0.994	40.82
1219	1.147	47.63
1372	1.529	56.70
1524	1.835	65.77
1829	3.135	197.32
2134	4.282	242.68
2438	5.199	290.30
2743	7.493	360.61
3048	9.480	433.19
3658	15.368	569.27
4572	26.747	825.55

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 $f'_c = 27613$ kPa.The air entraining agent shall be approved by the engineer prior to acceptance for use. All exposed edges shall be chamfered 19 mm 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 0.9144 meters, and in no case less than 0.3048 meters.

Piping: When using previous 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.

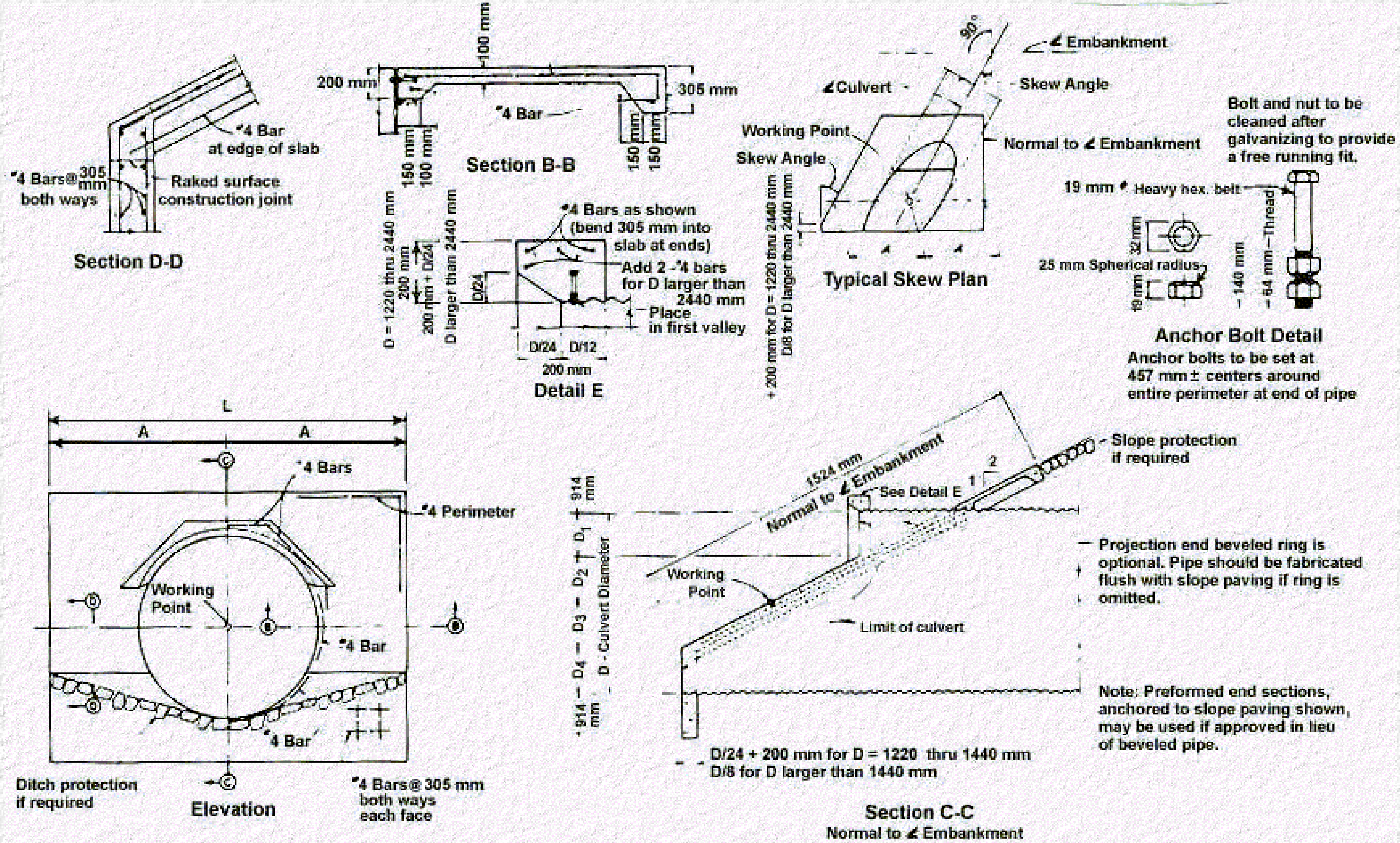
Skew: When culvert is skewed to embankment, the embankment may be contoured as shown on Sheet No. 4.

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

U.S. Department of Transportation
Federal Highway Administration
Washington, D.C.

Circular Culvert End Treatment
Inlet Structures for Concrete and Corrugated
Metal Culverts Sizes 457 mm to 4570 mm Diameter

Figure 2-A-1. Inlet Structures for Concrete and Corrugated Metal Culverts



Section D-D
#4 Bar at edge of slab
#4 Bars@305 mm both ways
Raked surface construction joint
Section B-B
200 mm
100 mm
305 mm
#4 Bar
150 mm
100 mm
Detail E
#4 Bars as shown (bend 305 mm into slab at ends)
Add 2 - #4 bars for D larger than 2440 mm
Place in first valley
D = 1220 thru 2440 mm
D larger than 2440 mm
200 mm + D/24
D/24
D/12
200 mm
Typical Skew Plan
Embankment
Culvert
Skew Angle
Working Point
Normal to Embankment
90°
+ 200 mm for D = 1220 thru 2440 mm
D/8 for D larger than 2440 mm
Anchor Bolt Detail
Bolt and nut to be cleaned after galvanizing to provide a free running fit.
19 mm Heavy hex. belt
25 mm Spherical radius
32 mm
19 mm
140 mm
64 mm—Thread
Anchor bolts to be set at 457 mm± centers around entire perimeter at end of pipe
Elevation
L
A
A
#4 Bars
#4 Perimeter
Working Point
#4 Bar
#4 Bar
Ditch protection if required
#4 Bars@305 mm both ways each face
Section C-C
Normal to Embankment
1524 mm
Normal to Embankment
See Detail E
Slope protection if required
Working Point
Limit of culvert
914 mm
914 mm
D - Culvert Diameter
Projection end beveled ring is optional. Pipe should be fabricated flush with slope paving if ring is omitted.
Note: Preformed end sections, anchored to slope paving shown, may be used if approved in lieu of beveled pipe.
D/24 + 200 mm for D = 1220 thru 1440 mm
D/8 for D larger than 1440 mm

TYPICAL DIMENSIONS AND QUANTITIES

Culvert Dia mm	S m	Square Headwall A m	L m	Concrete m^3	Reinf. kg	15 deg. Skew A m	L m	Concrete m^3	Reinf. kg
1,219	4.09	1.22	2.44	2.14	81.65	1.30	2.59	2.14	83.92
1,524	4.60	1.52	3.05	2.83	102.06	1.63	3.25	2.83	106.60
1,829	5.11	1.83	3.66	3.59	124.74	1.96	3.91	3.67	133.81
2,743	6.65	2.74	5.49	6.80	217.73	2.95	5.89	7.11	231.34
3,658	8.18	3.66	7.32	11.62	315.25	3.91	7.82	12.31	337.93
4,572	9.71	4.57	9.14	18.12	426.38	4.90	9.80	19.50	462.67

Culvert Dia mm	S m	30 deg. Skew A m	L m	Concrete m^3	Reinf. kg	45 deg. Skew A m	L m	Concrete m^3	Reinf. kg
1,219	4.09	1.63	3.25	2.45	99.79	2.44	4.88	3.44	145.15
1,524	4.60	2.03	4.06	3.36	131.54	3.05	6.10	4.82	190.51
1,829	5.11	2.44	4.88	4.36	163.30	3.66	7.32	6.35	238.14
2,743	6.65	3.66	7.32	8.64	285.77	5.49	10.97	12.84	421.85
3,658	8.18	4.88	9.75	15.37	424.12	7.32	14.63	23.17	632.77
4,572	9.71	6.10	12.19	24.54	582.88	9.14	18.29	37.23	882.25

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 = 27613 kPa.The air entraining agent shall be approved by the engineer prior to acceptance for use. All exposed edges shall be chamfered 19 mm unless otherwise noted. Slope paving surface variations shall not exceed 9 mm in 3 m.

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 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 one meter.

Piping: When using previous 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 alternative and the embankment contoured as shown on Sheet No. 4.

U.S. Department of Transportation
Federal Highway Administration
Washington, D.C.

Circular Culvert End Treatment
Inlet Structures for Corrugated Metal Culverts
Sizes 1200 mm to 4500 mm Diameter

Figure 2-A-2. Inlet Structures for Corrugated Metal Culverts Sizes 1200 mm to 4500 mm in Diameter

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 pipes 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 materials 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 special subdrainage system.

2A1. 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 p.
(Hydraulic Engineering Circular No. 5).

2A2. 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).

2A3. Harrison, L. J., Morris, J. L., Norman, 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 Engineering Circular No. 13).

2-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;

1. local scour
2. 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 the 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 tail-water and the full barrel velocity for high tail-water. Where the discharge leaves the barrel at critical depth, the velocity will usually be in the range of 3 to 6 meters 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 4, 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.

2B1. Normann, J. M.,
Design of Stable Channels with Flexible Linings,
Federal Highway Administration, October 1975

2B2. AASHTO, Highway Drainage Guidelines,
Guidelines for the Hydraulic Design of Culverts,
Vol. IV, 1975, 45 pp.

2-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 2-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 2C1.

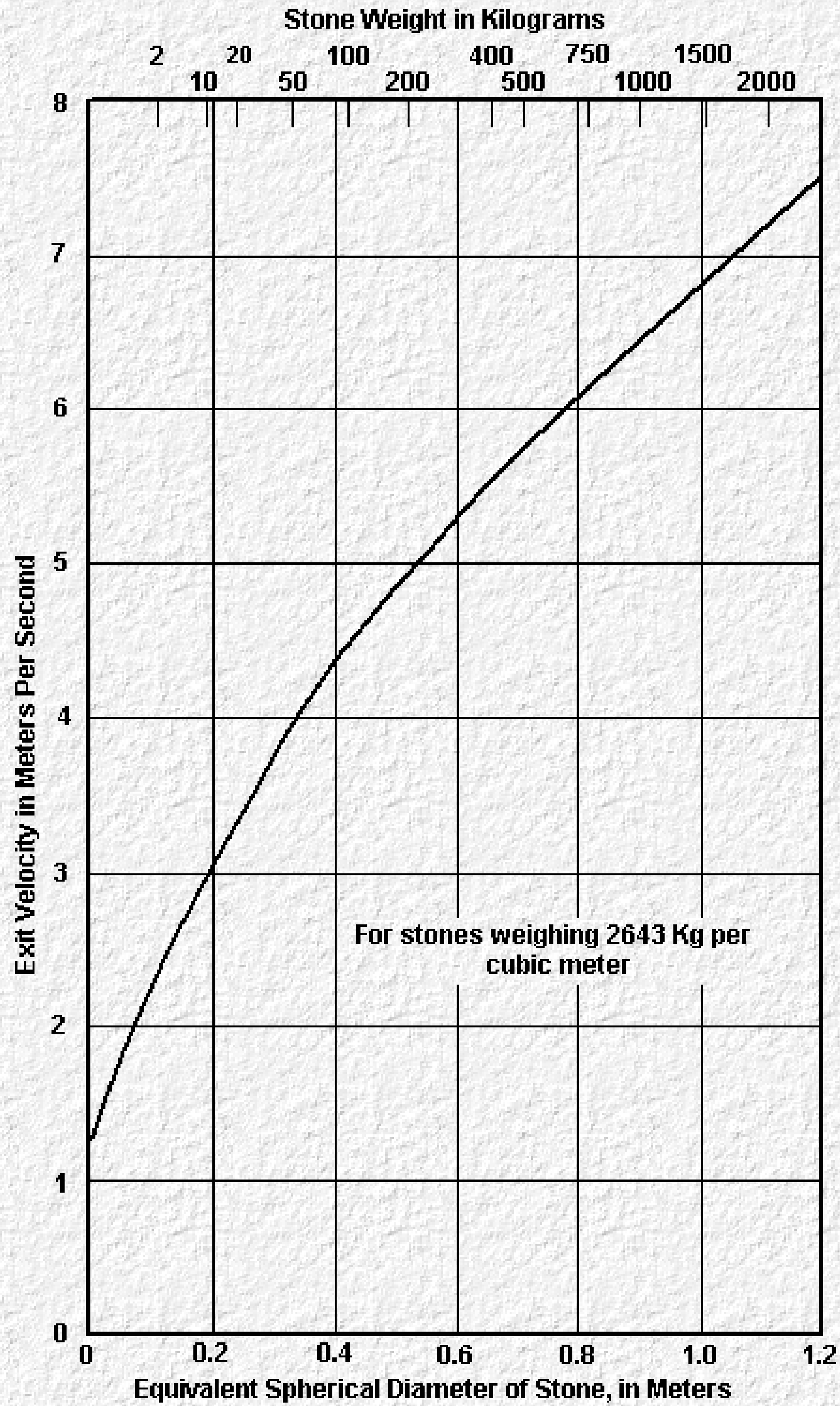


Figure 2-C-1. Riprap Size for Use Downstream of Energy Dissipators (from Reference 2C1)

2C1. Searcy, James K.,

Use of Riprap for Bank Protection
Federal Highway Administrations,
Washington, D.C., 1967, pp. 43.

Go to Chapter 3

Chapter 3 : HEC 14

Culvert Outlet Velocity and Velocity Modification

Go to Chapter 4

Culvert Outlet Velocity

Culvert outlet velocity is one of the primary indicators of erosion potential. Outlet velocities are seldom less than 3.05 m/s and range up to 9 m/s 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 3-1, taken from HEC No. 5 (3-1), indicates the four types of flow for culverts on mild slopes, i.e., culverts flowing with outlet control.

Figure 3-1a 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	900 mm	1200 mm	1500 mm	1800 mm
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 Figure 3-1b, Figure 3-1c, or Figure 3-1d 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 3-1b 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. Figure 3-3,

Figure 3-4, Figure 3-5, Figure 3-6, Figure 3-7, and Figure 3-8 are included for convience in determining critical depth for various shapes of culverts.

Example: A 900 mm CMP discharging 2.832 m^3/s, flowing full with a tailwater of 0.610 m.

Critical depth (y_c) exceeds 0.914 meters. (see Figure 3-4).

Therefore, the barrel is flowing full to the end. From Table 3-2 with d/D=1, A/D^2 =0.785, and v=2.832/.785$(0.914)^2$ = 4.38m/s.

Changing to a 1200 mm CMP, changes y_c to 0.945 meters which is less than D so y_c controls outlet velocity.

$$V_{FULL} = \frac{2.832}{.785(1.219)^2} = 2.43 \text{ m/s and } y_c/D = 0.945/1.219 = 0.78$$

V/V_{FULL} = 1.13 from Figure 7-C-3 and V = 1.13(2.43) = 2.74 m/s.

This is a reduction of about 37 percent instead of the approximate 44 percent indicated in the previous example.

When culverts discharge as in Figure 3-1c and Figure 3-1d 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 steps to compute normal depth (tailwater) in the outlet channel, (see Table 3-1 and Table 3-2) facilitate normal depth calculations with this, Figure 3-9 and Figure 3-10 may be used directly to determine outlet brink depths for rectangular and circular sections. These figures are dimensionless rating curves which indicate the effect on brink depth of tailwater for culverts on mild or horizontal slopes. Values of 1.881 $Q/BD^{3/2}$ and 1.881 $Q/D^{5/2}$ for use with Figure 3-9 and Figure 3-10 are included as Table 3-3.

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 Figure 3-9 and Figure 3-10. As the tailwater increases, the depth at the brink increase, at a variable rate, along the 1.811$Q/BD^{3/2}$ or 1.811$Q/D^{5/2}$ curve, until a point where the tailwater and brink depth vary linearly at the 45º line on Figure 3-9 and Figure 3-10. Using these figures, the effects of changing culvert size may be determined. For example.

		Q = 1.698 m^3/s (constant) TW = 0.610 m (constant)		
D	$D^{5/2}$	1.881 $Q/D^{5/2}$	TW/D	y_o /D
1.07	1.18	2.61	.57	.63
1.22	1.64	1.88	.50	.54
1.37	2.20	1.39	.44	.46
1.52	2.85	1.09	.40	.41

y_o/D	y_o	y_c	A/D^2	A	V= Q/A	D
.63	0.67	0.73	0.52	0.595	2.83	1.07
.54	0.66	0.70	0.43	0.640	2.65	1.22
.46	0.63	0.70	0.35	0.657	2.57	1.37
.41	0.62	0.67	0.30	0.693	2.46	1.52

Changing culvert diameter from 1.07 to 1.52 meters, a 42 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 in Figure 3-4, Figure 3-5, Figure 3-6, Figure 3-7, and Figure 3-8. 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 3-2, it is convenient to determine normal flow conditions by the use of Manning's equation. The charts and tables of reference 3-1 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 1500 mm diameter concrete pipe with constant slopes as a base, the velocity in a 900 mm pipe will be 1.03 larger and the velocity in an 2400 mm 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 1500 mm concrete pipe

> n = 0.012, on a 1 percent slope (S_o = 0.01), discharging at 2.83 m^3/s
> V_n = 4.21 m^3/s
> 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 2.6 m/s

Manning's equation:

$$V = (R^{2/3}S^{1/2}) /n$$

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 7 contains detailed discussion and specific design information for increasing barrel resistance.

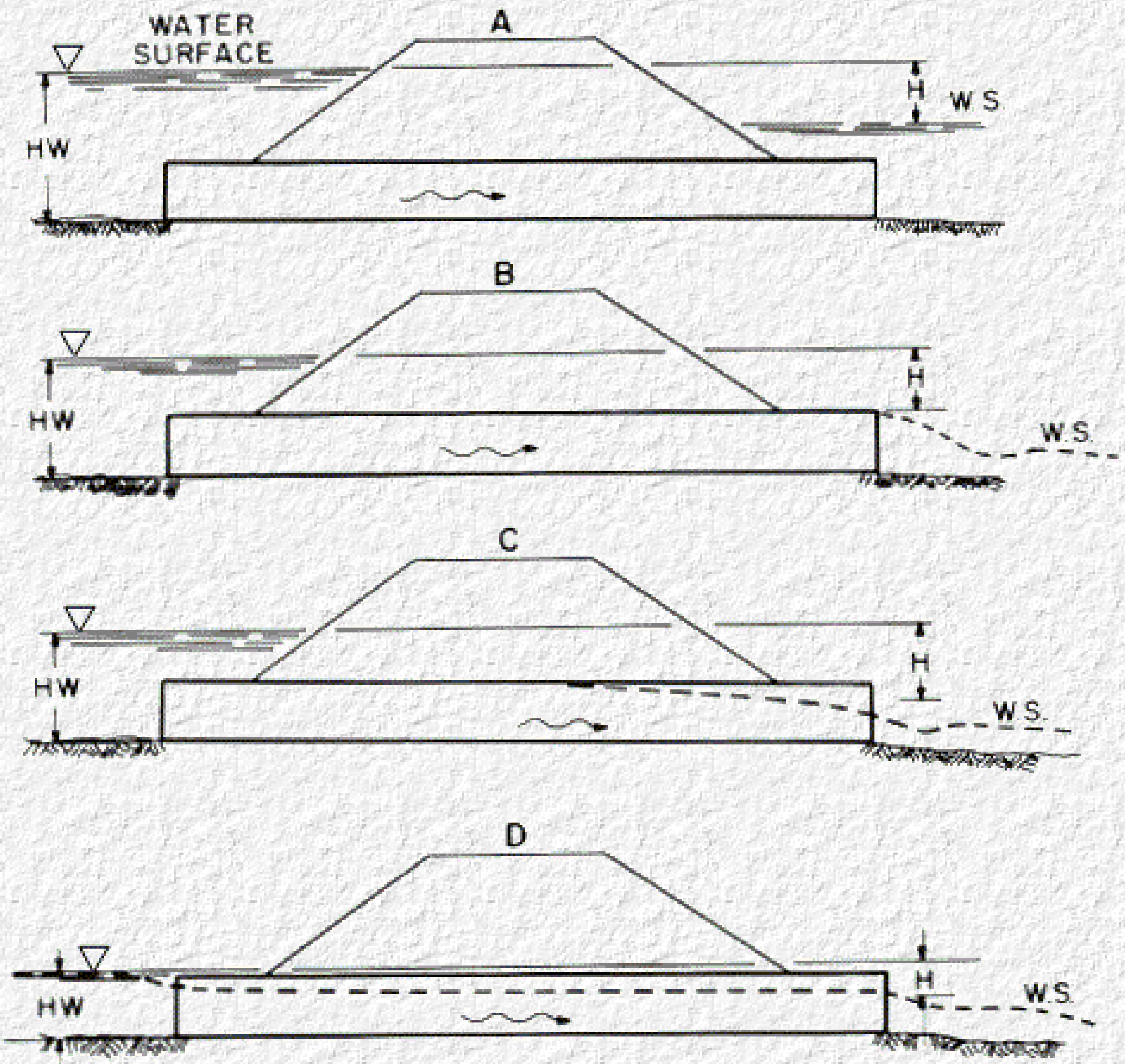


Figure 3-1. Outlet Control Flow Types

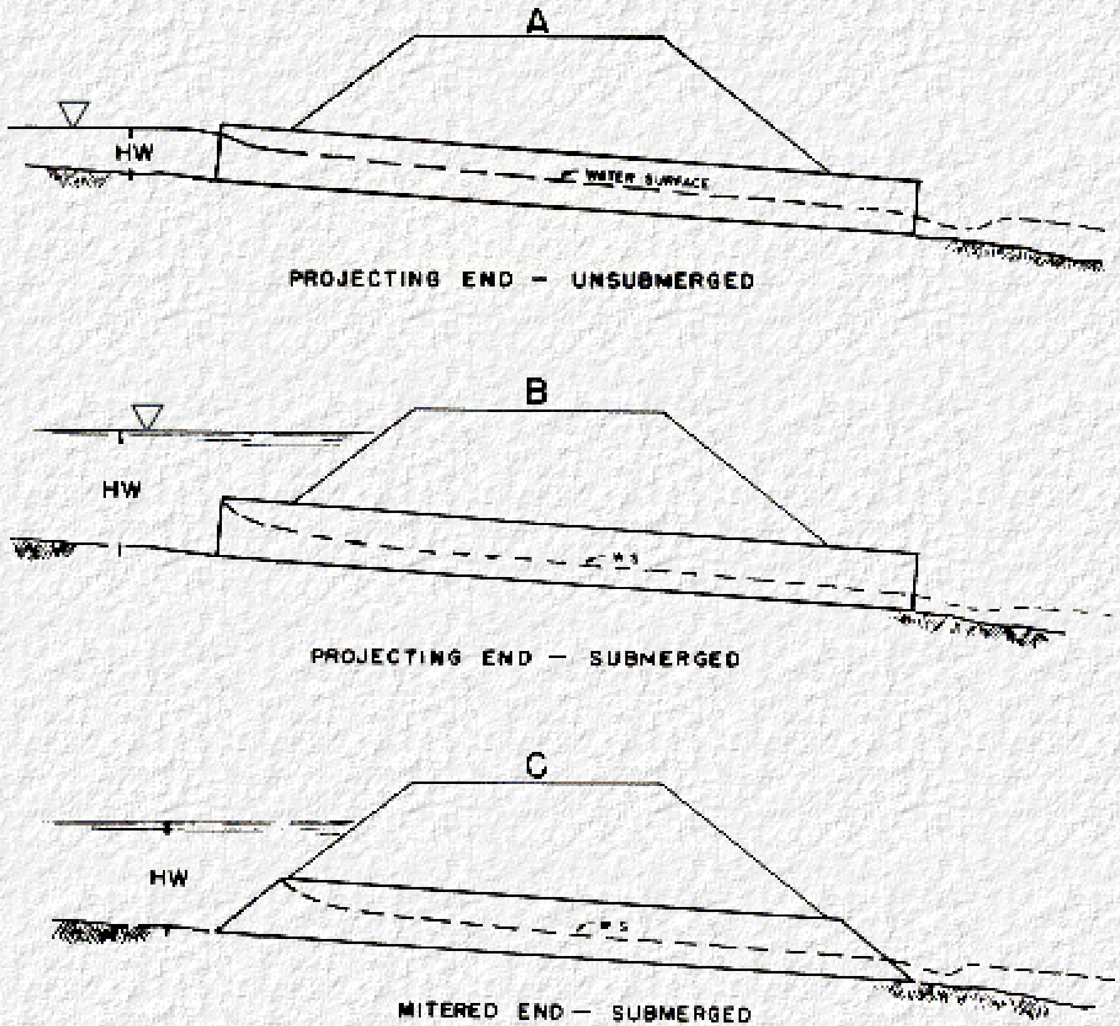
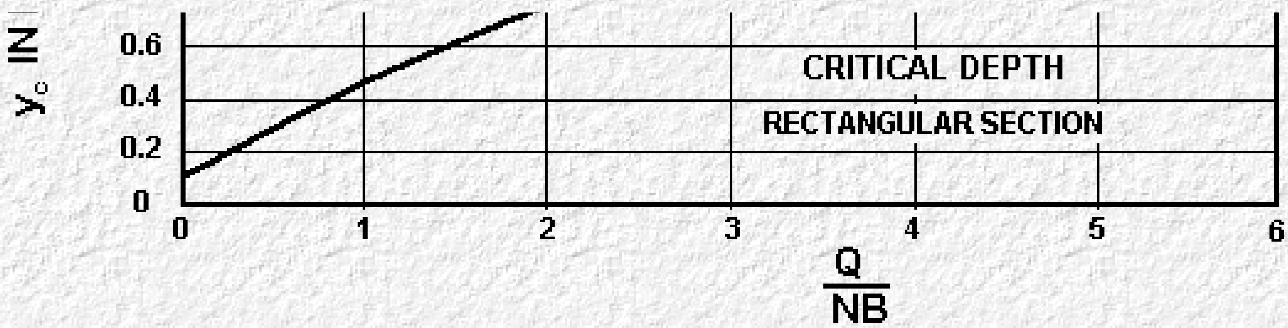


Figure 3-2. Inlet Control Flow Types


CRITICAL DEPTH
RECTANGULAR SECTION
Y_c IN
Q / NB
0.6
0.4
0.2
0
0
1
2
3
4
5
6

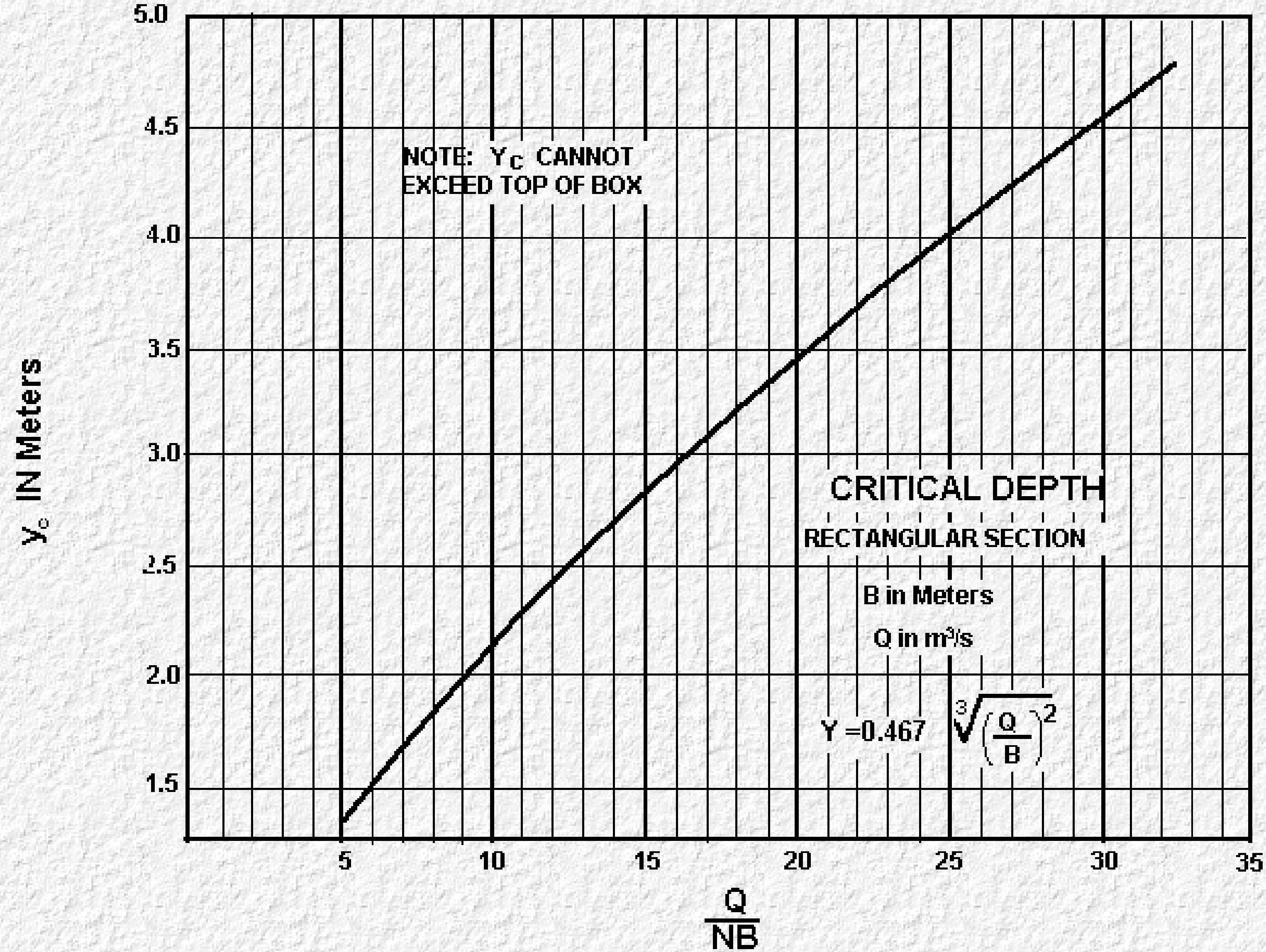
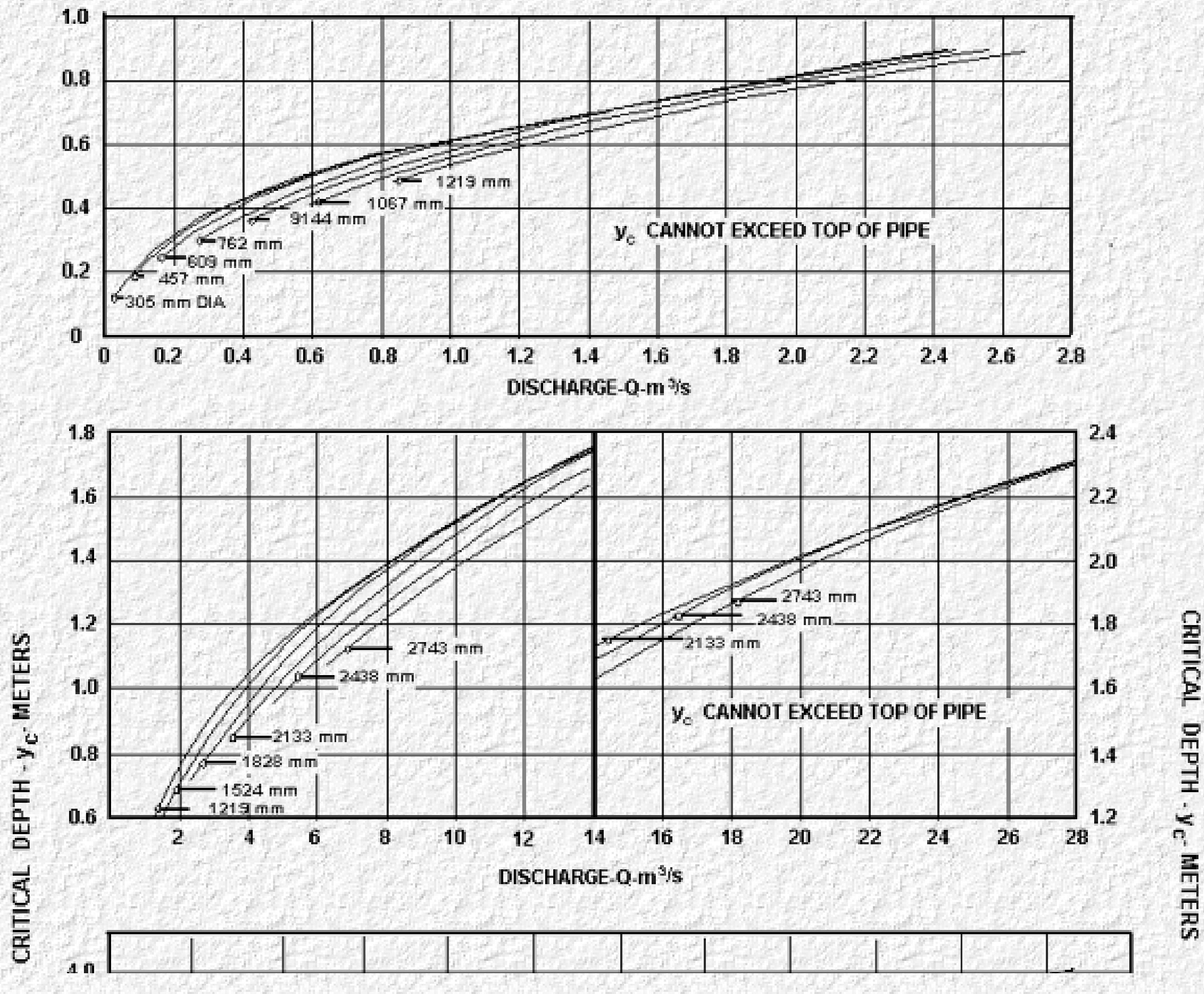

NOTE: Y_C CANNOT EXCEED TOP OF BOX
CRITICAL DEPTH
RECTANGULAR SECTION
B in Meters
Q in m^3/s
$Y = 0.467 \sqrt[3]{\left(\frac{Q}{B}\right)^2}$
Y_c IN Meters
Q / NB
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
5
10
15
20
25
30
35

Figure 3-3. Critical Depth Rectangular Section



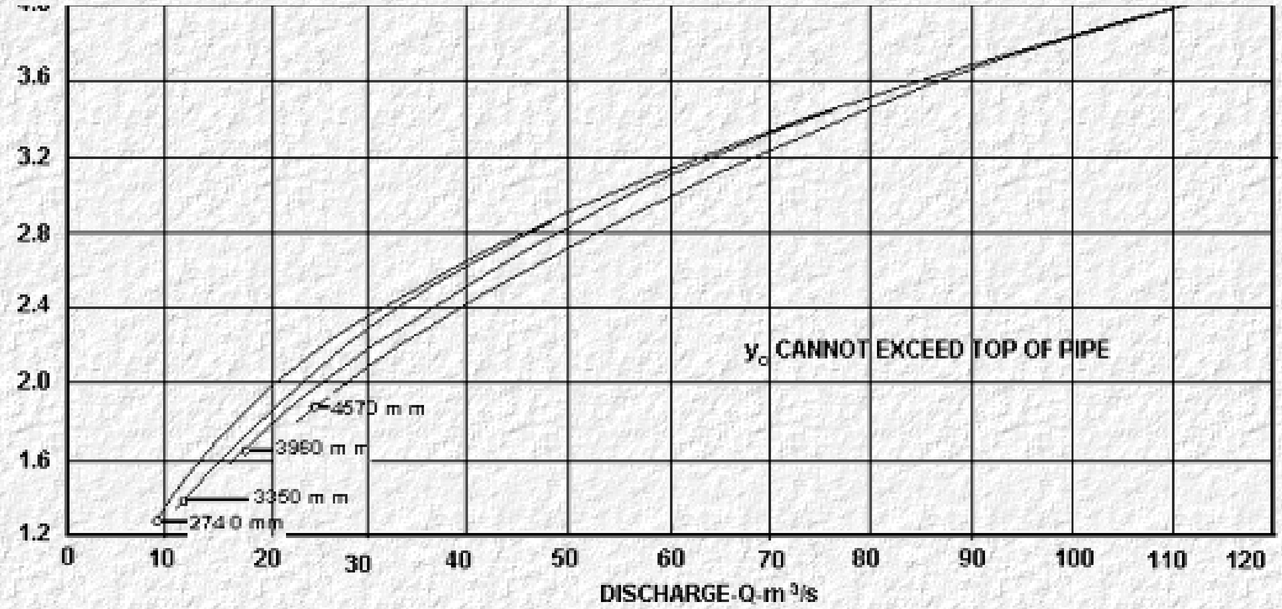
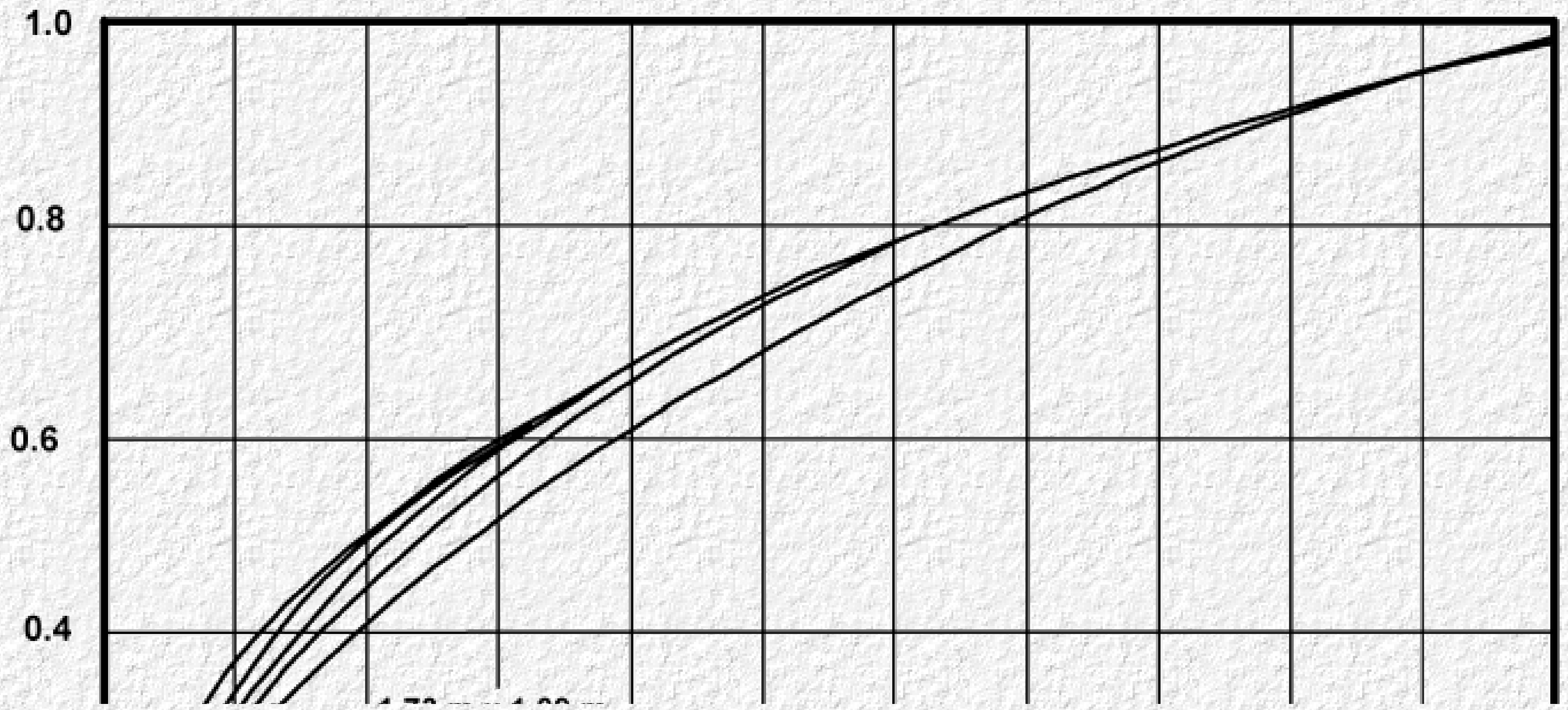
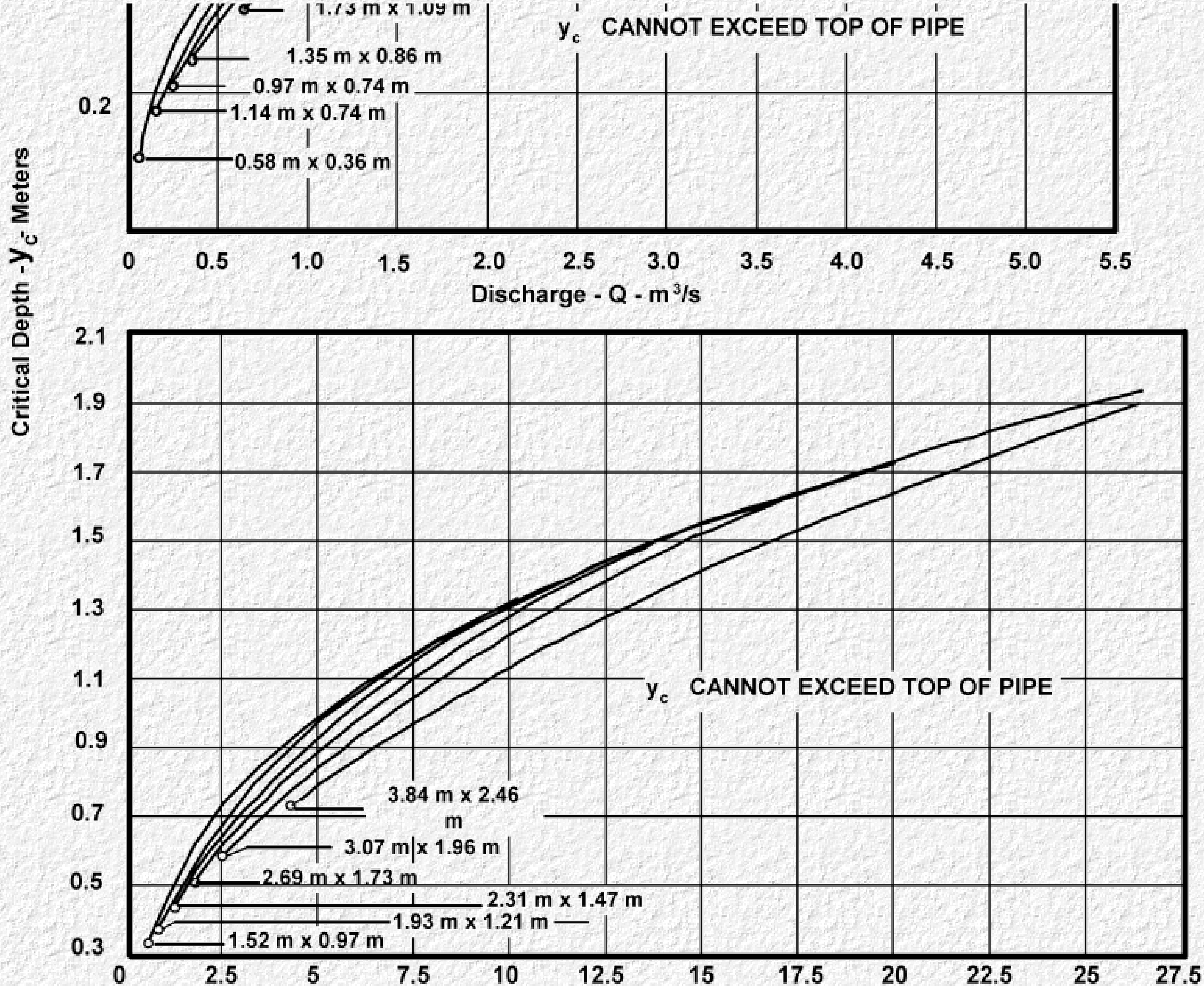


Figure 3-4. Critical Depth Circular Pipe

Figure 3-4. Critical Depth of Circular Pipe

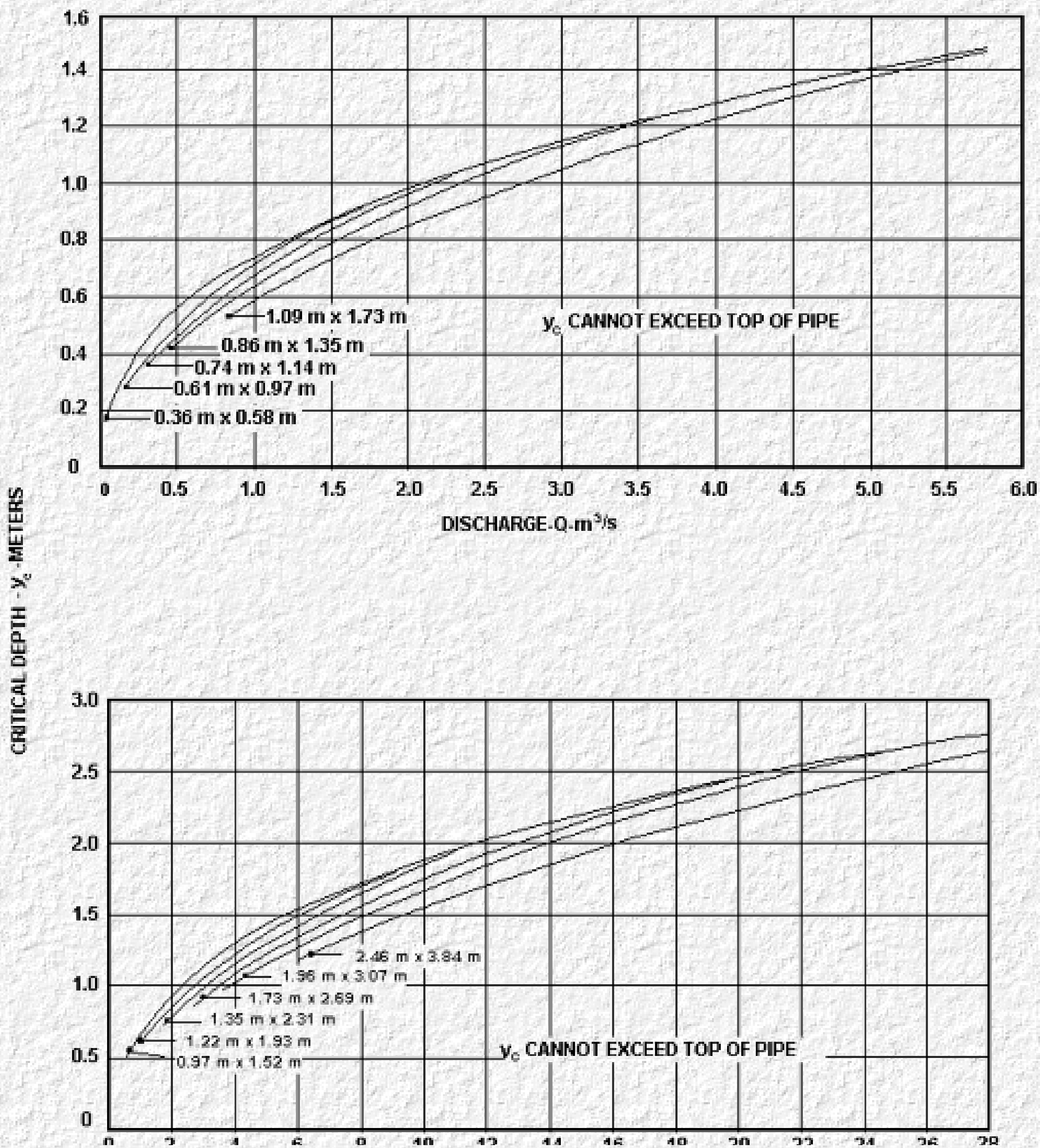




1.73 m x 1.09 m
y_c CANNOT EXCEED TOP OF PIPE
1.35 m x 0.86 m
0.97 m x 0.74 m
0.2
1.14 m x 0.74 m
0.58 m x 0.36 m
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
Discharge - Q - m^3/s
Critical Depth - y_c - Meters
2.1
1.9
1.7
1.5
1.3
1.1
0.9
0.7
0.5
0.3
y_c CANNOT EXCEED TOP OF PIPE
3.84 m x 2.46 m
3.07 m x 1.96 m
2.69 m x 1.73 m
2.31 m x 1.47 m
1.93 m x 1.21 m
1.52 m x 0.97 m
0
2.5
5
7.5
10
12.5
15
17.5
20
22.5
25
27.5

Discharge - Q - m^3/s

Figure 3-5. Critical Depth Oval Concrete Pipe Long Axis Horizontal



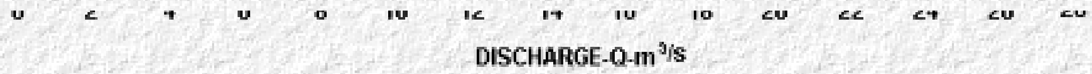
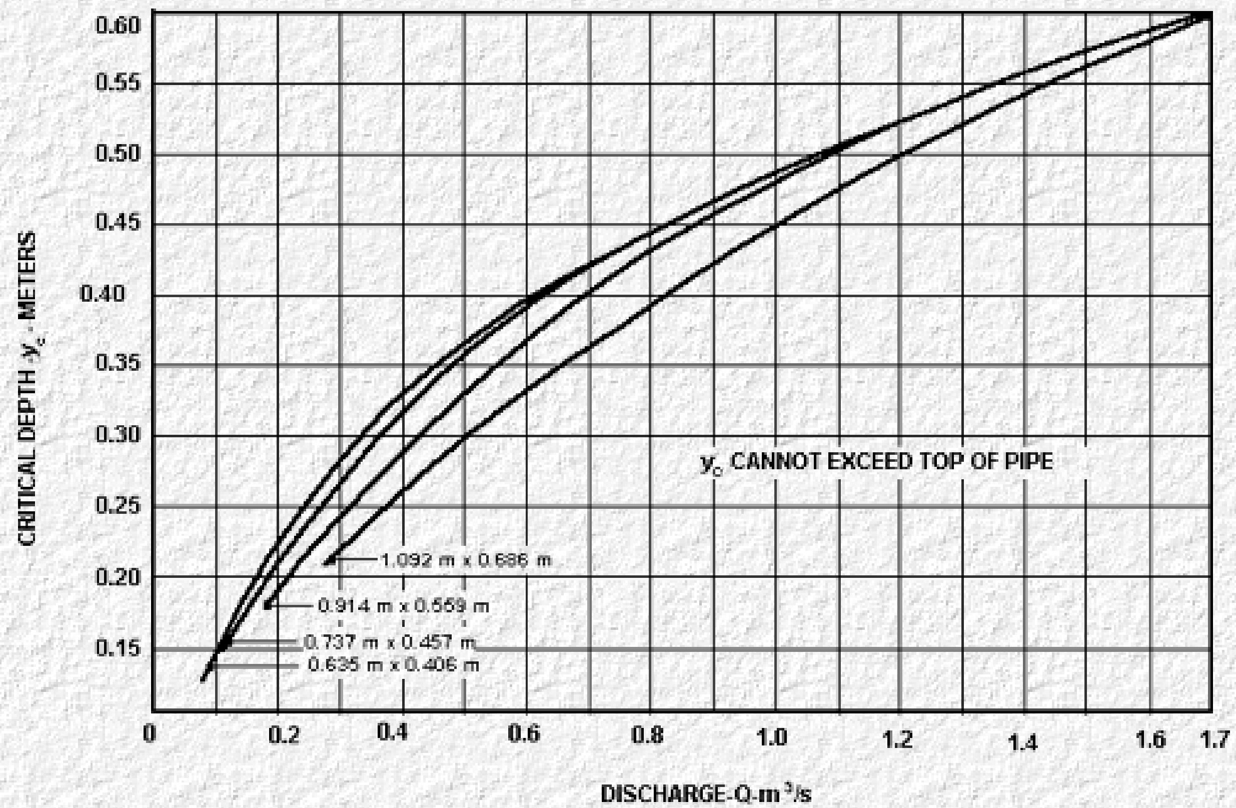
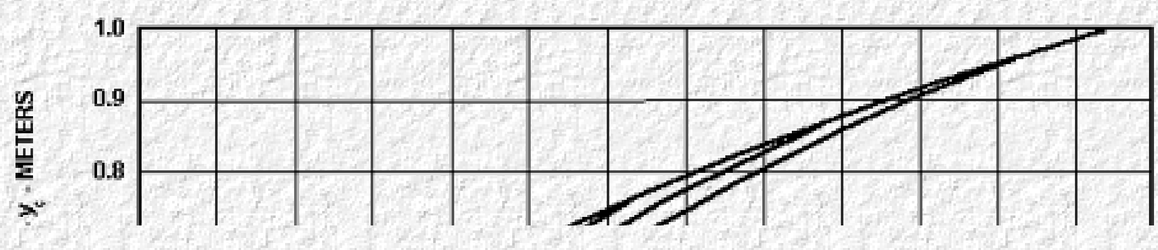


Figure 3-6. Critical Depth Oval Concrete Pipe Long Axis Vertical

Figure 3-6. Critical Depth Oval Concrete Pipe Long Axis Vertical





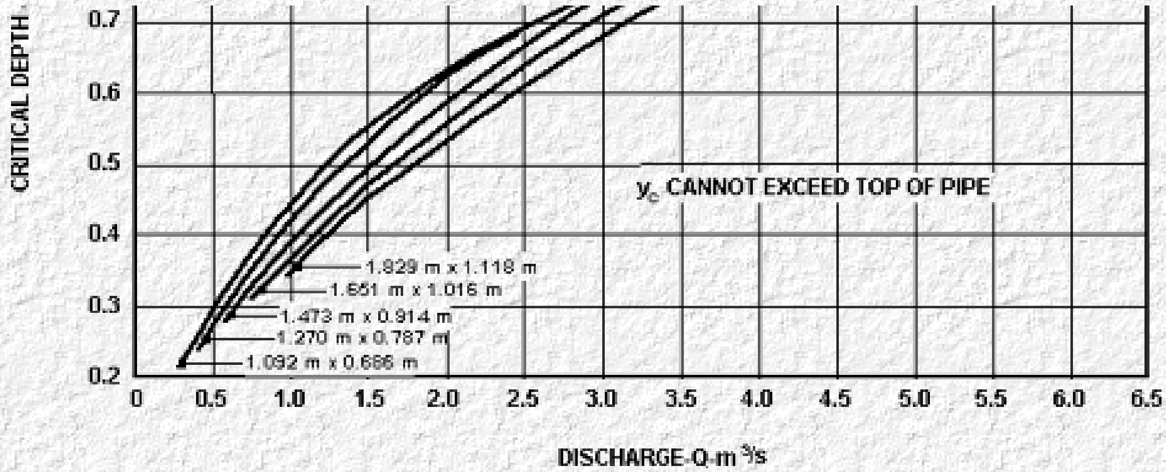
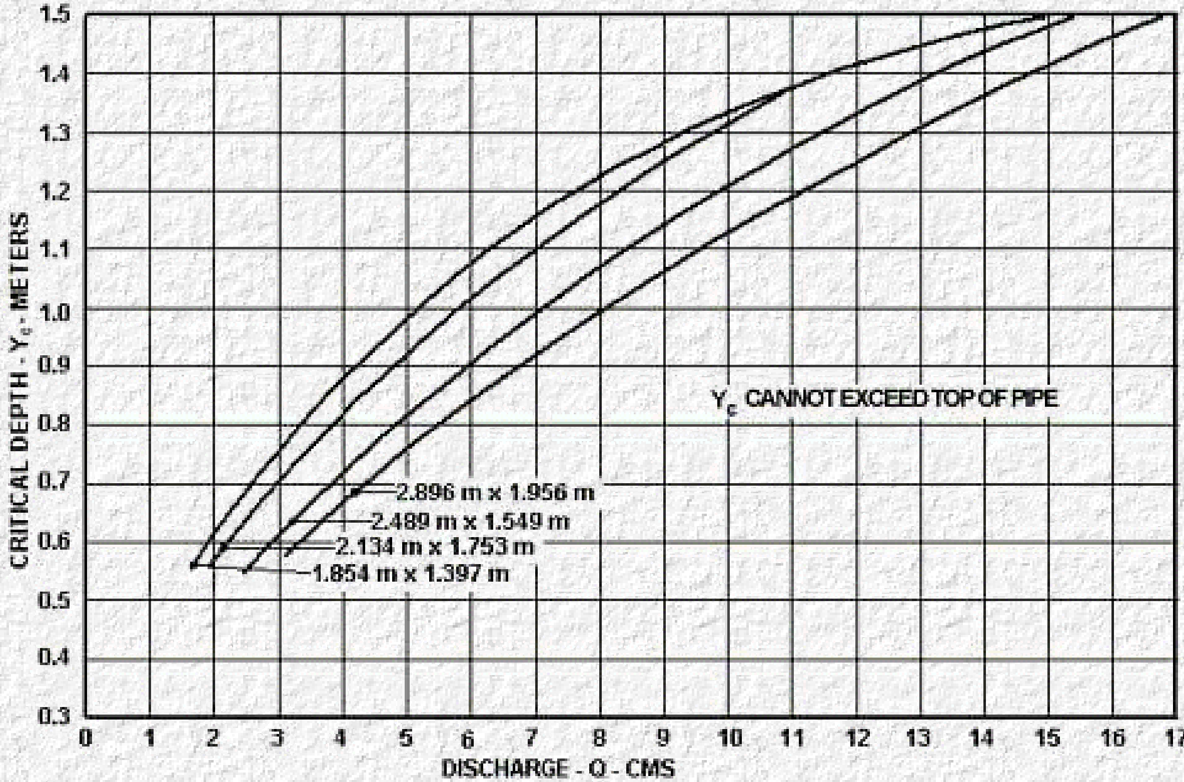
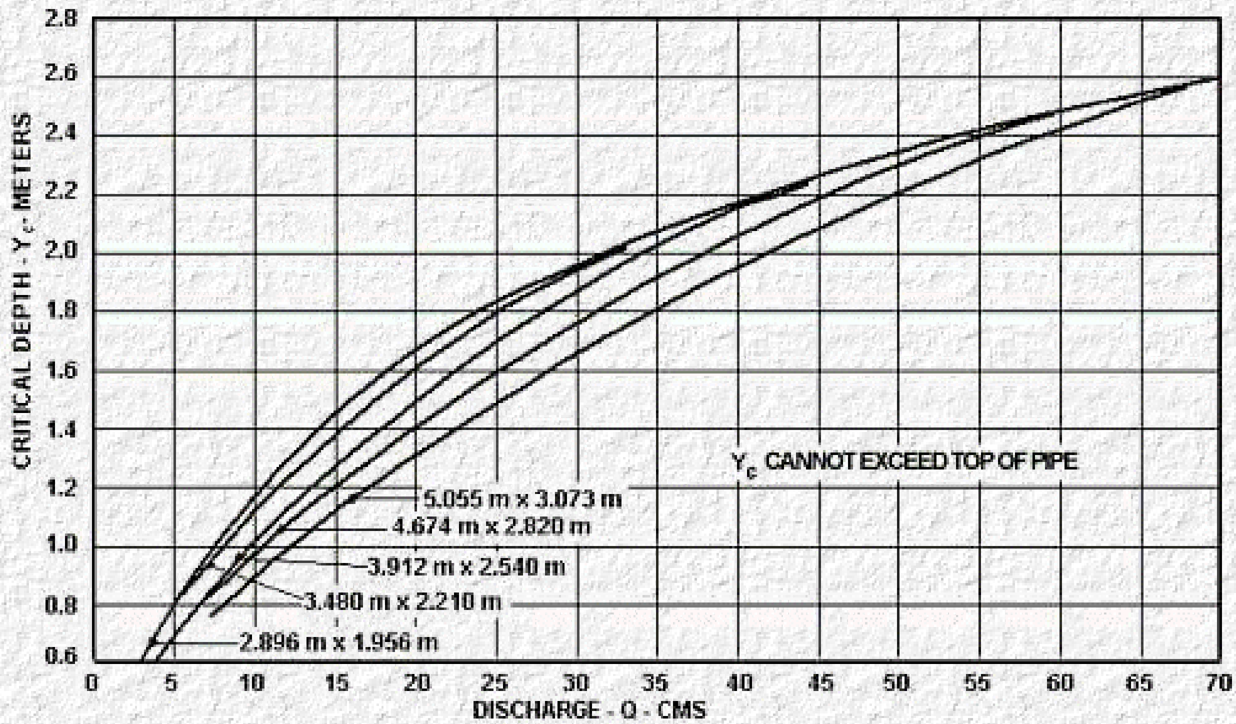


Figure 3-7. CRITICAL DEPTH Standard C.M. Pipe-Arch

Figure 3-7. Critical Depth Standard C.M. Pipe-Arch



CRITICAL DEPTH - Y_c - METERS
DISCHARGE - Q - CMS
Y_c CANNOT EXCEED TOP OF PIPE
2.896 m x 1.956 m
2.489 m x 1.549 m
2.134 m x 1.753 m
1.854 m x 1.397 m



CRITICAL DEPTH - Y_c - METERS
DISCHARGE - Q - CMS
Y_c CANNOT EXCEED TOP OF PIPE
5.055 m x 3.073 m
4.674 m x 2.820 m
3.912 m x 2.540 m
3.480 m x 2.210 m
2.896 m x 1.956 m

Figure 3-8. Critical Depth Structural Plate C.M. Pipe-Arch

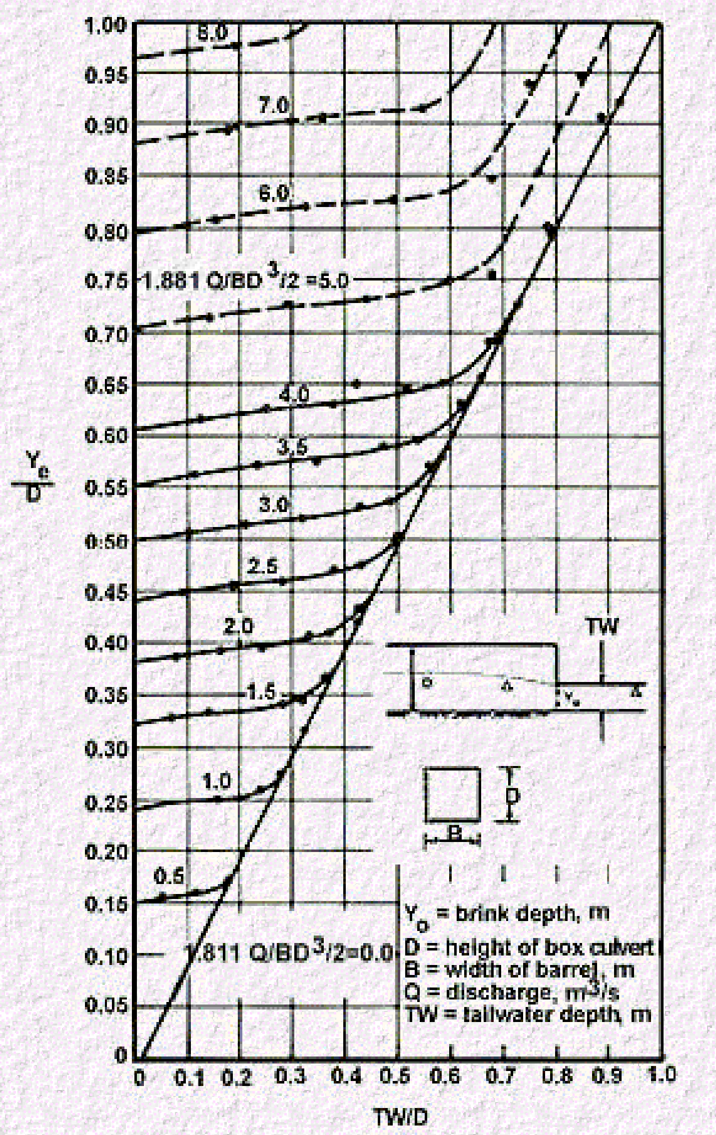


Figure 3-9. Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes (from reference 3-2)

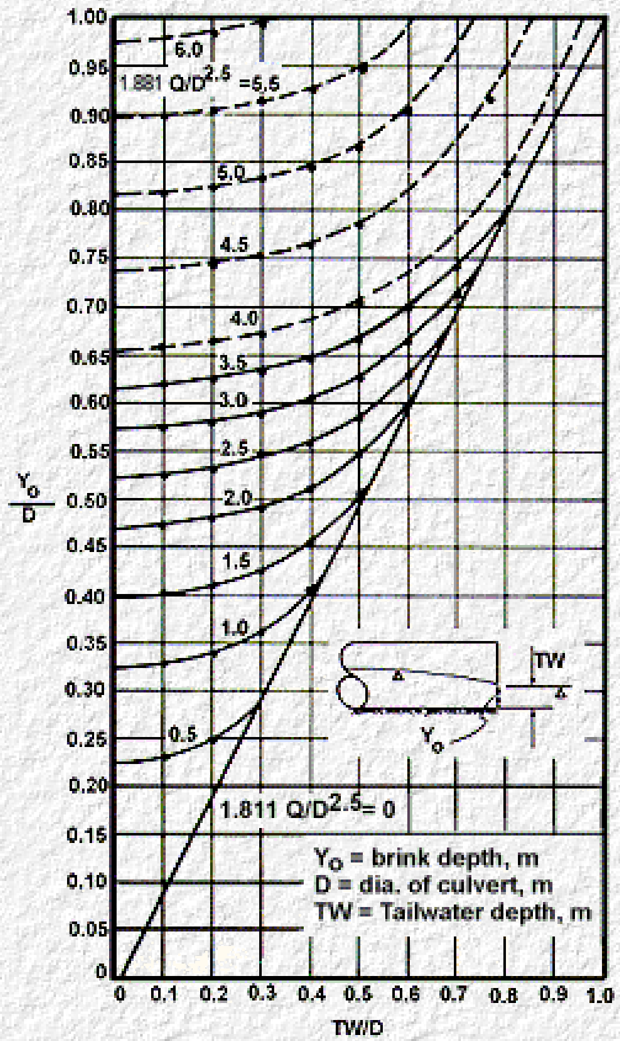


Figure 3-10. Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes (from reference 3-2)

To view the following tables click on the hyperlinks below:

- Table 3-1. Uniform Flow in Trapezoidal Channels by Manning's Formula
- Table 3-2. Uniform Flow in Circular Sections Flowing Partly Full.
- Table 3-3. Values of $BD^{3/2}$, $D^{3/2}$, and $D^{5/2}$

3-1. 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)

3-2, 7A1, 11-2. 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.

3-3. U. S. Department of the Interior, Bureau of Reclamation
Design of small Canal Structures,
1974, pp. 127-130.

Go to Chapter 4

Chapter 4 : HEC 14
Flow Transitions

Go to Chapter 5

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 4-1. The most common flow transitions are the abrupt headwall and the straight-line wingwall transitions.

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 are designed as submerged transitions and do not fall within the intended limits of open channel transitions discussed in this chapter (see reference 2A3).

Special inlet transitions are useful when the conservation of energy flow is essential because of allowable headwater considerations such as an irrigation structure in subcritical flow (see Section 4-A-2) or where it is desirable to maintain a small cross section with supercritical flow in a steep channel. (Section 4-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. See Chapter 7.

Transition designs fall into two general categories:

1. Those applicable to culverts in outlet control (subcritical flow)
2. Those applicable to culverts in inlet control (supercritical).

For design, see Section 4-A for culverts in outlet control and Section 4-B for culverts in inlet control.

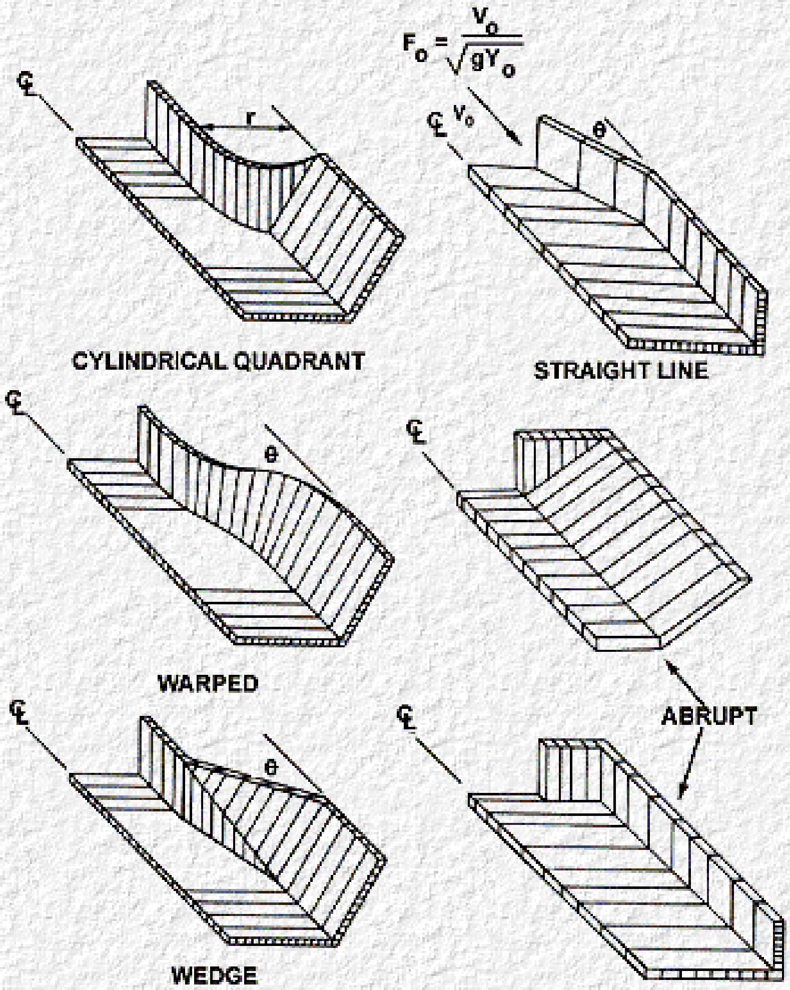


Figure 4-1. Transition Types

4-A Culverts in Outlet Control

Two types of design problems apply to culverts in outlet control:

1. abrupt expansions
2. gradual 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 4-A-1). As 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.

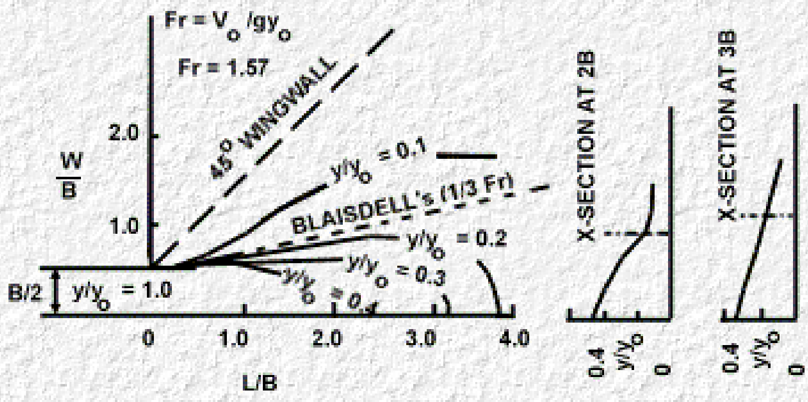


Figure 4-A-1. Dimensionless Water Surface Contours (from reference 4A1)

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 was developed by Watts (reference 4A1). Watts' experimental data has been converted to a family of curves relating the Froude number (Fr) to the average depth--brink depth ratio (y_A/y_o), Figure 4-A-4 and Figure 4-A-5, and Fr or $\sqrt{gD^5}$ to V_A/V_o, Figure 4-A-2 and Figure 4-A-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 tail-water 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 tanθ = 1/3Fr. As shown in Figure 4-A-5 flaring the wingwall more than 1/3Fr--45° for example provides unused space which is not completely filled with water.

Design Procedure

- Step 1. Determine the flow conditions at the culvert outlet: (V_o) and (y_o) see Chapter 3.

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

- Step 3. Find the optimum flare angle (θ) using tanθ = 1/3Fr. If the chosen wingwall flare (θ_w) is greater than (θ), consider reducing θ_w to θ.

- Step 4. Use Figure 4-A-4 for boxes and Figure 4-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).

Step 5. Use Figure 4-A-2 for boxes and Figure 4-A-3 for pipes to find average velocity (V_A).

Step 6. Calculate the downstream width (W_2) using:

$W_2 = W_o + 2L\tan\theta$ 4-A-1

$\tan\theta = 1/3Fr$ 4-A-2

if $\theta_w > \theta$ use θ_w in Equation 4-A-1.

Step 7. If θ was used in Equation 4-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:

1524 mm x 1524 mm RCB
$L = 60.96$ m
$S_o = 0.002$ m/m

$Q = 7.65\ m^3/s$
$1.881\ Q/BD^{3/2} = 4.83$
$d_c = 1.37$ m

Wingwall flare $\theta_w = 45°$ with 3.1 m apron

Find:

Flow Condition at end of apron - y and v.

Solution:

1. Find outlet velocity from Figure 3-9 with

$1.881\ Q/BD^{3/2} = 4.83$ and $TW/D \cong 0$
$y_o/D = 0.68$
$y_o = 0.68(1.524) = 1.036$
$v_o = Q/A = 7.65/1.036(1.524) = 4.84$ m/s

2. Find outlet Froude number

$$Fr_o = v_o / \sqrt{gy_o} = 4.84/\sqrt{9.81(1.036)} = 1.52$$

3. Find θ

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

4. Apron Length/Diameter = 3.1/1.524 = 2. Use Figure 4-A-4 for average depth, y_A.

$y_A = 0.26(1.036) = 0.269$ meters

- 5. From Figure 4-A-2 the average velocity v_A is:

  $v_A/v_o = 1.2$
  $v_A = 4.84(1.2)$
  $v_A = v_2 = 5.82$ m/s

- 6. $\theta_w > \theta$ use θ_w

  $W_2 = W_o + 2L \tan(\theta_w)$
  $W_2 = 1.524 + 2(3.048)(1.0) = 7.62$ m

- 7. θ_w was used:

  $y_2 = y_A = 0.269$ meters
  $W_A = Q/V_A\, y_A = 7.65/(5.82)(.269)$
  $W_A = 4.89$ m < 7.62 m

Alternate Solutions Using Energy Equation

- 1. Assume W_2 = full width between wingwalls at the end of the apron.

  $W_2 = Wo + 2L \tan 45^o = 7.62$ m
  $A_2 = W_2\, y_2 = 7.62\, y_2$, $V_2 = Q/A_2 = 7.641/7.62y_2 = 1.003/y_2$
  $z_o + y_o + V_o^2/2g = z_2 + y_2 + V_2^2/2g + H_f$
  $H_f = 0$ and $z_o = z_2$
  $1.036 + (4.84)^2/2(9.81) = y_2 + (1.003/y_2)^2/2(9.81)$
  $1.036 + 1.194 = y_2 + 0.0513/y_2^2$
  $2.230 = y_2 + 0.0514/y_2^2$

  $y_2 = .157$ meters, which is 41% lower than first solution.
  $V_2 = 1.004/0.157 = 6.39$ m/s which is 10% higher than the first solution.

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

  $W_2 = W_o + 2L \tan 12.41$
  $= 1.524 + 6.096(.22) = 2.87$ meters
  $A_2 = 2.87\, y_2$, $V_2 = 7.65/2.87y_2 = 2.66/y_2$
  $2.23 = y_2 + 0.360/y_2^2$

  $y_2 = 0.45$ meters, which is 68% higher than first solution
  $V_2 = 2.66/0.45 = 5.91$ m/s, 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 4-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°, (4.5H:1V or flatter), reference 4A3. 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, 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:

$$H_{LC} = C_c (V_2^2 /2g - V_1^2 /2g) \qquad \text{4-A-3}$$

and in the expansion

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

where C_c and C_e are found from Table 4-A-1.

Table 4-A-1. Transition Loss Coefficients (4A4)

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 4-A-6) The streambed elevation is equal to z.

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

$$H_{f2} \cong 0,\ V_3 = Q/W_3 y_3,\ V_4 = Q/W_4 y_4$$

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

$$z_4 + y_4 + (1 - C_e) V_4^2 /2g = z_3 + y_3 + (1 - C_e) v_3^2 /2g$$

$$z_4 - z_3 + y_4 + (1 - C_e) (Q/W_4 y_4)^2 /2g = y_3 + (1 - C_e) (Q/W_3 y_3)^2 /2g \qquad \text{4-A-5}$$

When known values are used in Equation 4-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 solved quickly by trial and error.

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

$$z_2 + y_2 + V_2^2 /2g + H_{LC} + H_{f1} = z_1 + y_1 + V_1^2 /2g$$
$$z_2 + y_2 + V_2^2 /2g + C_c (V_2^2 /2g - V_1^2 /2g) = z_1 + y_1 + V_1^2 /2g$$
$$z_2 + y_2 + (1 + C_c) V_2^2 /2g = z_1 + y_1 + (1 + C_c) V_1^2 /2g$$
$$z_2 - z_1 + y_2 + (1 + C_c) (Q/W_2 y_2)^2 /2g = y_1 + (1 + C_c) (Q/W_1 y_1)^2 /2g \quad \text{4-A-6}$$

These depths are approximate because friction loss was neglected. They should be checked by computing the water surface profile. Since the channel width is changing, the standard step method (4A5) 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 energy balance.

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

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.

$$H_f = S_f \Delta L \quad \text{4-A-8}$$

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:

$$H_L = H_{Le} (\Delta L/L_T) \text{ or } H_{LC} (\Delta L/L_T) \quad \text{4-A-9}$$

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

$$Z=z+y \quad \text{4-A-10}$$

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

$$H=z+y+V^2/2g=Z+V^2 /2g \quad \text{4-A-11}$$

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 supercritica1 flow.

$$H_k + H_f + H_L = H_x \quad \text{4-A-12}$$

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

$$Z_x = Z_k + S_o \Delta L \quad \text{4-A-13}$$

and

$$y_x = Z_x - z_x \quad \text{4-A-14}$$

With y_x known, V_x can be found by the continuity equation Q=AV. Hx is determined from Equation 4-A-11 Using y_x and V_x in Equation 4-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 4-A-8 to find H_f. Equation 4-A-9 provides H_L. If we assumed Z_x correctly, H_x (Equation

4-A-12) should be equal to $H_k + H_f + H_L$. If not, choose another Z_x and repeat the procedure. When Equation 4-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

Step 1. Find y_4, and V_4 knowing S_o, n, and approach geometry using Manning's equation or Table 3-3.

Step 2. Calculate critical depth (y_c) using $y_c = 0.467(Q/W)^{2/3}$ which is valid for rectangular channel. See Figure 3-3.
Compare y_c with y_c to insure subcritical flow.

Step 3. Choose transition type and C_c and C_e from Table 4-A-1.

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

Step 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.

Step 6. Calculate y_1 by assuming $y_2 = y_3$ in Equation 4-A-6.

Step 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 4-A-5 as reduced in step 3. Then use y_3 in Equation 4-A-6 as reduced in step 5.

Step 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 $4.5(W_1 - W_2)/2$.

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

Example Problem

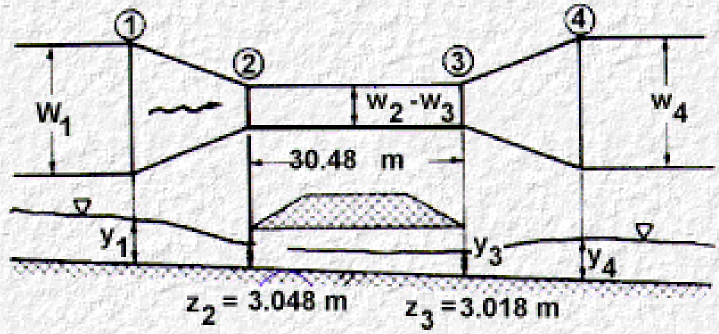
Given:

3.048 meters wide rectangular irrigation canal on a slope (S_o) of 0.001 m/m. The canal is designed to convey 8.495 m^3/s with a Manning's roughness (n) = 0.02. A rural highway will cross the canal requiring a 30.48 meter long culvert.

Find:

The culvert and transition dimensions.

Solution:



- Step 1. Assume normal depth at section 4

  $y_n = y_4 = 1.953$ m from Manning's equation and trial and error.
  $V_n = V_4 = 1.426$ m/s
  $H_n = H_4 = y_4 + V_4^2/2g = 2.057$ m

- Step 2. Determine critical depth (y_c) for section 4

  $y_c = 0.467\ (Q/W)^{2/3} = 0.467\ (8.495/3.048)^{2/3} = 0.925$ m

- Step 3. Use Straight line transition $C_e = 0.5$, $C_c = 0.3$

- Step 4. Calculate minimum w_3 using [Equation 4-A-5] and $y_c = 0.467\ (Q/W_3)^{2/3} = y_3$ also $z_4 - z_3 = S_o L_T \cong 0$ even if $L_T = 15.24$ m, $S_o L_T$ would only be 0.015 m

  $z_4 - z_3 + y_4 + (1-C_e)\ (Q/w_4\ y_4)^2/2g = y_3 + (1-C_e)\ (Q/W_3\ y_3)^2/2g$
  $0+1.953+0.5[8.495/3.048(1.953)]^2/2g = y_3+0.5(8.495^2/(W_3 y_3)^2)/2g$
  $1.953 + 0.0518 = 2.005 = y_3 + 1.839/(W_3\ y_3)^2$

  Try $W_3 = 1.219$ m (4 ft)
  $y_3 = 0.467(8.495/1.219)^{2/3} = 1.704$ m
  $y_3 + 1.839/(w_3\ y_3)^2 = 1.704+1.839/[1.219(1.704)]^2 = 2.130\ \text{m} > 2.005\text{m}$

  Try $W_3 = 1.524$ m (5 ft)
  $y_3 = 0.467\ (8.495/1.524)^{2/3} = 1.468$ m
  $y_3+1.839/(w_3 y_3)^2 = 1.468+1.839/[1.524(1.468)]^2 = 1.835\text{m} < 2.005\text{m}$

  Try $W_3 = 1.335$ m since 1.219 was high and 1.524 was low.
  $y_3 = 0.467\ (8.495/1.335)^{2/3} = 1.604$ m
  $y_3 + 1.839/(W_3\ y_3)^2 = 1.604 + 1.839/\ [1.335(1.604)]^2 = 2.005$ m
  $y_3 = 1.604$ m

- Step 5. Choose $y_3 = 1.1\ y_c$ to insure subcritical flow and recalculate W_3, $y_3 = 1.1(1.604) = 1.764$ m

  $2.005 = 1.764 + 1.839/[w_3\ (1.764)]^2$
  $0.231\ W_3^2\ (3.094) = 1.839$
  $W_3 = 1.566$ m

Use W_3 =1.524 m (5 ft)

$2.005 = y_3 + 1.839/[1.524\ y_3]^2 = y_3 + 0.792/y_3^2$
$y_3 = 1.745$ m
$V_3 = 3.194$ m/s

Step 6. Assume $y_2 = y_3$. Calculate y_1 using Equation 4-A-6.

$z_2 - z_1 + y_2 + (1+C_c)(Q/W_2\ y_2)^2 /2g = y_1 + (1+C_c)(Q/W_1\ y_1)^2 /2g$
$0+1.745+1.3[8.495/1.524(1.745)]^2/19.62=y_1+1.3(8.495/3.048y_1)^2/2g$
$1.745 + 0.676=2.421=y_1 +0.515/y_1^2$
y_1 =2.326 m which is 0.373 m above y_n

Step 7. Backwater of 0.373 m is all right since 0.61 meters of freeboard is available. If backwater was too high, return to step (4), choose another w_3 to use in $2.005 = y_3 + 1.839/(W_3\ y_3)^2$, solve for y_3 then use Equation 4-A-6 in step 6. $y_2 + 1.3(8.495/w_2\ y_2)^2 /19.62 = y_1 + 0.515/y_1^2$

For example try W_3 =1.829 meters:

$2.005 = y_3 +1.839/(1.829\ y_3)^2 = y_3 +0.550/y_3^2$
y_3 =1.843 m
$1.843 + 1.3[8.495/1.829(1.843)]^2 /19.62=y_1 +0.515/y_1^2$
$2.264 = y_1 +0.515/y_1^2$
y_1 =2.153 meters or 0.200 meters of backwater

Step 8 Transition length use 4.5:1

$L_T = 4.5(W_1 - W_2)/2 = 4.5(3.048-1.524)/2 = 3.429$ m. Use 3.353 m (11 ft)

Step 9 Calculate the water surface profile.

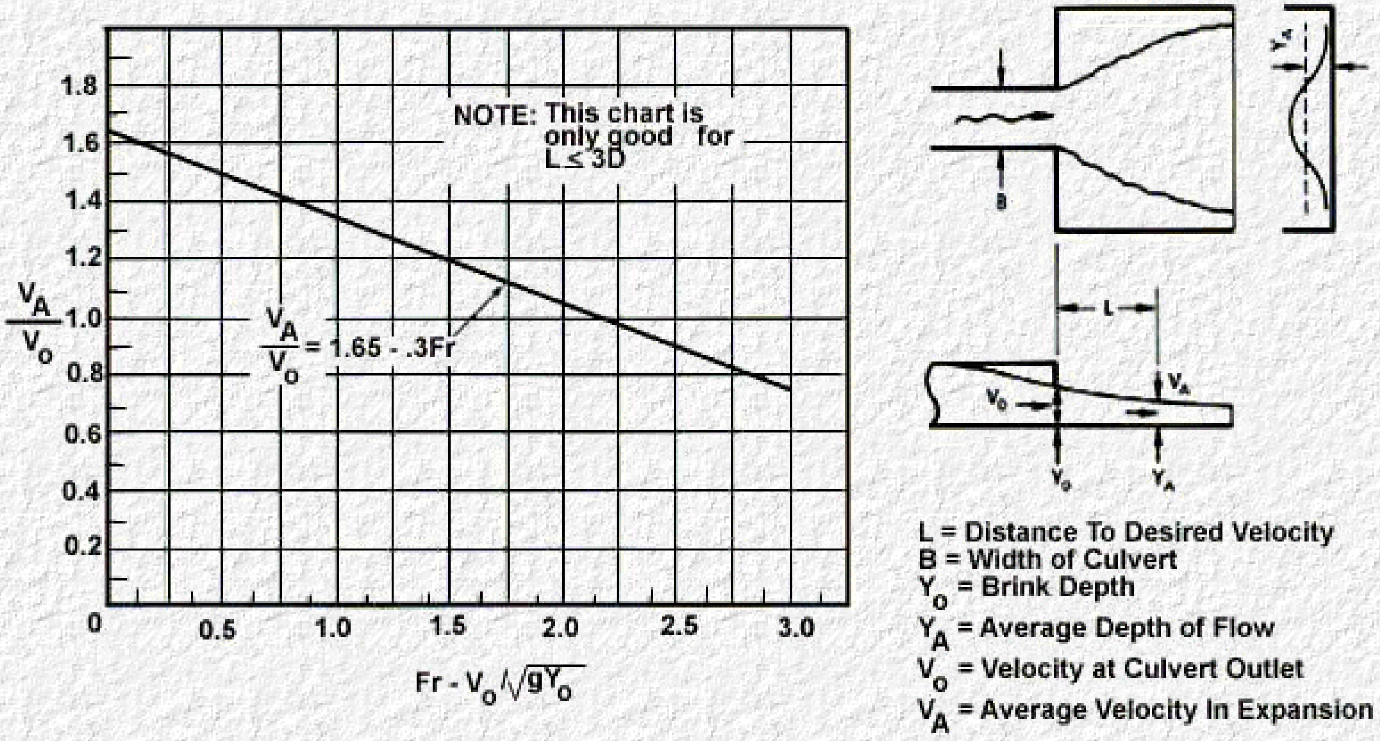


Figure 4-A-2. Average Velocity for Abrupt Expansion Below Rectangular Outlet (from reference 4A1)

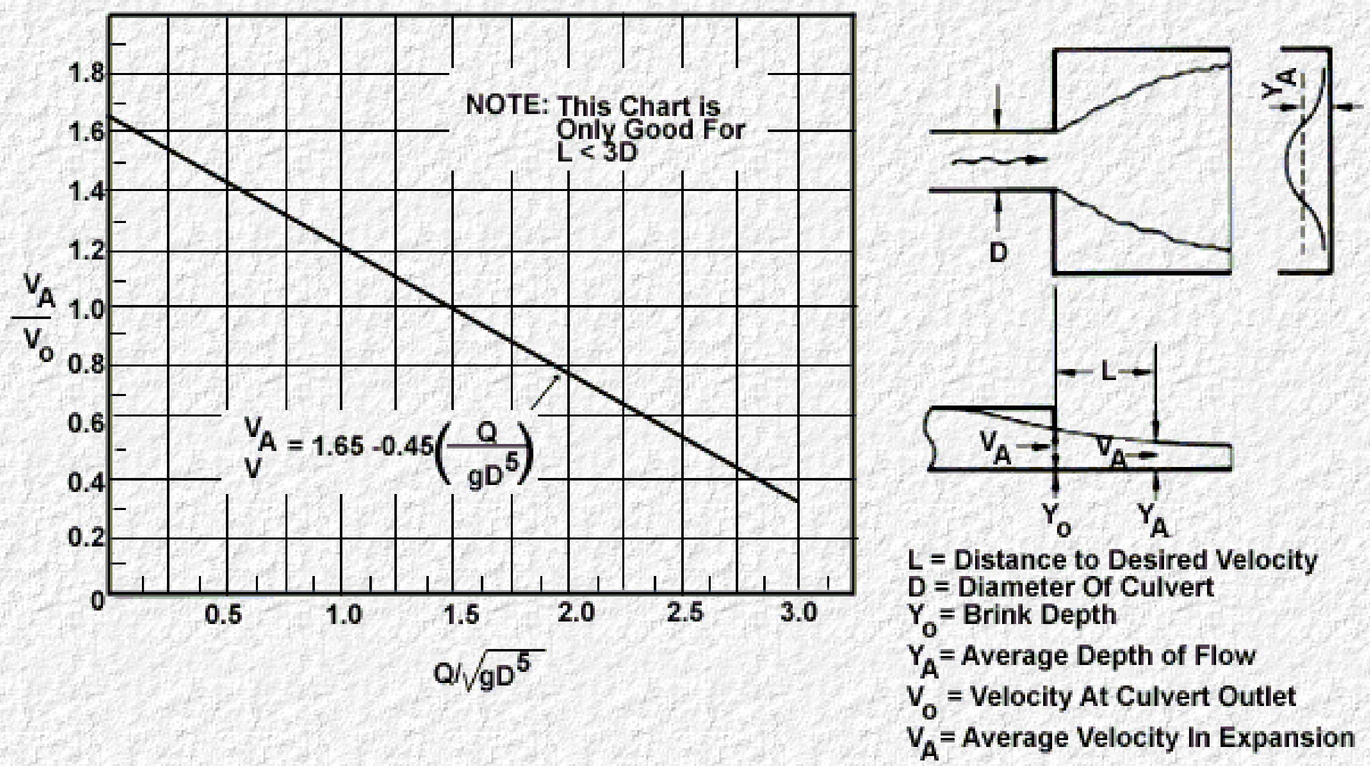


Figure 4-A-3. Average Velocity for Abrupt Expansion Below Circular Outlet (from reference 54)

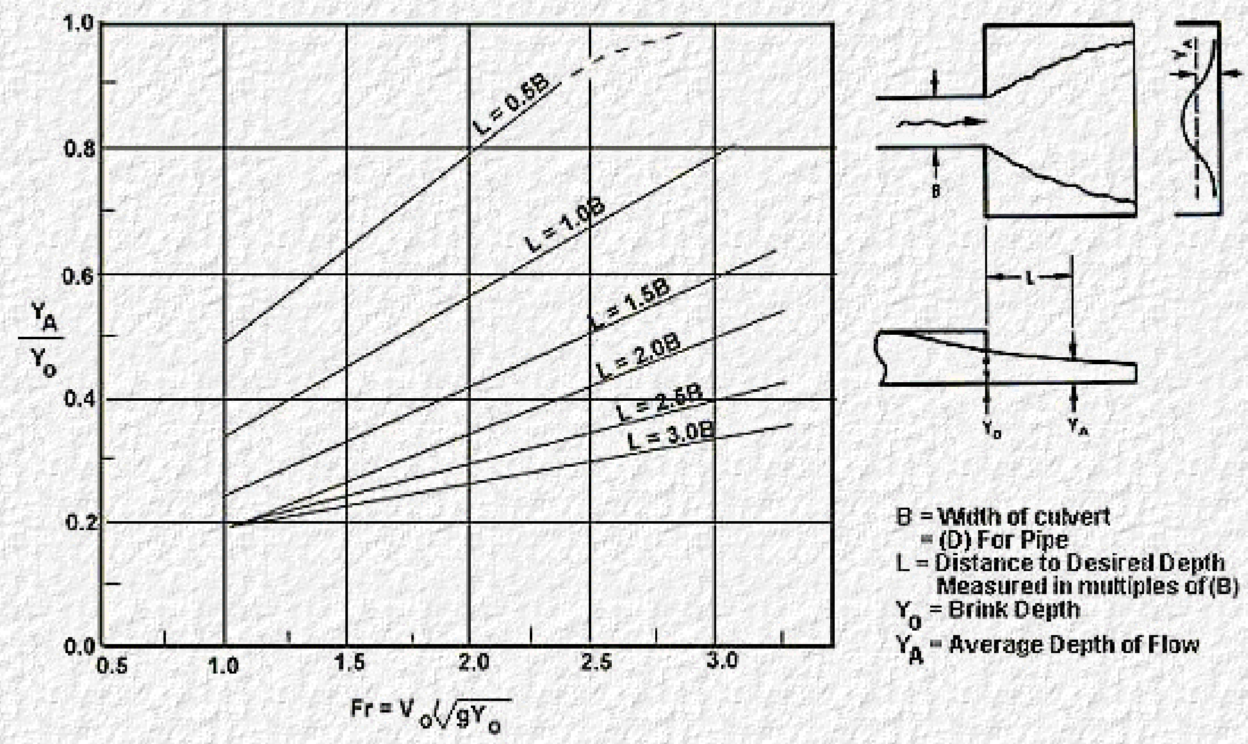


Figure 4-A-4. Average Depth for Abrupt Expansion Below Rectangular Culvert Outlet

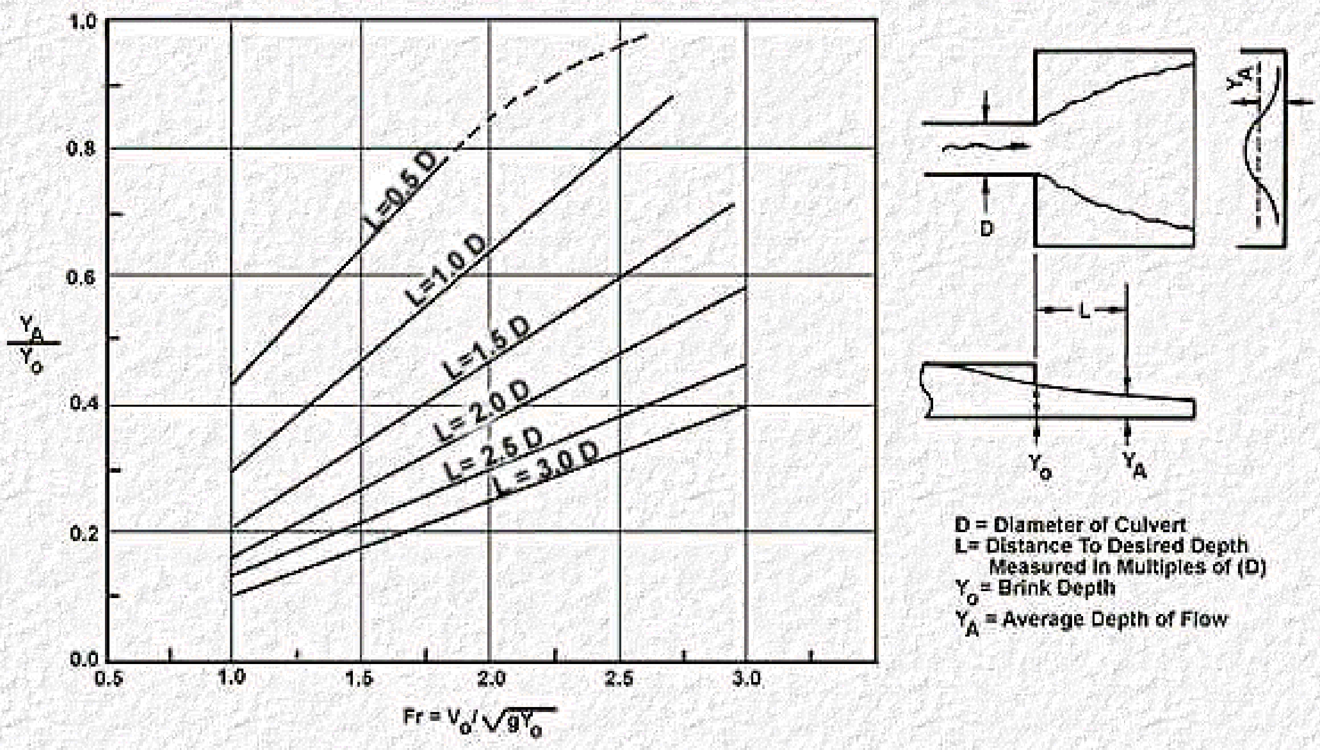


Figure 4-A-5. Average Depth for Abrupt Expansion Below Circular Culvert Outlet

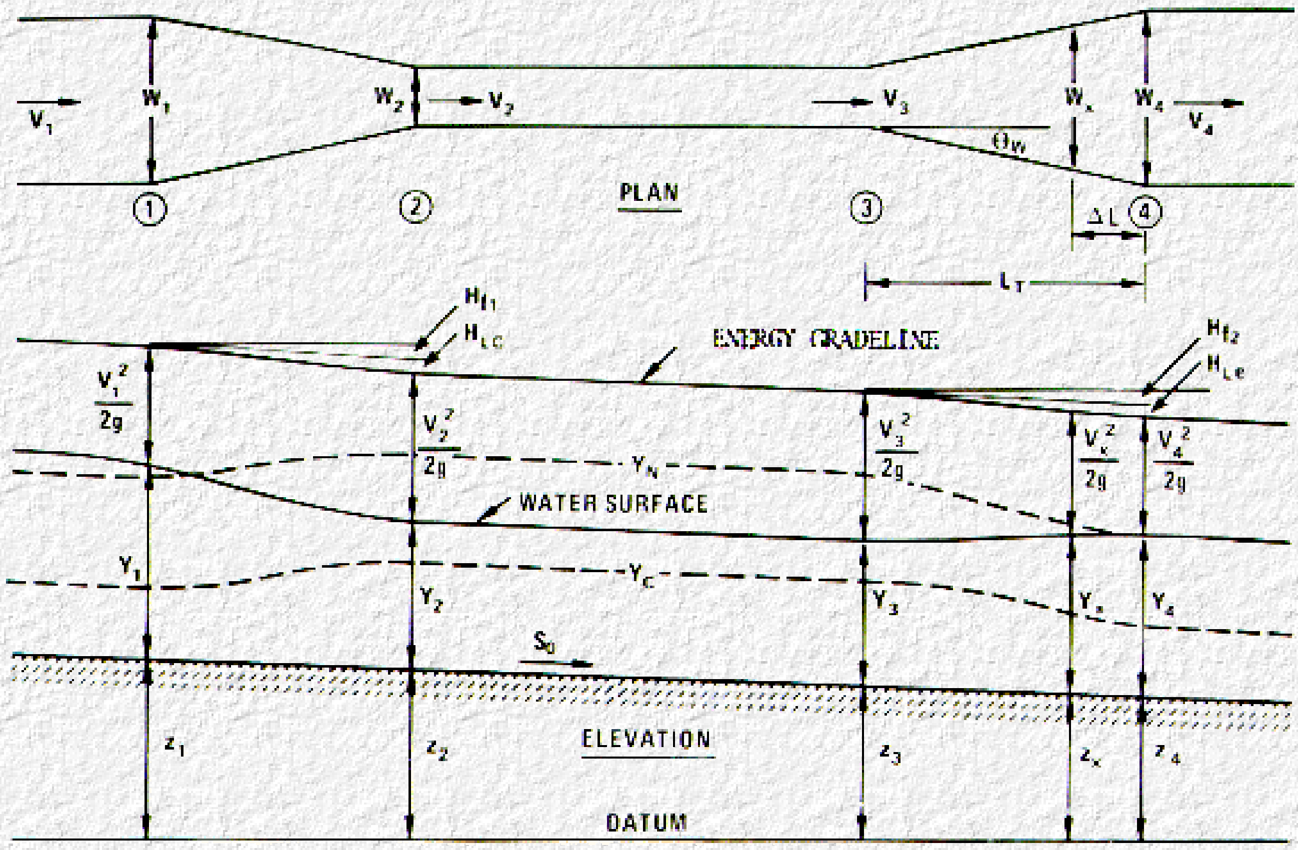


Figure 4-A-6. Definition Sketch

4-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

A smooth transition of 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 4-B-1. This design, which must be accomplished in a rectangular channel, yields a long transition. The design approach is outlined in reference 4A4 and 4A5. The length (L) is defined by (W_1 -W_2), the channel contraction, and the wall deflection angle (θ_w):

$$L=(W_1 - W_2)/2\tan\theta_w \qquad \text{4-B-1}$$

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

$$L_1 = W_1 / 2\tan \beta_1 \qquad \text{4-B-2}$$

$L_2 = W_2 / 2\tan(\beta_2 - \theta_w)$ 4-B-3

$$\tan\Theta_w = \frac{\tan\beta 1\left(\sqrt{1+8Fr_1^{\,2}\sin^2\beta_1}-3\right)}{2\tan^2\beta 1+\sqrt{1+8Fr_1^{\,2}\beta_1}-1} \qquad \text{4-B-4}$$

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

$$y_2 / y_1 = \left[\sqrt{1+8Fr_1^2\sin^2\beta_1}-1\right]/2 \qquad \text{4-B-5}$$

Then

$$Fr_2^2 = (y_1/y_2)[Fr_1^2 - (y_1/2y_2)(y_2/y_1-1)(y_2/y_1+1)^2] \qquad \text{4-B-6}$$

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 4-B-5 and Equation 4-B-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, Figure 4-B-2 and Figure 4-B-3 are a graphical solution of Equation 4-B-4, Equation 4-B-5, and Equation 4-B-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

Step 1. The flow conditions (y_n, V_n, Fr) in the approach channel should be computed using Table 3-1 or other design aids. If the channel is irregular, choose the trapezoidal section which best matches.

Step 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.

Step 3. Assume a trial culvert width W_2 .

Step 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 4-B-2 is equal to $L_1 + L_2$ from Equation 4-B-2 and Equation 4-B-3.

Select trial θ_W

. Calculate L using Equation 4-B-1.

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

c. Calculate L_1 using Equation 4-B-2.

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

e. Calculate L_2 using Equation 4-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: $y_3 = y_1 (y_2/y_1)(y_3/y_2)$.

Step 5. Compare the depth y_3 and width w_2 to see if a culvert of regular dimension (i.e., 1829 mm x 1829 mm, 2134 mm x 1829 mm) 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:

$Q = 8.495\ m^3/s$ in a 1.829 meter bottom trapezoidal channel with 2H:1V side slopes, $S_o = 0.02$ m/m and n= 0.012.

Find:

culvert size and transition dimensions.

Solution:

1. $1.49\ Qn/b^{8/3} S^{1/2} = (1.49)(8.495)(0.012)/(1.829)^{8/3} (0.02)^{1/2} = 0.2141$
   $d/b = y_n/b = 0.278$ from Table 3-1.
   $y_n = 0.508$ m $V_n = 5.852$ m/s

$$Fr = v/\sqrt{g(A/T)} = 5.852/\sqrt{9.81(1.445/3.861)} = 3.05$$

2. $(T+b)/2 = (3.861+1.829)/2 = 2.845$ m
   use $w_1 = 3.048$ m (10 ft) rectangular channel
   $1.49\ Qn/b^{8/3} S^{1/2} = (1.49)(8.495)(0.012)/3.048^{8/3} (0.02)^{1/2} = 0.0548$
   $d/b = y_n/b = 0.154$ from Table 3-1.
   $y_n = 0.469$ m, $v_n = 5.938$ m/s

$$Fr_1 = v/\sqrt{gy} = 5.938/\sqrt{g(0.469)} = 2.77$$

- 3. Assume W_2 =1.524 m. (5 ft)
- 4. Try θ_w=15° for Fr_1 =2.8

  . L=(3.048 -1.524)/2tan15°=2.844 m.
  B. β_1 =36°, y_2/y_1 =1.9, Fr_2 =1.8.
  C. L_1 =3.048/2tan 36°=2.098 m.
  D. β_2 = 58°, y_3/y_2 =1.7, Fr_3 =1
  E. L_2 =1.524/2tan(58°-15°) = 0.817 m
  F. L_1+L_2 =2.915 m > L

Try θ_w =10° for Fr =2.8

  . L=(3.048 -1.524)/2tan10°=4.322 m
  B. β_1 =31°, y_2/y_1 =1.6, Fr_2 =2.1
  C. L_1 = 3.048/2tan 31°=2.536 m
  D. β_2 =39°, y_3/y_2 =1.5, Fr_3 =1.5
  E. L_2 =1.524/2tan(39°-10°)=1.375
  F. L_1+L_2 =2.536 + 1.375 = 3.911 m < 4.322 m
  G. y_3 =0.469(1.5)(1.6)=1.126 m

Try θ_w =14° for Fr =2.8

  . L=(3.048 -1.524)/2tan 14°=3.056 m
  B. β_1 =35°, y_2/y_1 =1.8, Fr_2 =1.8
  C. L_1 =3.048/2tan 35°=2.176 m
  D. β_2 =55°, y_3/y_2 =1.6, Fr_3 =1.1
  E. L_2 =1.524/2 tan (55°-14°)=0.876 m
  F. L_1+L_2 =2.176 + 1.524 = 3.052 m ≈ L, O.K.
  G. y_3 =0.469(1.6)1.8=1.351 m

Use θ_w =14°,
  y_3 =1.351 m
  V_3 =4.126 m/s
  Fr_3 =1.1
  L= 3.048 m.

- 5. Since y_3 = 1.351 m and W_3 = 1.524 m, a 1524 mm x 1524 mm box culvert will be satisfactory.

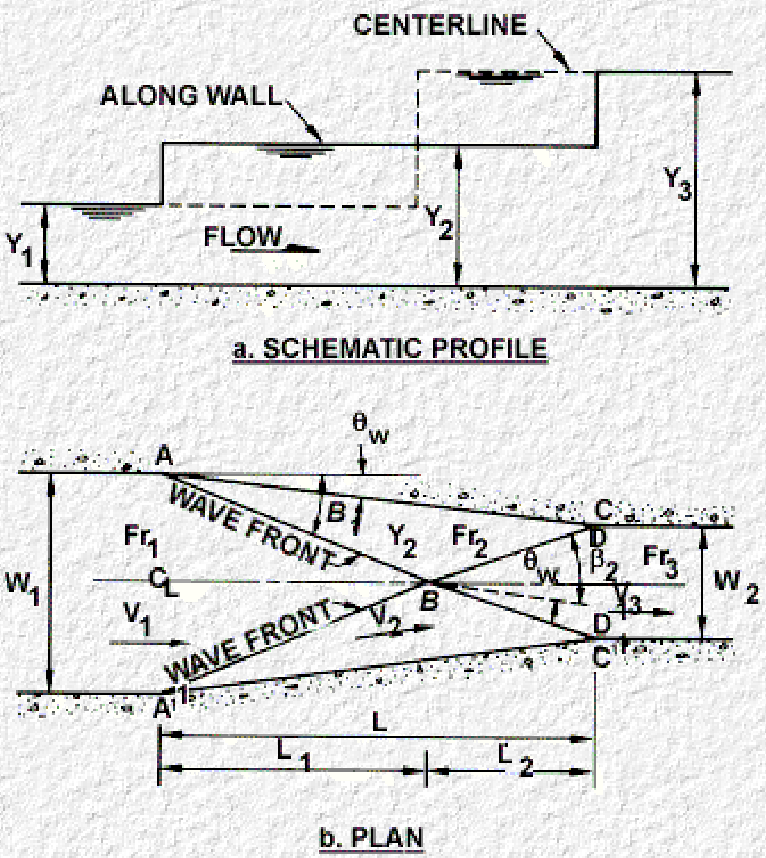


Figure 4-B-1. Supercritical Inlet Transition for Rectangular Channel (from reference 4A4)

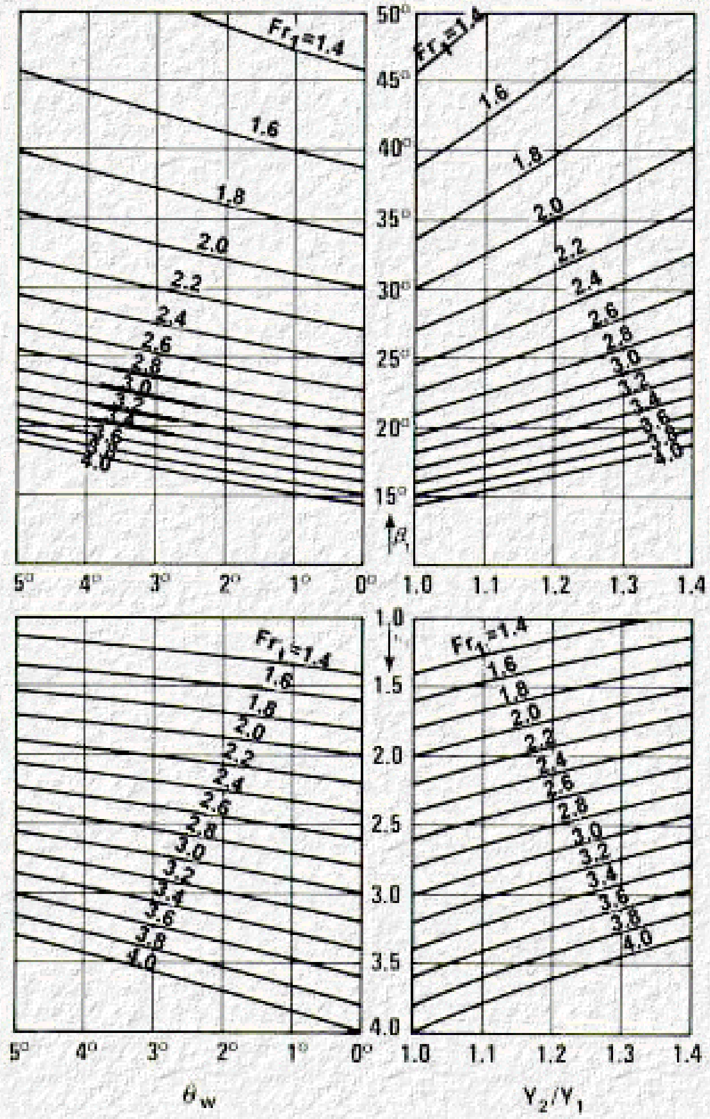


Figure 4-B-2. Supercritical Inlet Transition Design Curves for Rectangular Channels (from reference 4A4)

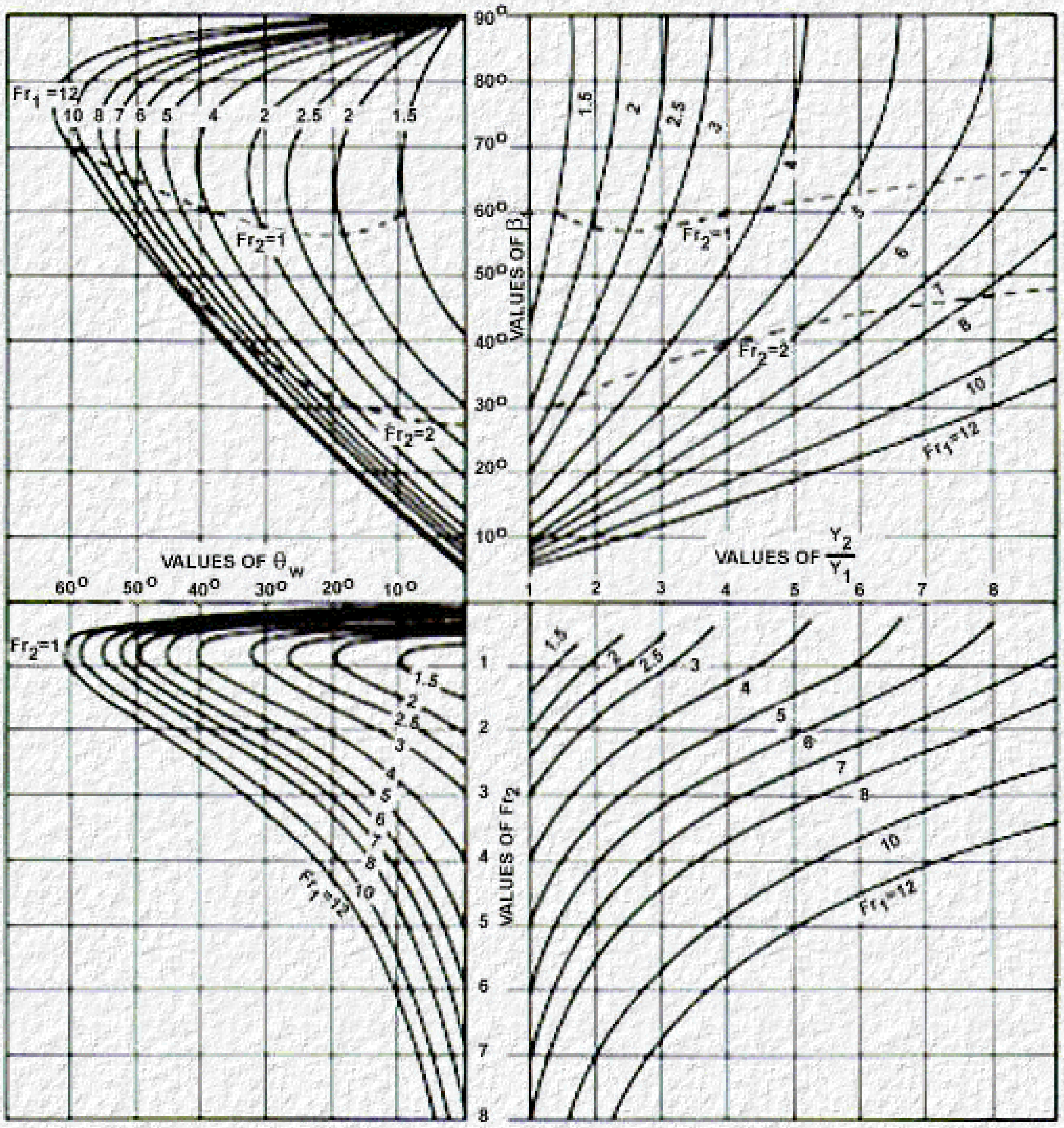


Figure 4-B-3. Supercritical Inlet Transition Design Curves for Rectangular Channels (from reference 4A5)

Supercritical Flow Expansions

Supercritical expansion design has in part been discussed in Section 4-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 S_o is less than 10%

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 4-B-4 is used to convert depth or potential energy at the culvert outlet to kinetic energy by allowing the flow to expand, drop, or both. The result is:

1. the depth decreases,
2. the velocity increases,
3. the Froude number increases.

The higher Froude number (Fr) results in a more efficient jump and a shorter basin.

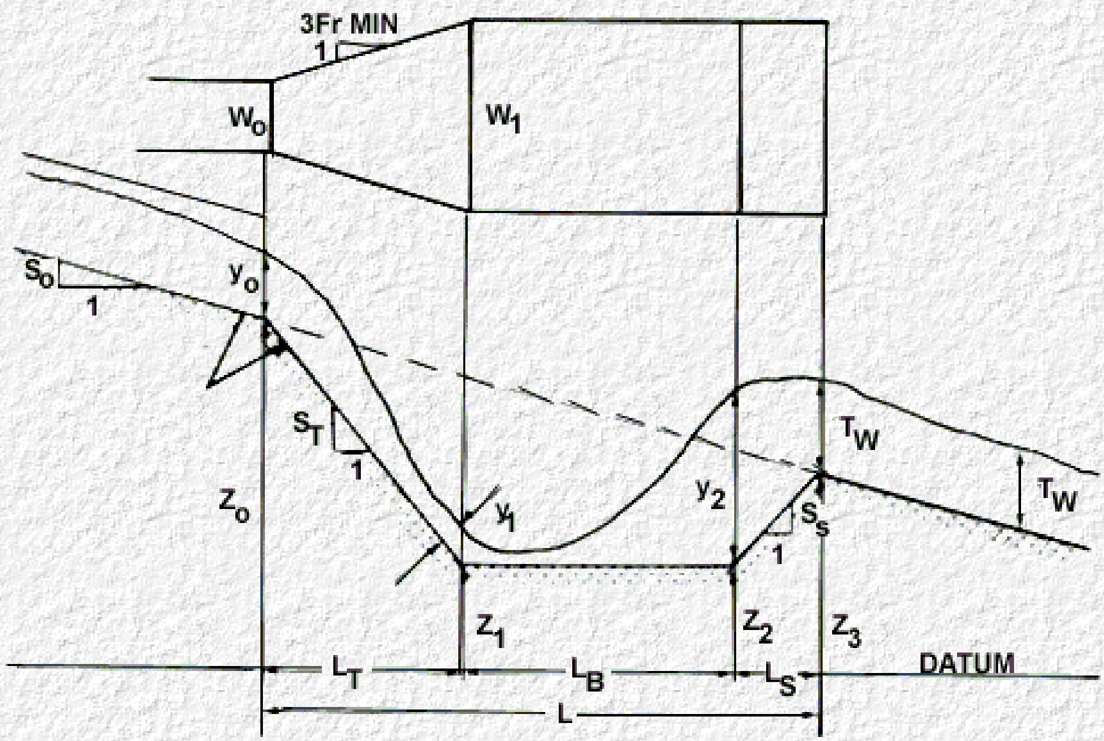


Figure 4-B-4. Definition Sketch for Basin Transition

The energy balance is written from the culvert outlet to the basin. Substituting $Q/y_1\, w_B$ for V_1 and solving for Q results in:

$$Q = y_1 W_B \left[2g(z_o - z_1 + y_o - y_1) + V_o^2\right]^{1/2} \qquad \text{4-B-7}$$

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.

$$W_B < W_o + 2L_T\sqrt{S_T^2 + 1} / 3Fr_o \qquad \text{4-B-8}$$

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:

$$y_2 = c_1 y_1\left[\sqrt{1 + 8Fr_1^2} - 1\right] / 2 \qquad \text{4-B-9}$$

Where:

$C_1 = TW/y_2$ ratio.

For USBR basins, C_1 is found on Figure 8-D-2: For the hydraulic jump, C_1=1.0 and for

SAF basin, C_1 varies with Fr (see Section 7-G 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:

$$L_T = (z_o - z_1)/S_T \quad \text{4-B-10}$$

$$L_s = (z_3 - z_2)/S_s \quad \text{4-B-11}$$

$$L_B = f(y_1, Fr_1) \quad \text{4-B-12}$$

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

Solving for z_3 yields

$$z_3 = z_o - (L_T + L_B - z_2/S_s)S_o]/(S_o/S_s + 1) \quad \text{4-B-13}$$

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

Step 1. Calculate culvert brink depth y_o using Figure 3-9 or Figure 3-10, velocity V_o, and $Fr_o = V_o / \sqrt{g y_o}$.

Step 2. Determine y_n (tailwater, TW) in downstream with the aid of Table 3-1.

Step 3. Find y_2 using Equation 4-B-9.

Step 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.

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

Choose Trial basin elevation, z_1

. Choose basin width, W_B and Basin slope S_T and S_s. A slope of 0.5 (2H:1V) or 0.33 (3H:1V) is satisfactory for either S_T or S_s.

B. Check W_B using Equation 4-B-8.

C. Calculate y_1 by trial and error using Equation 4-B-7 and calculate V_1.

D. Calculate $Fr_1 = V_1 / \sqrt{g y_1}$.

E. Determine y_2 using Equation 4-B-9 with C_1 corresponding to basin type.

F. Find z_3 using Equation 4-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 step 5 until balance is reached.

- Step 6. Calculate L_T, L_S, and L_B using Equation 4-B-10, Equation 4-B-11, and Equation 4-B-12. The horizontal distance downstream to the sill crest, L, is $L_T + L_S + L_B$.

- Step 7. Determine radius to use between culvert and transition from Figure 4-B-5.

Example Problem

Given:

3048 mm x 1829 mm RCB, Q=11.809 m^3/s, S_o =6.5%, Elevation outlet invert z_o =30.480 m, V_o =8.473 m/s, y_o = 0.457 meters Downstream Channel is a 3.048 m bottom trapezoidal channel with 2H:1V side slopes and n=0.03

Find:

Dimensions for hydraulic jump basin.

Solution:

- 1. V_o =8.473 m/s, y_o =0.457 m

$$Fr_0 = 8.473 / \sqrt{9.81(0.457)} = 4$$

- 2. $1.49Qn/b^{8/3} S^{1/2} =(1.49)11.809(0.03)/3.048^{8/3} (0.065)^{1/2} =0.1060$

  $d/b=y_n /b=0.19$ y_n =TW=0.579 m V_n =4.846 m/s

- 3. $$y_2 = C_1 y_1 \left[\sqrt{1+8Fr^2} - 1\right] / 2 = (1.0)0.457\left[\sqrt{1+8(4)^2} - 1\right] / 2 = 2.367 \text{ m}$$

- 4. Since y_2 >TW, 2.367>0.579 m, the basin is too high.

- 5. Try z_1 =28.651 m (94 ft) since y_2 -TW=1.788 m

  . W_B = 3.048 m, $S_T = S_s = 0.5$

  B. $$W_B = W_0 + 2(z_0 - z_1)\sqrt{S_T^2 + 1} / 3Fr_0 S_T$$

  $$W_B = 3.048 + 2(30.48 - 28.651)\sqrt{0.5^2 + 1} / 3(4)(0.5) = 3.730 \text{ m} > 3.048 \text{ m O.K.}$$

  C. $Q = y_1\ 3.048[2g(30.480-28.651+0.457-y_1)+ 8.473^2]^{1/2}$

  $Q = 3.048\ y_1\ [19.62(2.286-y_1) +71.792]^{1/2}$

  Try y_1 = 0.305 m (1 ft), Q=9.779 m^3/s -low

  y_1 = 0.610 m, Q=19.022 m^3/s -high

  y_1 = 0.370 m, Q=11.801 m^3/s -O.K.

  V=11.801/0.370(3.048)=10.464 m/s

  D. $$Fr_1 = 10.464 / \sqrt{9.81(0.372)} = 5.49$$

E. For C_1 = 1, $y_2 = 0.370\left[\sqrt{1+8(5.49)^2}-1\right]/2 = 2.693$ m

F. L_B =45 (0.370) = 16.65 m   Figure 6-11
$L_T =(z_o - z_1)/S_T$=(30.480 -28.651)/0.5=3.658 m
z_3 =[30.480-(3.658+16.65 -28.651/0.5).065]/(0.065/0.5+1)
z_3 =[30.480 + 2.404]/1.13=29.101 m

G. $y_2 + z_2$ =2.693 + 28.651=31.344 m
z_3 +TW=29.101 + 0.579 = 29.680 m
31.344 > 29.680 try z_1 = 27.432 m (90 ft)

Try z_1 =27.432 m (90 ft)

. W_B =3.048 m., $S_T = S_s$ =0.5

B. W_B =3.048 m. O.K.

C. Q=3.048y_1 [19.62 (3.505 - y_1)+71.792]$^{1/2}$
y_1 =0.335 m, V_1 =11.557 m/s

D. $Fr_1 = 11.557/\sqrt{g(0.335)} = 6.38$

E. $y_2 = 0.335\left[\sqrt{1+8(6.38)^2}-1\right]/2 = 2.860$ m

F. L_B =53(0.335) =17.755 m
L_T = (30.480 -27.432)/0.5=6.096 m
z_3 = [30.480 -(6.096 + 17.755 -27.432/0.5).065]/(0.065/0.5+1)
=28.757 m

G. $y_2 + z_2$ =2.860 + 27.432 = 30.292 m
z_3 +TW=28.757 + 0.579 = 29.336 m

Since 30.287 > 29.336 m Try z_1 = 25.908 m (85 ft)

. W_B =3.048 m, $S_T = S_s$ =0.5

B. W_B =3.048 m. O.K.

C. Q=3.048 y_1 [19.62 (5.029 - y_1)+71.792]$^{1/2}$
y_1 =0.302 m, v_1 =12.820 m/s

D. $Fr_1 = 12.820/\sqrt{g(0.302)} = 7.45$

E. $y_2 = 0.302\left[\sqrt{1+8(7.45)^2}-1\right]/2 = 3.034$ m

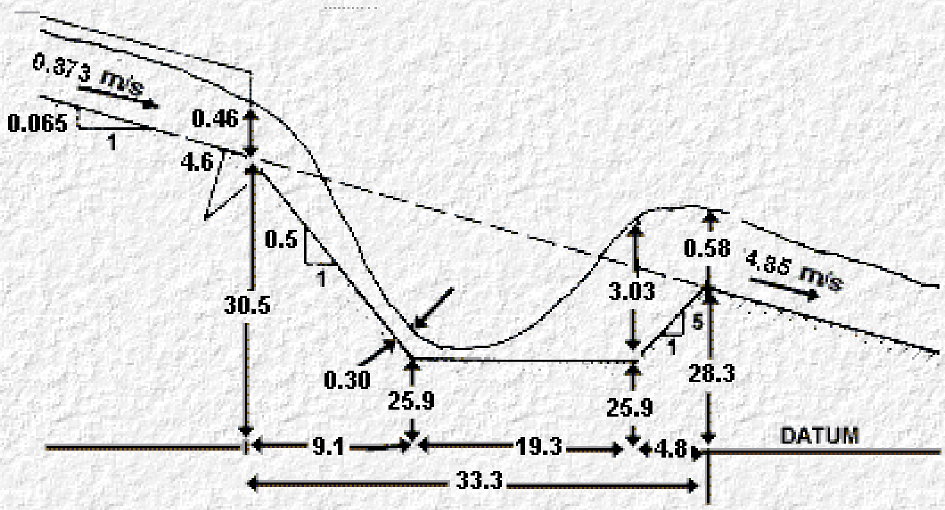
F. L_B =64(0.302) =19.328 m
L_T = (30.480 -25.908)/0.5=9.144 m
z_3 = [30.480 - (9.144 + 19.328 -25.908/0.5).065]/1.13
z_3 =28.316 m

G. $y_2 + z_2$ =3.034 + 25.908 = 28.942 m
z_3 +TW=28.316 + 0.579 = 28.895 m O.K. use z_1 =25.908 m

- 6. L_T =9.144 m,
  L_B = 19.328 m
  $L_S = (z_3 - z_2)/ S_s$ = (28.316 -25.908)/0.5 = 4.816 m

  L = 9.144 + 19.328 + 4.816 = 33.288 m

- 7. Fr_o = 4, from [Figure 4-B-5](#) y_o /r=.10



Example Problem Sketch (4-B)

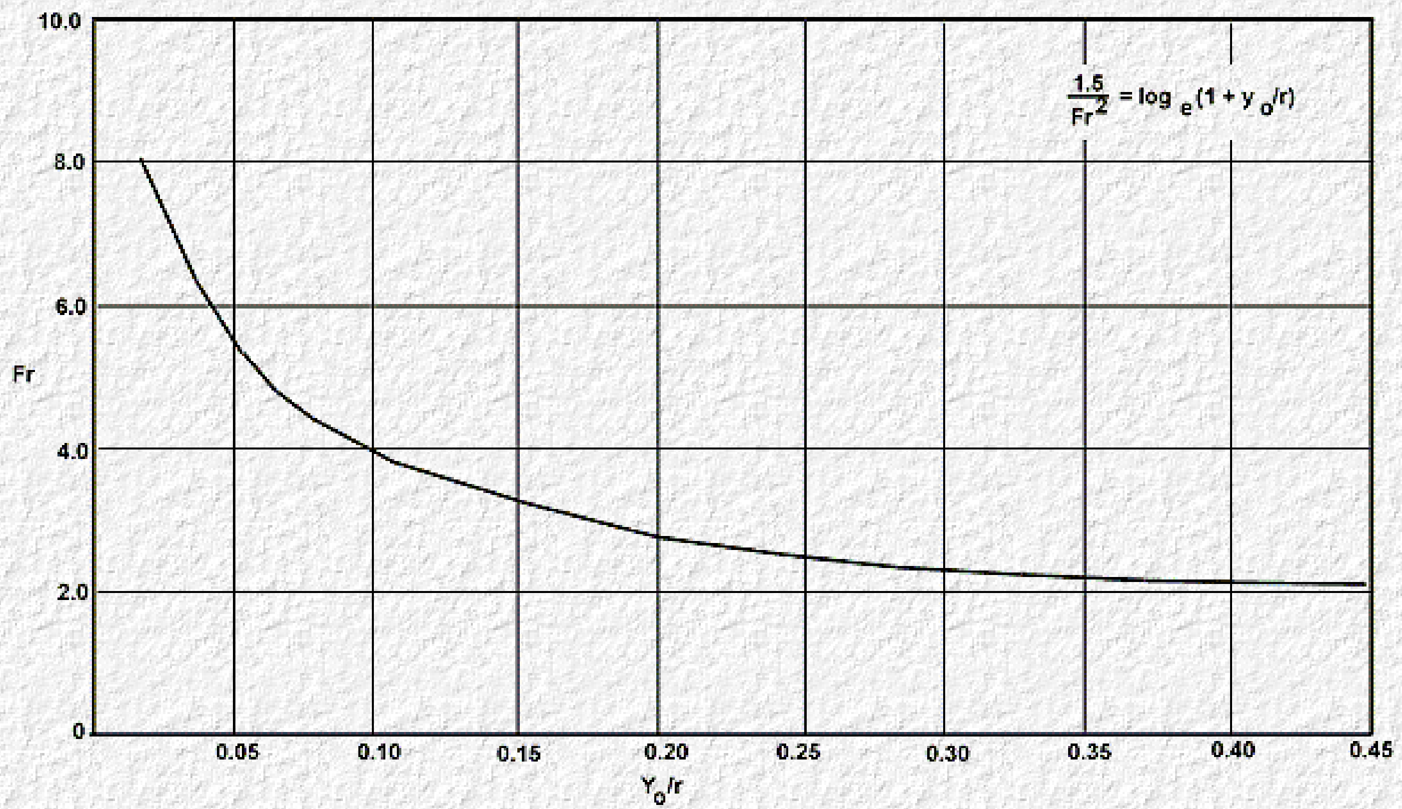


Figure 4-B-5. Fr vs. y_o/r for Transition (from reference 4B1)

4A1, 7B9. Watts, F. J.,

Hydraulics of Rigid Boundary Basins,
Ph.D. Dissertation,
Colorado State University, Fort Collins,
Colorado, August 1968

4A2. Blaisdell, F. W.,
Flow Through Diverging Open Channel Transitions At Supercritical Velocities,
U.S. Department of Agriculture,
SCS Report-SCS-TP-76, April 1949

4A3. Hinds, J.,
The Hydraulic Design of Flume And Siphon Transitions, Transactions,
ASCE, vol. 92, pp. 1423-1459, 1928

4A4. U.S. Army, Corps of Engineers,
Hydraulic Design of Flood Control Channels,
Engineering and Design Manual EM 1110-2-1601, July 1970, pp. 20-26

4A5. Ippen, A. T.,
Mechanics of Supercritical Flow, ASCE
Transactions Volume 116, pp. 268-295, 1951

4A6 Chow, Ven Te,
Open-Channel Hydraulics,
McGraw-Hill Book Company, Inc., New York, 1959.

4B. Meshgin, K., Moore, W. L.,
Design Aspects and Performance Characteristics of Radical Flow Energy Dissipators,
University of Texas at Austin, August 1970.
Research Report 116-2F

Go to Chapter 5

Chapter 5 : HEC 14

Estimating Scour at Culvert Outlets

Go to Chapter 6

Estimating erosion at culvert outlets is difficult because of the many complex factors affecting erosion. Some of these factors are the discharge, culvert shape, soil type, duration of flow, culvert slope, culvert height above the bed, and tailwater depth. In addition, the magnitude of the total scour can consist of local scour and channel degradation, the two types of erosion discussed in Section 2-B. Maintenance history, site reconnaissance and data on soils, flows and flow duration provide the best estimate of the potential scour 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, flow data, and culvert geometry. This scour prediction procedure is intended to serve together with the maintenance history and site reconnaissance information for determining energy dissipator needs.

Investigations (5-1), (5-3) indicate that the scour hole geometry varies with tailwater conditions with the maximum scour geometry occurring at tailwater depths less than half the culvert height (5-1); and that the maximum depth of scour (h_s) occurs at a location approximately 0.4 L_s downstream of the culvert outlet (5-3) where L_s is the length of scour.

Empirical equations define 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:

$$\text{Dimensionless Scour Geometry} = \frac{\alpha}{\sigma^{1/3}}\left(\frac{Q}{\sqrt{g}R_h^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta} \qquad \text{5-1}$$

Is where:

Dimensionless Scour Geometry is $\frac{h_s}{R_h}, \frac{W_s}{R_h}, \frac{L_s}{R_h}, \text{or} \frac{V_s}{R_h^3}$

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

R_h is the hydraulic radius

Q is the discharge

g is the acceleration of gravity

t is the time in minutes

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

σ is the material standard deviation

The values of the coefficients α, β, and θ in Equation 5-1 are presented in Table 5-1.

Gradation

The bed-material grain-size distribution is determined by performing a sieve analysis (ASTM DA22-63) . The standard deviation (σ) is computed as:

$$\sigma = \left(\frac{d_{84}}{d_{16}}\right)^{1/2} \qquad 5\text{-}2$$

where the values of d_{84} and d_{16} are extracted from the grain sizedistribution. If $\sigma < 1.5$, the material is considered to be uniform; if $\sigma > 1.5$, the material is classified as graded.

Cohesive Soils

If the soil is cohesive in nature, Equation 5-1 should not be used to determine the scour hole dimensions. Since Equation 5-1 does not include soil characteristics, it should only be used for cohesionless soils. Shear number expressions, which 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 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. The shear number expressions for circular culverts are:

$$\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{or } \frac{V_s}{D^3} = \alpha\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta} \qquad 5\text{-}3$$

and for other shaped culverts:

$$\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{ or } \frac{V_s}{{y_e}^3} = \alpha_e \left(\frac{\rho V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{t_o}\right)^{\theta} \quad \text{5-4}$$

where:

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

$\frac{\rho V^2}{\tau_c}$ = modified shear number

V =outlet mean velocity

τ_c= critical tractive shear stress

ρ = fluid density

$\alpha_e = \frac{\alpha}{(0.63)}$ for h_s , W_s , and L_s

$\alpha_e = \frac{\alpha}{(0.63)^3}$ for V_s

A = Cross-sectional area of flow
D= Culvert diameter

The values of the coefficients α, β, θ, and α_e are presented in Table 5-1. The critical tractive shear stress is defined in Equation 5-5.

$$\tau_c = 0.001\ (S_v + 8618) \tan\ (30 + 1.73\ PI) \quad \text{5-5}$$

where:

S_v = the saturated shear strength in N/m^2
PI = the Plasticity Index from the Atterberg Limits.

It is recommended that Equation 5-3 and Equation 5-4 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 Equation 5-1, Equation 5-3, and Equation 5-4. 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 5-1 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 (5-6) 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.

Drop Height

The scour hole dimensions will vary with the distance the culvert invert extends above the bed. The scour hole shape becomes deeper, wider, and shorter, as the culvert invert height is increased (5-10). The coefficients are derived from tests where the pipe invert is adjacent to the bed In order to

$$[\text{Dimensionless Scour Geometry}] = C_h \frac{\alpha}{\sigma^{1/3}} \left(\frac{Q}{\sqrt{g} R_h^{5/2}} \right)^{\beta} \left(\frac{t}{t_o} \right)^{\theta} \qquad 5\text{-}6$$

compensate of an elevated culvert invert, Equation 5-1 can be modified to where C_h, expressed in pipe diameters, is a coefficient for adjusting the compound scour hole geometry. The values of C_h are presented in Table 5-3.

Slope

The scour hole dimensions will vary with culvert slope. The scour hole becomes deeper, wider, and longer as the slope is increased (5-11). The coefficients presented are derived from tests where the pipe invert is adjacent to the bed. In order to compensate for a sloped culvert, Equation 5-1 can be modified to:

$$[\text{Dimensionless Scour Geometry}] = C_s \frac{\alpha}{\sigma^{1/3}} \left(\frac{Q}{\sqrt{g} R_h^{5/2}} \right)^{\beta} \left(\frac{t}{t_o} \right)^{\theta} \qquad 5\text{-}7$$

Where C_s is a coefficient adjusting scour hole geometry. The values of C_s are present in Table 5-4.

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 (5-1), and Colorado State University (5-5) through (5-11).

Design Procedure for Cohesionless Materials

- Step 1. Determine the magnitude and duration of the peak discharge. Express the discharge in m^3 /s and the duration in minutes.

- Step 2. Compute the modified discharge at the peak discharge.

  The modified discharge intensity (D.I.*) is:

$$D.I.^* = \frac{Q}{\sqrt{g} R_h^{5/2}} \text{ for all culvert shapes}$$

- Step 3. Determine scour coefficients from Table 5-1.

- Step 4. Obtain a soil sample at the proposed culvert location, conduct a sieve analysis, and determine the material standard deviation.

- Step 5. Determine the coefficients for culvert drop height and slope if appropriate.

- Step 6. Compute the scour hole dimensions from:

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

Design Procedure for Other Cohesive Materials with PI From 5 to 16

- Step 1. Determine the magnitude and duration of the peak discharge. Express the discharge in m^3/s and the duration in minutes.

- Step 2. Compute the culvert outlet velocity in m/sec.

- Step 3. Obtain a soil sample at the proposed culvert location.

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

  b. Saturate a sample and perform an unconfined compressive test (ASTM D211-66-76) to determine the saturated shear stress, S_v, in newtons per square meter.

- Step 4. Compute the critical tractive shear strength, τ_c, from Equation 5-1a.

- Step 5. Compute the modified shear number $\frac{\rho V^2}{\tau_c}$.

- Step 6. Determine scour coefficients from Table 5-2.

- Step 7. Compute the desired scour hole dimensions for a circular culvert from:

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{or } \frac{V_s}{D^3}\right] = \alpha_e C_h C_s \left(\frac{\rho V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{316}\right)^{\theta}$$

for noncircular culverts:

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

where:

$$Y_e = \left[\frac{A}{2}\right]^{1/2}$$

Cohesionless Material Example Problem

Determine the scour geometry-maximum depth, width, length and volume of scour-for a proposed circular 762 mm C.M.P. discharging an estimated 1.416 m^3/s when flowing full. The downstream channel is composed of graded gravel material with σ =2.10. The culvert barrel is horizontal and adjacent to the bed.

1. The duration of the peak discharge of 1.416 m^3/s is not known. Therefore, a peak flow duration of 30 minutes will be estimated.

The hydraulic radius (R_h) of a circular, 762 mm CMP is:

$$R_h = \frac{\text{Area}}{\text{Wetted Perimeter}} = \frac{\pi r^2}{2\pi r} = \frac{3.14159(0.1452)}{2(3.14159)0.381} = 0.1906 \text{ m}$$

3. The circular, 762 mm C.M.P. at 1.42 m^3/s will have a discharge intensity of

$$D.I.^* = \frac{1.416}{\sqrt{g}(0.1906)^{5/2}} = \frac{1.416}{(3.132)(0.1906)^{5/2}} = 28.50$$

4. The coefficients of scour obtained from Table 5-1, Table 5-2, and Table 5-3 are:

	α	β	θ
Depth of scour	2.27	0.39	0.06
Width of scour	6.94	0.53	0.08
Length of scour	17.10	0.47	0.10
Volume of scour	127.08	1.24	0.18

$C_h = 1.0$

$C_s = 1.0$

5. Scour hole dimensions:

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

Depth: $h_s=1.0(1.0)\ \frac{2.27}{1.28}\ (28.50)^{0.39}(0.095)^{0.06}\ (0.1906)$; $h_s=1.08$ m

Width: $W_s=1.0(1.0)\ \frac{6.94}{1.28}\ (28.50)^{0.53}\ (0.095)^{0.08}\ (0.1906)$; $W_s=5.05$ m

Length: $L_s=1.0(1.0)\ \frac{17.10}{1.28}\ (28.50)^{0.47}\ (0.095)^{0.10}\ (0.1906)$; $L_s=9.72$ m

Volume: $V_s = 1.0(1.0)\ \frac{127.08}{1.28}\ (28.50)^{1.24}\ (0.095)^{0.18}\ (0.1906)^3$; $V_s = 28.7\ m^3$

6 The location of the maximum scour.

$0.4(L_s) = .4\ (9.72) = 3.9$ m downstream of the culvert outlet.

Cohesionless Material Example Problem

Determine the scour geometry-maximum depth, width, length and volume of scour for a proposed circular 457 mm CMP, discharging at 0.764 m^3/s. The downstream channel is composed of graded sand material with a standard deviation of 1.87. The existing outlet pipe has a barrel slope of 2 percent and is suspended .9144 m above the bed because of channel degradation.

The general equation for computing the scour hole dimension is:

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

1. The duration of the peak discharge of 0.764 m^3/s is not known. Therefore, a peak flow duration of 30 minutes will be estimated.

2. The hydraulic radius (R_h) for a circular, 457 mm CMP is:

$$R_h = \frac{3.14159(0.05226)}{2(3.14159)0.2285} = 0.1143\ \text{m}$$

3. The circular, 457 mm CMP at 0.764 m^3/s will have a modified discharge of:

$$D.I.^* = \frac{0.764}{\sqrt{9.81}\ (0.1143)^{5/2}} = \frac{0.764}{(3.133)(0.1143)^{5/2}} = 55.2$$

$$H_d = \frac{0.9144}{0.4572} = 2$$

4. The Coefficients of scour obtained from Table 5-1, Table 5-3, and Table 5-4 are:

	α	β	θ	C_s	C_h
Depth of scour	2.27	0.39	0.06	1.03	1.26
Width of scour	6.94	0.53	0.08	1.28	1.54
Length of scour	17.10	0.47	0.10	1.17	0.73
Volume of scour	127.08	1.24	0.18	1.30	1.47

5. Scour hole dimensions are:

$$\text{Depth: } C_h C_s \frac{\alpha}{\sigma^{1/3}} \left(\frac{Q}{\sqrt{g} R_h^{5/2}} \right)^{\beta} \left(\frac{t}{316} \right)^{\theta} R_h$$

$$= 1.26(1.03)\frac{2.27}{1.23}(55.2)^{0.39}(0.095)^{0.06}(0.1142) = 1.14\text{m}$$

Length: $L_s = 0.73(1.17)\ \frac{17.10}{1.28}\ (55.2)^{0.47}(0.095)^{0.10}(0.1142) = 6.78\text{m}$

Width: $W_s = 1.54(1.28)\ \frac{6.94}{1.28}\ (55.2)^{0.53}(0.095)^{0.08}(0.1142) = 8.47\text{ m}$

Volume: $V_s = 1.47(1.30)\ \frac{127.08}{1.28}\ (55.2)^{1.24}(0.095)^{0.18}(0.1142)^3 = 26.74\text{ m}^3$

Cohesive Material Example Problem

Determine the scour geometry-maximum depth, width, length and volume of scour for a proposed circular 610 mm CMP, discharging at an estimated 1.133 m^3/s. The downstream channel is composed of graded sandy-clay material. The culvert barrel is horizontal and adjacent to the bed.

1. The duration of the peak discharge of 1.133 m^3/s is not known. Therefore, a peak flow duration of 30 minutes will be estimated.

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

$$V = \frac{Q}{A} = \frac{1.133}{0.292} = 3.88\text{m/s}$$

b-e. The sandy-clay material was tested and found to have a plasticity index (PI) of 12 and a saturated shear strength (S_v) of 23970 N/m^2

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

τ_c = 0.001 (23970+8629)tan [30+1.73(12)]

0.001(32599) tan(50.76)=39.9 N/m^2

f. The modified shear number $S_{nm} = \frac{\rho V^2}{\tau_c}$

$$S_{nm} = \frac{1000(3.88)^2}{39.9} = 377.3$$

3. The experimental coefficients α, β, and θ from Table 5-1, Table 5-2, and Table 5-4 are:

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

$C_h = 1.0$

$C_s = 1.0$

4. The scour hole dimensions are:

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

$=1.0(1.0)\ 0.86(377.3)^{.18}\ (0.09)^{.10}$; $h_s=1.97 \times .61 = 1.20$ m

$\frac{W_s}{D} =1.0(1.0)\ 3.55(377.3)^{.17}\ (0.09)^{.07}$; $h_s=8.23 \times .61 = 5.02$ m

$\frac{L_s}{D} =1.0(1.0)\ 2.82(377.3)^{.33}\ (0.09)^{.09}$; $W_s=16.09 \times .61 = 9.81$ m

$\frac{V_s}{D^3} =1.0(1.0)\ 0.62(377.3)^{.93}\ (0.09)^{.23}$; $V_s=88.8 \times 0.227 =20.16\ m^3$

5. Location of maximum scour depth is:

$0.4\ L_s =0.4(9.81)=3.92$ m downstream of culvert outlet.

Table 5-1. Coefficients for Culvert Outlet Scour-Cohesionless Materials

	α	β	θ
Depth, h_s	2.27	0.39	0.06
Width, W_s	6.94	0.53	0.08
Length, L_s	17.10	0.47	0.10
Volume, V_s	127.08	1.24	0.18

For All Shaped Culverts. Cohesionless Material

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

Table 5-2. Coefficients for Culvert Outlet Scour-Cohesive Sandy Material

	α	β	θ	α_e
Depth, h_s	0.86	0.18	0.10	1.37
Width, W_s	3.55	0.17	0.07	5.63
Length, L_s	2.82	0.33	0.09	4.48
Volume, V_s	0.62	0.93	0.23	2.48

For Circular Culverts. Cohesive Sandy Clay Material

$$\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{or } \frac{V_s}{D^3} = C_h C_s \alpha \left(\frac{\rho V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{316}\right)^{\theta}$$

For Other Culvert Shapes. Cohesive Sandy Clay with PI=5-16

$$\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{or } \frac{V_s}{y_e{}^3} = C_h C_s \alpha \left(\frac{\rho V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{316}\right)^{\theta}$$

Table 5-3. Coefficients (C_h) for Outlets above the Bed**

*H_d	Depth	Width	Length	Volume
0	1.00	1.00	1.00	1.00
1	1.22	1.51	0.73	1.28
2	1.26	1.54	0.73	1.47
4	1.34	1.66	0.73	1.55
*Height above bed in pipe diameters **Coefficient derived for sand bed materials				

Table 5-4. Coefficients (C_s) for Culvert Slope

Slope%	Depth	Width	Length	Volume
0	1.00	1.00	1.00	1.00
2	1.03	1.28	1.17	1.30
5	1.08	1.28	1.17	1.30
$\geq$ 7	1.12	1.28	1.17	1.30

5-1. Bohan, J. P.,
"Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets."
U.S. Army Engineer Waterways Experiment Station,
Vicksburg, Mississippi.

5-2 . Dunn, I. S.,
Tractive Resistance of Cohesive Channels.

Journal of the Soil Mechanics and Foundations,
Vol. 85, No. SM3, June 1959.

5-3. Fletcher, B. P. and Grace, J. L.,
Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets."
Misc. Paper H-72-5, U.S. Army Waterways Experiment Station, 1972
Vicksburg, Mississippi.

5-4. Lamb, T. W. and Whitman, R. V.,
Soil Mechanics.
John Wiley and Sons, Inc., 1969.

5-5. McGowan, M. J.,
"Scour in Uniform Sand at Culvert Outlets."
Master Thesis Working Papers,
Colorado State University, Fort Collins,
Colorado, April 1980.

5-6. Mendoza, C.,
"Headwall Influence on Scour at Culvert Outlets."
Masters Thesis,
Colorado State University, Fort Collins,
Colorado, April 1980.

5-7. Opie, T. R.,
"Scour at Culvert Outlets."
Master Thesis,
Colorado State University, Fort Collins,
Colorado, December 1967.

5-8. Ruff, J. F. and Abt, S. R., "
Scour at Culvert Outlets in Cohesive Bed Materials."
Prepared for the U.S. Department of Transportation, Federal Highway Administration,
CER79-80JFR-SRA61, May 1980.

5-9. Shaikh, A.,
"Scour in Uniform and Graded Gravel at Culvert Outlets."
Master Thesis,
Colorado State University, Fort Collins,
Colorado, May 1980.

5-10. Abt, S.R., Donnell, C.A., Ruff, J.F., and Doehring, F.K.,
Culvert Slope and Shape Effects on Outlet Scour.
Transportation Research Record 1017, pp. 24-30, 1985.

5-10. Doehring, F. K.
The influence of Drop Height on Outlet Scour
Master Thesis, Colorado State University, Fort Collins, Colorado, December, 1987.

5-11. AASHTO. Model Drainage Manual.
Task Force on Hydrology and Hydraulics
Chapter 11 (1991)

5-12. Abt, S.R., Ruff, J.F., and Doehring, F.K.,
Culvert Slope Effects on Outlet Scour.
Journal of Hydraulic Engineering, ASCE, 111(10), pp. 1363-1367.October 1985b

5-13. Abt, S.R., Donnell, C.A., Ruff, J.F., and Doehring, F.K.,
Influence of Culvert Shape on Outlet Scour.
Journal of Hydraulic Engineering, ASCE, (113)(3), p. 393-400, March 1987.

5-14. Blaisdell, Fred W., and Anderson, C. L.
A Comprehensive Generalized Study of Scour at Cantilevered Pipe Outlets, I
IAHR, Journal of Hydraulic Resistance, 26(4), p. 357-376. (1988a)

5-15. Blaisdell, Fred W., and Anderson, C. L.
A Comprehensive Generalized Study of Scour at Cantilevered Pipe Outlets, II
IAHR, Journal of Hydraulic Resistance, 26(5), p. 509-524. (1988b)

5-16 . Corry, M. L., Thompson, P. L., Wyatts, F. J., Jones, J. S., and Richards, D.C.
Hydraulic Design of Energy Dissipators for Culverts and Channels.
Chapter 5, Hydraulic Engineering Circular No. 14,
Office of Engineering, FHWA Washington D.C. December 1975.

5-17. Doehring F. K. and Abt, S. R.
Drop Height Influence on Outlet Scour,
Journal of Hydraulic Engineering, ASCE, 120(12), December 1994.

5-18. Donnell, C. A. and Abt, S. R.
Culvert Shape Effects on Localized Scour,
Hydrographic Engineering Publications CER82-83CAD-SRA42.
Colorado State University, Fort Collins, Co. 1983.

5-19. Laursen E. M.
Observations on the Nature of Scour
Proceedings of the fifth Hyd. Conference Bulletin 34,
University of Iowa City, IA, P. 179-197. 1952.

5-20. Robinson, A. R.
"Model Study of Scour From Cantilevered Outlets
Transitions, ASAE 14, p. 571-581. 1971.

5-21. Rouse H.
Experiments on the Mechanics of Sediment Suspension."
Proceedings of fifth International Congress For Applied Mechanics.
John Wiley and Sons, Inc. New York, N.Y. 1938.

5-22. Ruff, J. F., Abt, S. R., Mendoza, C., Shaikh, A. and Kloberdanz, R. (1982).
Scour at Culvert Outlets in Mixed Bed Materials,
Rep. No. FHWA/RD-82/011,
Offices of Research and Development, Washington D.C. 20590, September 1982.

Go to Chapter 6

Chapter 6 : HEC 14
Hydraulic Jump

Go to Chapter 7, Part I

6-1 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 (6-1) has related the jump form and flow characteristics to the Froude number, Figure 6-1. 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.

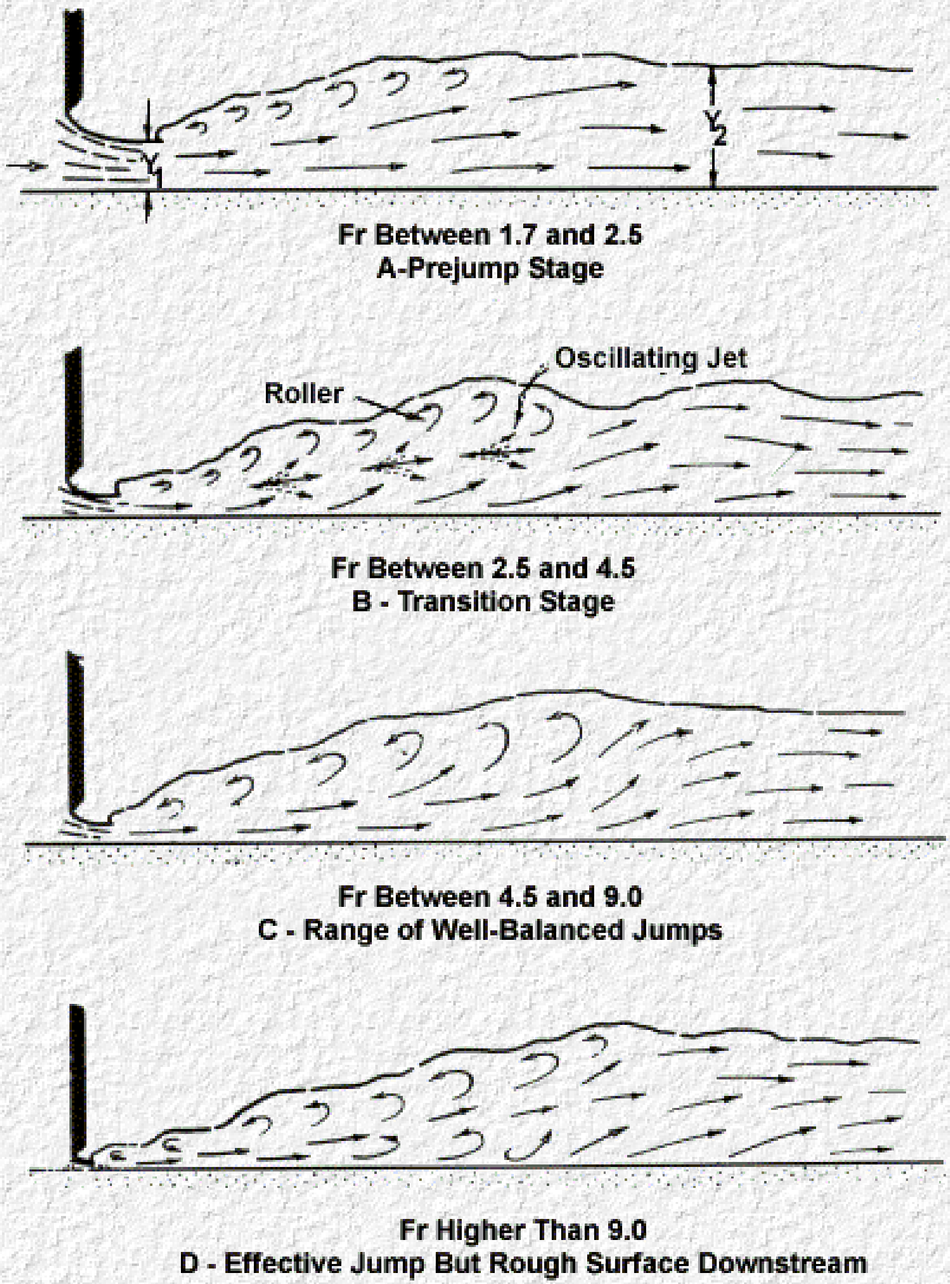


Figure 6-1. Jump Forms Related to Fr (from reference 6-1)

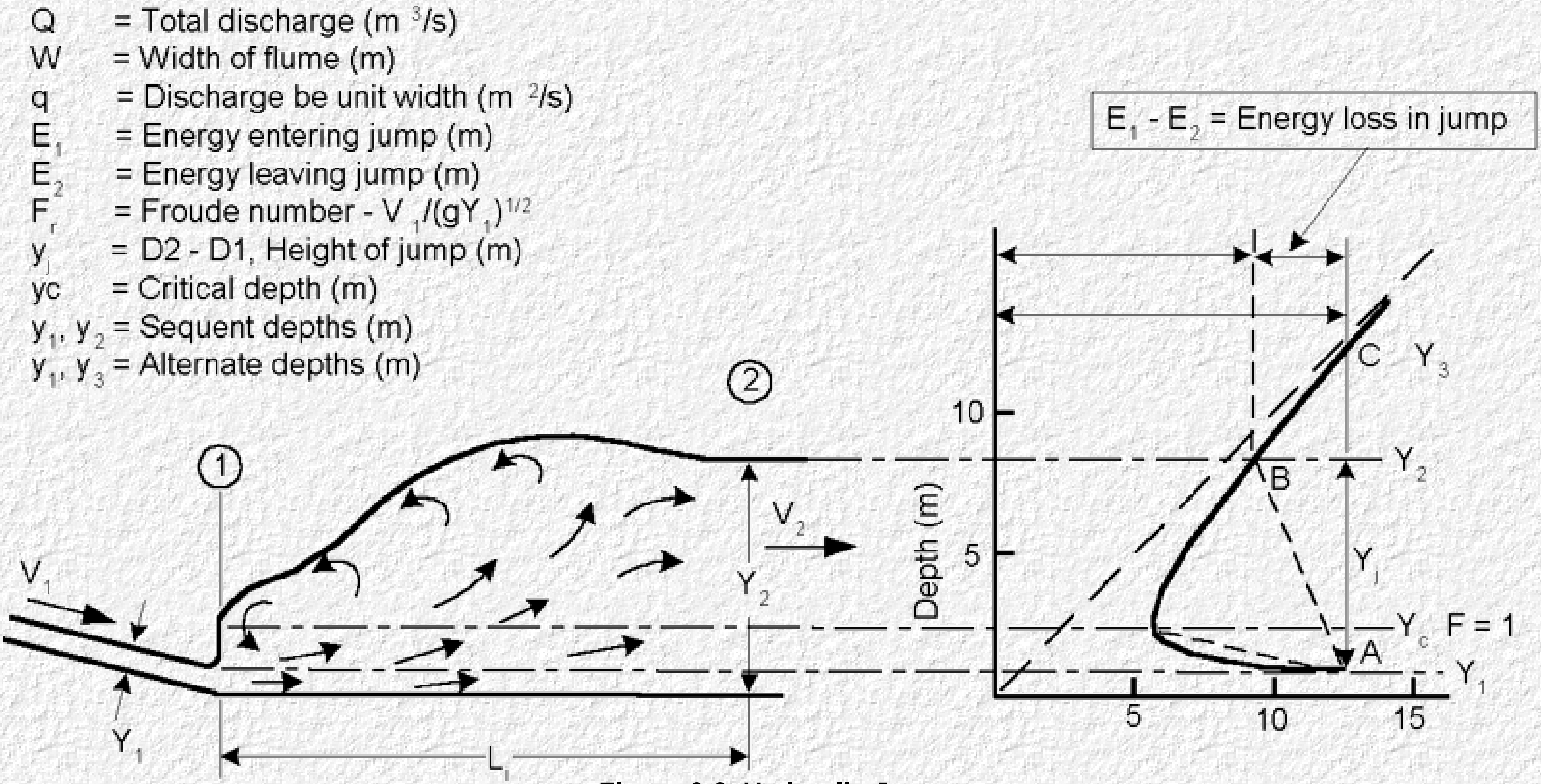


Figure 6-2. Hydraulic Jump

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 down-stream 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 6-2. 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.

6-2 Hydraulic Jump Expression--Horizontal Channels

The hydraulic jump in any shape of horizontal channel is relatively simple to analyze (6-2). Figure 6-3 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.

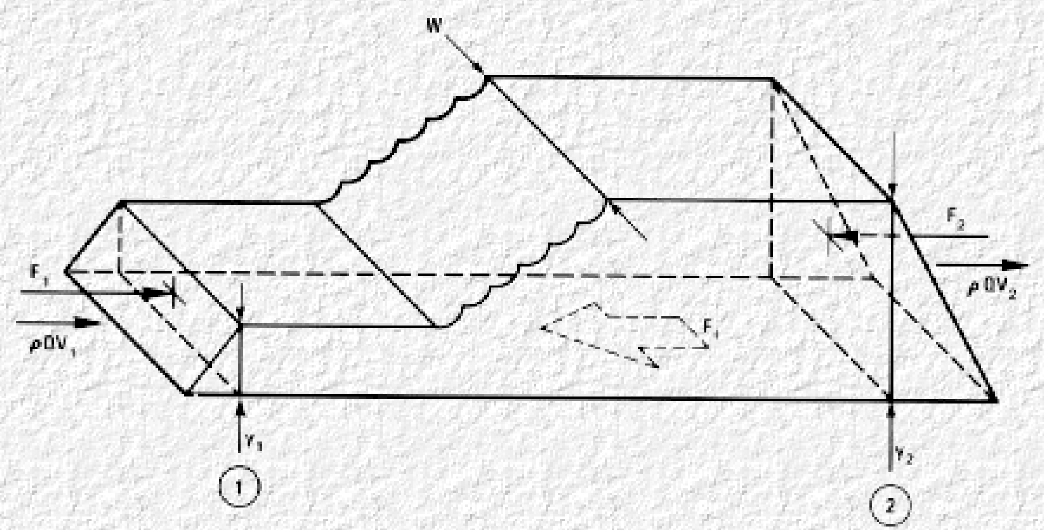


Figure 6-3. Hydraulic Jump in a Horizontal Channel

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,

$$Q^2/gA_1 + A_1y_1 = Q^2/gA_2 + A_2y_2 \qquad \text{6-1}$$

or

$$A_1 y_1 - A_2 y_2 = (1/A_2 - 1/A_1)Q^2/g$$

Letting the distance to the centroid from the water surface = Ky gives: $A_1 K_1 y_1 - A_2 K_2 y_2 = (1/A_2 - 1/A_1)Q^2/g$. Rearranging and using $Fr_1^2 = V_1^2 /gy_1 = Q^2 /A_1^2 gy_1$. Gives: $A_1 K_1 y_1 - A_2 K_2 y_2 = Fr_1^2 A_1 y_1 (A_1 /A_2 -1)$. Dividing this by $A_1 y_1$ provides:

$$K_2 A_2 y_2 /A_1 y_1 - K_1 = Fr_1^2 (1 - A_1 /A_2) \qquad 6\text{-}2$$

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:

$$J^2 -1 = 2Fr_1^2 (1 - 1/J) \qquad 6\text{-}3$$

Figure 6-4 is a plot of Equation 6-3.

For Triangular Channels

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

and

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

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

$$Fr_1 = (J^4 + J^3 + J^2)/ 3(J + 1) \qquad 6\text{-}4$$

Figure 6-5 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:

$$Fr_m = V / \sqrt{g y_m}$$

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 6-4.

For Parabolic Channels

$K_1 = K_2 = 2/5$, 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

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

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

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

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

For Circular Channels

Figure 6-7 and Figure 6-8 are 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:

1. where y_2 is greater than D, and
2. where y_2 is less than D .

For y_2 less than D

$$(K_2 y_2 C_2 / y_1 C_1) - K_1 = Fr^2(1 - C_1 / C_2) \qquad 6\text{-}7$$

For y_2 greater than or equal to D

$$(y_2 C_2 / y_1 C_1) - 0.5 (C_2 D / C_1 y_1) - K_1 = Fr^2 (1 - C_1 / C_2) \qquad 6\text{-}8$$

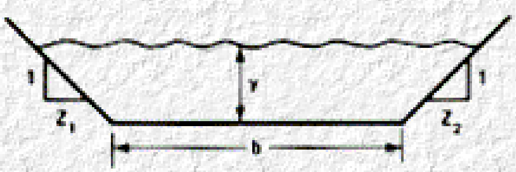
C and K are functions of y/D and may be evaluated from the following table:

Table 6-1

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 take on a more complex form:

$z=(z_1 + z_2)/2$, $t=b/z(y_1)$ or $z(y_1) = b/t$

$J=y_2/y_1$ or $y_2 = J(y_1)$

Area (1) = $z(y_1)^2 + b(y_1) = b(y_1)(1+t)/t$

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 equations 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:

$Fr^2 = J(J+t)]\ [3t(J+1)+2(J^2 + J+1)]/[6(J+t+1)(1+t)]$ 6-9

and using the hydraulic depth:

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

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

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

$(Fr_m)^2 = Fr_2^2 (1+t)/(2+t)$ 6-10

Figure 6-9 and Figure 6-10 represent plots of Equation 6-9 and Equation 6-10. 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:

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

$$K = \frac{\bar{y}}{y};\quad J = \frac{y_2}{y_1};\quad t = \frac{b}{zy_1}$$

Table 6-2. Area Ratio and K Factors

Channel Shape	Area Rations	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 6-1			
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, (Case A)

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 L_j 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 non-rectangular 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 non-rectangular 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 non-rectangular channels is still being actively debated, the recommendations presented are considered conservative if used within the indicated limits.

Figure 6-11, Figure 6-12, and Figure 6-13 may be used for the determination of jump lengths in rectangular, trapezoidal, parabolic, triangular, and circular channels, respectively. The circular channel curve (Figure 6-13) 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., $L_J = 7(y_2 - y_1)$.

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 2.134 meter wide box culvert discharging 11.327 m^3/s on a 0.2 percent slope.

1. Outlet flow conditions are:

v_1 = 5.79 m/s
y_1 = 0.914 meters
Fr = 1.9

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

$J = y_2 / y_1 = 2.2$
$y_2 = 2.2(0.914) = 2.0$ m.

3. Length of jump, L_j: See Figure 6-2

$L_j / y_1 = 9.0$
$L_j = .914(9.0) = 8.226$ m

4. After jump velocity

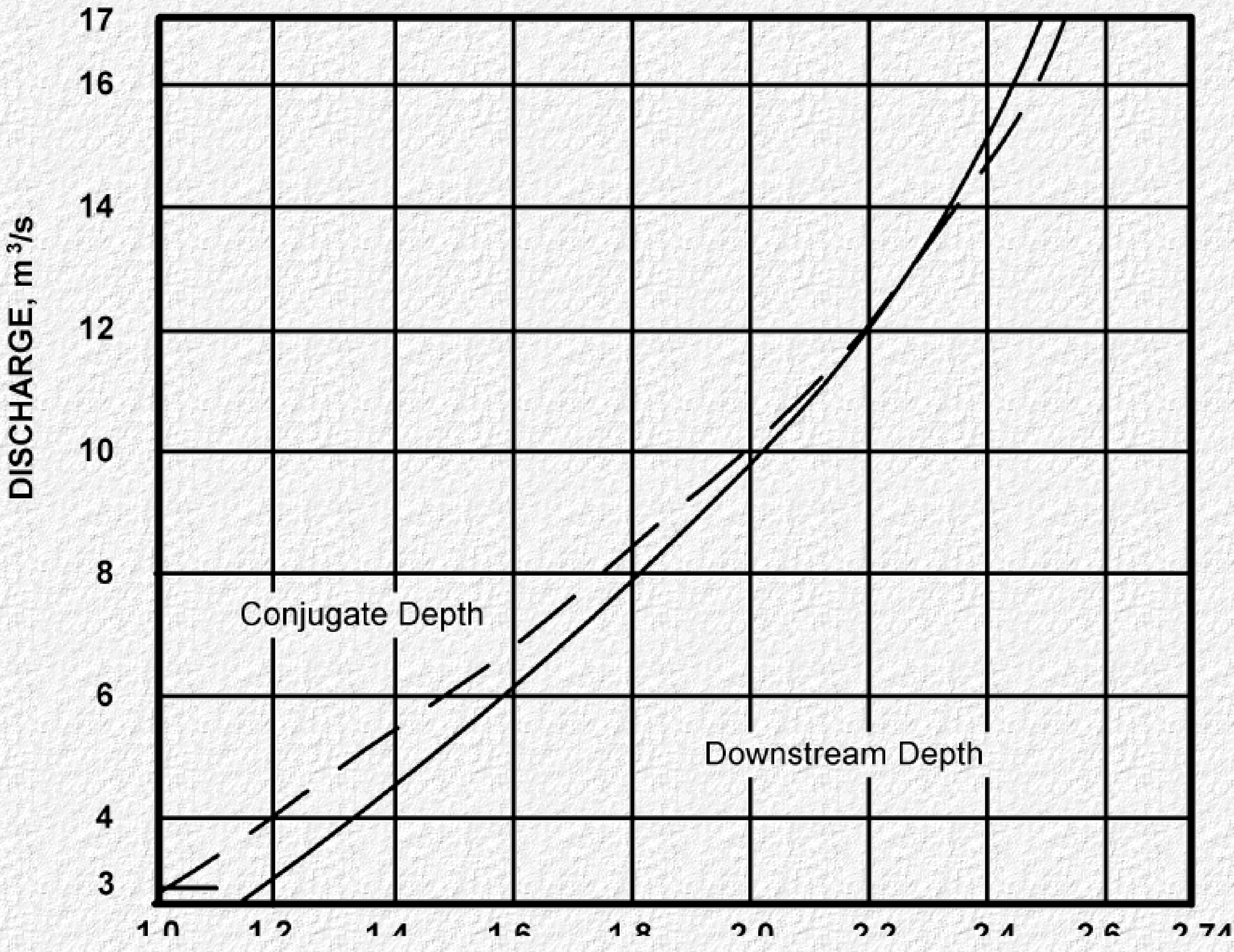
$V_2 = Q/A = 11.327/2.134(2.011) = 2.639$ m/s

5. Velocity reduction is more than 54%.

V(in) = 5.79 m/s
V(out) = 2.64 m/s

These answers could also be obtained using Figure 6-21.

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 3.04 meter bottom, is trapezoidal with 2H:1V side slopes, and n = 0.03. The bottom slope changes to 0.0004 m/m 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(3A1)or Table 3-1.



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 13.6 m^3/s. Beyond this point, the jump would begin to move downstream out of the basin.

6-3 Hydraulic Jump Expressions--Sloping Channels, Case B & Case D

Figure 6-14 from reference (6-3) 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:

$$Q(V_2 - V_1)/g = .5b(y_1^2 - y_2^2)\cos\theta + w\sin\theta/\gamma \qquad 6\text{–}11$$

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 (6-3) 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 6-15 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 6-14. Figure 6-16 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 6-17 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 6-18. This figure is for Case D type jump but it can be applied to Case B with negligible error. Figure 6-19 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.

6-4 Design Recommendations

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

- Step 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.

- Step 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 6-15 and Figure 6-17. 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.

- Step 3. The length of the jump for maximum or partial flows can be obtained from Figure 6-18. 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.

- Step 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 6-14), 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.

- Step 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.

- Step 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.

- Step 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 3H:1V to 2H:1V

- Step 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.

- Step 9. A model study is advisable when the discharge exceeds 46.45 m^3/s per meter 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.

6-5 Jump Efficiency

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

$$H_L/H_1 = 2-2(y_2)+Fr^2\,[1-A_1^2/A_2^2]/(2+Fr^2) \qquad \text{6-12}$$

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 6-20.

Even though this figure indicates that the non-rectangular 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.

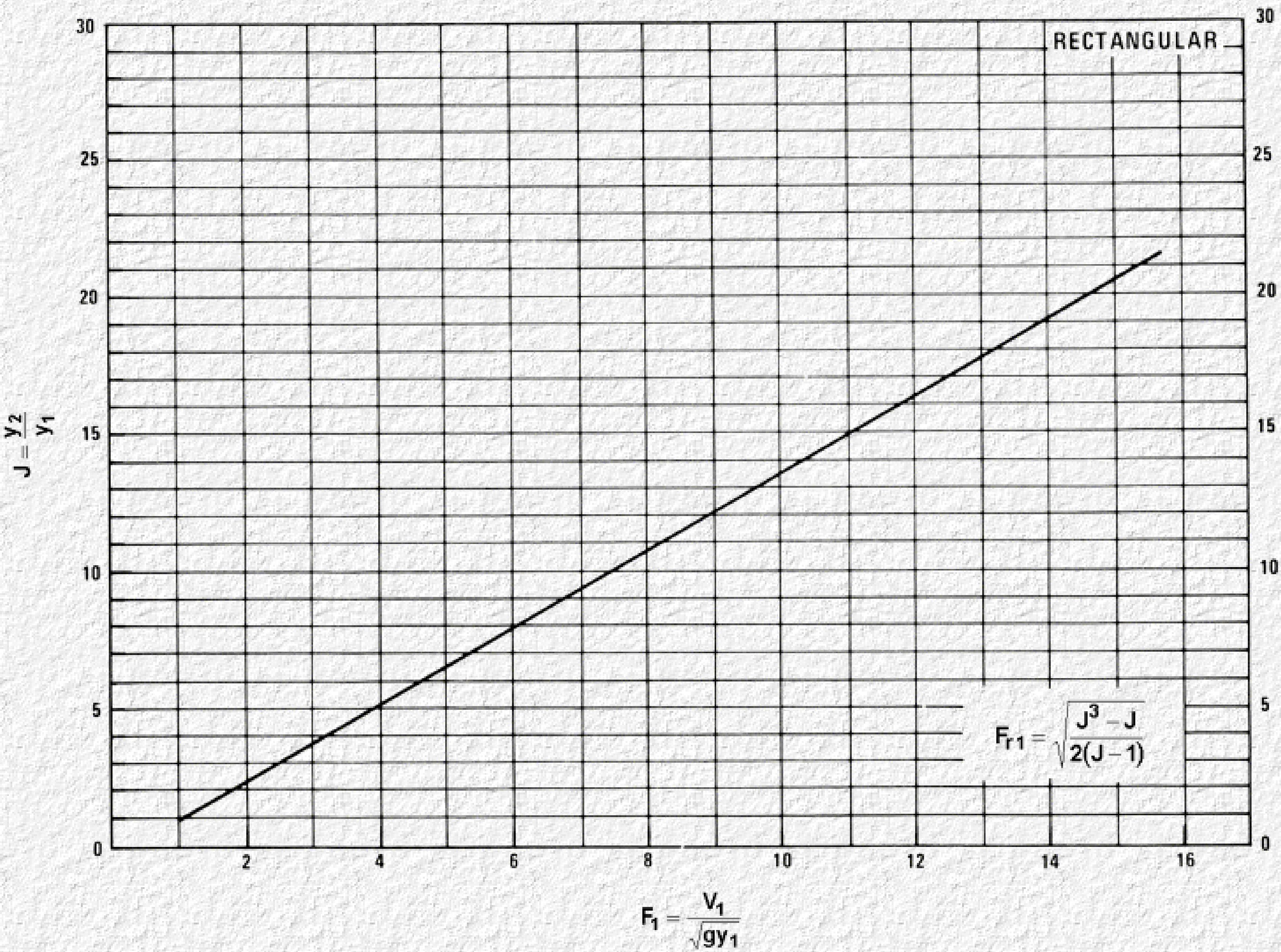


FIGURE VI-4 HYDRAULIC JUMP – HORIZONTAL, RECTANGULAR CHANNEL

Figure 6-4. Hydraulic Jump - Horizontal, Rectangular Channel

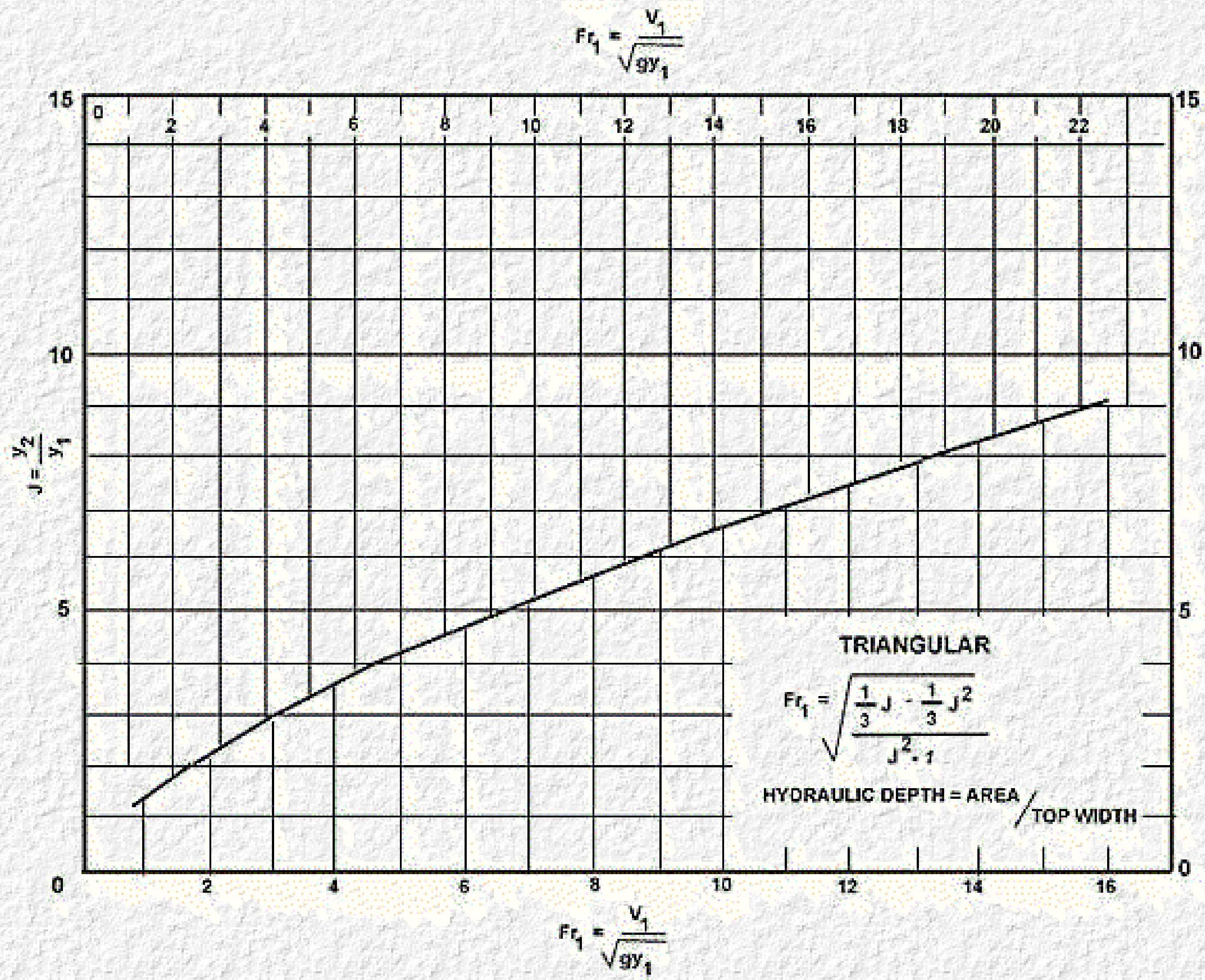


Figure 6-5. Hydraulic Jump - Horizontal, Triangular Channel

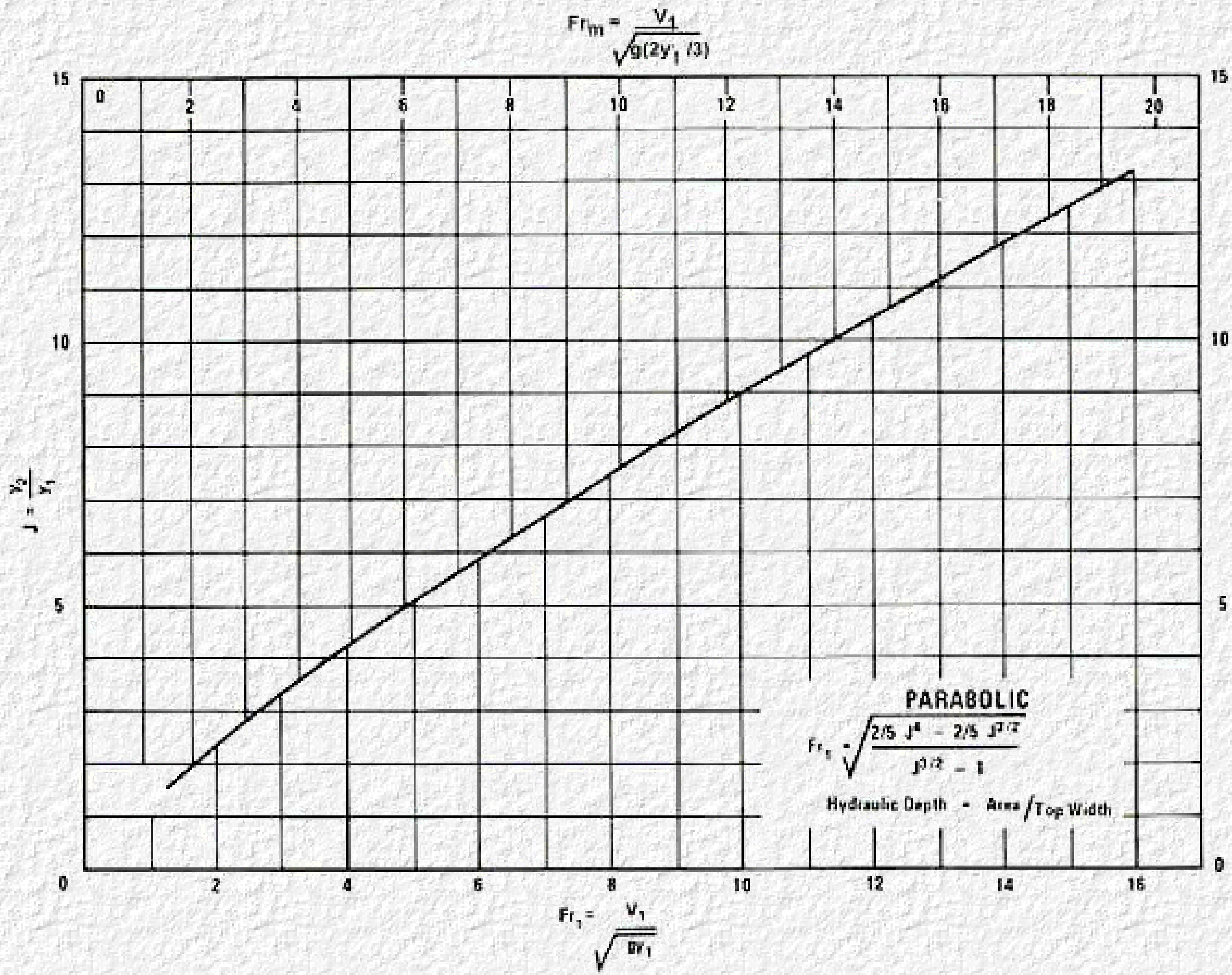


Figure 6-6. Hydraulic Jump - Horizontal, Parabolic Channel

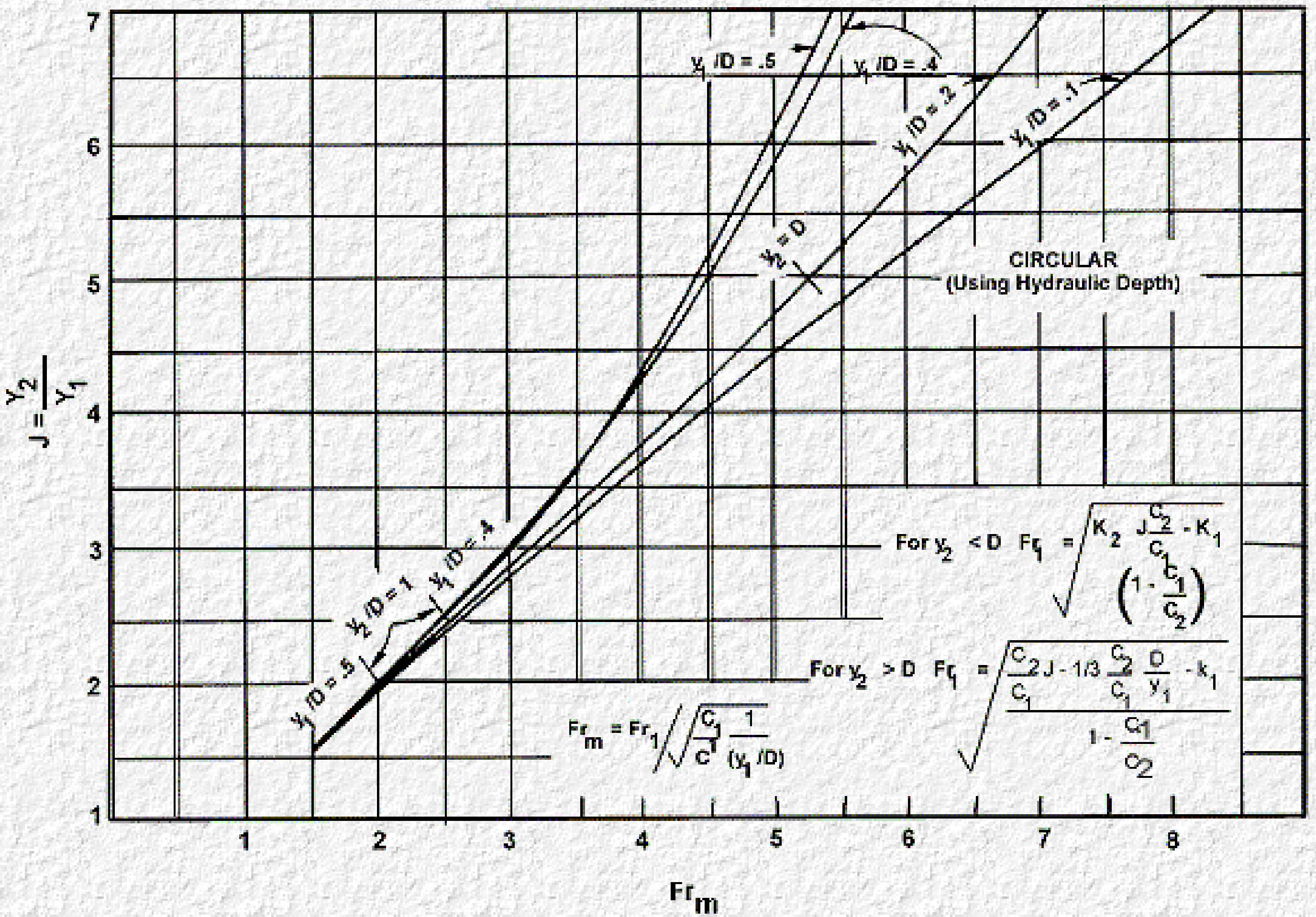


Figure 6-7. Hydraulic Jump - Horizontal, Circular Channel (hydraulic depth)

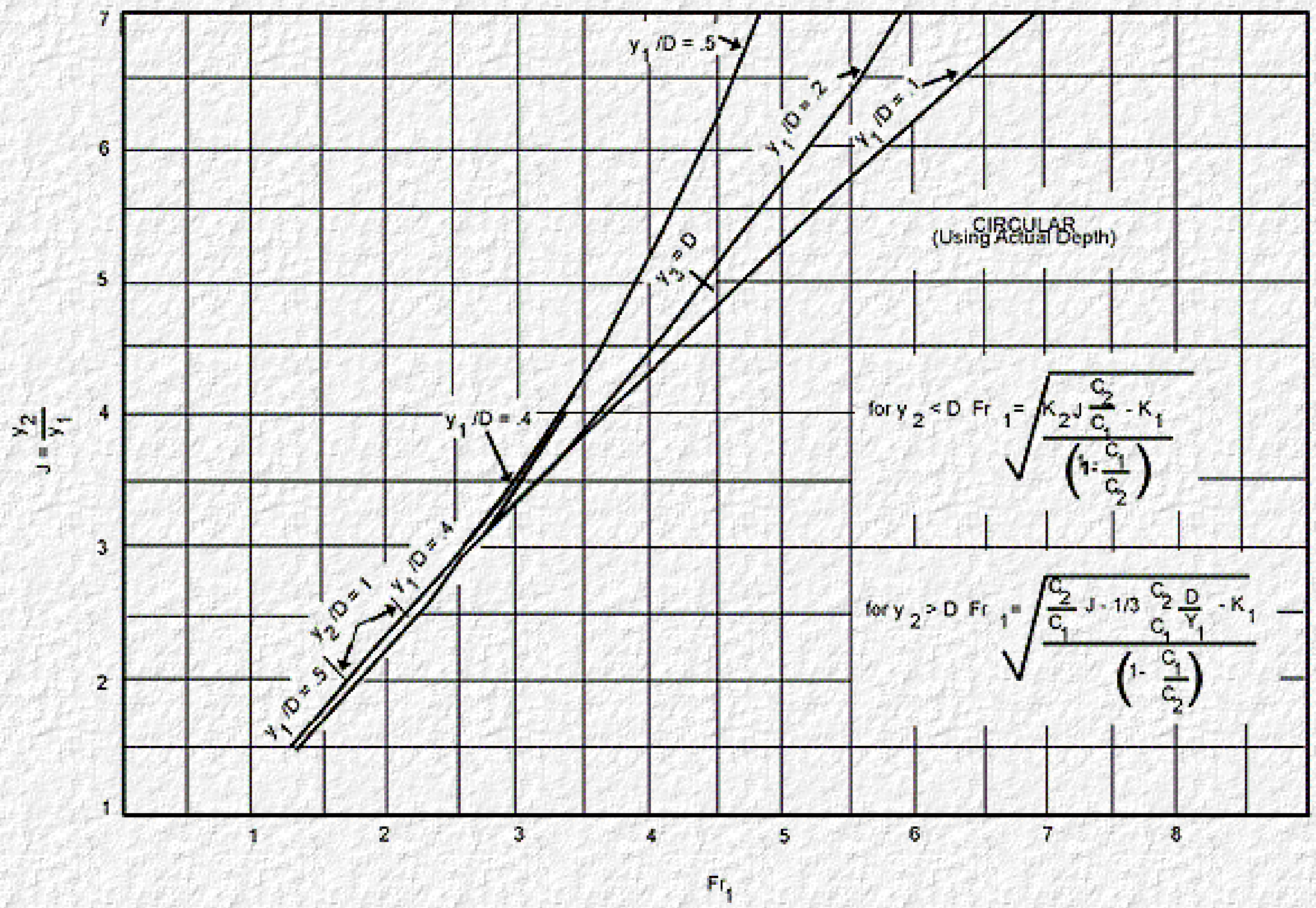


Figure 6-8. Hydraulic Jump - Horizontal, Circular Channel (actual depth)

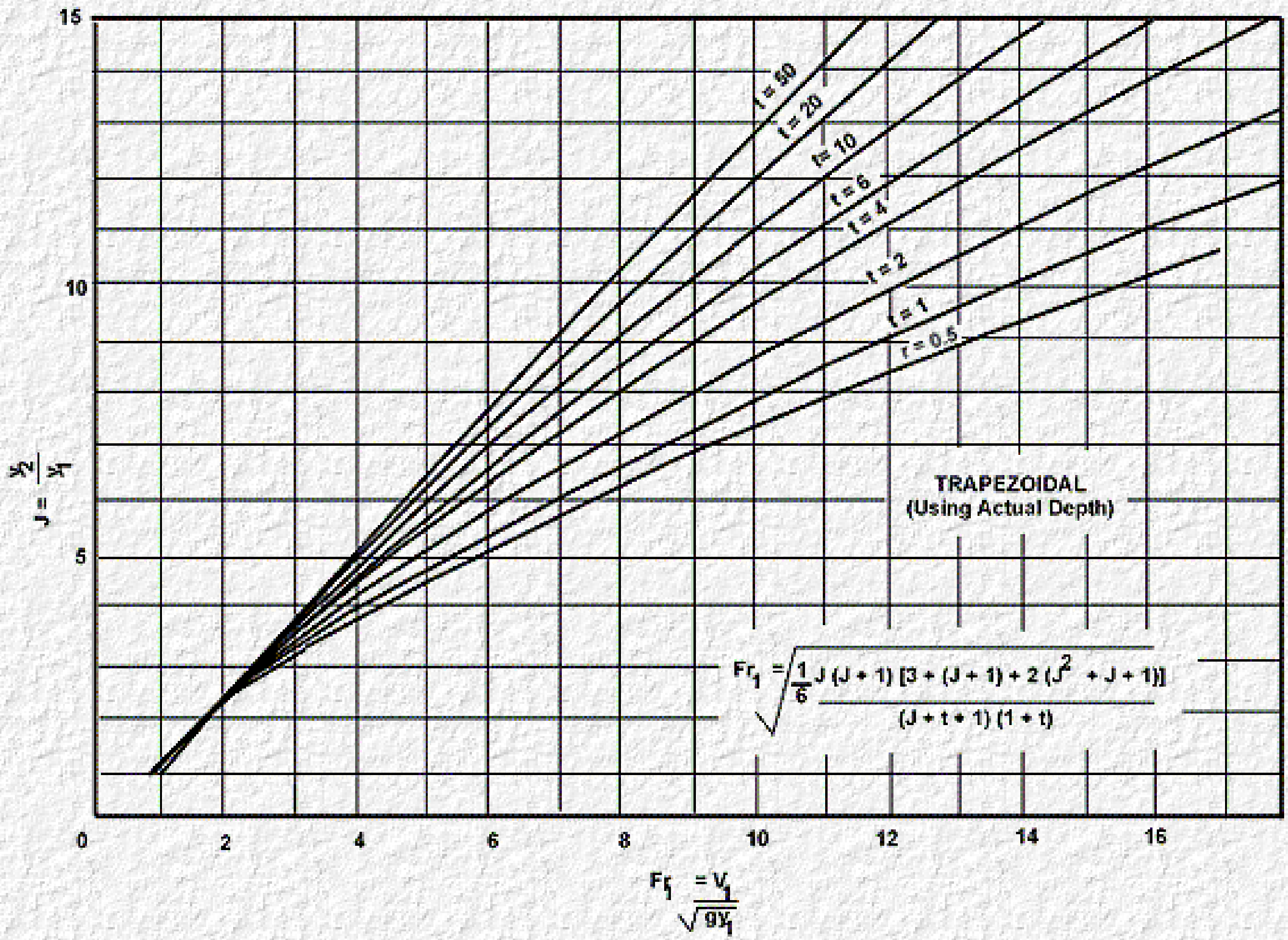


Figure 6-9. Hydraulic Jump - Horizontal, Trapezoidal Channel (actual depth)

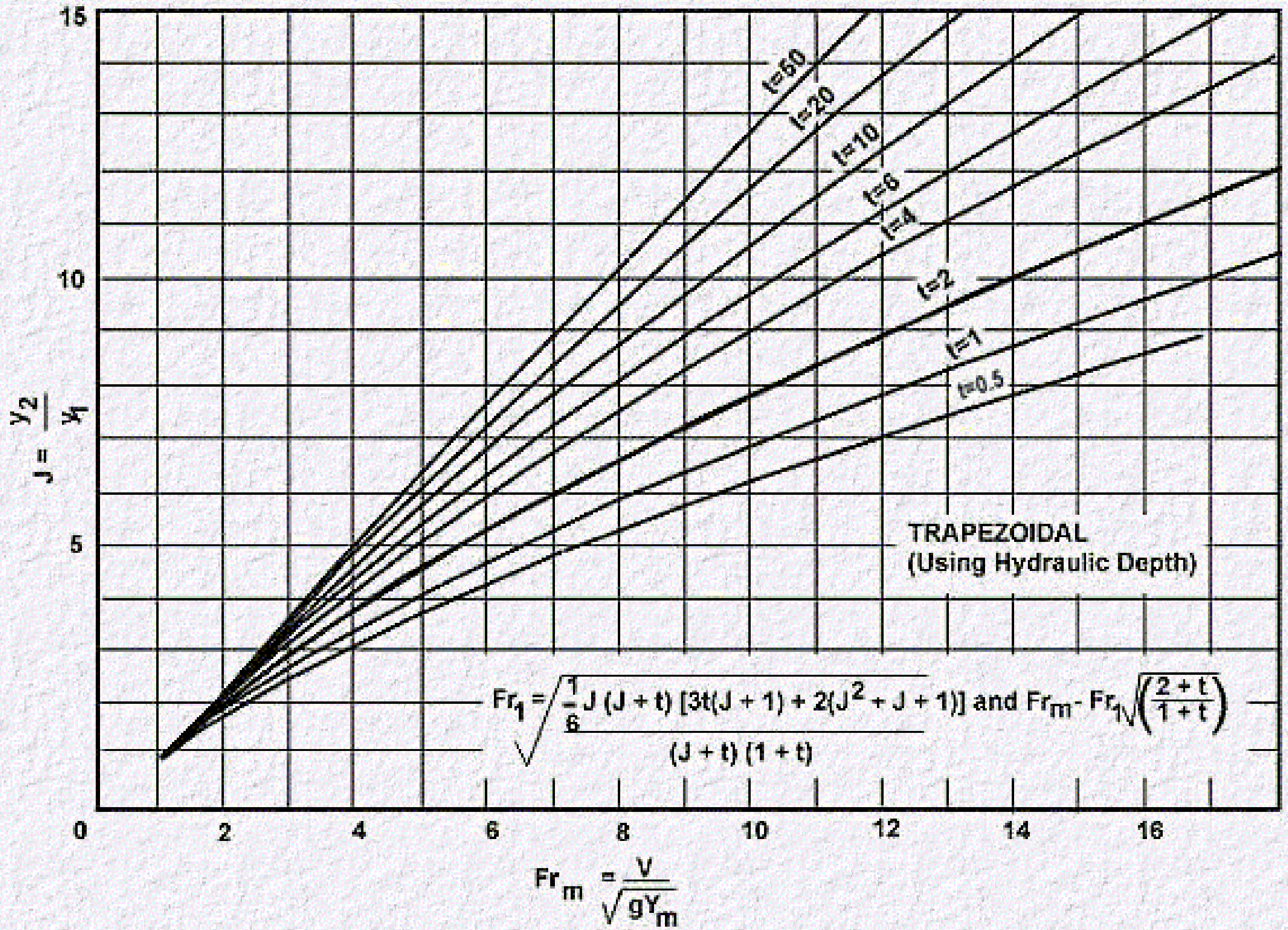


Figure 6-10. Hydraulic Jump - Horizontal, Trapezoidal Channel (hydraulic depth)

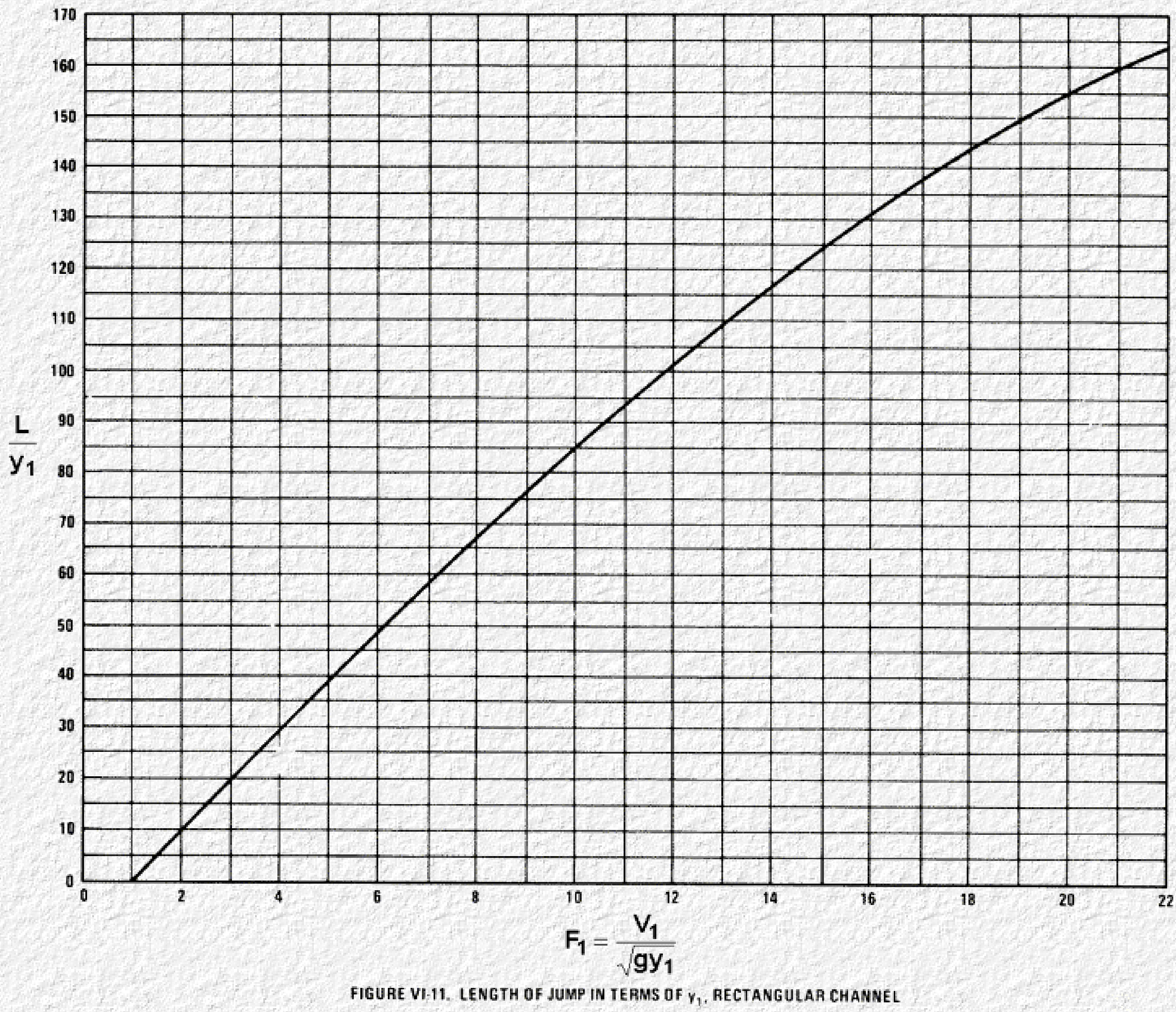


Figure 6-11. Length of Jump in Terms of y_1, Rectangular Channel

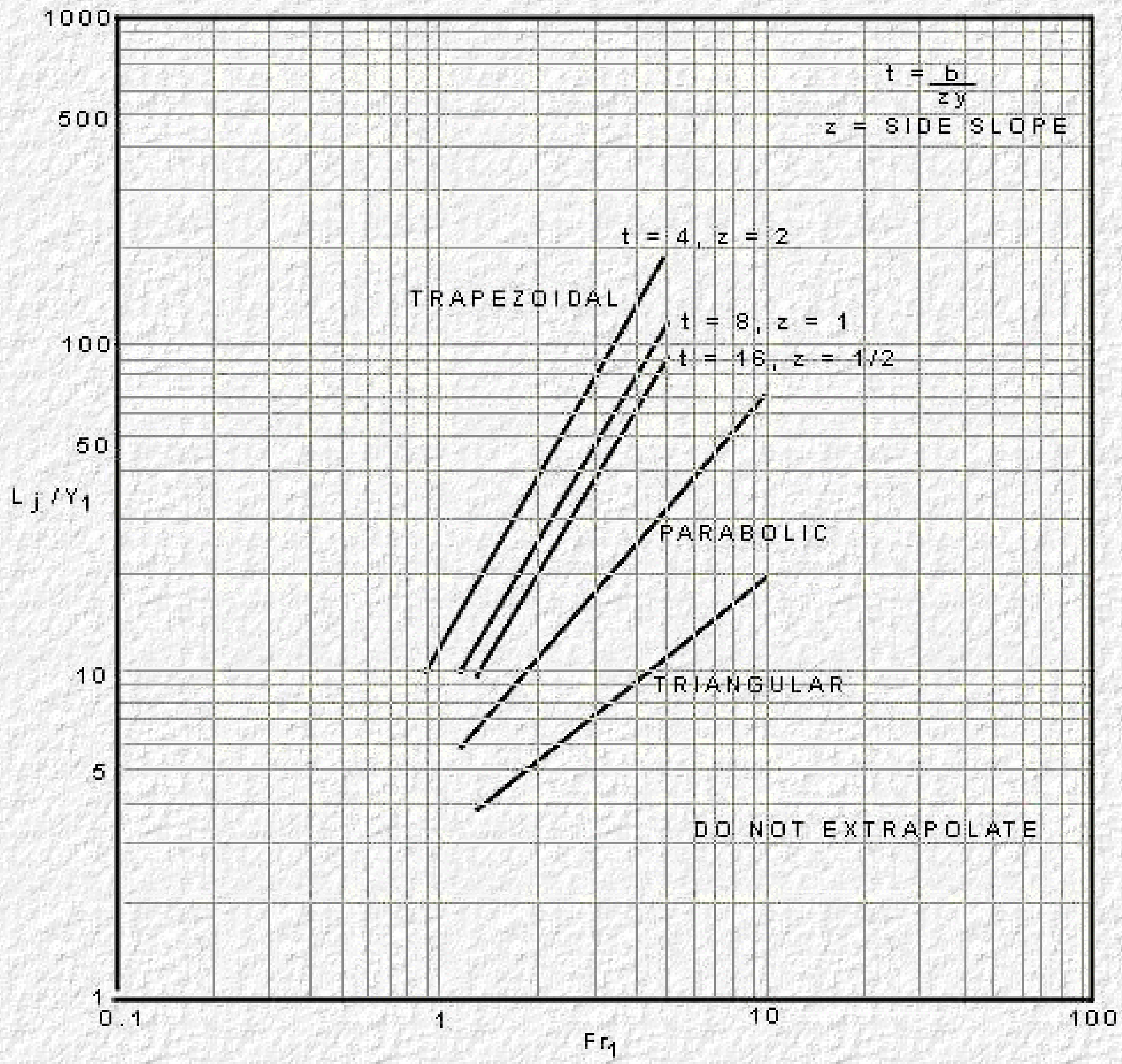


Figure 6-12. Hydraulic Jump Length for Non-rectangular Channels

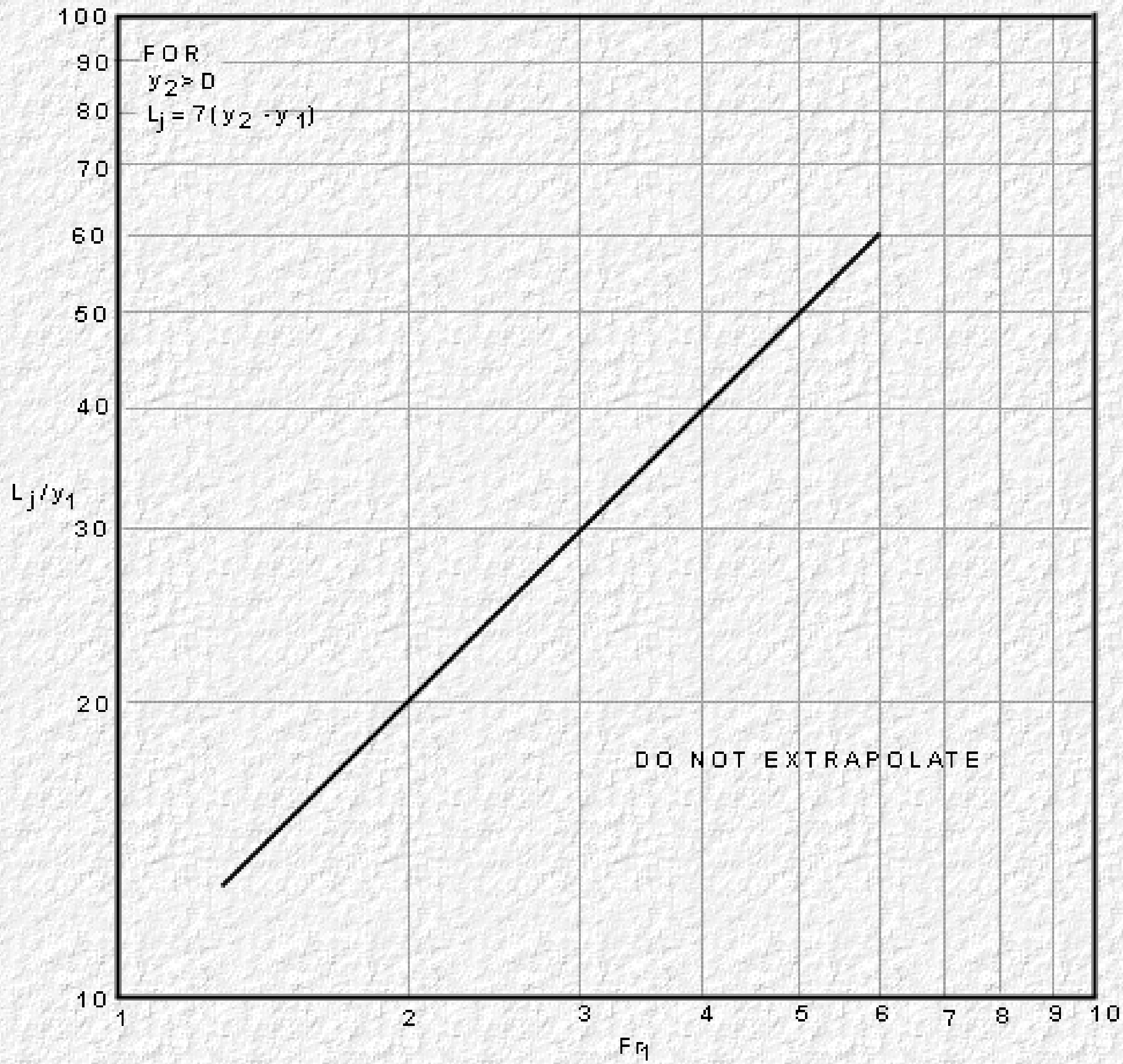


Figure 6-13. Jump Length Circular Channel with y_2 < D

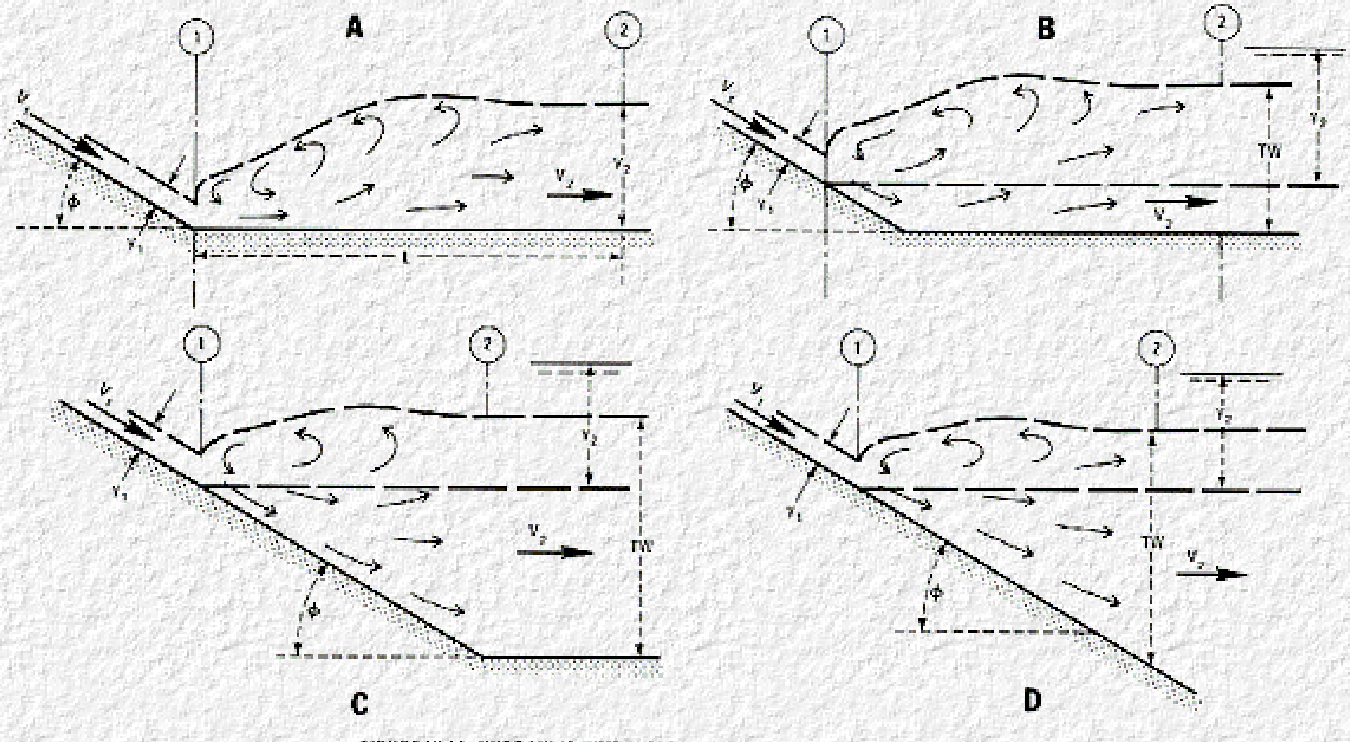


Figure 6-14. Hydraulic Jump Types Sloping Channels (from reference 6-3)

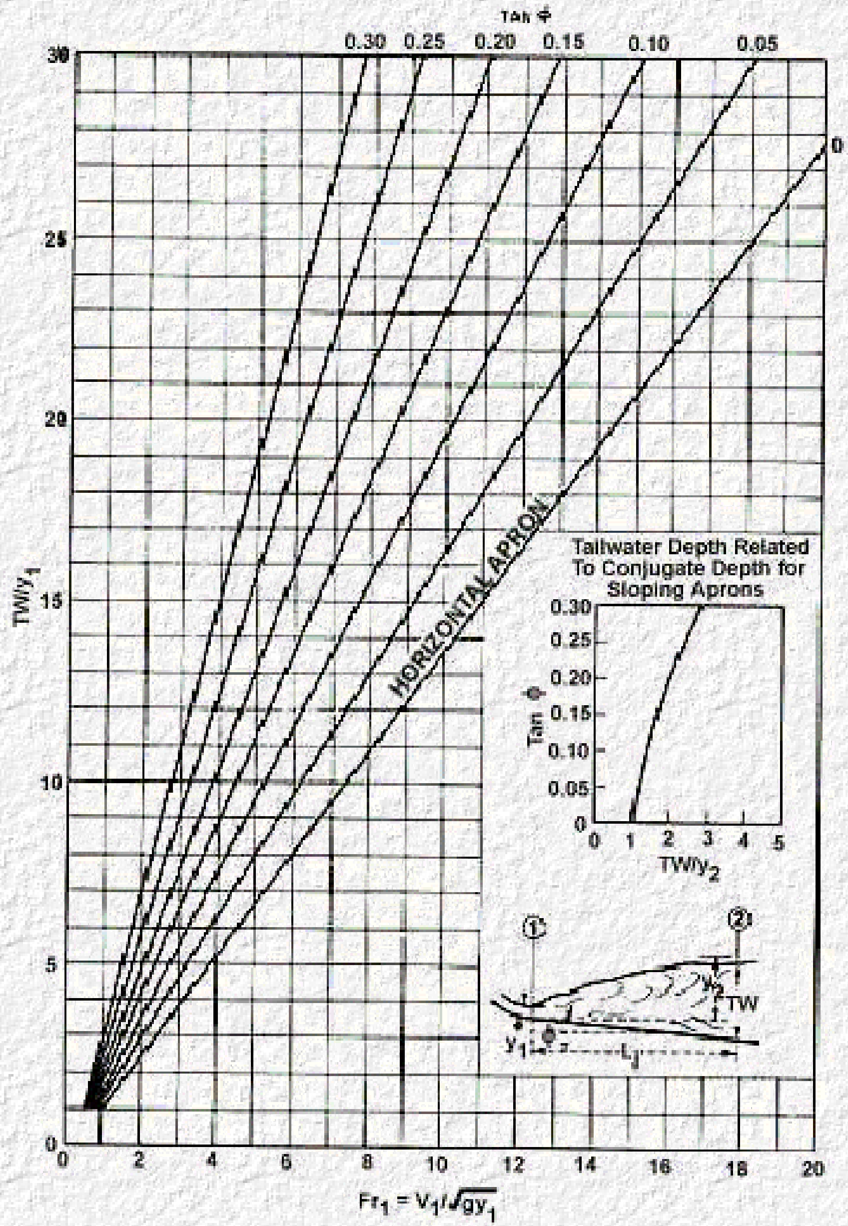


Figure 6-15. Stilling Basin, Case D, Hydraulic Jump on Sloping Apron Ratio of Tailwater Depth to y_1 (from reference 6-3)

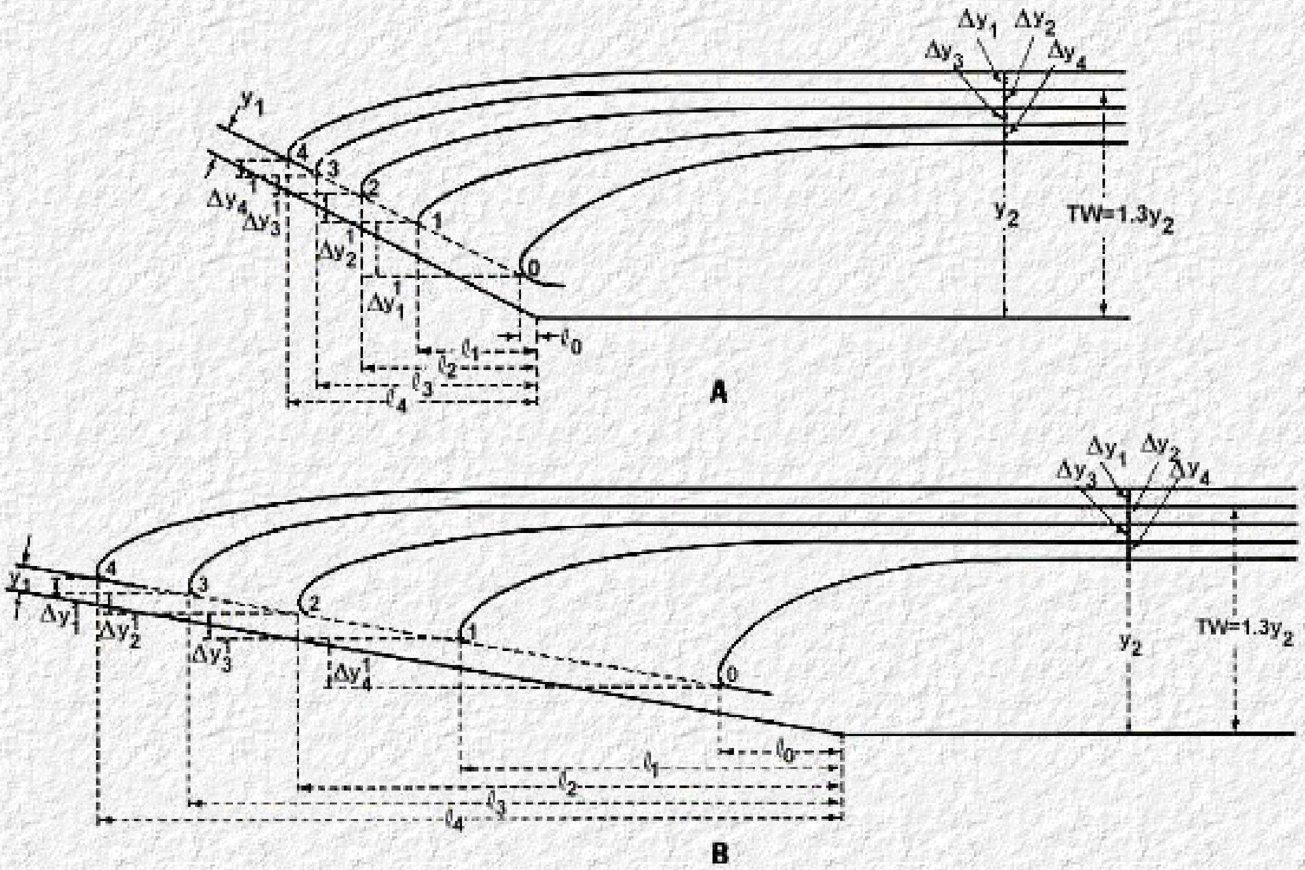


Figure 6-16. Stilling Basin, Case B, Profile Characteristics (from reference 6-3)

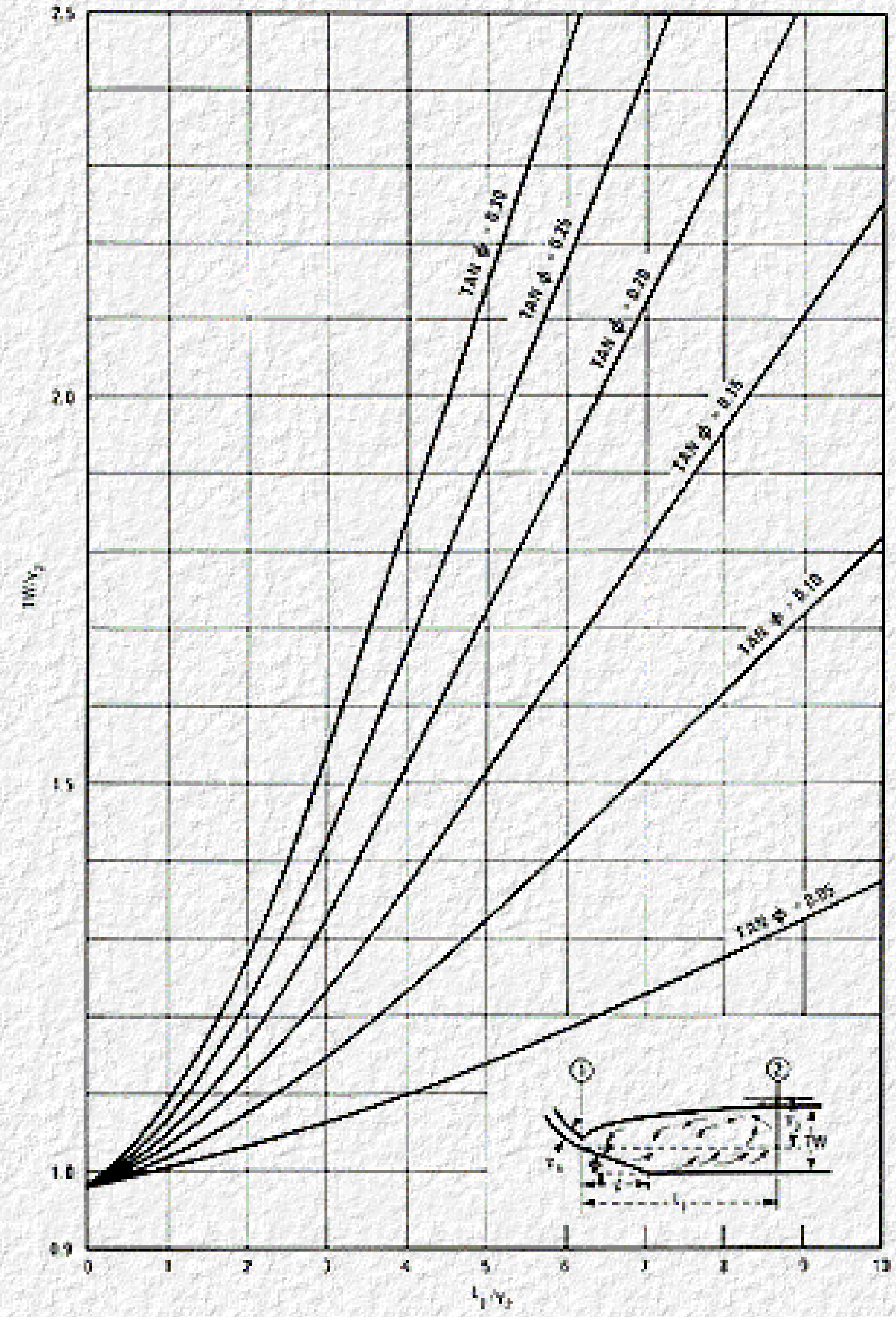


Figure 6-17. Stilling Basin Tailwater Requirement for Sloping Aprons - Case B (from reference 6-3)

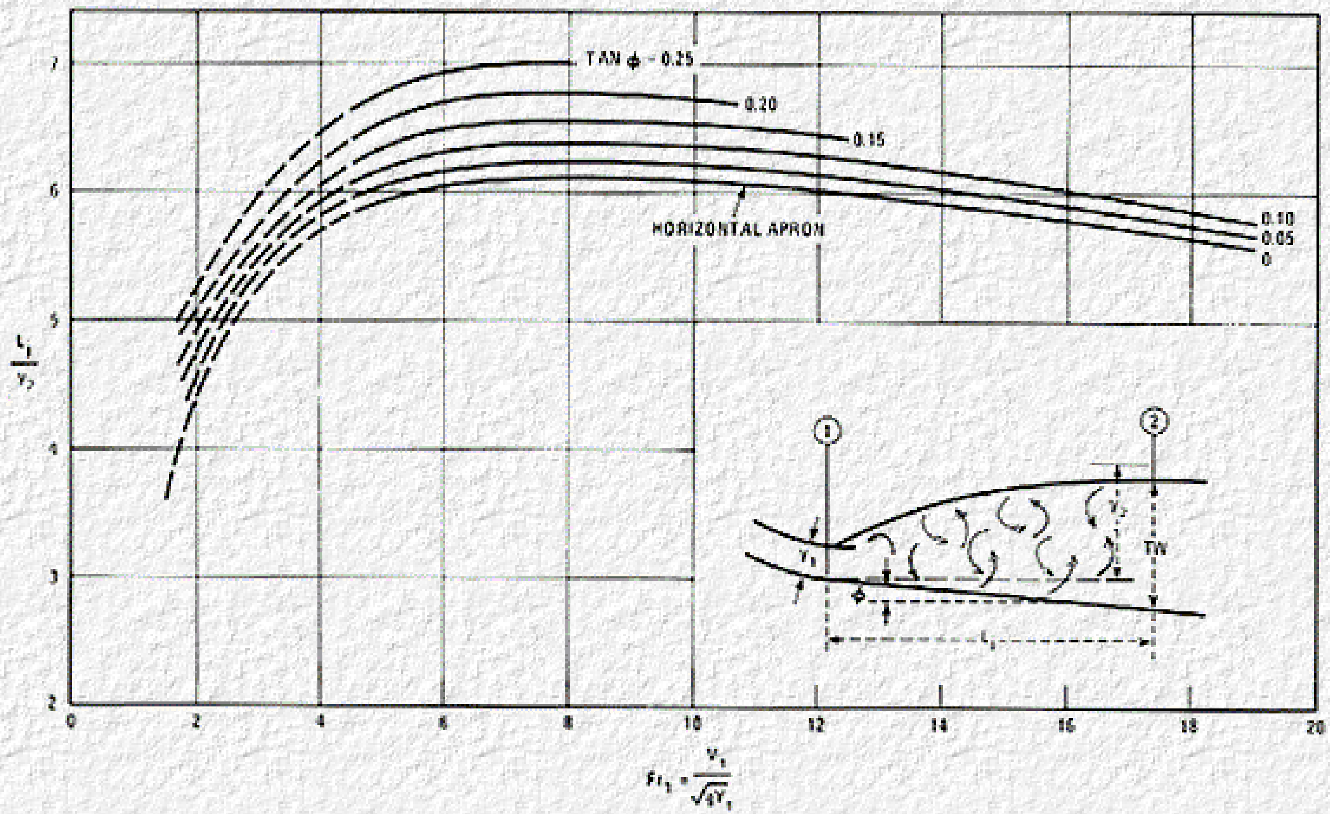


Figure 6-18. Stilling Basin, Case D, Length of Jump in Terms of Conjugate Depth, y_2 (from reference 6-3)

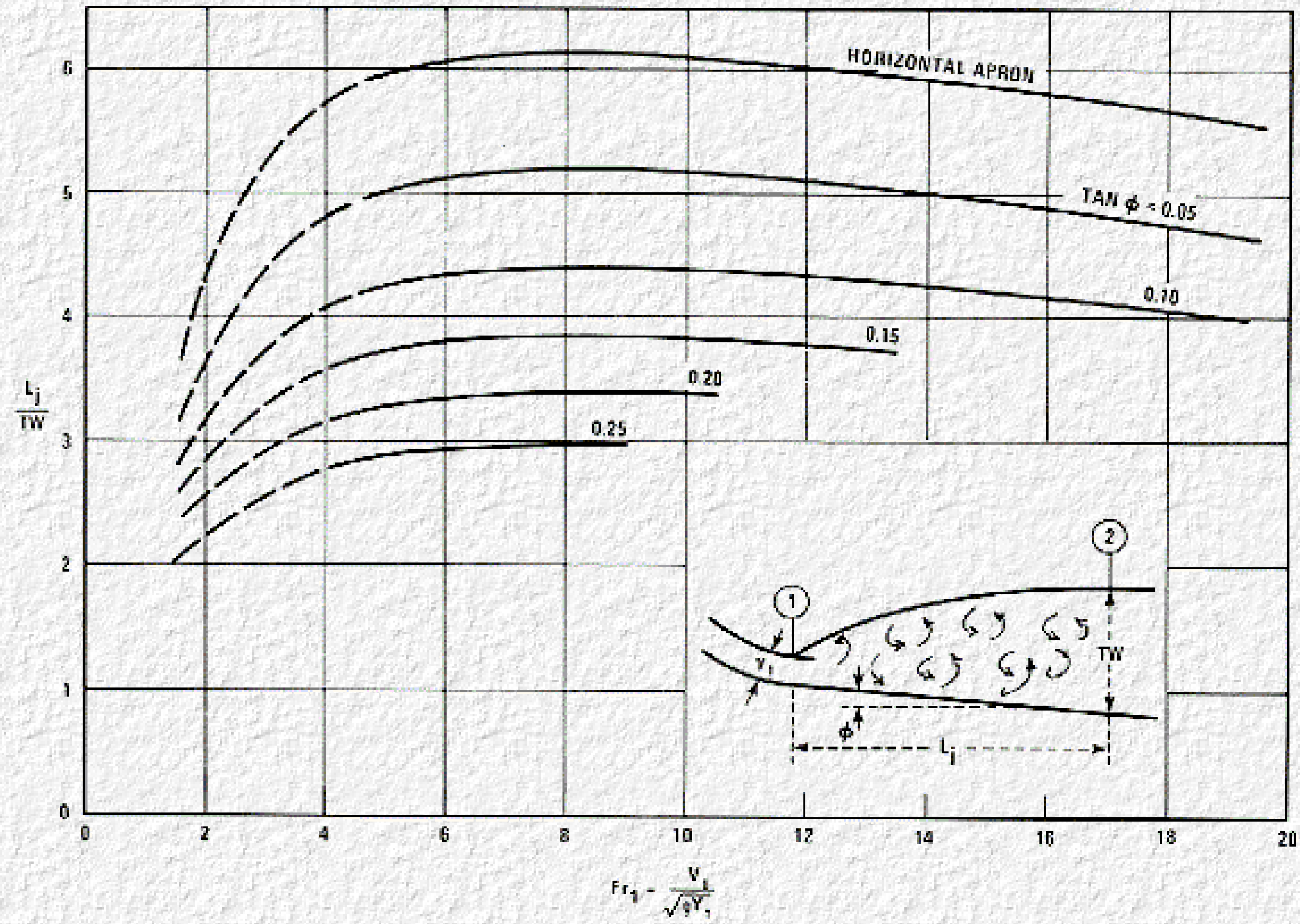


Figure 6-19. Stilling Basin, Case D, Length of Jump in Terms of TW Depth (from reference 6-3)

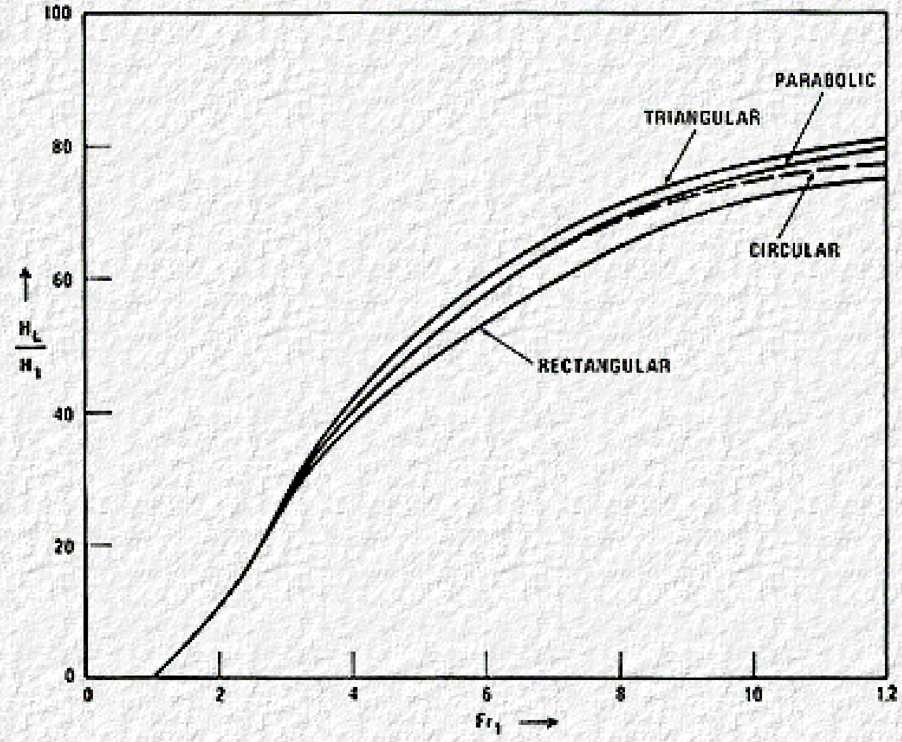
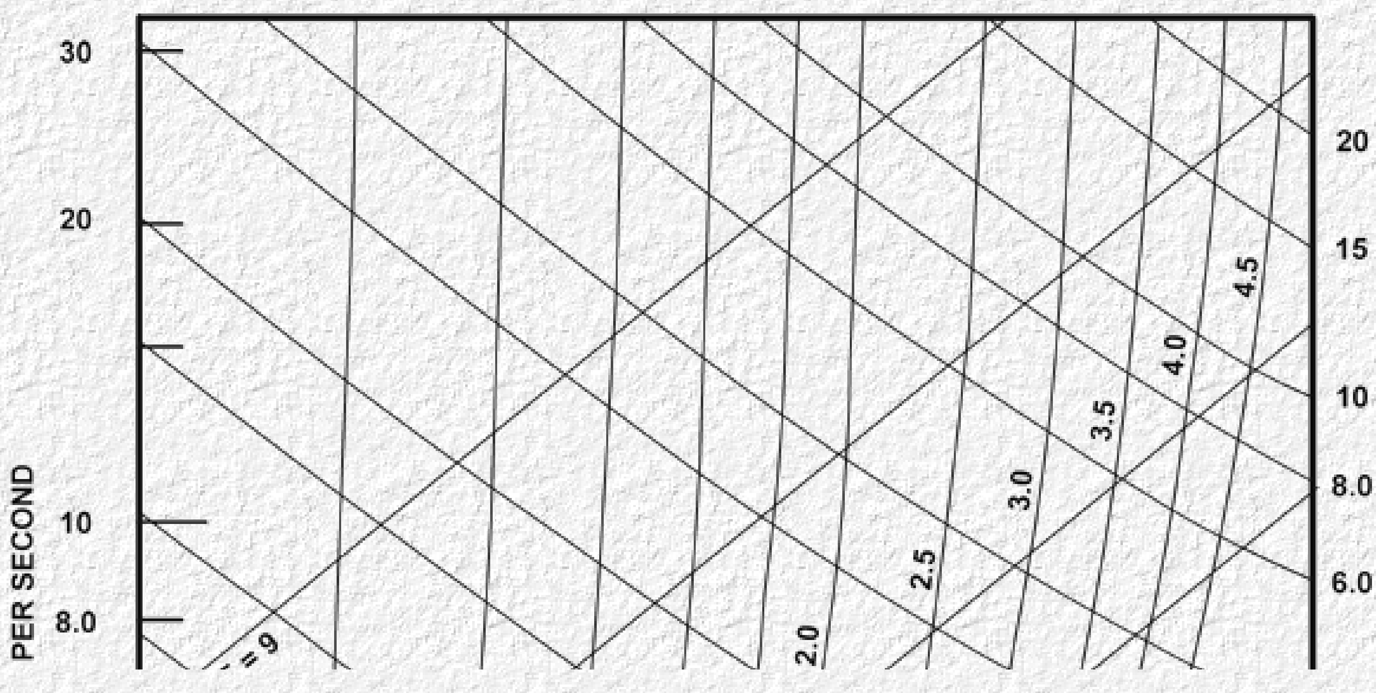


Figure 6-20. Relative Energy Loss for Various Channel Shapes



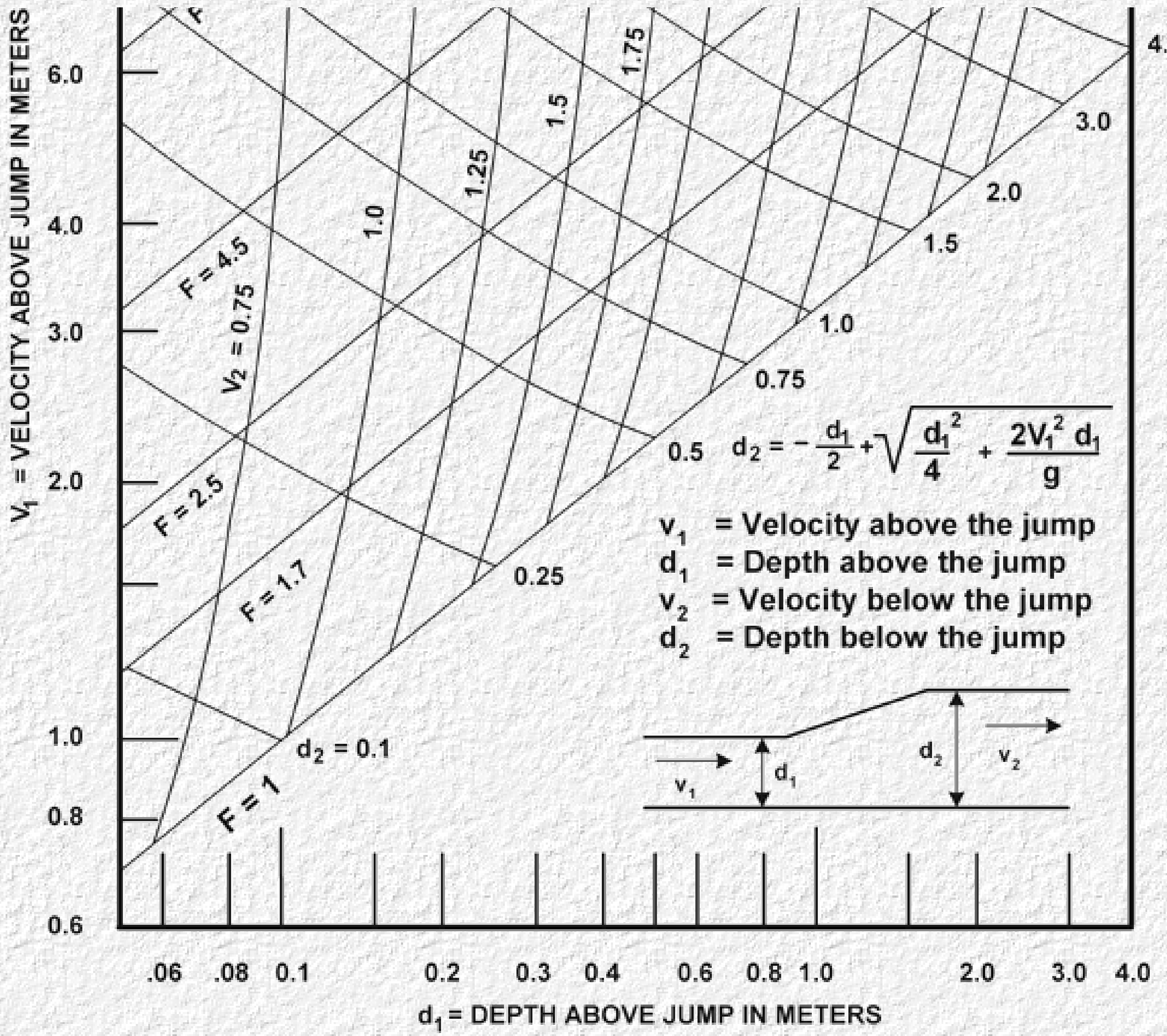


Figure 6-21. Relations between Variables in Hydraulic Jump for Rectangular Channel (from reference 3-3)

6-1. U.S. Bureau of Reclamation,
Design of Small Dams, Second Edition
1973, GPO

6-2. Sylvester, R.,
Hydraulic Jump In All Shapes Of Horizontal Channels,
Jourl. Hyd. Div. ASCE, Vol. 90, HY1. Jan 1964, pp. 23-55.

6-3. 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.

Go to Chapter 7, Part I

Chapter 7 : HEC 14

Forced Hydraulic Jump Basins
Part I

Go to Chapter 7, Part II

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:

1. Limitations
2. Design guidance
3. 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 (reference 7-1) maximum coefficient of drag, C_D, for a structural angle or a rectangular block is 1.98. Using 1000 kg/m^3 for the density of water, the drag force becomes:

$$F_D = 990 A_F V_a^2 \qquad \text{7-1}$$

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 7-1.

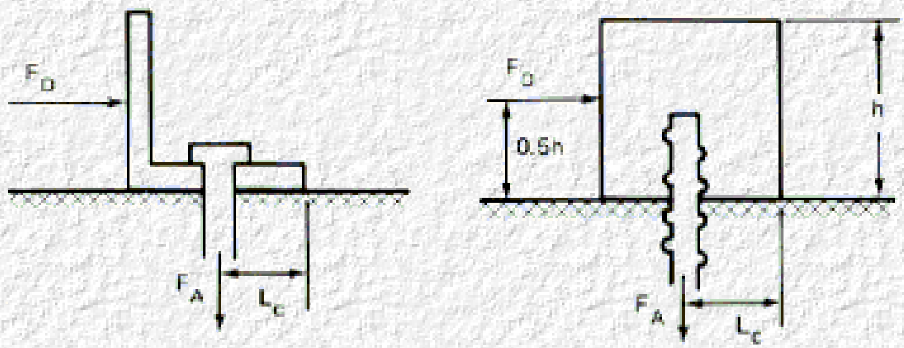


Figure 7-1. Forces Acting on a Roughness Element

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

$$F_A = hF_D/2L_c = 495\ (h/L_c)\ A_F V_a^2 \qquad 7\text{-}2$$

Where:

F_A = total force on anchors
F_D = drag force on roughness element
h = height of roughness
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.

7-A C.S.U. Rigid Boundary Basin

The Colorado State University rigid boundary basin (7A1) which uses staggered rows of roughness elements is illustrated in Figure 7-A-1.

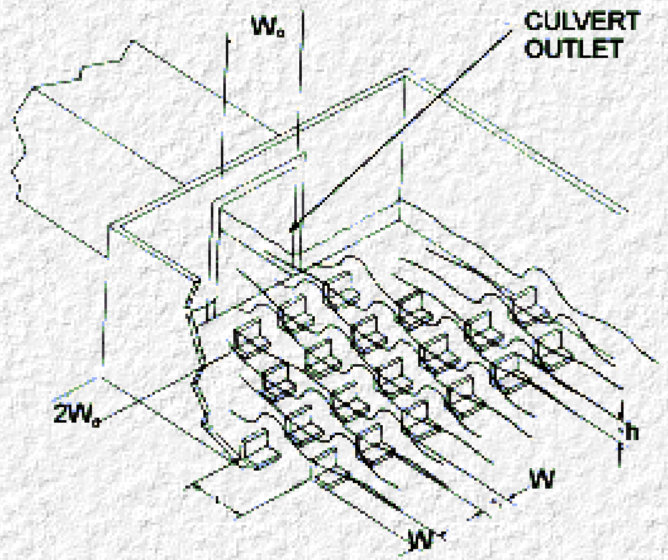


Figure 7-A-1. Sketch of C.S.U. Rigid Boundary Basin

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 7-A-2, and compute the drag coefficient from the momentum equation by balancing the forces acting on the volume of fluid.

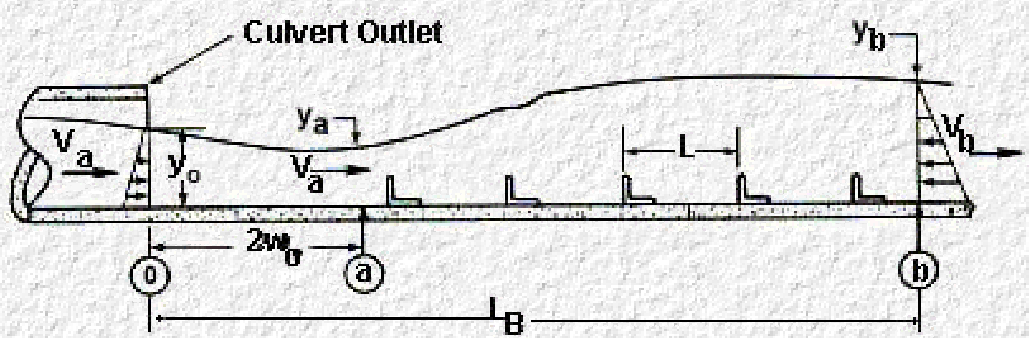


Figure 7-A-2. Definitive Sketch for the Momentum Equation

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 (7A2) 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 7-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 7-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:

$$\rho V_o Q + C_p \gamma (y_o^2 /2) W_o = C_B A_F N \rho V_A^2 /2 + \rho V_B Q + \gamma Q^2 /(2 V_B^2 W_B) \qquad \text{7-A-1}$$

Where:

C_p is the momentum correction coefficient for the pressure at the culvert outlet Figure 7-A-4.

γ and ρ are unit weight (9810 N/m^3) and density (1000 Kg/m^3) of water, respectively

y_o = depth, V_o = Velocity, W_o = culvert width, at the culvert outlet.

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

V_B = exit velocity, and W_B = 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 7-A-2 includes the weight component by assuming a straight-line water surface profile across the jump:

$$C_P \gamma y_o^2 W_o /2+ \rho V_o Q+w(\sin\theta)=C_B A_F N \rho V_A^2 /2+ \gamma Q^2 /(2V_B^2 W_B)+ \rho V_B Q \qquad 7\text{-}A\text{-}2$$

where:

w = weight of water within the basin.

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

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 Figure 4-A-2, Figure 4-A-3, Figure 4-A-4, and Figure 4-A-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) .

$$2W_o S_o +y_A +(0.25) (Q/W_A y_A)^2 /2g=y_o +0.25(V_o^2 /2g) \qquad 7\text{-}A\text{-}3$$

where:

$W_A =W_o [4/3Fr+1]$

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:

$$F.B. = h+y_1 +0.5(V_A \sin\phi)^2 /9.81 \qquad 7\text{-}A\text{-}4$$

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

Another solution is a splash shield, which has been investigated in the laboratory (7A3). This involves suspending a plate with a stiffener between the first two rows of roughness elements as shown in Figure 7-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.

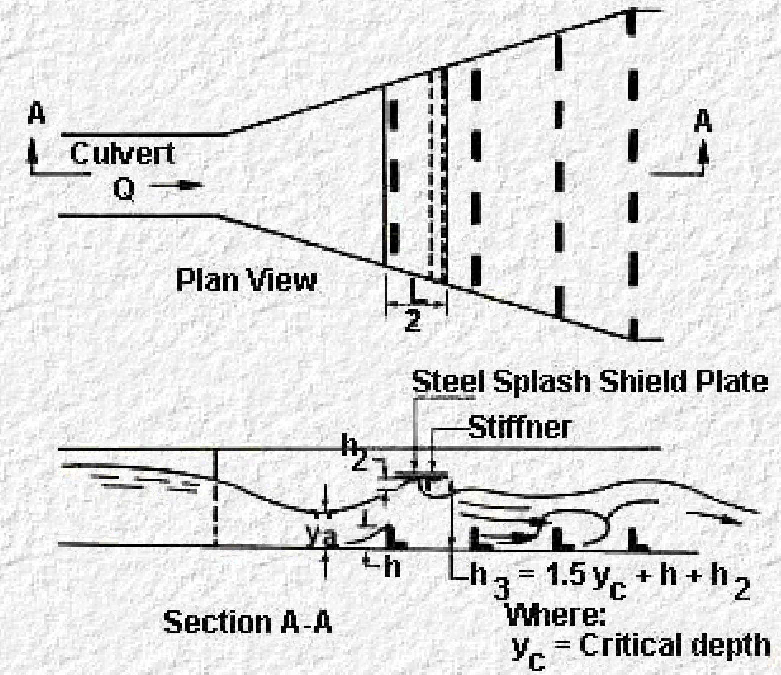


Figure 7-A-3. Splash Shield

Design Discussion

The initial 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 7-A-3 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 4-A-2 or 4-A-3 to find V_A and Figure 4-A-4 or Figure 4-A-5 to find y_A .For basins with expansion ratios between 2 and 4, use Figure 4-A-2 or Figure 4-A-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 7-A-3 to determine both V_A and y_A .

Select the trial roughness height to depth ratio h/y_A from Figure 7-A-5 and determine:

- roughness element height, h;
- longitudinal spacing 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, $A_F = W_1 h$;
- C_p from Figure 7-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 7-A-1 or Equation 7-A-2 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 7-A-1 or Equation 7-A-2 for C_B A_F N.

Using the C_B, A_F, and N values found in Figure 7-A-5 also compute C_B A_F N. This last value should be equal to or larger than the C_B A_F N value obtained from Equation 7-A-1 or Equation 7-A-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 7-A-1 or Equation 7-A-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 sub-critical, 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 7-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 2 contains a section on "Riprap Protection" and Figure 2-C-1 may be used to size the required riprap.

Design Procedure

- Step 1. Compute:

  a. V_o

  b. y_o

  c. Fr

- Step 2 Select a basin from Figure 7-A-5 that fits site geometry. Choose W_B /W_o, number of rows, N_r, N, h/y_A and L/h.

- Step 3. Determine:

  a. V_A

  b. y_A

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

  For $W_B /W_o <4$ use Figure 4-A-2 or Figure 4-A-3 for V_A and compute y_A by Equation 7-A-3.

  For slopes > 10 percent use Equation 7-A-3 to find both V_A and y_A

- Step 4. Determine dissipator parameters:

  . 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. $A_F = W_1 h$--element frontal area

  h. C_P --from Figure 7-A-4

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

- Step 5.

  a. If the downstream channel width is approximately equal to W_B, compute from Equation 7-A-1 or Equation 7-A-2.

  $C_B\ A_F\ N$

  Also compute $C_B\ A_F\ N$ from values in step 4 If the latter value is equal to or greater than value from Equation 7-A-1 or Equation 7-A-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 7-A-1 or Equation 7-A-2 and compare with downstream flow to determine controlling V_B. Compute y_B and compare with TW. If $TW > Y_B$ TW controls.

- Step 6. Sketch the basin:

. 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

Step 7. Determine riprap protection requirement downstream of basin. Chapter 2 provides guidance and Figure 2-C-1 design information.

Example Problem

Given:

2438 x 2438 mm Box culvert:

length = 71.6 meters
slope = 0.02
Q Design = 39.64 m^3/s.
Assumed n = 0.013
computed critical depth, y_C = 2.987 m
normal depth, y_n = 1.829 meters.

Natural channel:

width = 12.497 meters
Q = 39.64 m^3/s.

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 meters downstream of the basin could be assumed. Using the standard step method, a backwater profile was plotted and the tailwater depth determined as TW= 1.001 meter.

Find:

Design a CSU basin to provide a transition from the 2.438 m wide culvert to the 12.497 m wide natural channel and reduce velocities to approximately the downstream level.

Solution:

1. Culvert outlet flow conditions
   . $y_o = y_n = 1.829$ m Reference 3-1.
   2. $V_o = V_n = 8.870$ m/s
   3. $Fr = V_o / \sqrt{gy_o} = 8.870 / \sqrt{9.81(1.829)} = 2.1$

2. Select basin configuration Figure 7-A-5.

   Channel Width/Culvert Width=12.497/2.438=5.12

   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 2 x 2.438=4.876 m from culvert exit.
   . $V_A / V_o = 1.05$ from Figure 4-A-2.
   b. $y_A / y_o = 0.33$ Figure 4-A-4.
   $V_A = 8.870(1.05) = 9.314$ m/s.
   $y_A = 1.829(0.33) = 0.604$ meter

4. Determine dissipator parameters,

. h/y_A =0.71; h=0.71(0.604)=0.429 meters

b. L/h=6; L=6(0.429)=2.574 m

c. W_B /W_o =5; W_B =5(2.438)=12.190 m

d. W_1 /W_o =0.63; W_1 =0.63(2.438)=1.536 m; use 1.524 m (5 ft)

e. u_e=4/7+10L/7W_o =4/7+10(2.574)/(2.438)7=2.07 use 2

f. C_B =0.42

g. A_f =(1.524)(0.429)=0.65 sq. meters

h. C_P =0.7

i. L_B =2(2.438)+4(2.574)=15.172 meters.

5. Since channel and dissipator are approximately equal in width,12.190 m versus 12.497 m, the C_B A_F N value will be computed directly from Equation 7-A-1.

y_n Downstream=1.001 m

V_B =39.64/12.190(1.001)=39.64/12.178=3.255 m/s

$\rho V_o Q+C_p \gamma Yo^2 W_o /2=C_B A_F N \rho V_A{}^2/2+\rho V_B Q+ \gamma Q^2 /2V_B{}^2 W_B$

ρ=1000 kg/m^3

Terms with V_o and y_o: 1000(8.870) (39.64) + 0.7(9810) (1.829)^2 (2.438)/2 =379609

Terms with V_B : 1000(3.255) (39.64) +9810(39.64)^2 /2 (3.225)^2 (12.190) =189820

C_B A_F N (1000)(9.314)^2 /2=43375 C_B A_F N

(379609 - 189820 =43375C_B A_F N

C_B A_F N =4.40

From step 4

C_B =0.42

N =15

A_F =0.65

so, C_B NA_F =4.12 < 4.40

try 5-rows same h/yA and return to step 4.

. h=0.429 m

b. L=2.574 m

c. W_B =12.190 m

d. W_1 =1.524 m

e. u_e =2

f. C_B =0.38

g. A_F =0.654 m^2

h. C_p =0.7

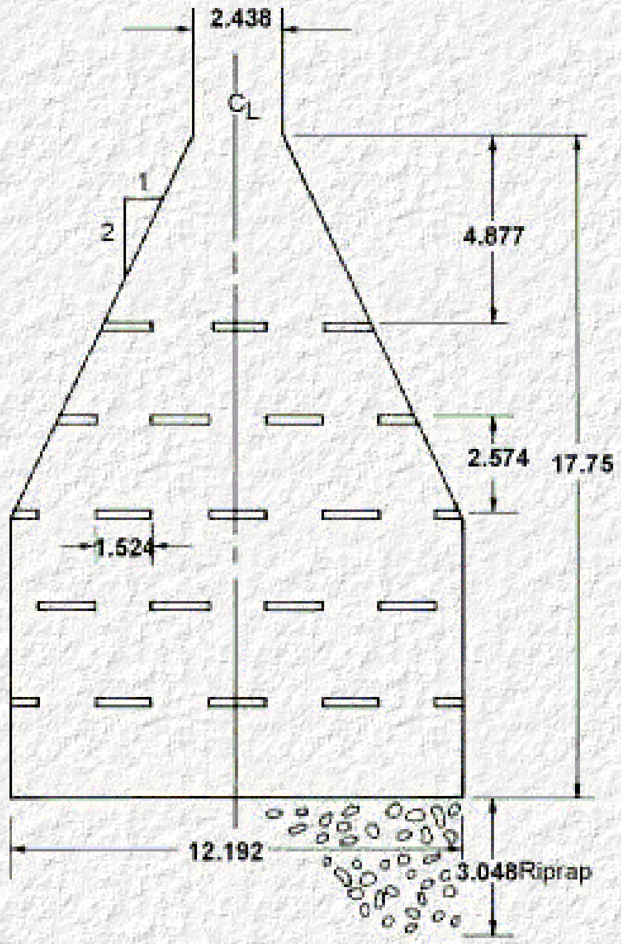
i. L_B =17.75 m

$C_B A_f N$ =0.654(19)(0.38)=4.72>4.40 o.k.

6. Sketch basin and distribute roughness elements.

W_1/h=1.524/0.429=3.55 between 2 and 8 o.k

7. Since the design matches the downstream conditions, minimum riprap will be required. From Figure 2-C-1, place stone with 0.213 meters mean diameter in a .457 meter layer for 3.05 meters downstream of dissipator exit. Design required filter from reference 52.



2.438
C_L
1
2
4.877
2.574
17.75
1.524
12.192
3.048Riprap

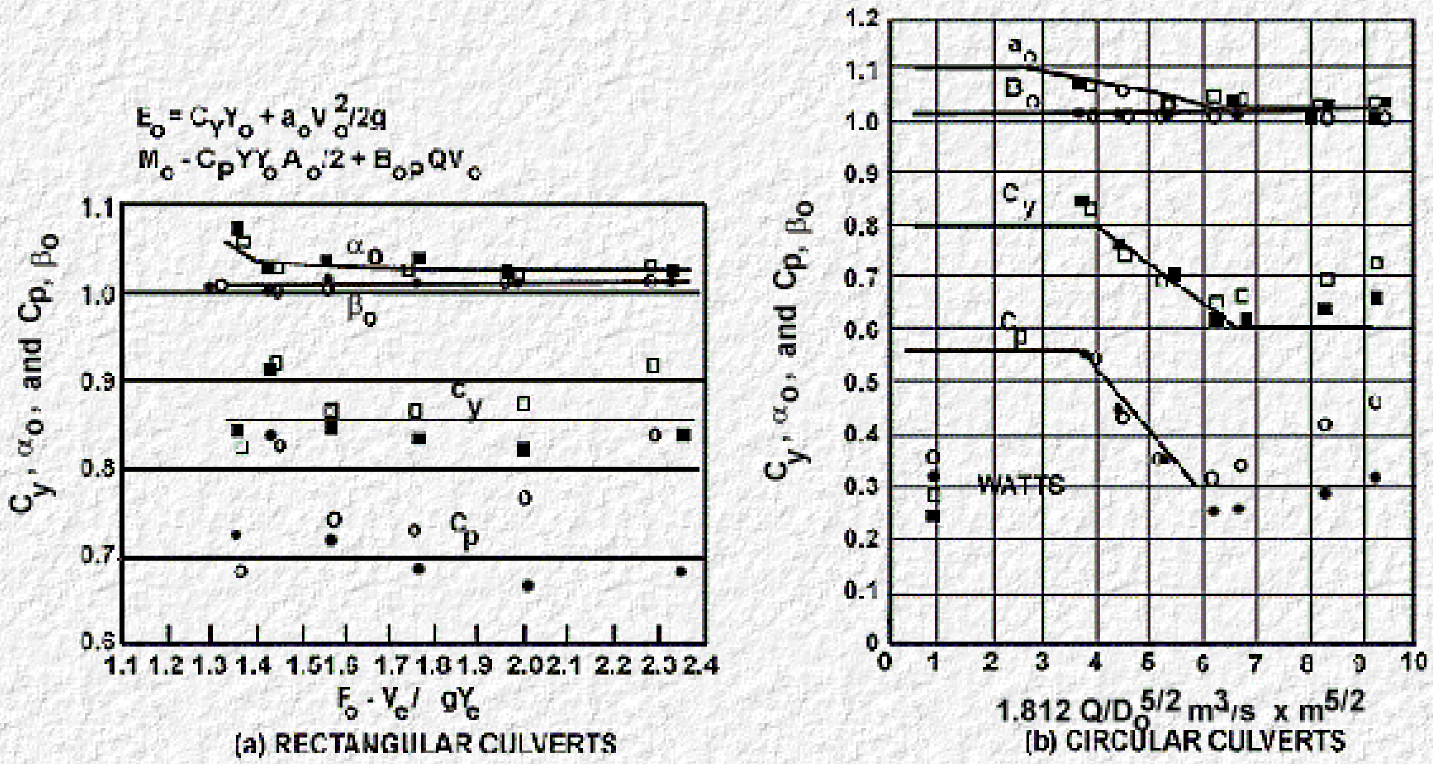
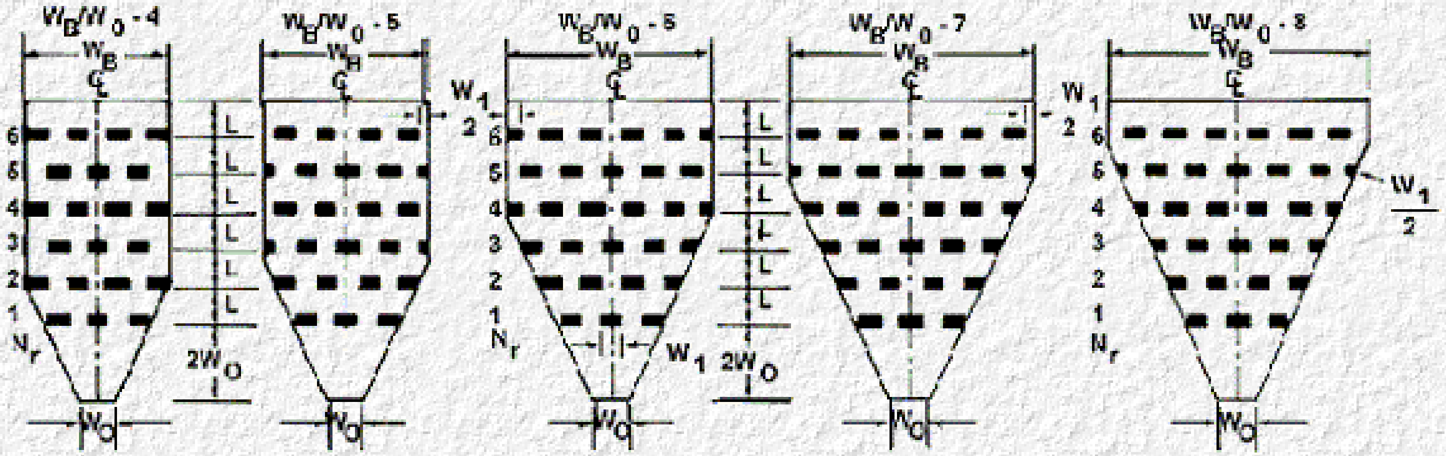


Figure 7-A-4. Energy and Momentum Coefficients (from Watts and Simons reference 7A1)



W_B/W_O			2 to 4			5			0			7		8
W_1/W_O			.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_B	L/h	C_B FOR ROUGHNESS ELEMENT DISSIPATORS											
RECTANGULAR	0.91	6	0.32	0.28	0.24	0.32	0.28	0.24	0.11	0.27	0.23	0.26	0.22	0.22
	0.71	6	0.44	0.40	0.37	0.47	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	0.16	0.38	0.35
	0.37	12	0.68	0.65	0.65	0.65	0.62	0.50	0.62	0.58	0.55	0.54	0.50	0.46
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	0.77	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	0.47	0.40	0.38	0.36	

Figure 7-A-5. Design Values for Roughness Element Dissipators

---

7-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 sometimes 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 (7B1).

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), (7B2, 7B3, 7B4, 7B5). Slopes up to 20 percent were tested at VPI and up to 35 percent in subsequent tests by the Federal Highway Administration (7B6).

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 provisions 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 predominant 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 tumbling 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 7-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.

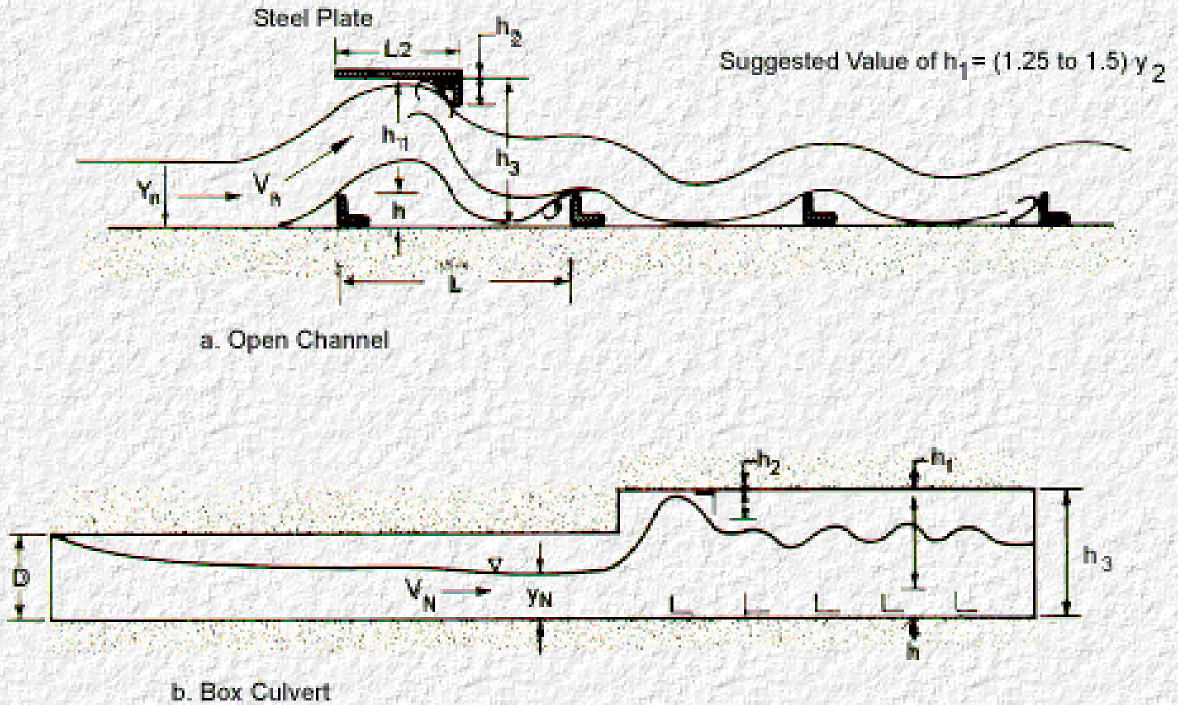


Figure 7-B-1. Splash Shield Definitive Sketch for Tumbling Flow

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

- Step 1. Check culvert control. If inlet control governs, tumbling flow may be a good choice for dissipating energy.
- Step2. 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 3

c. Normal depth y_n and velocity V_n Table 3-2 or Table 3-1

- Step 3. Select type of roughness configuration.

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

$$h = y_c /(3-3.7S_o)^{2/3} \qquad \text{7-B-1}$$

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 hydraulic jump:

$$y_2 = (Fr)y_n (S_o + 0.153)/0.133 \qquad \text{7-B-2}$$

where :

y_n and F_r are the approach conditions at the toe of the jump.

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

$$h_i > y_2 - y_C \qquad \text{7-B-3}$$

This element is followed by four smaller elements with a height computed by Equation 7-B-1.

- Step 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:

$$L_1 = 2h + \frac{q^{2/3}}{g^{1/3}}\left(\frac{\cos\Psi}{\cos\theta}\right)^2 \left\{(\tan\Psi\cos\theta + \sin\theta) + \left[(\tan\Psi\cos\theta + \sin\theta)^2 + \frac{2g^{1/3}}{q^{2/3}}\left(\frac{\cos\theta}{\cos\psi}\right)^2 h_i\cos\theta\right]^{1/2}\right\}$$

Where:

$y = \phi - \theta = 45^o$ is used.

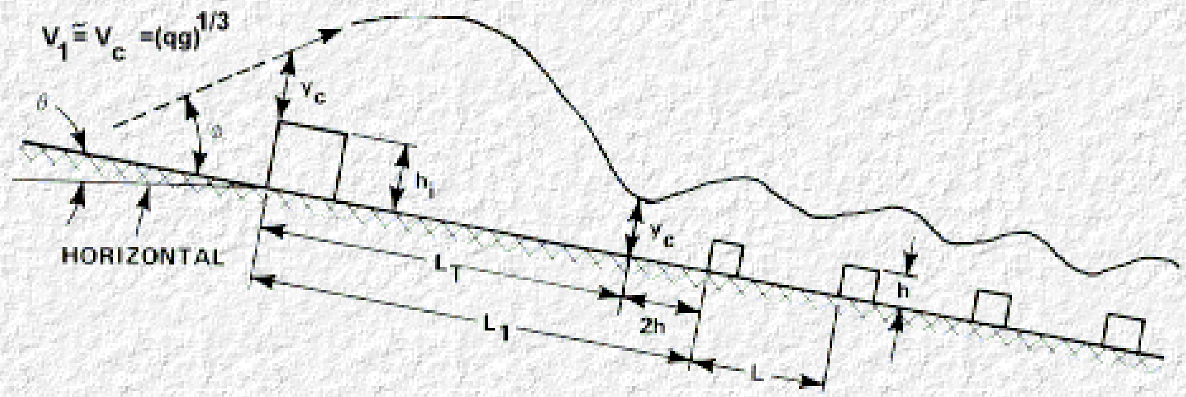


Figure 7-B-2. Trajectory for Flow over the Large Leading Element

Step 5. Slots may be provided in the roughness elements as shown in Figure 7-B-3. To pass fine sediment and to allow for drainage during low flow. The slot width (W_2) should be:

$W_2 = 0.5h$ 7-B-4

where:

h = the height of the small elements.

The slot configuration shown in Figure 7-B-2 is recommended.

$W_1 = (W - N_S\ W_2)/3$ 7-B-5

where:

N_S = the number of slots .

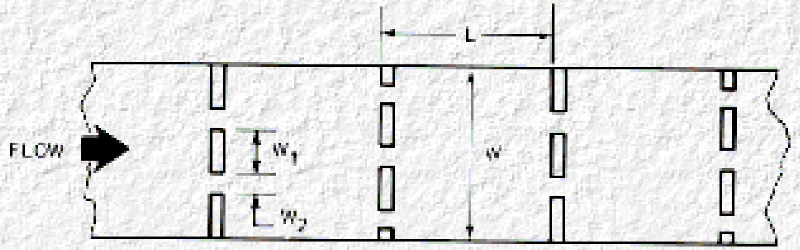


Figure 7-B-3. Definition Sketch for Slotted Roughness Elements

- Step 6. When the recommended roughness configuration is used, (see step 3a), a splash shield may be desirable.

  . For open channels use the splash shield as indicated on Figure 7-B-1 with the splash guard height (h_2) equal to 50 mm or as dictated by structural requirements. The length, position and height are shown on Figure 7-B-1.

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

  $$h_1 = 1.25 y_C \qquad \text{7-B-6}$$

  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 7-B-1b

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

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

- Step 7. The outlet velocity can be computed by:

  $$V_c = (qg)^{1/3} \qquad \text{7-B-8}$$

Design Procedure Summary

- Step 1. Check culvert control.

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

- Step 3. Compute h using Equation 7-B-1
  Compute y_2 for alternate design and $h_i > y_2 - y_c$.

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

- Step 5. Calculate W_1 and W_2 from Equation 7-B-4 and Equation 7-B-5 and arrange elements as indicated in Figure 7-B-3.

- Step 6. Splash shields:

  a. For open channels h_2 = 50 mm 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_i from Equation 7-B-6. If $D<h_1 +h$, make $h_3 =h_1 +h$. If $D>h_1 +h$ use Equation 7-B-7 to find h_2 required.

- Step 7. Calculate V_c using Equation 7-B-8.

Example Problem

Design a tumbling flow energy dissipator for:

0.610 meters wide concrete channel
Manning's n = 0.015
$Q = 0.566\ m^3/s$.
S_o = 10 %

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

- 2. $q = Q/W = 0.566/0.610 = 0.928\ m^3/s/m$
  $y_C =(q^2/g)^{1/3} = (0.928^2 /9.81)^{1/3} =0.444$ m
  V_C =2.088 m/s

y_n = 0.186 meters
V_n =4.999 m/s

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

  $h=y_C/(3.-3.7S_o)^{2/3}$
  =0.444/1.91=0.232 use 0.229 meters (0.75 ft)

- 4. Spacing L , using L/h =8.5

  L=8.5(0.229)=1.95 meters, use 1.98 meters (6.5 ft.)

- 5. Slots W_2

  W_2 =0.5(h)=0.5(0.229)=0.115, use 0.122 meters (0.4 ft)

  Segment width W_1
  1st, 3rd, and 5th rows W_1 = [0.610-2 (0.122)] /3 = 0.122 meters (0.4 ft)
  2nd, and 4th rows W_1 = [0.610 - 3 (0.122)] /3 = 0.081 meters, use 0.914 m (0.3 ft.)

- 6. h_2 = 0.050 m
  $h_1 = 1.25\ y_C$ =1.25 (0.444) = 0.555 m
  $h_3 = h + h_1$ = 0.229 + 0.555 = 0.784 m

  Splash shield length = L/2 = 1.98 /2 = 0.99 m

- 7. V_C at end of dissipator.
  $V_C=(qg)^{1/3} = (0.928\ x9.81)^{1/3}$ =2.088 m/s

Tumbling Flow in Circular Culverts

Tumbling flow in circular culverts can be attained by inserting circular rings inside the barrel, Figure 7-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(7B7,8). Their experiments for tumbling flow in circular culverts were run with a 152 mm plexiglass model and a 457 mm concrete prototype culvert. Slopes ranged from 0 to 25 percent, h/D_1 ranged from 0.06 to 0.15 and L/D_1 ranged from 0.3 to 3.0 (L/h from 5 to 20). The experimental variables are illustrated in Figure 7-B-4. The variables that determine whether or not tumbling flow will occur are: roughness height= h, spacing = L, slope = S_o, discharge = Q, and diameter = D_1.

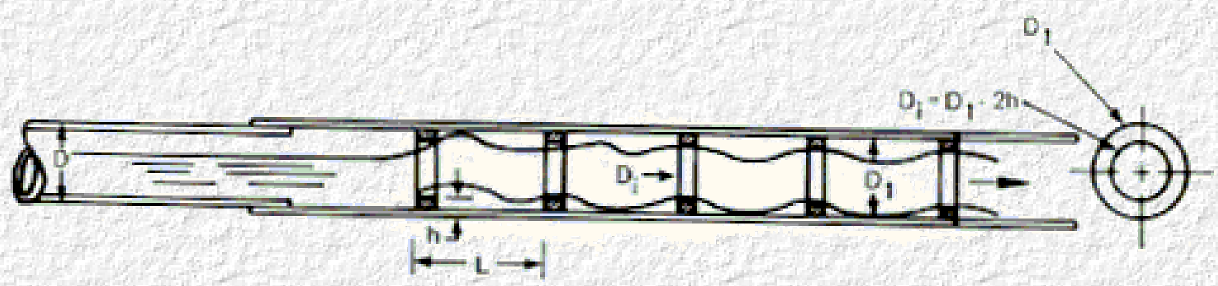


Figure 7-B-4. Definition Sketch for Tumbling Flow in Circular Culverts

A functional relationship for the roughness height can be described as:

$$h=f\ (L, S, Q, D_1, g) \qquad \text{7-B-9}$$

Establishing dimensionless groupings yields:

$$h/D_1 = f(L/D_1, S, Q/(gD_1^5)^{1/2} \qquad \text{7-B-10}$$

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 7-B-10 can be rewritten:

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

or

$$Q/(gD_1^5)^{1/2} = f(S) \qquad \text{7-B-11}$$

Theoretically f(S) in Equation 7-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 7-B. "Roughness Elements for Increased Flow Resistance." Equation 7-B-11 can be rewritten to yield

$$0.21<Q/(g\,D_1^{5})^{1/2} <0.32 \qquad \text{7-B-12}$$

or

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

Equation 7-B-12 is the basic design equation for tumbling flow in steep circular culverts. If the diameter of the roughened section of the culvert is sized according to this equation, tumbling flow will occur and the outlet velocity will be approximately critical velocity. Equation 7-B-12 is limited to the following conditions:

$L/D_1 \cong 2.0$ (tolerance $\pm$ 25%)

$h/D_1 \cong 0.125$ (tolerance $\pm$ 20%)

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 (7B9) 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 7-B-12 resulted from the 152 mm model data rather than from the 457 mm 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 tends to be self-cleansing. In their original list of possible applications, Peterson and Mohanty (7B1) 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 7-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 (7B7) 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 7-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, y_c, for the inside diameter of the roughness rings. Critical flow for an open channel of any shape will occur when

$$Q^2 T/gA_C{}^3 = 1 \qquad \text{7-B-13}$$

Referring to Figure 7-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 $y_c < r$:

$y_1 = r - y_c$
$\beta = \text{Same}$
$A_C = \pi r^2 \beta/360 - y_1 (r^2 - y_1{}^2)^{1/2}$
$T = 2 (r^2 - y_1{}^2)^{1/2}$

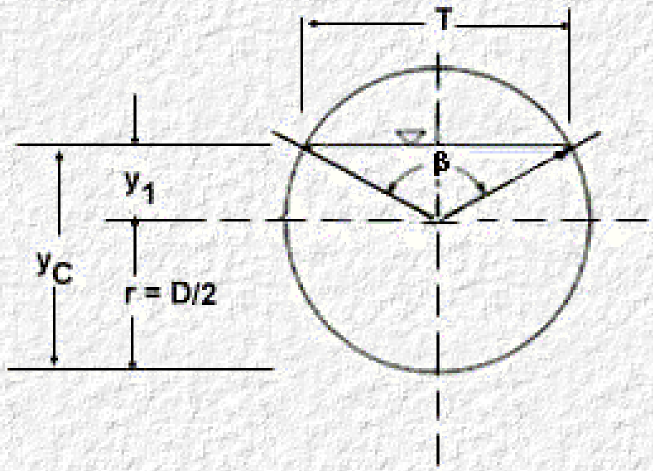


Figure 7-B-5. Definition Sketch for Critical Flow in Circular Pipes

Figure 3-4 can be used to determine critical depth using D_i.

Table 3-2 can be used to determine the critical area, A_C. Outlet velocity can be computed from:

$$V_C = Q/A_C$$

Design Procedure

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

- Step 2. Determine the Diameter, D_1, of the roughened section of pipe to sustain tumbling flow. Use Equation 7-B-12,

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

- Step 3. Compute h and L from:

$$h/D_1 = 0.125 \pm 20\%$$

$$L/D_1 = 2.0 \pm 25\%$$

- Step 4. Compute the internal diameter of the roughness rings:

$D_i = D_1 - 2h$

Step 5. Determine the critical depth, y_c, using: Figure 3-4 from design discharge for Q and D_1 for diameter.

Step 6. Compute y_c / D_i.

Step 7. Determine A_c from Table 3-2 use y_c / D_i for d/D and read A/D^2 which equals A_c / D_i^2

$$A_C = (A_C / D_i^2) D_i^2$$

Step 8. Compute the outlet velocity:

$$V_o \cong V_C = Q / A_C$$

Example Problem

Given:

1219 mm diameter culvert, 60.96 m long
n = 0.012, 6% slope
Q-Design = 2.266 m^3/s
V_o = 7.315 m/s
The Culvert is governed by inlet control.

Required:

Determine size and spacing of roughness elements for tumbling flow.

Solution:

1. Inlet control governs.

2. Equation 7-B-12

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

$1.4\ m < D_1 < 1.7$

Use $D_1 = 1.524$ m (5 ft)

3. Compute h and L.

$h/D_1 = 0.125 \pm 20\%$

$h=0.125(1.524)=0.191$ m

$0.15<h<0.23$ Use h = .178 m (7 inches)

$L/D_1 = 2 \pm 25\%$

$L=3.048$

2.9 m < L<3.8 m, Use L=3.048 m (10 ft.)

4. Compute D_i.

$D_i = D_1 - 2h = 1.524 - (0.382) = 1.142$ m

5. Determine y_c from Figure 3-4 .

$y_c = 0.853$ m

6. Compute y_c/D_i.

$y_c/D_i = 0.853/1.142 = 0.747$ m =d/D for Table 3-2

7. Determine A_c from Table 3-2.

$A/D^2 = A_c/D_i^2 = 0.629$

$A_c = 0.629(1.142)^2 = 0.820\ m^2$

8. Compute the outlet velocity.

$V_c = Q/A_c = 2.266/0.820 = 2.763$ m/sec

This is a reduction from V=7.315 m/s in the original culvert of:

100(7.315 - 2.763)/7.315 =62%

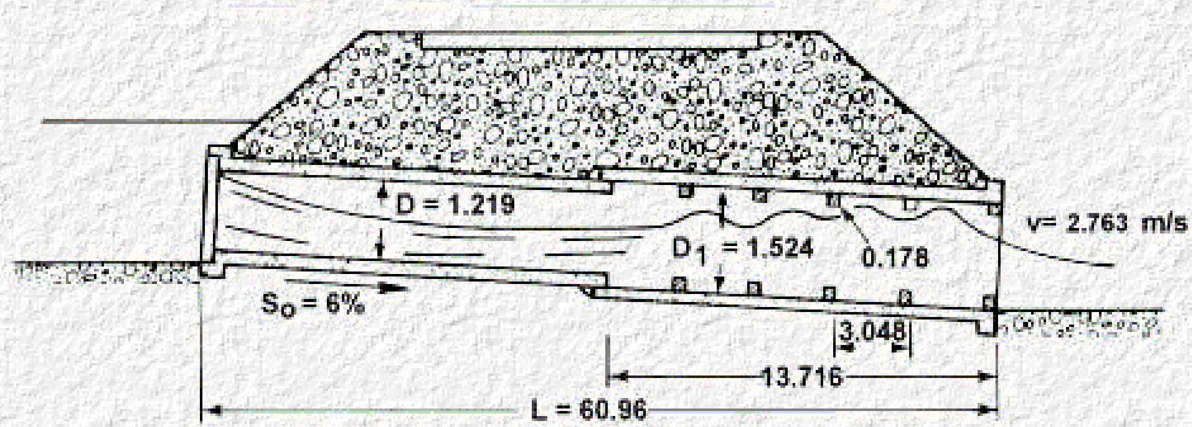


Figure 7-B-6. Sketched Solution for the Tumbling Flow Design Example for Circular Culverts

7-1. Horner, S. F.,
Fluid Dynamic Drag
Published by author, 2 King Lane, Greenbriar, Bricktown, N.J. 08723, 1965.

7A1. 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.

7A2, Morris, H. M.,
Hydraulics Of Energy Dissipation In Steep Rough Channels,
Virginia Polytechnical Institute Bulletin 19, VPI & SU, Blacksburg, VA., Nov. 1968.

7A3, Jones. J. S.,
FHWA In-House research

7B1. Peterson, D. F. & P. K. Mohanty,
Flume Studies of Flow In Steep Rough Channels,
ASCE Hydraulics Journal, HY-9, Nov. 1960.

7B2. Morris, H. M.,
Hydraulics Of Energy Dissipation In Steep Rough Channels,
Virginia Polytechnical Institute Bulletin 19, VPI & SU, Blacksburg, VA., Nov. 1968.

7B3. 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.

7B4. Mohanty, P. K.,
The Dynamics Of Turbulent Flow In Steep, Rough, Open Channels,
Ph.D. Dissertation,
Utah State University, Logan, Utah, 1959.

7B5. Morris,
Bulletin 19,
pp. 50, 54 & 56.

7B6. Jones. J. S.,
FHWA In-House research

7B7. 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.

7B8. Culvert Velocity Reduction By Internal Energy Dissipators,
Concrete Pipe News, pp. 87-94,
American Concrete Pipe Association, Arlington, VA, Oct. 1972.

7B9. Watts, F. J.,
Hydraulics of Rigid Boundary Basins,
Ph.D. Dissertation,
Colorado State University, Fort Collins,
Colorado, August 1968

Go to Chapter 7, Part II

Chapter 7 : HEC 14

Forced Hydraulic Jump Basins
Part II

Go to Chapter 7, Part III

7-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 Section 7-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.

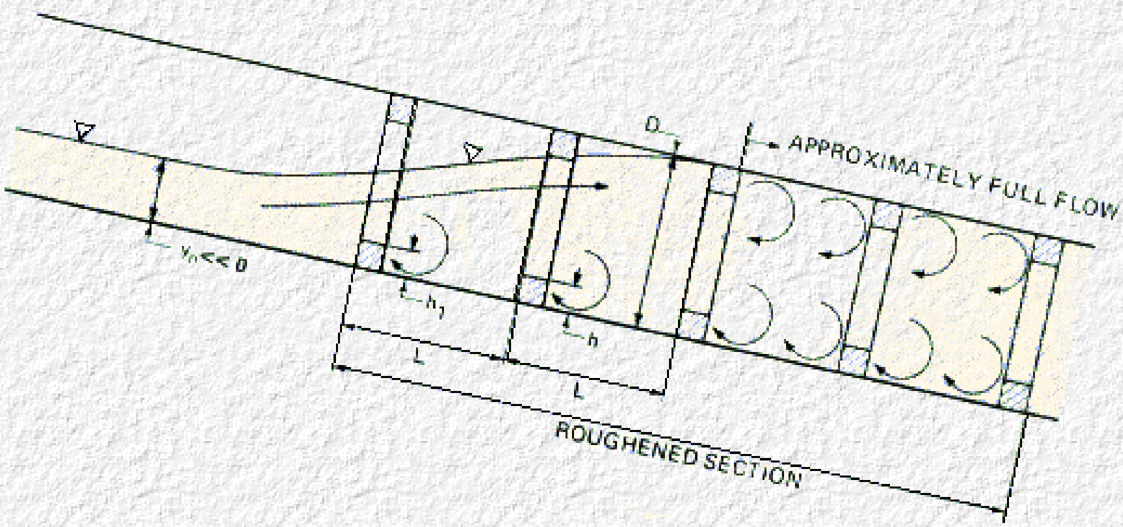


Figure 7-C-1. Conceptual Sketch of Roughness Elements to Increase Resistance

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:

$$Q=AV=\pi(D^{2/4})(1/n)\,(D/4)^{2/3}S_f^{1/2}=(0.31/n)D^{8/3}S_f^{1/2} \qquad \text{7-C-1}$$

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 7-C-3, to compute partial flow parameters. Designing roughness elements is basically a matter of manipulating Equation 7-C-1 and Figure 7-C-3 in conjunction with empirical graphs for determining "n" in roughened pipes.

Wiggert and Erfle (7B7) 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 establish 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 (7B8) 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 (7C4) 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:

1. Quasi-smooth flow-Occurs only when there are depressions or when roughness elements are spaced very close ($L/h \cong 2$).Quasi-smooth flow is not important for this discussion.
2. Hyper-turbulent 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.
3. 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 7-C-2.

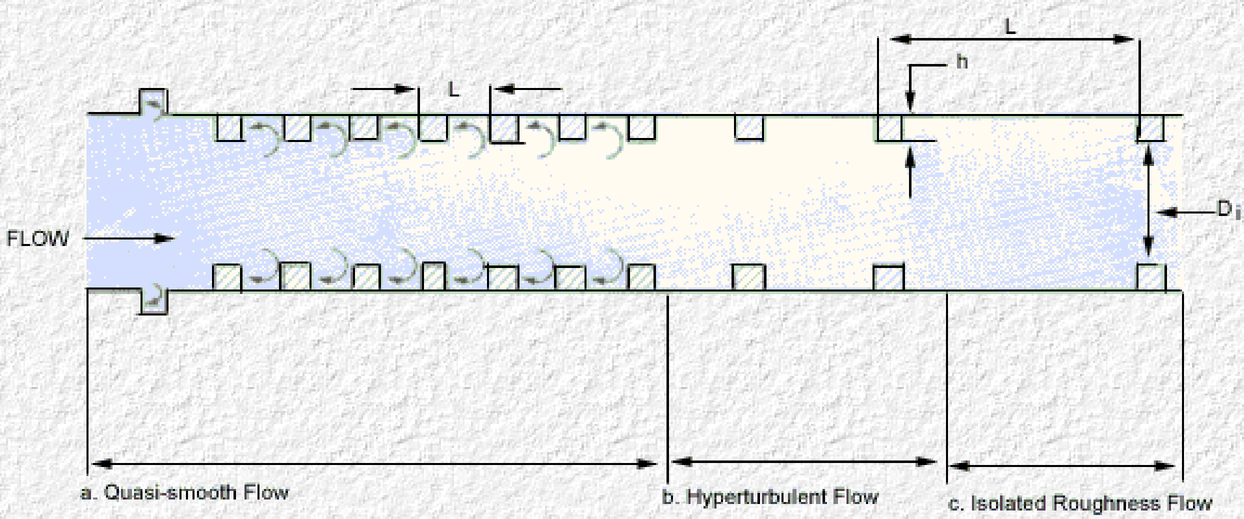


Figure 7-C-2. Flow Regimes in Rough Pipes

Isolated-Roughness Flow

The overall friction or resistance, f_{IR}, is made up of two parts:

$f_{IR} = f_s + f_d$ 7-C-2

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 spacing and the velocity impinging on the roughness elements. Morris related the velocity to surface drag and derived the following equation:

$$f_{IR} = f_s\,[1+67.2C_D\,(L_r/P)(h/r_i)(r_i/L)] \qquad \text{7-C-3}$$

where:

C_D = drag coefficient for the roughness shape,

L_r/P = ratio of total peripheral length of roughness elements to total wetted perimeter,and

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 7-C-3 can be converted to a Manning's "n" expression as follows:

$$f_s = 123.96\,(n/D^{1/6})^2$$
$$f_{IR} = 123.96\,(n_{IR}/D_i^{1/6})^2$$
$$n_{IR} = n\,(D_i/D)^{1/6}\,[1+67.2\,C_D\,(L_r/P)(h/L)]^{1/2} \qquad \text{7-C-4}$$

where:

n_{IR} =overall Manning's 'n" for isolated roughness flow.
n = Manning's "n" for the culvert surface 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 7-C-4 is a graphical solution to Equation 7-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

$$1/\sqrt{f_{HT}} = 2\log_{10}(r_i/L) + 1.75 + \phi \qquad \text{7-C-5}$$

where:

f_{HT} = overall friction for hyper-turbulent flow.
ϕ = function of Reynolds number, element shape, and relative spacing

By restricting application of Equation 7-C-5 to sharp edged roughness rings and to spacing greater than the pipe radius, ϕ can be neglected.

Substituting:

$$D_i^{1/6} / n_{HT} = \sqrt{123.96 / f_{HT}}$$

into Equation 7-C-5 and rearranging terms yields:

n_{HT}=0.0898 $D_i^{1/6}$/(1.75 - 2$\log_{10}$ L/r_i) 7-C-6

The effect of the roughness height, h, is included inherently in D_i. From Figure 7-C-5, it can be seen that n_{HT} increase as the spacing increases for this regime.

Regime Boundaries

Since resistance increases when the spacing increases for the hyper-turbulent 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 7-C-3 to f_{HT} in Equation 7-C-5. All the boundary curves in Figure 7-C-6 are based on sharp-edged rectangular roughness elements (C_D=l.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

- Step 1. Compute 0.820 n/$D^{1/6}$, where "n" is Manning's coefficient for smooth culvert and "D" is the diameter.

- Step 2. Select L/D_i in the range 0.5 to 1.5. (1.0 is suggested as a starting point)

- Step 3. Select h/D_i in the range 0.05 to 0.10. Use sharp edged roughness rings.

- Step 4. Determine the flow regime from Figure 7-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 0.820 n/$D^{1/6}$ value. If the point is below, the flow is hyper-turbulent" (H.T.). Isolated roughness flow is the most common for large culverts.

- Step 5. Determine the rough pipe resistance (n_r =n_{IR} or n_{HT})

  . For isolated roughness flow obtain (n_{IR}/n) from Figure 7-C-4 or from Equation 7-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 7-C-4 must be used since Figure 7-C-4 is based on L_r/P=1.0.

  2. For hyper-turbulent flow obtain n_r=n_{HT} from Figure 7-C-5 directly or from Equation 7-C-6.

- Step 6. Compute the crest to crest roughness ring diameter,

  D_i =D-2h=D/(1+2h/D_i)

- Step 7.Compute full flow characteristics based on D and n:

  Q(FULL)=(0.31/n_r)$D_i^{8/3}$ $S_o^{1/2}$

and $V(FULL)=(0.40/n_r)D_i^{2/3} S_o^{1/2}$

Step 8. Determine outlet velocities:

. 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 headwater 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.

3. If Q(FULL) is greater than Design Q, use Figure 7-C-3 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)

Step 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:

. 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 $n=0.40 D_i^{2/3}So^{1/2}/(V(DESIRED)/1.15))$ and using an estimated value of D_i slightly greater than expected. With n_r known, selecting a corresponding h/D_i from Figure 7-C-4 or Figure 7-C-5 is relatively straightforward.

2. 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 above in the rough section and repeat steps 1 through 8.

3. Use a tumbling flow design as described in Section 7-B.

4. Use another type of dissipator either in lieu of or in addition to the roughness rings.

Step 10. Determine the size and spacing of the roughness rings.

. $D_i = D/(1+2(h/D_i))$

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 7-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:

1200 mm Culvert flowing under inlet control.

Diameter = 1.219 m
Design Q = 2.830 m^3/s
n = 0.012
Slope = 4%
Length = 60.960 m

Also

Q(FULL) = 8.892 m^3/s
V(FULL) = 7.620 m/s
Q(DESIGN)/Q(FULL) = 2.830/8.892 = 0.32
V(DESIGN)/V(FULL) = 0.88 (from Figure 7-C-3)
V_o= 6.706 m/s
y_o= 0.457 meters

Find:

the size and spacing of roughness element and the diameter of an enlarged end section (if required) to reduce the outlet velocity to 4.572 m/s (15 ft/s).

Solution:

1. Compute 0.820 $n/D^{1/6}$.
 0.820 $n/D^{1/6}$ = 0.820/(0.012)/$1.219^{1/6}$ =0.0095

2. Select L/D_i =1.5.

3. Try h/D_i =0.05

4. From Figure 7-C-6, the regime is isolated roughness flow. (0.820 $n/D^{1/6}$ is less than the point defined by the h/D_i and L/D_i ratios).

5. Determine rough pipe resistance from Figure 7-C-4.

n_r /n=2.3
n_r =(2.3)0.012=0.0276

6. Determine D_i :

D_i =D/(1+2h/D_i)=1.219 /(1+0.10)=1.108 m

7. Full flow computations for the rough pipe.

Q(FULL) = (0.31/.0276)$(1.108)^{8/3}$ $\sqrt{.04}$ = 2.954 m^3/s
V(FULL) = 3.063 m/s

8. Compare full flow with design flow.

Q(DESIGN)/Q(FULL)=2.830/2.954=0.96
V/V(FULL)=1.14 from Figure 7-C-3.
V(OUTLET)=1.14(3.063)=3.492 m/s < 4.572 which meets design conditions

9. The roughness size could be reduced slightly since velocities up to 4.6 m/s can be tolerated. From Figure 7-C-3 it can be seen that:

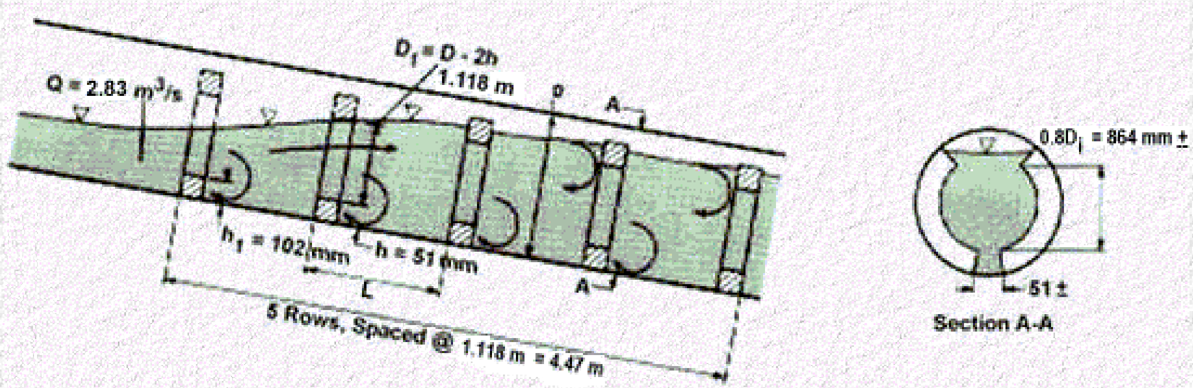
$\max(V/V(FULL)) \cong 1.15$;

The resistance, n, could be estimated such that
$V = 4.572$ m/s and $V(FULL) = 4.572/1.15$: i.e.
$n_r = 0.40\, D_i^{2/3}\, S_o^{1/2} / (4.572/1.15)$

Using $D_i = 1.158$, $n_r = .022$; 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 7-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:

$D_i = D/(1+2h/D_i) = 1.219/1.1 = 1.108$ m
$h = (h/D_i)D_i = (.05)1.108 = .0.055$ m use 51 mm (2 in)
$L = (L/D_i)D_i = (1.5)1.108 = 1.662$ m use 1.5m -1.676 m
$h_1 = 2h =$ 102 mm or .10 m
Use five roughness rings.



Sketch of Design Example For Increased Resistance In Circular Culverts

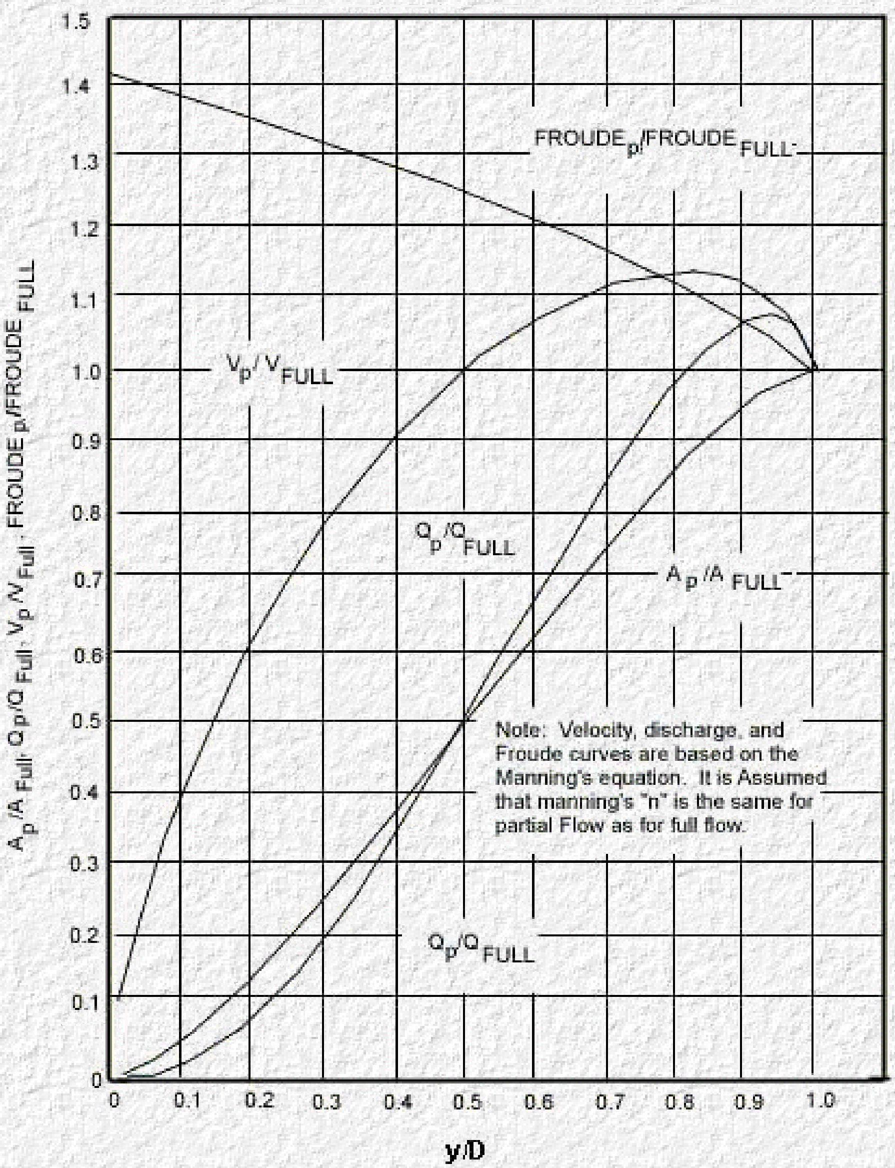


Figure 7-C-3. Hydraulic Elements Diagram for Circular Culverts Flowing Part Full

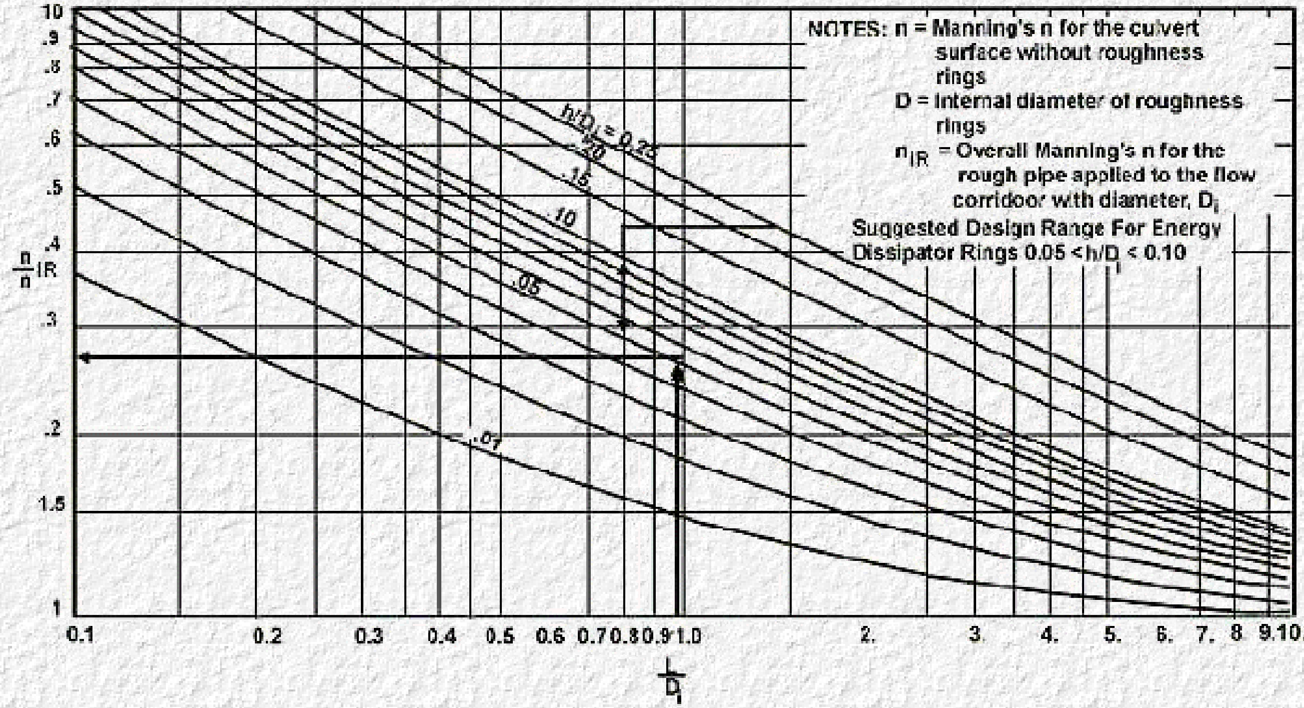


Figure 7-C-4. Relative Resistance Curves for Isolated Roughness Flow

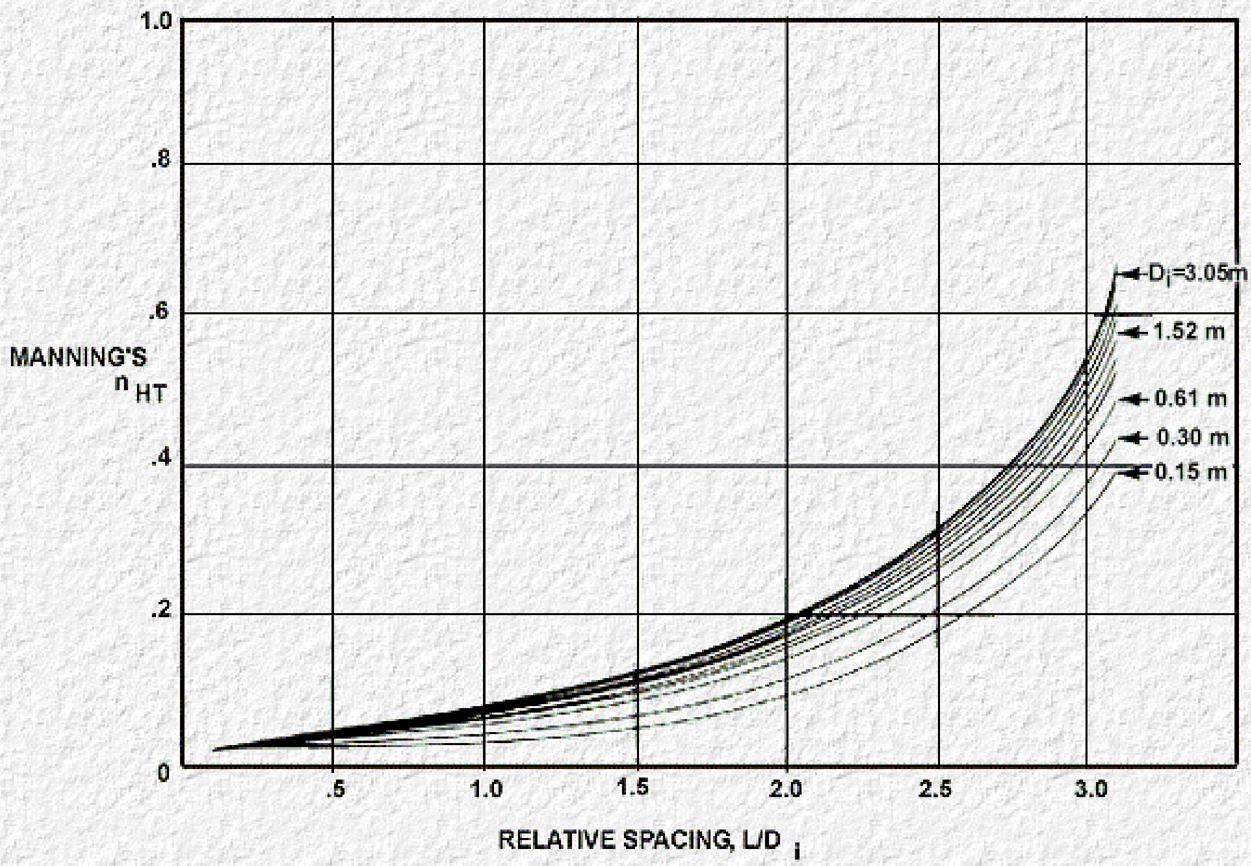


Figure 7-C-5. Resistance Curves for Hyperturbulent Flow

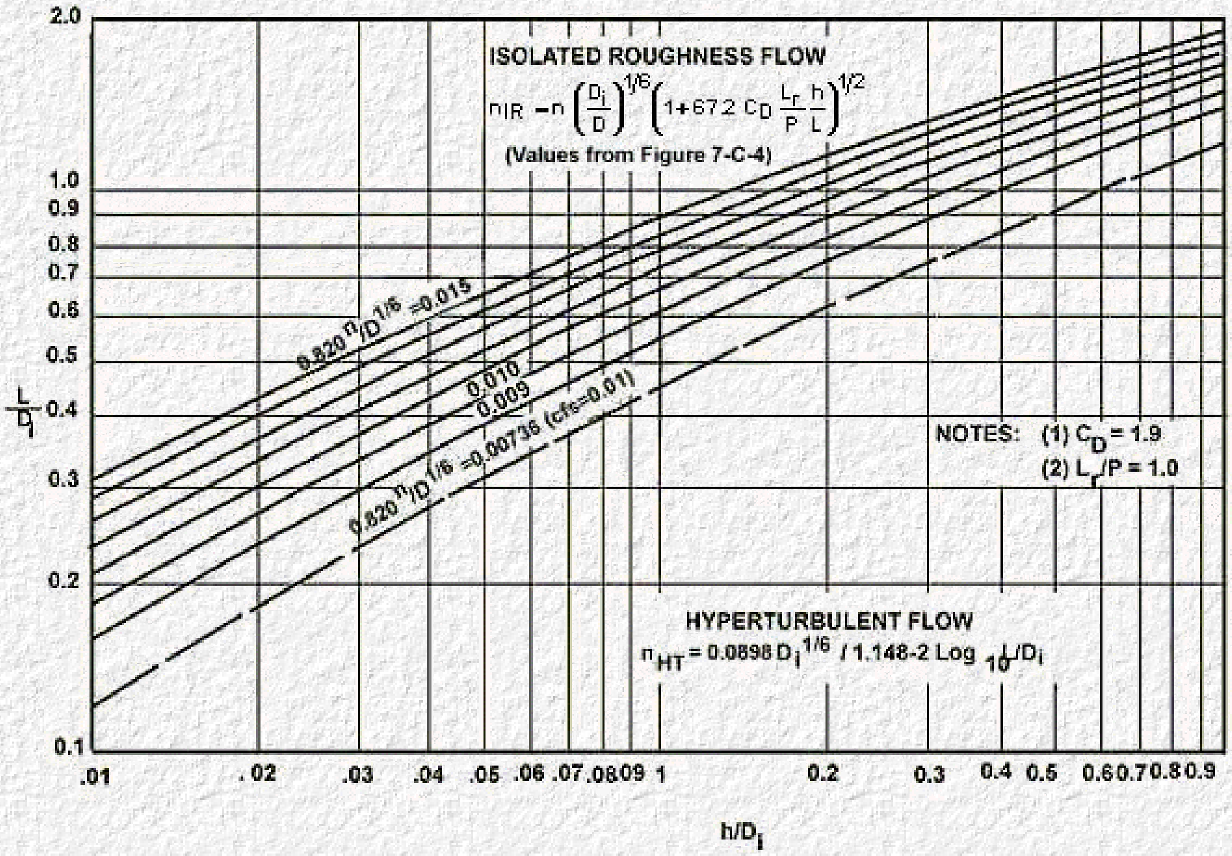


Figure 7-C-6. Flow Regime Boundary Curves

Increased Resistance in Box Culverts

Material for this section was drawn primarily from a preliminary FHWA report on fish baffles in box culverts (7C1). This report used Morris' (7C4) 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 (7C2) 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 7-C-7.

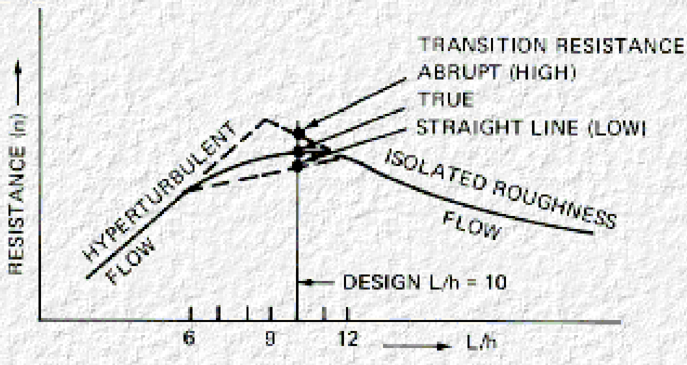


Figure 7-C-7. Transition Curves between Flow and Regimes

Observations by Powell (7C3) 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 the upper and lower design curves through a procedure similar to the one shown in Figure 7-C-7. The upper and lower design curves can be used to conservatively compute resistance for culvert sizing and outlet velocities.

$(n_{IR}/n)_{LOW} = [1+ 200\ (h/L)(L_r/P)]^{1/2}$ 7-C-7

$(n_{IR}\ /n)_{HIGH}=[1+390(h/L)(L_r\ /P)]^{1/2}$ 7-C-8

$(n_{HT}/n) = \{(1-L_r/P)+70.6(L_r/P)/[2\log(R_i/h)(h/L)+1.75]^2\}^{1/2}$ 7-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 7-C-7 and the higher value in Equation 7-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 equations and since roughness elements may be relatively small using this method, it is necessary to compute the length of the 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 not exceed 6 percent.

Design Procedure

Note: Steps 1-7 are concerned with computing outlet velocity to evaluate scouring potential and/or to design additional outlet protection.

- Step 1. Use $L/h=10$

- Step 2. Select h/R_i from 0.10 to 0.40

- Step 3. Computer L_r/P where: L_r = peripheral roughness length including slots. $L_r/P=1$ when roughness length extends through the flow or when the culvert is flowing full with a roughness length equal to the circumference. P = the total wetted perimeter of the culvert.

Slots are provided for low flow drainage. Their width should not exceed h/2. In this step assume the culvert flows full; so

$P=2(B+D)$

and

$L_r=B$ for bottom roughness only.

- Step 4. Determine (n_r/n) from the lower set of curves of Figure 7-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.

- Step 5. Determine the flow depth, y_i, measured from the roughness element crests:

  a. Assume a value of h from $h \cong (h/R_i)BD/2(B+D)$
  b. Assume a trial value of y_i. Initially assume $y_i=D-h$. Compute $C=S_o^{1/2}/n_r$.
  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:

     $Q=[(1.0/n_r)S_o^{1/2}]A_i R_i^{2/3}$

     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.

- Step 6.

  1. Compute the velocity, V, using A_i from the last iteration.

     $V=Q/A_i$

  2. 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.

- Step 7. Compute the required number of rows of roughness elements from the momentum equation written as follows:

$$0.5\gamma B y_n^2 + \rho Q V_n = (N) C_D A_f \rho V_w^2/2 + 0.5\gamma B y_i^2 + \rho Q V_i \qquad \text{7-C-10}$$

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.
ρ =1000 kg/m^3
V_w =average wall velocity acting on the roughness elements
≅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.

Step 8. Determine (n_r /n) from the upper set of curves of Figure 7-C-8, using L_r/P from step 3. Compute "n_r."

Step 9. Check adequacy of culvert height and compute flow depth if necessary by trial and error:

. Compute h using R_i from step 5.

$h=(h/R_i)R_i$

b. Try y_i =D-h Compute

$C = S_o^{1/2}/n_r$

c. Compute A_i and R_i.

$A_i = By_i$
$Ri = A_i/2(B+y_i)$

d. Compute "Q" from the Manning equation.

$Q=[(1/n_r)S_o^{1/2}]A_i R_i^{2/3}$

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.

Step 10. Use the last value of D as the height of the culvert for the roughened section.

Step 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:

1.219 m x 1.219 m box culvert, 61.0 meters long

n=.013, 6% slope

Q(DESIGN) 2.830 m^3/s
Allowable outlet velocity=4.572 m/s
$V_o=V_n$ =6.797 m/s
$y_o=y_n$ =0.341 m

Solution:

- 1. Use L/h=10.

- 2. Select h/R_i =0.10; use bottom roughness only.

- 3. L_r =B=1.219 m
  P=2(B+D)=4.876 m
  L_r /P=.25

- 4. (n_r /n)=2.25 from the lower set of curves of Figure 7-C-8,
  n_r =2.25x.013=0.029.

- 5. Flow depth is:
  . h $\cong$ 0.10(1.219 x 1.219)/2(1.219 +1.219)=0.030 m
  b. Try y_i =1.219 - 0.030=1.189 m, C=8.45 m
  c. A_i =1.219(1.219)=1.486 m^2
  R_i =1.486/(1.219+2.378)=0.413 m
  d. $Q=[(1/n_r)S_o^{1/2}] A_i R_i^{2/3}$ =[8.45](1.486)(0.554)= 6.956 m^3/s >2.830 m^3/s
  e. try smaller y_i

Trial y_i	A_i	R_i	Q
1.189			6.959
0.762	0.929	0.339	3.816
0.610	0.744	0.305	2.848

- 6. $V=Q/A_i$ =2.830/0.744=3.804 m/sec < 4.572 m/s OK.

- 7. $0.5(9810)(1.219)(0.341)^2 + 1000(2.830)(6.797) =$
  $N(1.9)(1.219)(0.030)(1000)[(6.797+3.804)/6^2]/2$
  $+0.5(9810)(1.219)(.610)^2/2 + 1000(2.830)3.804$

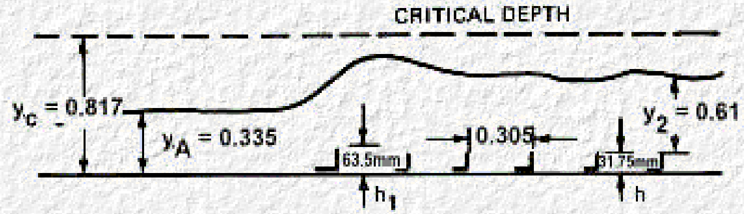
  695.27 +19235.51 =108.45N+2224.86 +10765.32
  N=64
  use 64 rows

- 8. (n_r /n)=2.5 from the upper curves Figure 7-C-8.
  n_r =2.5x0.013=0.033

- 9. a. h=0.305(3)=0.030 m
  b. Try y_i =1.219-(0.03)=1.189, C=7.42
  c. A_i =1.219(1.189)=1.449
  R_i =1.449/(1.219+2.378)=0.403
  d. $Q=[7.42](1.449)(0.403)^{2/3}$ =5.866 m^3/s
  e. Q>Q(DESIGN); therefore the culvert size is adequate.

- 10. D=1.219 m

- 11. h=0.03, use 31.75 mm angles
  L=10h=0.30 m; use 0.30 m (1 foot)
  h_1 =2h; use 63.5 mm angles



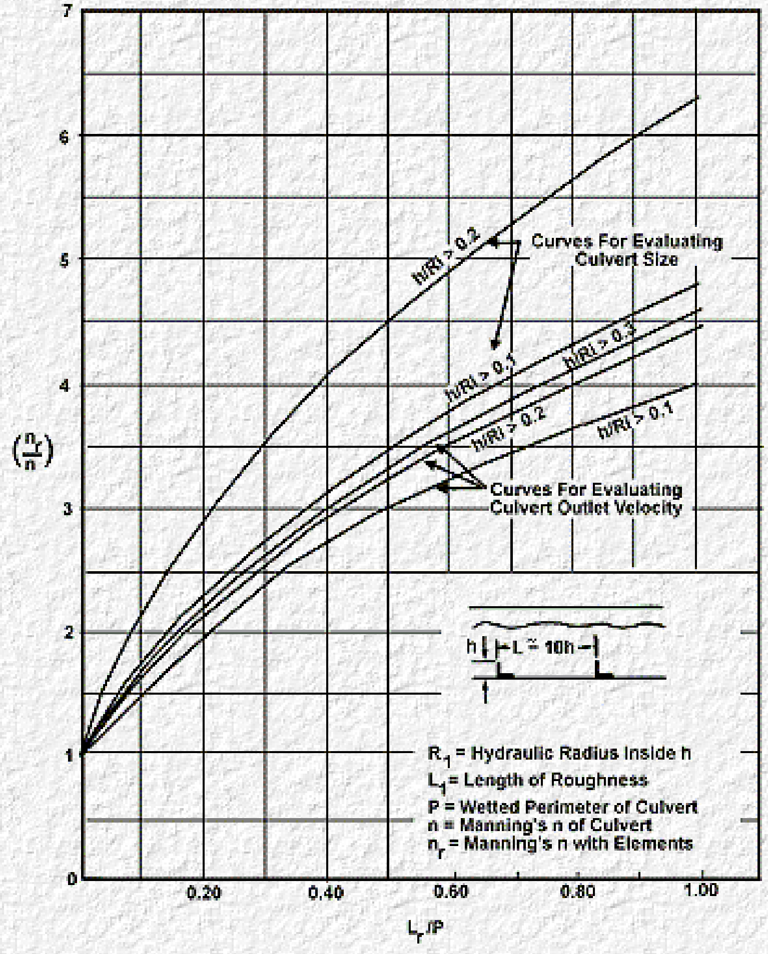


Figure 7-C-8. Relative Resistance Curves for Sharp Edged Rectangular Roughness Elements Spaced at L/h=10

7C1. Normann, J. M.,
Hydraulic Aspects of Fish Ladder Baffles in Box Culverts,
Preliminary FHWA Hydraulics Engineering Circular, Jan. 1974.

7C2. Shoemaker, R. F.
Hydraulics of Box Culverts With Fish Ladder Baffles,
Proceedings of the 35th Ann. Meeting of the HRB,
Washington, D.C., 1956, pp.196-209

7C3. Powell, R. W.,

Flow in A Channel of Definite Roughness,
ASCE Transactions, Figure 10, p. 544, 1946.

7C4. Morris, H. M.,
Applied Hydraulics In Engineering,
The Ronald Press Company, New York, 1963.

Go to Chapter 7, Part III

Chapter 7 : HEC 14

Forced Hydraulic Jump Basins
Part III

Go to Chapter 8

7-D USBR Type II Basin

The Type II basin was developed by the United States Bureau of Reclamation (7D1). The design is based on model studies and evaluation of existing basins.

The basin elements are shown in Figure 7-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 5 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 7-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 7-D-2.

The height of the chute blocks (h_1) is equal to the depth of the incoming flow, y_1, Figure 7-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 2H:1V. 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 perceptible effect on stilling basin action. Their test slopes varied from 0.6H:1V to 2H:1V. If the chute slope is 2H:1V or greater, a reasonable radius curve should be incorporated into the chute design, see Figure 4-B-5.

The length of the basin (L_B) may be obtained from Figure 7-D-3.

These design recommendations will result in a conservative stilling basin for flows up to 46.45 m^3/s per meter of basin width.

Design Procedure

Step 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 4-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E, use C =1.1 or Figure 7-D-2 to find y_2. For step 5F, use Figure 7-D-3 to find L_B.

Step 2. The chute block height (h_1), width (W_1), and spacing (W_2) are all equal to the incoming depth.

$$w_1 \cong w_2 \cong h_1 = y_1$$

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 = $W_1/2$

Step 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.

$$h_2 = 0.2y_2$$
$$W_3 \cong W_4 \cong 0.15y_2$$

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 example problem in Section 4.6.3, Supercritical Expansion.

> 3048 mm x 1829 mm (10' x 6') RCB,
> $Q = 11.809\ m^3/s$, $S_o = 6.5\%$
> Elevation outlet = 30.480 m (100 ft)
> $V_o = 8.473$ m/s, $y_o = .457$ m
> Downstream channel is a 3.048 m bottom trapezoidal channel with 1V:2H side slopes and n=0.03.

Find: Dimensions for a USBR Type II basin

Solution:

1. Determine basin elevation using design procedure outlined in Section 4-B, Supercritical Expansion Into Hydraulic Jump Basins.

Steps from Section 4.B:

1. $V_o = 8.473$ m/s, $y_o = 0.457$ m, $Fr_o = 4$
2. In channel $TW = y_n = 0.579$ m, $V_n = 4.849$ m/s
3. $y_2 = c_1 y_1[\sqrt{1+8Fr^2} - 1]/2 = (1.1)(0.457)[\sqrt{1+8(4)^2} - 1]/2$ =2.603 m
4. y_2 >TW, 2.603 > 0.579 drop the basin.
5. Use $z_1 = 25.756$ m (84.5 ft) $= z_2$
   . $W_B = 3.048$ m, $S_T = S_s = 0.5$
   B. W_B - OK no flare
   C. $Q = y_1(3.048)[2g(30.480-25.756+0.457-y_1)+8.437^2]^{1/2}$
      $Q = 3.048y_1\ [19.62(5.181-y_1)+71.183]^{1/2}$
      $y_1 = 0.299$ m OK
      $V_1 = 11.809/0.299(3.048) = 12.958$ m/s
   D. $Fr = 12.958/\sqrt{g(0.299)} = 7.57$
   E. For $C_1 = 1.1$, $y_2 = 1.1(0.299)\left[\sqrt{1+8(7.57)^2} - 1\right]/2 = 3.360$ m

F. From Figure 7-D-3 $L_B/y_2 = 4.3$, $L_B = 14.448$ m
$L_T = (z_o - z_1)/S_T = (30.480-25.756)/0.5 = 9.448$ m
$z_3 = [30.480-(14.448+9.448-25.756/0.5)0.065]/1.13$
$z_3 = 28.563$ m

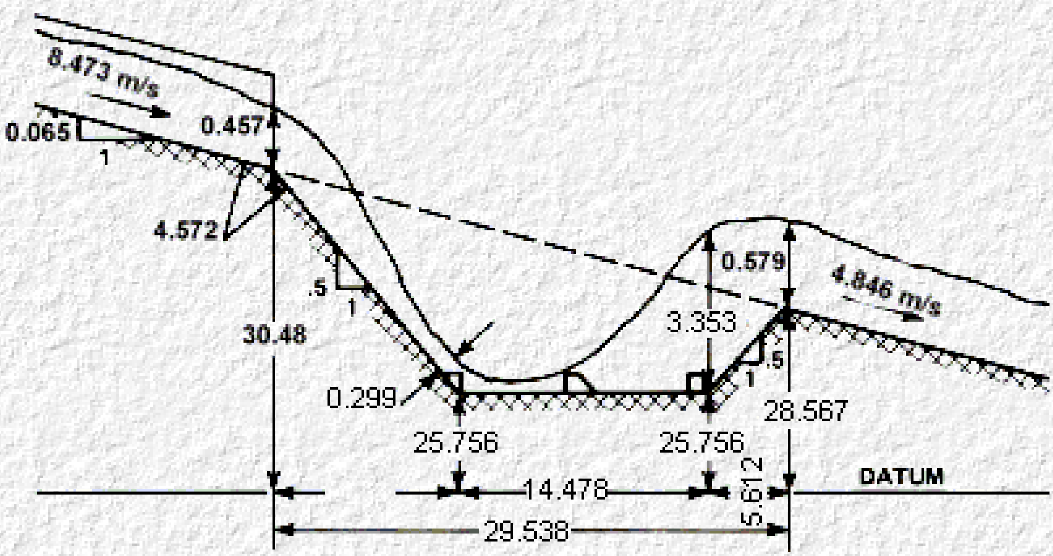
G. $y_2 + z_2 = 29.116$ m
$z_3 + TW = 29.147$ m OK

6. $L_T = 9.448$ m, $L_B = 14.478$ m
$L_S = (z_3 - z_2)/S_s = (28.563-25.756)/0.5 = 5.612$ m
$L = 9.448+14.478+5.612 = 29.493$ m

7. $Fr_o = 4$ from Figure 4-B-5 $y_o/r = 0.1$
$r = 0.457/0.1 = 4.570$ m
Basin width, $W_B = 3.048$ m
Basin elevation, $z_1 = 25.756$ m
Basin length, $L_B = 14.478$ m
Total length, $L = 29.538$ m
Incoming depth, $y_1 \cong 0.305$ m (1 ft)
Incoming Froude number, $Fr_1 = 7.6$
Jump height, $y_2 \cong 3.353$ m (11 ft.)

2. Chute Blocks:
$h_1 = W_1 = W_2 = y_1 = 0.305$ m
$N_C = 3.048/2(0.305) = 5$-OK whole number
$W_1 = W_2 = 3.048/(2 \times 5) = 0.305$ m
Sidewall spacing = 0.152 m

3. Dentated Sill:
$h_2 = 0.2y_2 = 0.2(3.353) = 0.671$ m
$W_3 = W_4 = 0.15y_2 = 0.503$ m
$N_s = W_B/W_3 = 3.048/0.503 \cong 6$, Use 5
which makes 3 blocks and 2 spaces each 0.610 m (2 ft).



EXAMPLE PROBLEM SKETCH (VII-G)

Note: See the USBR and SAF design comparison at the end of Section 7-G.

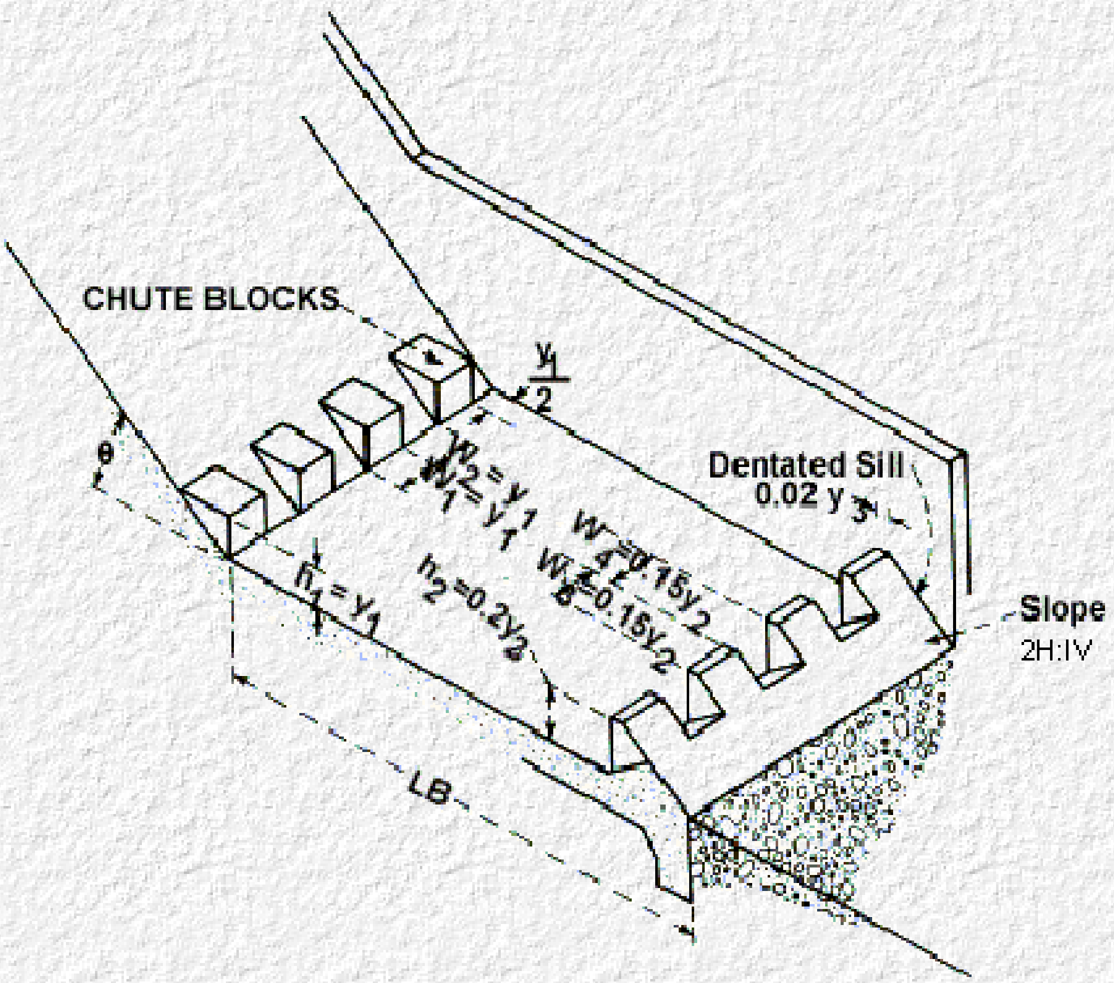


Figure 7-D-1. USBR Type II Basin

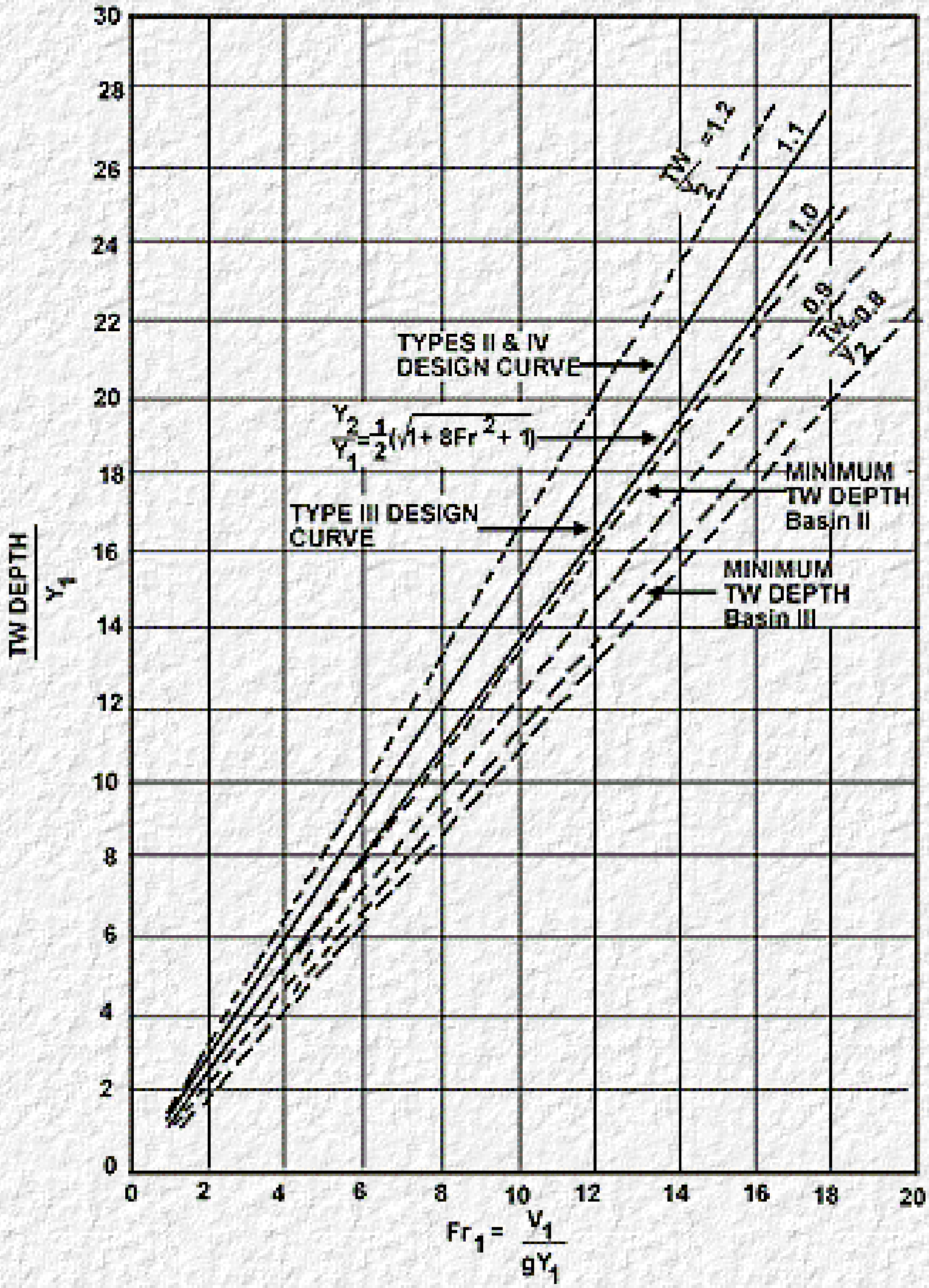


Figure 7-D-2. Tailwater Depth (Basin II,III, IV)

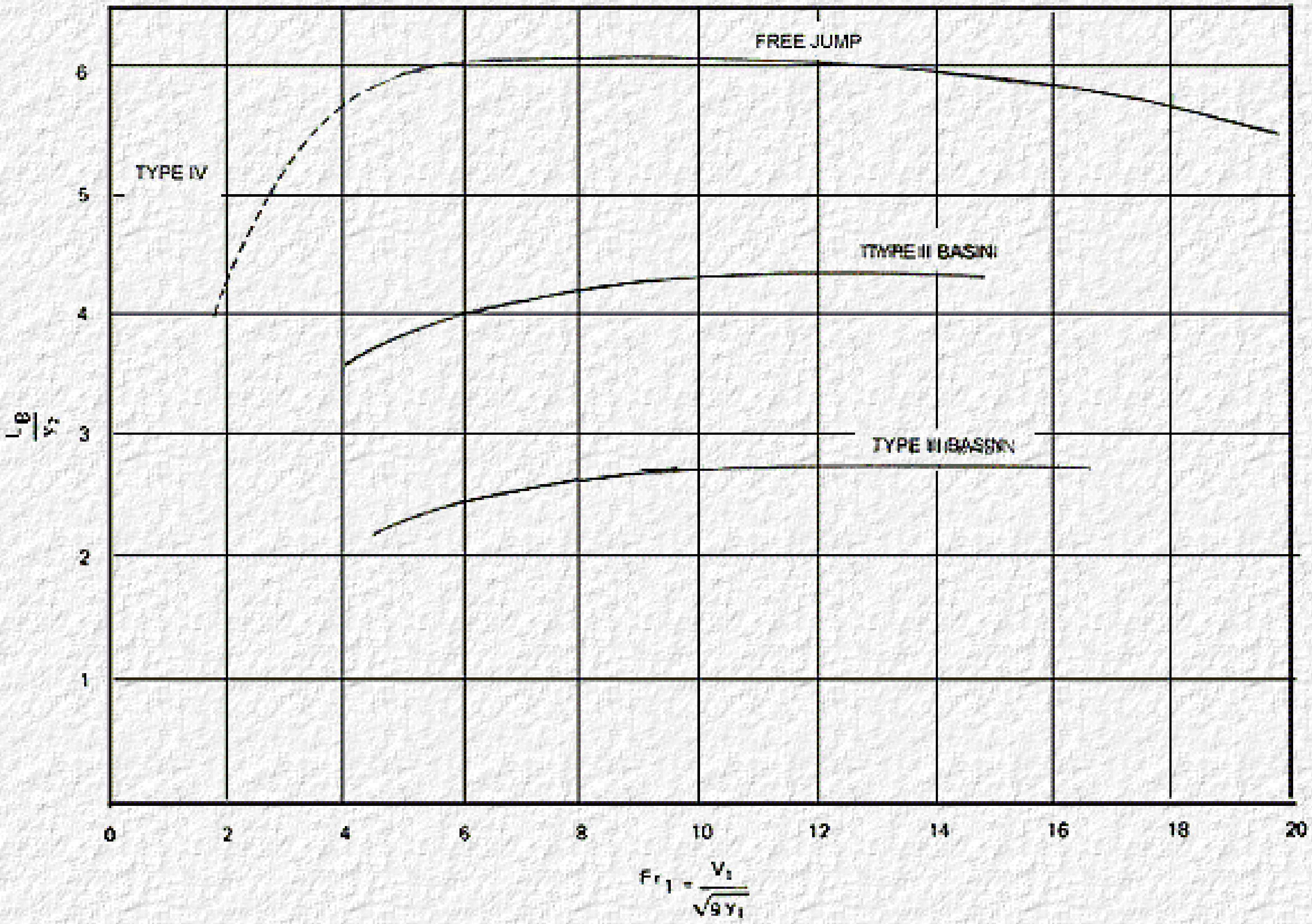


Figure 7-D-3. Length of Jump on Horizontal Floor

7-E USBR Type III Basin

This basin, which originated with the U.S. Bureau of Reclamation (7D1), is intended for discharges up to 18.58 cubic meters per second per meter of basin width and velocities up to 15.2 to 18.3 meters 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 7-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 7-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 7-D-2. This provides a 15 to 18 percent factor of safety.

Design Recommendations

Length of basin can be obtained from Figure 7-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 0.2 m, make blocks 0.2 m high.

The height, width, and spacing of the baffle piers are shown on Figure 7-E-1. The width and spacing may be reduced, for narrow structures, 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 7-E-2.

If the chute slope is 2H:1V or greater, use a circular curve at the intersection with the basin floor. See Figure 4-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

Step 1. Determine basin width (W_B), elevation (z_1), length (L_B), total length (L), incoming depth (y_1), incoming Froude number (Fr_1), jump height (y_2)

by using the design procedure in Section 4-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E, use C_1=1.0 or Figure 7-D-2 to find y_2. For step 5F, use Figure 7-D-3 to find L_B.

Step 2. The chute block height (h_i), width (W_1), and spacing (W_2) are all equal to the incoming depth

$$W_1 \cong W_2 \cong h_1 = y_1$$

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 = $W_2 / 2$.

Step 3. The baffle block height (h_3) is found by using Figure 7-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=$0.8y_2$.

Step 4. The end sill height (h_4) is found by using Figure 7-E-2 to determine h_4 / y_1, knowing Fr_1.

Example Problem

Given:

Same Conditions as Section 4-B, 3048 mm x 1829 mm RCB, Q=11.809 m^3/s, S_o= 6.5%.Elevation outlet invert z_o= 30.48 m V_o=8.473 m/s, yo=0.457 m. Downstream channel is a 3.1 m bottom trapezoidal channel with 2H:1V side slopes and n=.03.

Find:

Dimensions for a USBR Type III basin

Solution:

1. Determine basin elevation using design procedure outlined in Section 4-B , Supercritical Expansion Into Hydraulic Jump Basins

Steps from Section 4-B:

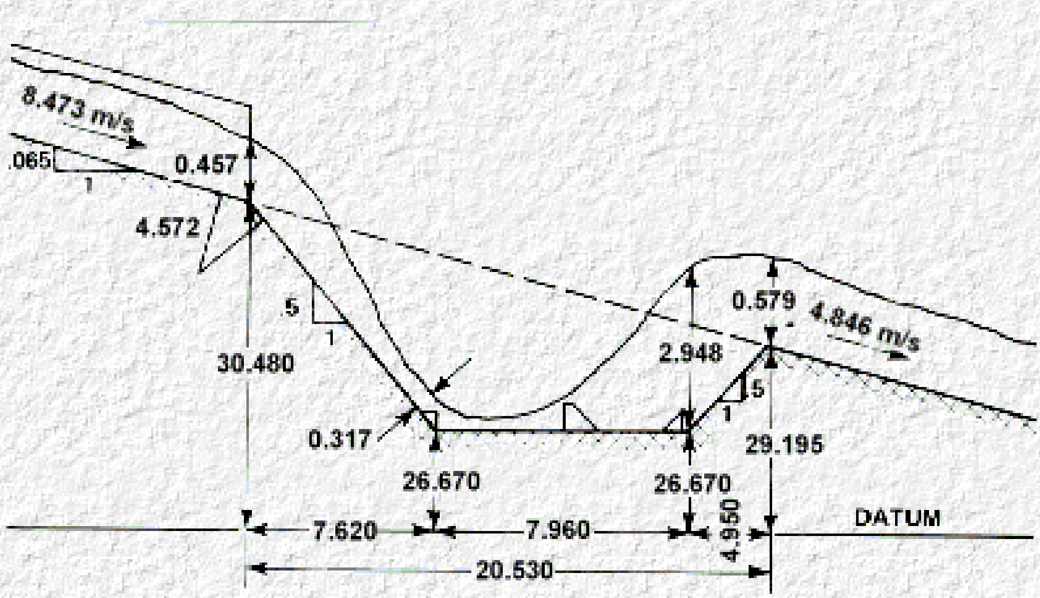
1. V_o=8.437 m/s, y_o=.457 m, Fr_o=4
2. downstream channel TW=y_n =.579 m, V_n =4.846 m/s
3. $y_2 = c_1 y_1 \left[\sqrt{1+8Fr^2} - 1\right]/2 = 1.0(0.457)\left[\sqrt{1+8(4)^2} - 1\right]/2 = 2.367 \text{ m}$
4. Since y_2>TW, 2.367>.58 drop the basin.
5. Use z_1 =26.670 m
   . W_B = 3.048 m, S_T = S_s =.5
   b. W_B - OK no flare
   c. Q= 3.048y_1 [2g(30.480-26.670+.457-y_1)+8.473^2]$^{1/2}$
   Q=3.048y_1 [19.620(4.267-y_1)+71.792]$^{1/2}$
   y_1 =0.317m
   V_1 =11.809/0.317(3.048)=12.223 m/s
   d. $Fr_1 = 12.223/\sqrt{g(0.317)} = 6.93$
   e. For $C_1 = 1, y_2 = 0.317\left[\sqrt{1+8(6.93)^2} - 1\right]/2 = 2.948$
   f. From Figure 7-D-3 L_B /y_2 =2.7, L_B = 7.960 m
   L_T =(z_o-z_1)/S_T =(30.480-26.670)/.5=7.620 m
   z_3 =[30.480-(7.960+7.620 - 26.67/.5)0.065]/1.13=29.145 m
   g. y_2 +z_2 =2.948 +26.670=29.618 m
   TW + z_3 =0.579 +29.145 =29.724 OK
6. L_T =7.620 m, L_B =7.960 m
   L_S =(z_3 -z_2)/S_s =(29.145 - 26.670)/0.5=4.950 m
   L=7.620 + 7.960 + 4.950 = 20.530 m
7. Fr_o = 4 from Figure 4-B-5 y_o/r=0.1
   r =0.457/0.1=4.570 m

Basin width, W_B =3.048 m
Basin elevation, z_1 = 26.670 m
Basin length, L_B =7.960 m
Total length, L=20.530 m
Incoming depth, y_1 =0.317 m
Jump height, y_2 =2.948 m
Incoming Froude number, Fr_1 =6.9

- 2. Chute Blocks:
  $h_1 = W_1 = W_2 = y_1 = 0.317$ m say 0.305 m (1 ft.)
  $N_C = W_B / 2y_1 = 3.048/2(0.305) = 5$-OK whole number
  Space at wall=$W_2 /2 = 0.305/2 = .152$ m (0.5 ft)

- 3. Baffle Blocks:
  From [Figure 7-E-2](), $h_3 / y_1 = 1.75$ for $Fr_1 = 6.9$
  $h_3 = 1.75(0.305) = 0.555$ m
  $W_3 = W_4 = .75h_3 = .75(.555) = 0.418$ m
  $N_B = W_B / 1.5h_3 = 3.048/(1.5(0.555) = 3.7 \cong$ 4 blocks
  adjusted $W_3 = W_4 = W_B / 2N_B = 3.048/2(4) = 0.381$ m
  Space at wall = $W_3 /2 = 0.418/2 = 0.209$ m
  Length from chute block = $0.8y_2 = 0.8(2.948) = 2.358$ m

- 4. End Sill:
  From [Figure 7-E-2](), $h_4 / y_1 = 1.4$ for $Fr_1 = 6.9$
  $h_4 = 1.4(0.317) = .443$ m



Note: See the USBR and SAF design comparison in [Table 5-4]().

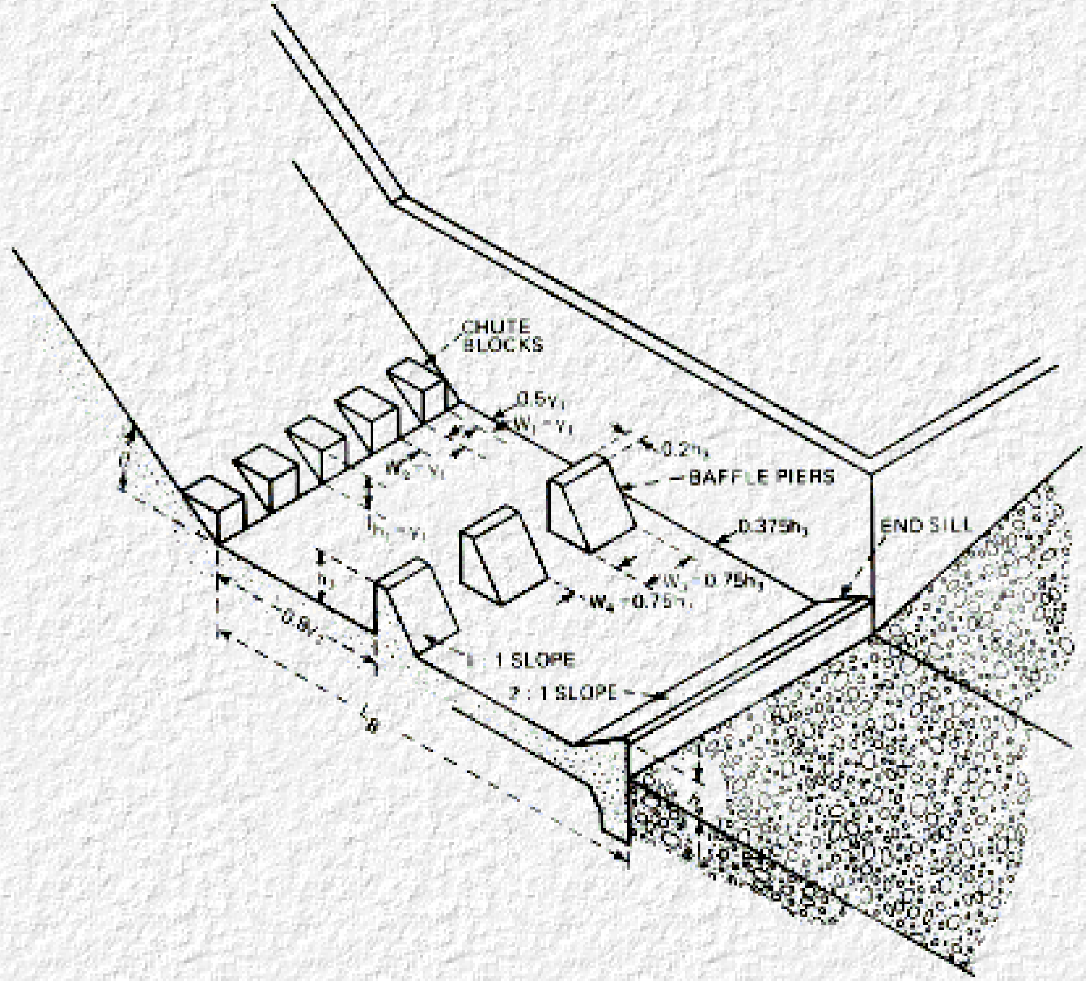


Figure 7-E-1. USBR Type III Basin

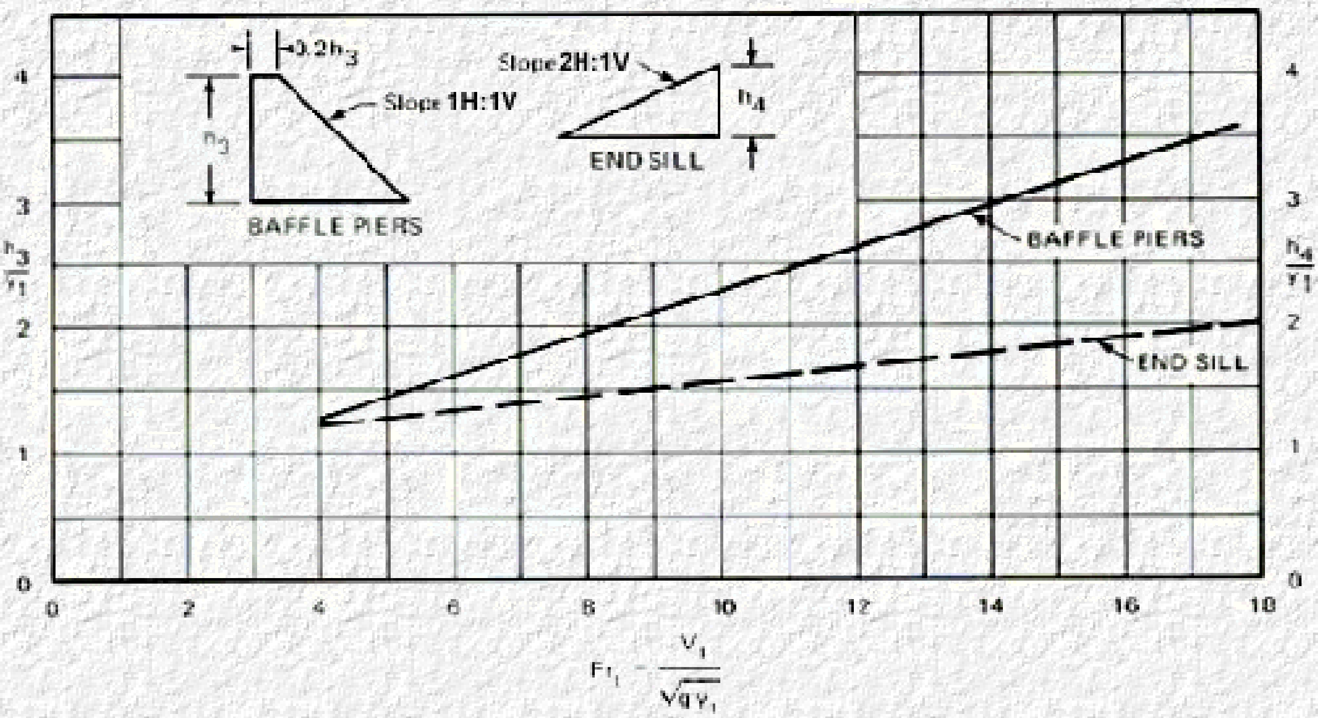


Figure 7-E-2. Height of Baffle Piers and End Sill (Basin III) (reference 7D1)

7-F USBR Type IV Basin

The Type IV U.S. Bureau of Reclamation basin (7D1) 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 4. 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 7-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, $1.1y_2$, Figure 7-D-2.

The basin length can be obtained from the dashed portion of the free jump curve of Figure 7-D-3

The end sill used with the type III basin is also recommended for this basin, Figure 7-E-2.

The Type IV basin is for use in rectangular channels only. See Chapter 4 for details on the design of transition sections.

Design Procedure

Step 1. Determine

- basin width (W_B)
- elevation (z_1)
- length (L_B)
- total length (L)
- incoming depth (y_1)
- incoming Froude number (Fr_1)
- jump height (y_2)

by using the design procedure in Section 4-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E: use C_1 =1.0 or Figure 7-D-2 to find y_2. For step 5F: use Figure 7-D-3 to find L_B .

Step 2. The chute block height (h_1) $=2y_1$
width (W_1) $=0.75\ y_1$
spacing (W_2)$=2.5\ W_1$

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 = $1.25W_1$

Step 3. The end sill height (h_4) is determined by using Figure 7-E-2 to find h_4/y_1

knowing Fr_1.

Example Problem

Given:

Same Conditions as Section 4-B, 3048 mm x 1829 mm RCB, Q=11.809 m^3/s S_o= 6.5%. Elevation outlet invert z_o=30.48 m; V_o=8.473 m/s, y_o=0.5 m. Downstream channel is a 3.05 m bottom trapezoidal channel with 2H:1V 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 4-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 7-G).

Steps from Section 4-B:

1. V_o=8.473 m/s, y_o=0.457 m, Fr_o=4
2. TW=y_n =0.579 m, V_n =4.846 m/s
3. y_2 =2.367 m
4. Since y_2 >TW drop the basin
5. Use z_1 =25.908 m (85 ft.)
   . W_B =3.048 m, S_T =S_S = 0.5
   b. W_B =3.048 m-OK no flare
   c. y_1 =0.302 m, V_1 =12.832 m/s
   d. Fr_1 =7.46 which exceeds the 2.5 to 4.5 design range
   e. y_2 =3.036 m
   f. L_B =19.202 m, L_T =9.144 m, z_3 = 28.323 m

g. $y_2 + Z_2 = 28.944$ m
$z_3 + TW = 28.902$ m OK

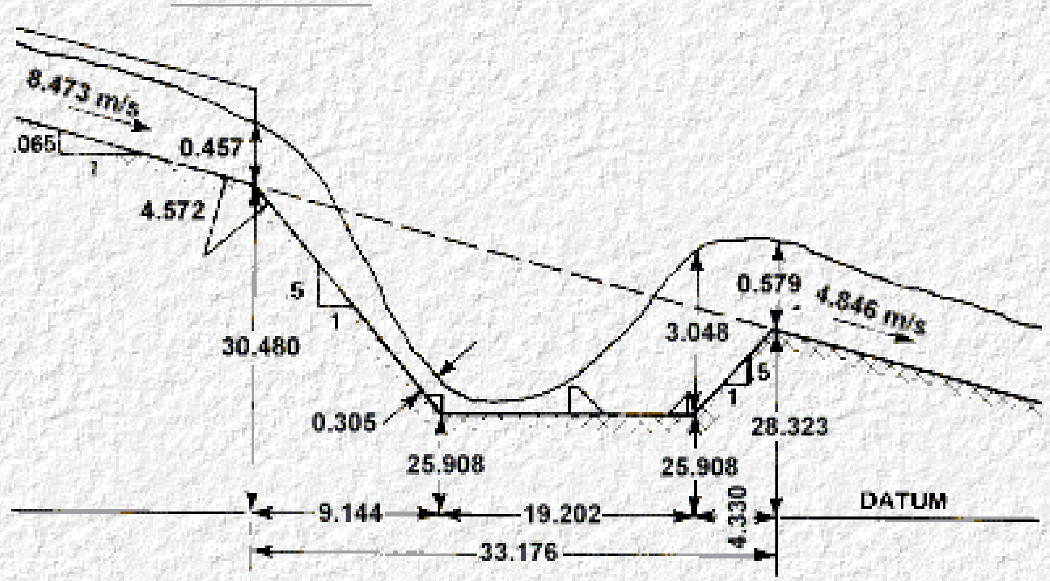
6. $L_T = 9.144$ m, $L_B = 19.202$ m, $L_S = 4.330$ m, $L = 33.176$ m

7. $r = 4.572$ m

Basin Width, $W_B = 3.048$ m
Basin elevation, $z_1 = 25.908$ m
Basin length, $L_B = 19.202$ m
Total length, $L = 33.176$ m
Incoming depth, $y_1 \cong 0.305$ m
Incoming Froude number, $Fr_1 \cong 7.5$
Jump height, $y_2 \cong 3.048$ m

- 2. Chute Blocks:
  $h_1 = 2y_1 = 2(0.305) = 0.610$ m
  $W_1 \cong 0.75y_1 = 0.229$ m (0.75 ft)
  $W_2 \cong 2.5W_1 = 0.572$ m
  $N_C = W_B / 3.5\, W_1 = 3.048/3.5(0.229) = 3.8$, Use 4,
  Adjusted $W_1 = W_B / 3.5N_C = 3.048/3.5\,(4) = 0.218$ m
  $W_2 = 2.5W_1 = 2.5(0.218) = 0.545$ m
  Side wall spacing = $1.25\, W_1 = 1.25(0.218) = 0.272$ m

- 3. End Sill: $h_4 / y_1 = 1.4$ from Figure 7-E-2 with $Fr_1 = 7.5$
  $h_4 = 1.4(0.305) = 0.427$ m



Note: See USBR and SAF design comparison in Table 5-4.

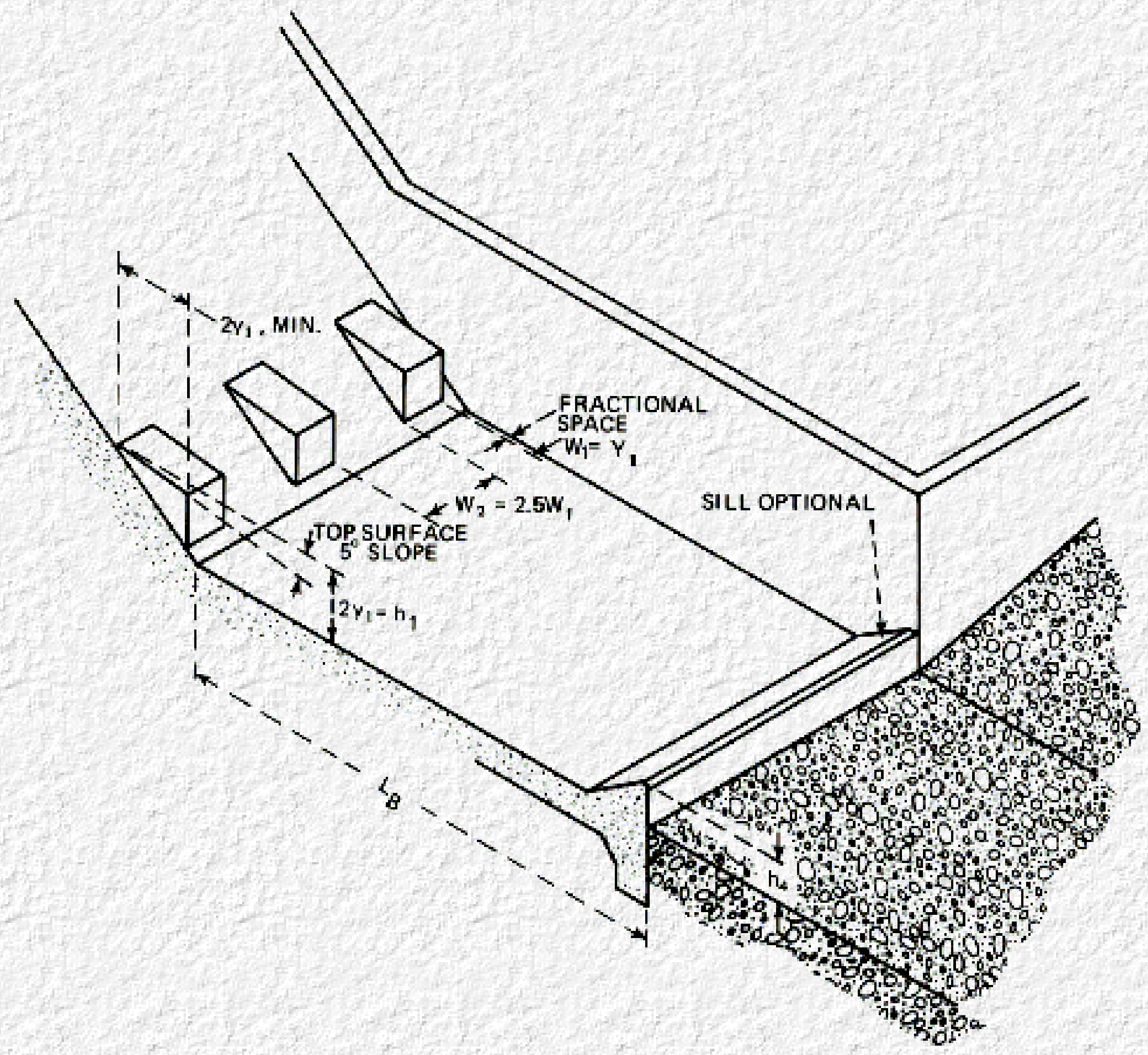


Figure 7-F-1. USBR Type IV Basin

7-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 (7G1).

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 dissipater 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 Recommendation

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

$$W_B = 1.7\ D(Q/g^{0.5}D^{2.5}) \tag{7-G-1}$$

The basin can be flared to fit an existing channel as indicated on Figure 7-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

$$y_j = y_1(\sqrt{1+8Fr_1^2} - 1)/2 \tag{7-G-2}$$

and

$$L_B = 4.5y_j/Fr^{0.76} \tag{7-G-3}$$

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 spacing 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 sequence depth corresponding to y_1.

The depth of tailwater y_2 above the stilling basin floor is:

$$Fr=1.7 \text{ to } 5.5,\ y_2 = (1.1 - Fr_1^2/120)y_j \tag{7-G-4}$$

$$Fr=5.5 \text{ to } 11,\ y_2 = 0.85y_j \tag{7-G-5}$$

$$Fr=11 \text{ to } 17,\ y_2 = (1.0 - Fr_1^2/800)y_j \tag{7-G-6}$$

Wingwalls should be equal in height and length to the stilling basin sidewalls. The top of the wingwall should have a 1H:1V 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

- Step 1. Choose basin configuration and flare dimension, z.

- Step 2. Determine basin width (W_B), elevation (z_1), length (L_B), total length (L), incoming depth (y_1), incoming Froude number (Fr_1), jump height (y_2) by using the design procedure in Section 4-B, Supercritical Expansion Into Hydraulic Jump Basins. For step 5E: y_2 is found from Equation 7-G-4, Equation 7-G-5, or Equation 7-G-6 and y_1 from Equation 7-G-2. For step 5F: use Equation 7-G-3 for L_B .

- Step 3. Chute Block:
  height, $h_1 = y_1$
  width, (W_1)=spacing, $(W_2) = .75y_1$
  Number, $N_C \cong W_B/2W_1$ rounded to a whole number
  Adjusted $W_1 = W_2 = W_B/2N_C$
  N_C includes the 1/2 block at each wall.

- Step 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, $N_B \cong W_{B2}/2W_3$ 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 = $L_B/3$

- Step 5. End Sill height, $h_4 = .07y_j$

- Step 6. Side wall height = $y_2 + y_j/3$

Example Problem

Given:

Same conditions as Section 4-B. 3.048 X 1.829 m (10' x 6') RCB, Q=11.809 m^3/s, S =6.5%. Elevation of outlet invert z_o=30.480 m (100 ft). V_o=8.473 m/s, y_o=0.457 m. Downstream channel is a 3.048 m bottom trapezoidal channel with 2H:1V 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 4-B , Supercritical Expansion Into Hydraulic Jump Basins.

Steps from Section 4-B:

1. V_o=8.473 m/s, y_o=0.457 m, Fr_o
2. Downstream channel TW=y_n =0.579 m, V_n =4.846 m/s
3. From Equation 7-G-2 and Equation 7-G-4:
   $y_j = y_1\,[(1+8Fr_1^{2})^{1/2} - 1\,]\,/2 = .457[(1+8 \times 16)^{1/2} - 1\,]\,/2 = 2.367$ m
   $y_2 = (1.1 - Fr_1^{2}/120)y_j = (1.1-16/120)2.367 = 2.288$ m
4. Since y_2 >TW, 2.288>0.579 drop the basin.
5. Use z_1 =27.889 m
   . W_B =3.048 m, $S_T = S_s$ =0.5
   b. W_B - OK no flare
   c. $Q=3.048y_1[2g(30.48 - 27.889 + 0.457 - y_1) + 8.473^2\,]^{1/2}$
      $Q=3.048y_1\,[19.62(3.048 - y_1\,) + 71.792]^{1/2}$
      y_1 =0.347 m
      V_1 =11.809/.347(3.048)=11.165 m/s
   d. Fr_1 =11.165/[9.81(0.347)]$^{1/2}$ =6.05
   e. $y_j = y_1\left[\sqrt{(1+8Fr^2)} - 1\right]/2 = 0.347\left[\sqrt{1+8(6.05)^2} - 1\right]/2$ =2.800
      Equation 7-G-5, $y_2 = 0.85y_j = 0.85(2.800) = 2.380$ m
6. Equation 7-G-3, $L_B = 4.5y_j\,/Fr^{0.76}$
   =4.5(2.800)/3.93=3.206 m

$L_T = (z_o - z_1)/S_T = (30.480-27.889)/0.5 = 5.182$ m
$z_3 = [30.480-(3.231+5.182-27.889/0.5)0.065]/1.13 = 29.698$m

7. $y_2 + z_2 = 30.269$
   $TW + z_3 = 30.277$ OK

6. $L_T = 5.182$ m, $L_B = 3.231$ m
   $L_S = (z_3 - z_2)/S_s = (29.698 - 27.889)/0.5 = 3.618$ m
   L=5.182 + 3.231 +3.618 = 12.031 m (39.5 ft)

7. $Fr_o = 4$ from Figure 4-B-5, $y_o/r = 0.1$
   r=0.457/0.1=4.57 m

Basin Width, $W_B = 3.048$ m
Basin Elevation, $z_1 = 27.889$ m
Basin Length, $L_B \cong 3.353$ m
Total Length, L $\cong$ 12.192 m
Incoming depth, $y_1 = .347$ m
Incoming $Fr_1 = 6.05$
Theoretical jump height, $y_j = 2.800$ m
Jump height, $y_2 = 2.380$ m

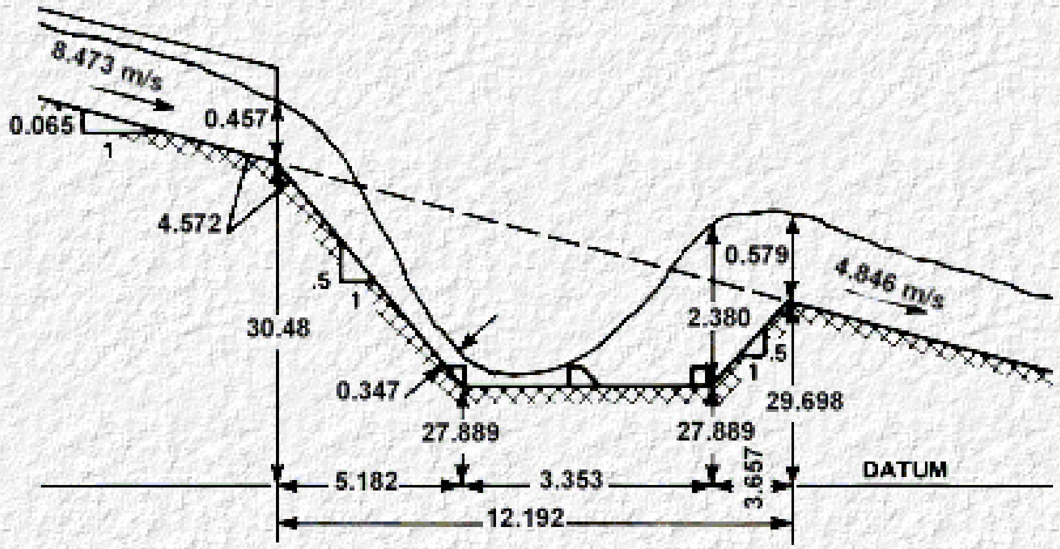
3. Chute Blocks:
$h_1 \cong 0.366$ m (1.2 ft)
$W_1 \cong 0.75y_1 = 0.274$ m$=W_2$
$N_C = W_B / 2W_1 = 3.048/2(0.274) = 5.6$, use 6 blocks.
Adjusted $W_1 = W_B / 2N_c = 3.048/(2 \times 6) = 0.254$ m. This gives 5 full blocks, 6 spaces, and a half block at each wall.

4. Baffle Blocks:
$h_3 \cong y_1 = 0.366$ m (1.2 ft)
$W_3 \cong 0.75y_1 = 0.274$ m$=W_4$
Basin Width, $W_{B2} = W_B + 2L_B / 3z = 3.048+0 = 3.048$ m
$N_B = W_{B2} / 2W_3 = 3.048/(2 \times 0.274) = 5.6$ m, use 6 blocks.
Adjust $W_3 = W_4 = 3.048/(2 \times 6) = 0.254$ m
Total block width = 6(0.254)=1.524 m
Check percent 1.524/3.048 = 0.50, 0.40<0.50 < 0.55 OK.
This gives 6 blocks, 5 spaces, and a half space at each wall
Distance from chute block = $L_B/3 = 3.353/3 = 1.118$ m.

5. End Sill:
$h_4 = 0.07y_j = 0.07(2.800) = 0.196$ m

6. Side Wall:

Height $= y_2 + y_j/3 = 2.380 + 2.800/3 = 3.313$ m



EXAMPLE PROBLEM SKETCH (VII-G)

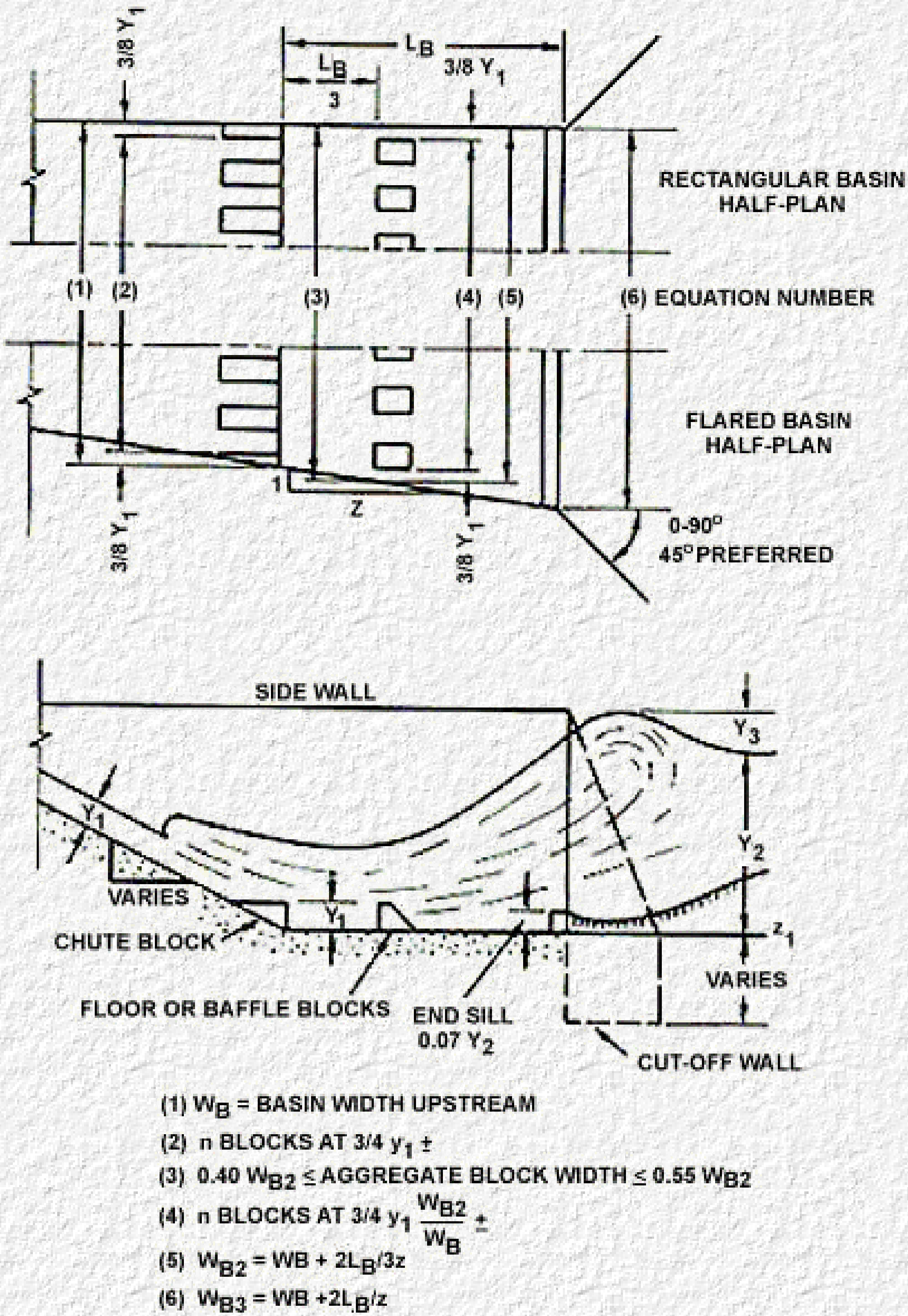


Figure 7-G-1. SAF Stilling Basin (from reference 7G1)

Comparison of Baffle Block Basins:

The three USBR Basins, Types II, III, IV, and the St. Anthony Falls Basin (SAF) were all designed using the same flow conditions:

Box Culvert 3.048 x 1.829
Discharge, Q=11.8 m^3 /s
Velocity, V_o-8.5 m/s
Depth, y_o=0.457 meters
Slope, S_o-6.5%
Froude Number, Fr_o=4.0

Type*	y_1	Fr_1	TW Req'd. Meters	L_B m	L m	Basin Elevation m**
USBR						
II	0.299	7.6	3.4	14.5	29.5	25.8
III	0.317	6.9	2.9	8.0	20.5	26.7
IV	0.305	7.5	3.0	19.2	33.2	25.9
SAF	0.347	6.0	2.4	3.4	12.2	27.9

* All Basins with constant cross section have same width, are 3.05 meters, and rectangular.
**The culvert outlet invert has a reference elevation of 30.5 meters.

7D1. U.S. Bureau of Reclamation
Design of Small Dams
2nd Ed 1973, pp393-439
U.S. Government Printing Office

7G1. Blaisdell, Fred W.,
The SAF Stilling Basin ,
U.S. Government Printing Office, 1959.

[Go to Chapter 8]

8-A Contra Costa Energy Dissipator

Introduction

The Contra Costa energy dissipator (8A1) 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 8-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, $Fr = V / \sqrt{g y_e}$. By entering Figure 8-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. h_e equation of the lines in Figure 8-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:

$$L_2 /(h_2 Fr^2) = 1.2(h_2 / y_o)^{-1.83}$$

has been revised to

$$L_2 /(h_2 Fr^2) = 1.35(h_2 / y_e)^{-1.83} \quad \text{8-A-1}$$

The remaining two equations, used for determining other dimensions of the dissipator, remain unchanged; these are given below and plotted on Figure 8-A-3 and Figure 8-A-4.

$$L_3 / L_2 = 3.75(h_2 / L_2)^{0.68} \quad \text{8-A-2}$$

$$y_2 / h_2 = 1.3(L_2 / h_2)^{0.36} \quad \text{8-A-3}$$

The three equations may be used for proportioning the dissipator but Figure 8-A-2, Figure 8-A-3, and Figure 8-A-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 8-A-2, the dimension L_3 can be obtained by entering Figure 8-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 8-A-2, 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 1H:1V. 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 $0.06y_2$ to $0.10y_2$. After obtaining satisfactory basin dimensions, the approximate maximum water surface depth, y_2, without tailwater, can be obtained from Figure 8-A-4.

Design Procedure

The dissipator design should only be applied within the design limitations:

$2.5 < L_2 / h_2 < 7.0$

$D < W < 3D$

$y_o < D/2$

side slopes of 1H:1V

The following steps outline the procedure for the design of the Contra Costa energy dissipator:

- Step 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.

- Step 2. Compute y_e:

  $y_e = y_o$; for rectangular

  $y_e = (A/2)^{1/2}$; for other shapes.

- Step 3. Compute the parameter $Fr^2 = V_o^2/gy_e$

- Step 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.

- Step 5. Assume a value of L_2/h_2 between 2.5 and 7 and with the aid of Figure 8-A-2 and Figure 8-A-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 $0.5h_2$.

- Step 6. The approximate maximum water surface depth without tailwater can be obtained for the final arrangement from Figure 8-A-4.

- Step 7. Riprap may be necessary downstream especially for the low tailwater cases. See Chapter 2 for design recommendations.Freeboard and a cutoff wall also should be considered to prevent overtopping and undermining of the basin.

Example Problem

Given:

Diameter of culvert 1.219 m

$Q = 8.490\ m^3/s$

$y_o = 0.701$ m

V_o =12.192 m/s

$A = 0.696\ m^2$

Find:

A Contra Costa energy dissipator dimensions.

Solution:

- 1. y_o =0.701 m ≈ D/2, OK.
- 2. y_e =$(0.696/2)^{1/2}$ =0.590 m.
- 3. $Fr^2 = V_o^2/g(y_e) = 12.192^2 / 9.81 \times 0.590 = 25.7$
- 4. W=2D=2.438 m
- 5. By assuming L_2/h_2=3.5 and entering Figure 8-A-2 with Fr^2 =25.7, a value of 3.50 is obtained for h_2 / y_e. Therefore,

  h_2 = 3.50(0.590) = 2.065 m and

  h_1 = 0.5(2.065) = 1.032 m

  L_2 = 3.5(2.065) = 7.228 m

  Entering Figure 8-A-3 with L_2 = 7.2 meters and L_2/h_2 =3.5, L_3 is found to be 11.6 meters. If the maximum rise in water surface, without tailwater, is desired, this can be obtained from Figure 8-A-4. Strictly speaking, the value of y_2 from this chart applies for a bottom width W/D = 2 and 1H:1V side slopes. For the problem at hand, these values are essentially correct.

- 6. Entering Figure 8-A-4 with h_2 = 2.1 m and L_2 / h_2 = 3.5, gives y_2 = 4.2 meters. If the height of end sill is based on this value,

  $h_3 = 0.09(y_2) = 0.09(4.2) = 0.4$ m

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 Meters				
	W	First Baffle	Second Baffle	End Sill
Hor. Distance	2.4	3.7	7.3	18.9
Height	--	1.1	2.1	0.4
Length (baffle)	--	2.4	2.4	3.2

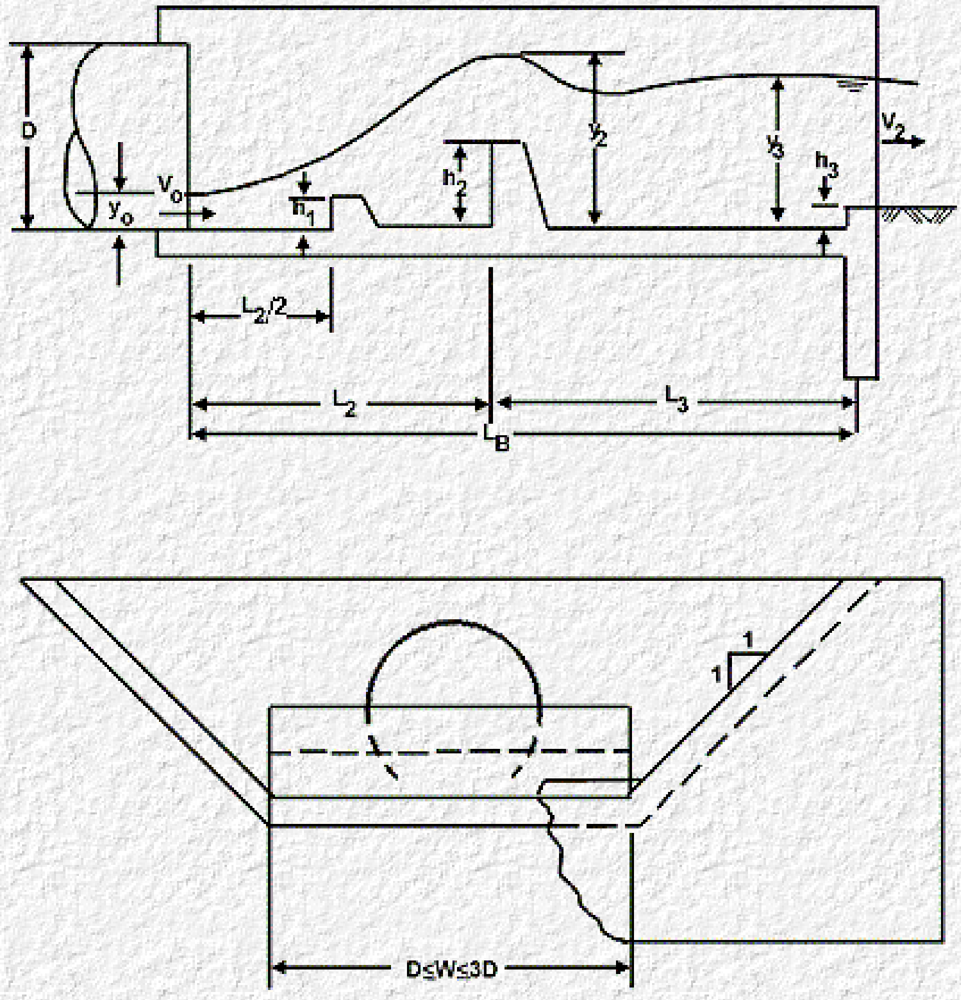


Figure 8-A-1. Contra Costa Energy Dissipator

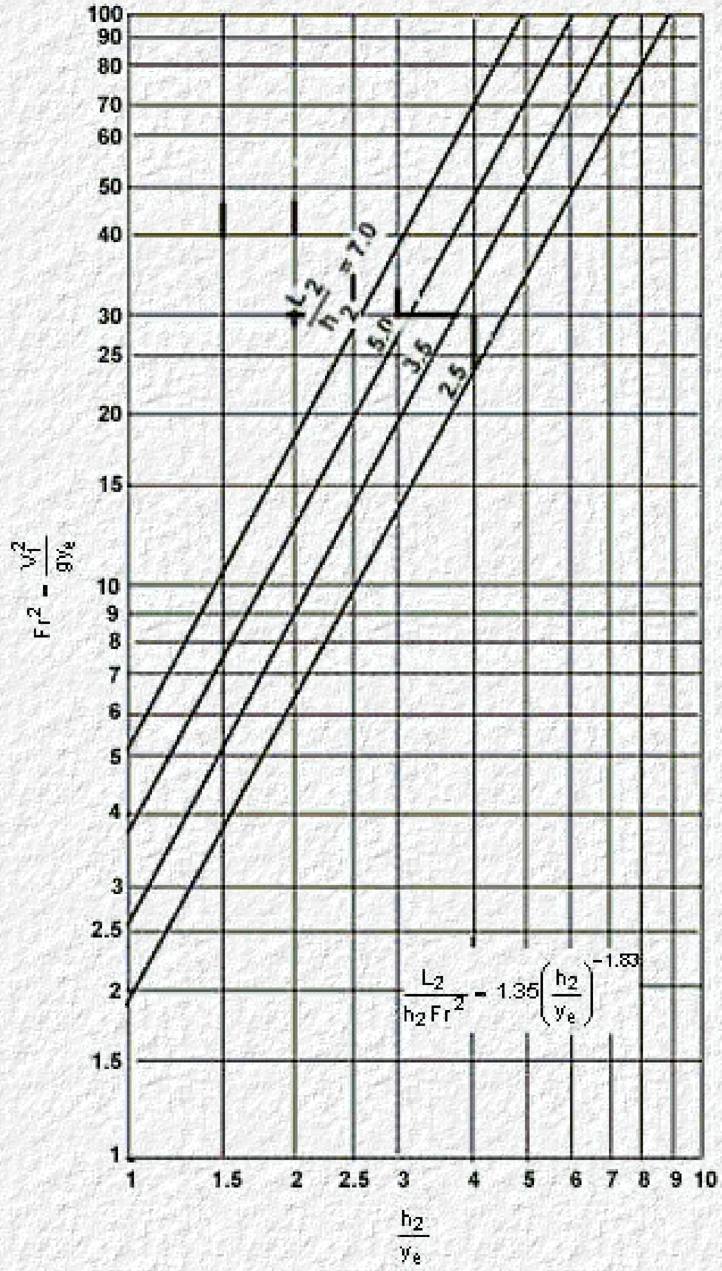
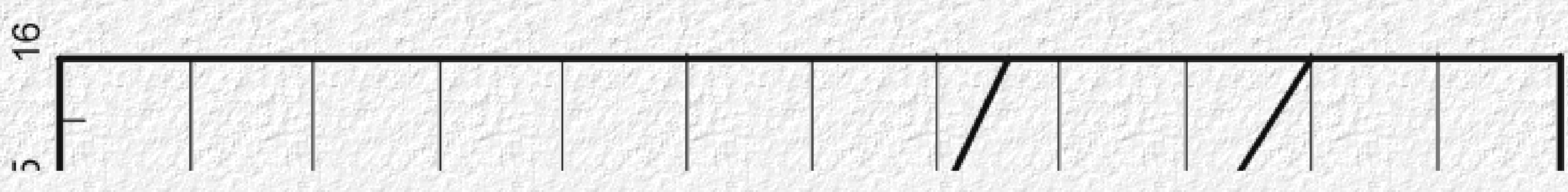
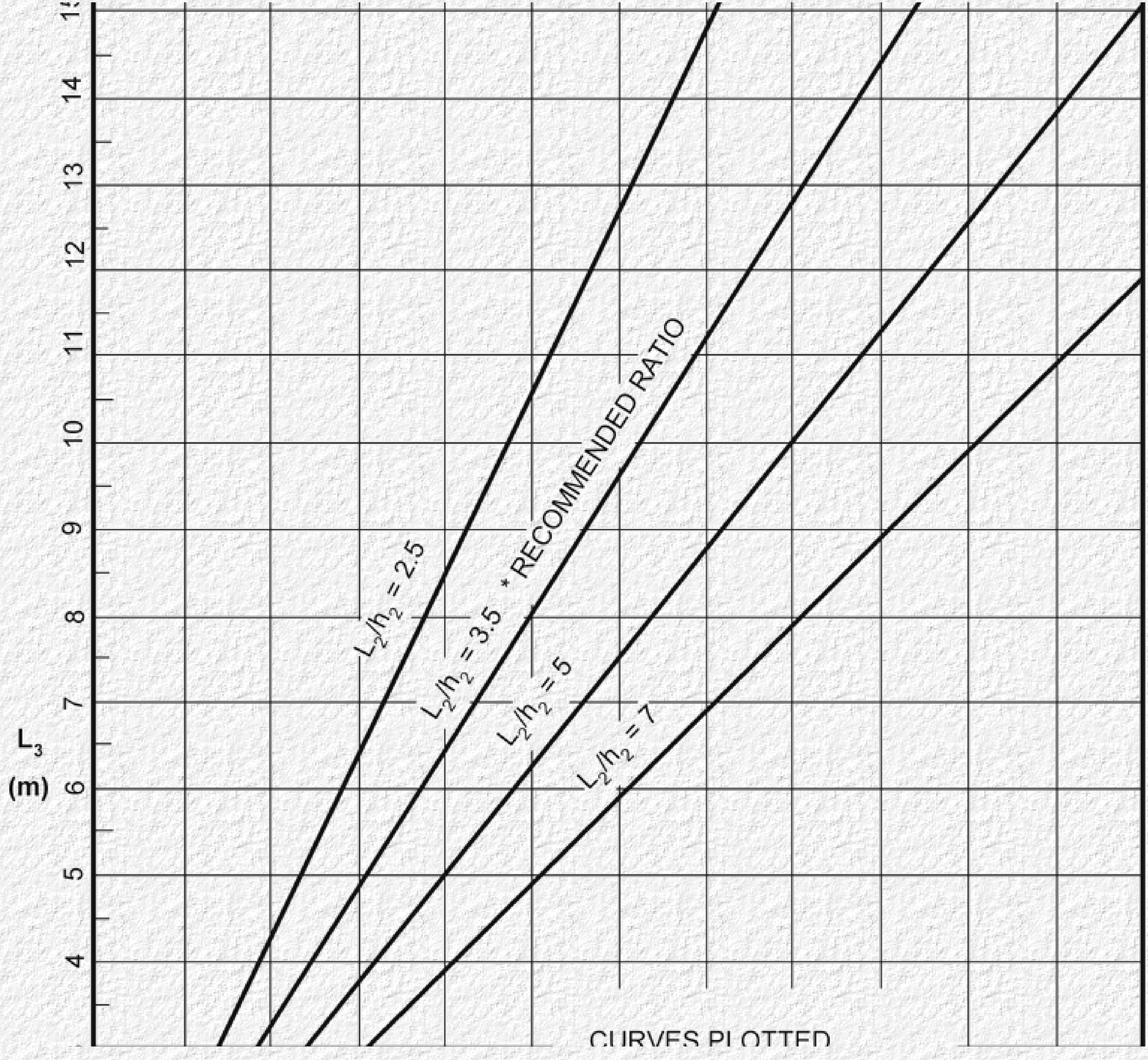


Figure 8-A-2. Baffle Height, Contra Costa Energy Dissipator





L_3
(m)
4
5
6
7
8
9
10
11
12
13
14
$L_2/h_2 = 2.5$
$L_2/h_2 = 3.5$
* RECOMMENDED RATIO
$L_2/h_2 = 5$
$L_2/h_2 = 7$
CURVES PLOTTED

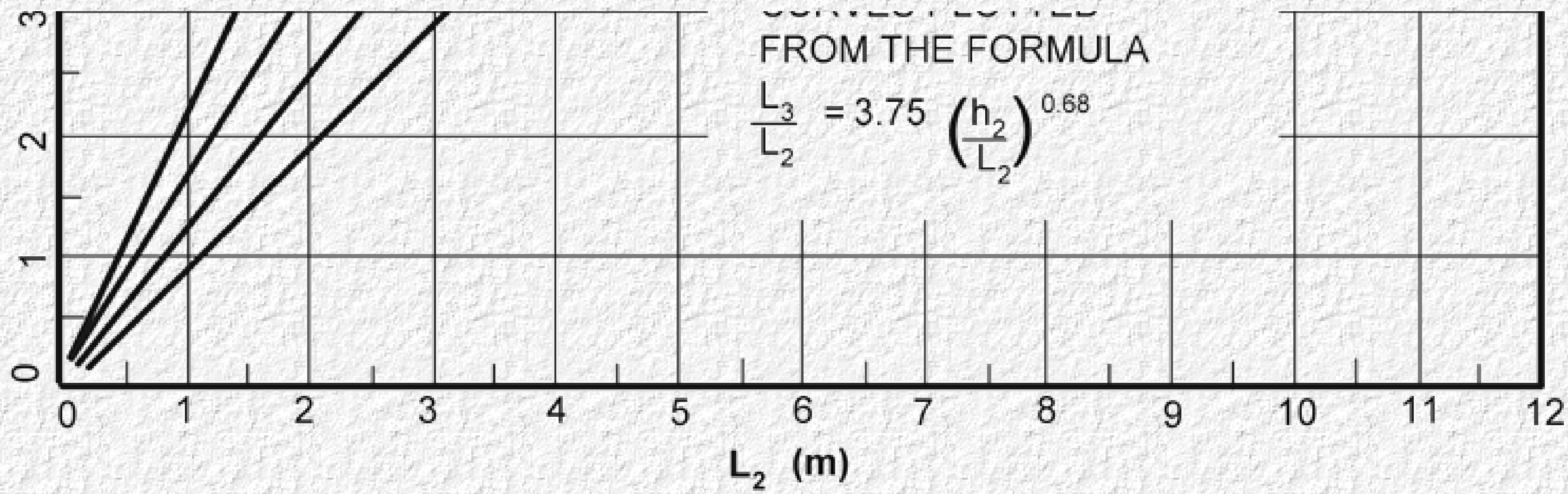
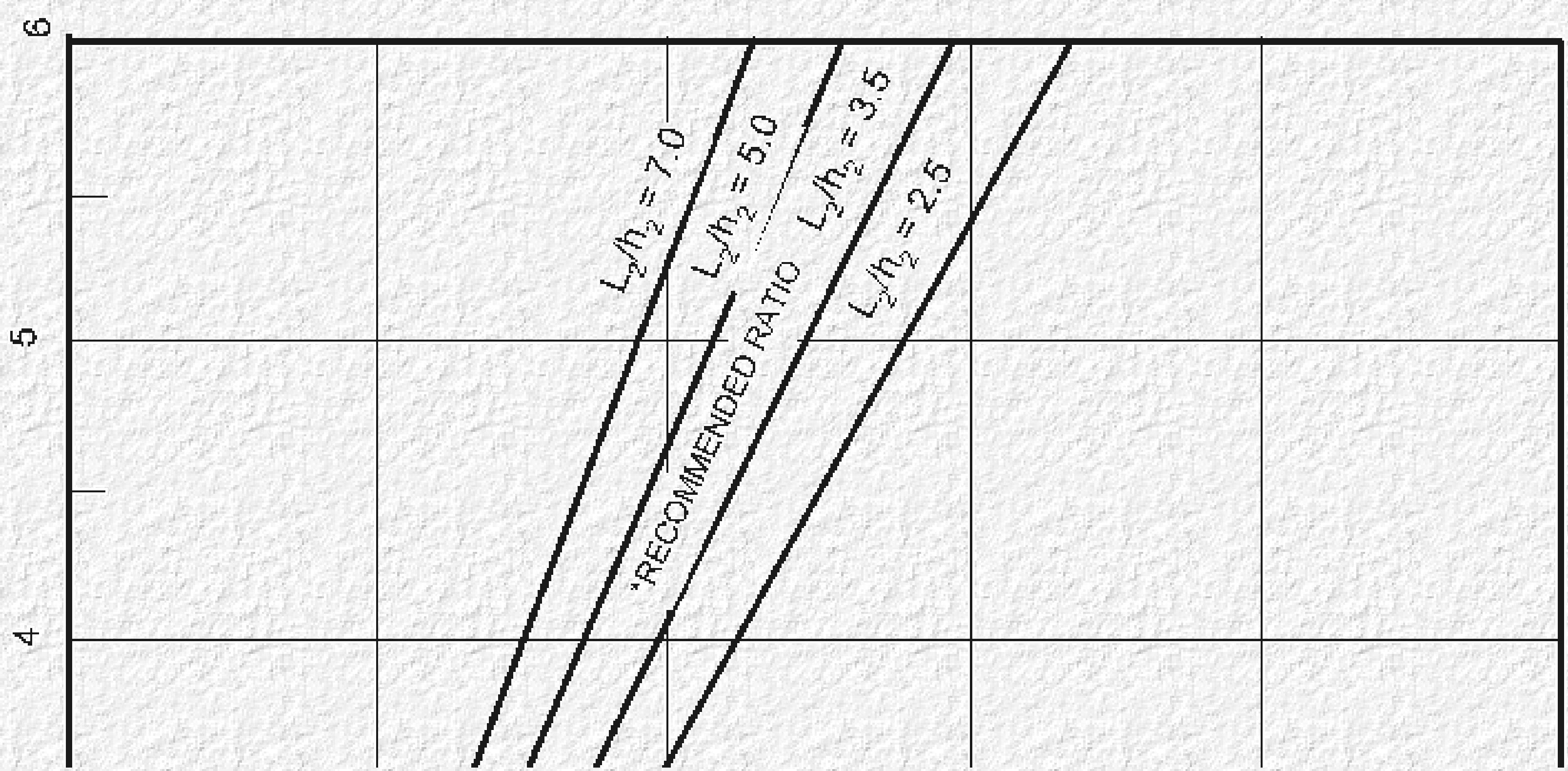


Figure 8-A-3. Length of Contra Costa Energy Dissipator

Figure 8-A-3. Length of Contra Costa Energy Dissipator



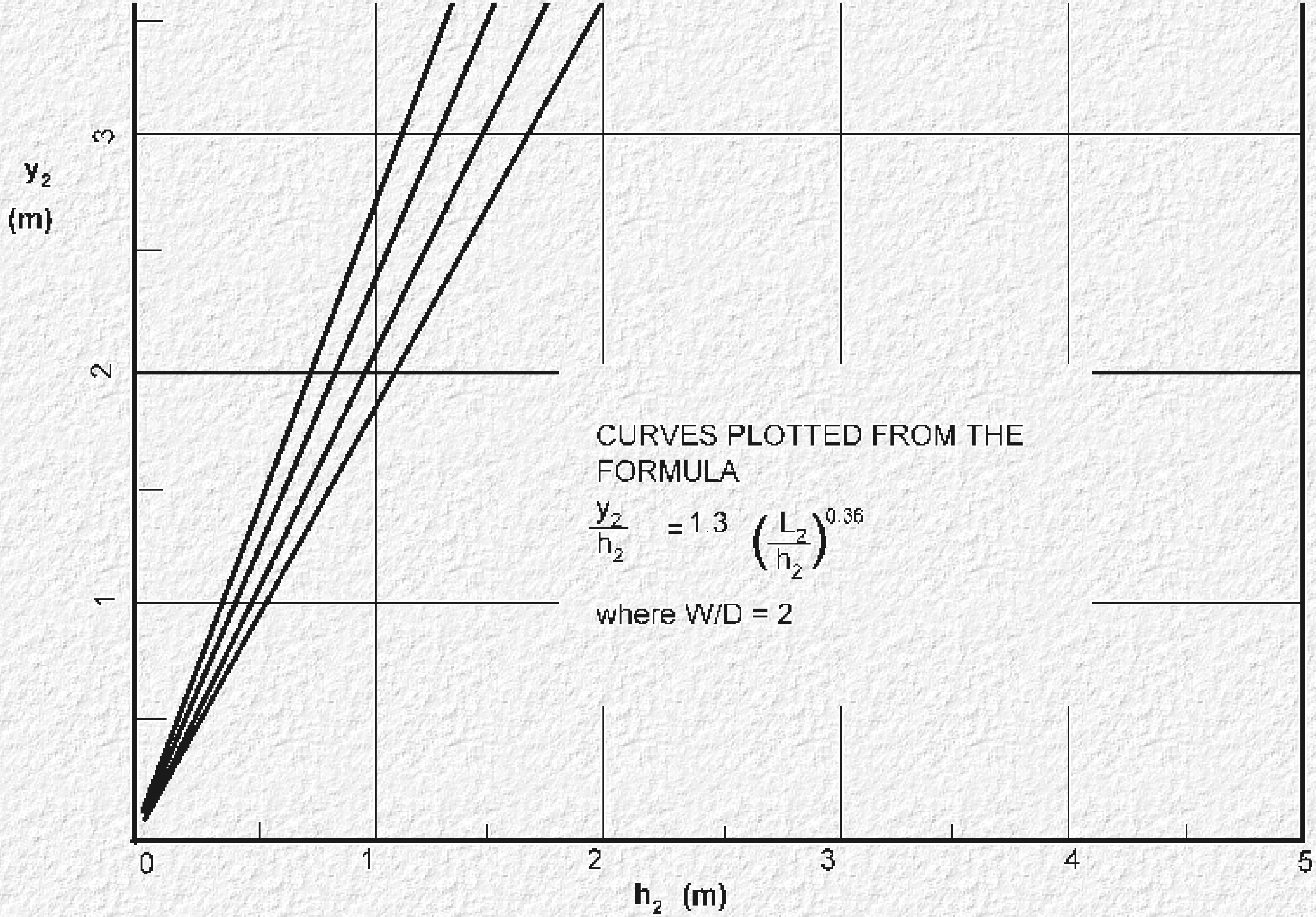


Figure 8-A-4. Depth over Baffle-Contra Costa Energy Dissipator

Figure 8-A-4. Depth over Baffle - Contra Costa Energy Dissipator

8-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 (8B1). 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 1.5H:1V at the end sill, and a tapered basin floor as shown in Figure 8-B-1. In the second study, the warped basin was replaced with a trapezoidal channel of constant cross section such as shown in Figure 8-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 α of 5.5 degrees per side ($\tan\alpha = 0.10$) 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 5.5 degrees = 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 8-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 2 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 8-B-1

- Step 1. Compute the culvert outlet conditions.

  a. Velocity, V_o
  b. Depth, y_e
  c. Froude number , $Fr = V_o / \sqrt{g y_e}$
  d. Check 1.8<Fr<3.0, if not within this range, select another type of dissipator

- Step 2. Compute the flow conditions in the downstream channel.

  a. Velocity, V_n
  b. Depth, y_n
  C. Ratio V_o / V_n

- Step 3. Select the tangent of the flare angle, α. The angle α =Arc tan (0.10) is recommended and determine L_B. Assume a value 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.

Step 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. $W_3 = (W_2 - W_4)/2$

Step 5. Compute other hook dimension: Figure 8-B-2.

a. $h_1 = y_e/1.4$
b. $h_2 = 1.3h_1$
c. $h_3 = y_e$
d. $\beta = 135^o$
e. $r = 0.4h_1$

Step 6. Assess scour potential downstream based on soil condition and outlet velocity. Find V_o/V_B from Figure 8-B-3 and compare with V_o/V_n from step 2c for Riprap projects see Chapter 2.

Step 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 3.658 meters wide and 3.658 meters from the floor to the crown of the semicircular arch.

$S_o = 0.020$

$n = 0.012$
$Q = 76.410\ m^3/s$
$y_e = 1.829$ meter
$V_o = 11.430$ m/s

The normal stream channel is trapezoidal with:

bottom width of 6.096 meters side slopes of 1.5H:1V
$S_o = 0.020$
$n = 0.030$
$V_n = 5.273$ m/s
$y_n = 1.676$ meters.

Find:

Hook energy dissipator dimensions.

Solution:

1. Calculate the Froude number at the culvert exit V_o=11.430 m/s, y_e=1.829 m.

$Fr_o = V_o / (g \times y_e)^{1/2} = 11.430/(9.81 \times 1.829)^{1/2} = 2.70$
1.8<2.70<3-OK

2. $V_n = 5.273$ m/s $y_n = 1.676$ m
$V_o / V_n = 11.430 / 5.273 = 2.17$

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 3.658 meters to a channel width of 6.096 meters, the basin must be 12.802 meters long or about 3.5 times the width of the culvert (see Figure 8-B-4).

4. Following the order of the design recommendations, the position, size, and spacing of the hooks can be determined:

. For distance to the two A hooks:

$L_1/L_B = 0.75$, or $L_1 = 0.75(12.802) = 9.602$ m
$W_1 = 2L_1 (\tan \alpha)+W_o = 2(9.602)(0.1)+3.658 = 5.578$ m

b. The spacing between the A hooks is:

$W_2 / W_1 = 0.66$, or $W_2 = 0.66(5.578) = 3.681$ m

c. The distance to the B hook is

$L_2 / L_B = 0.84$, or $L_2 = 0.84(12.802) = 10.754$ m

d. The width of opening in the end sill is

$W_5 / W_6 = 0.33$, or $W_5 = 0.33(6.096) = 2.012$ m

e. The best height of end sill is

$h_4 / y_e = 0.67$, or $h_4 = 0.67(1.829) = 1.225$ m

f. The same height of end sill is carried up the sloping sides to:

$h_5/h_6 = 0.94$ or $h_5 = 0.94[2(y_e)] = 3.438$m

g. For width of hooks, according to the recommendation,

$W_4/W_B = 0.16$ or $W_4 = 0.16(3.658) = 0.585$ m

h. $W_3 = (W_2 - W_4)/2 = 1.548$ m

9. $h_6 = 2y_e = 2(1.829) = 3.658$ m

5. The hook dimensions are proportioned according to the sketch on Figure 8-B-2:

. $h_i = y_e /1.4 = 1.306$ m

b. $h_2 = 1.28(h_1) = 1.673$ m

c. $h_3 = y_e = 1.829$ m

d. $r = 0.4(h_1) = 0.524$ m

e. $\beta = 135^o$

The dissipator design is shown on Figure 8-B-4, dimensioned in meters. The spacing between hooks is twice the hook width which is satisfactory.

From Figure 8-B-3, with the Froude number of 2.70 and $L = 3.5\ W_o$, V_o / V_B will be less than 1.9 making $V_B \cong 11.430/1.9 = 6.016$ m/s. This is somewhat higher than the normal velocity in the downstream channel indicating riprap 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 8-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 (7B1) 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 Figure 8-B-5 or Figure 8-B-6 for identification of symbols):

$L_B = 3.0\ W_o$ $\quad$ $W_5 / W_6 = 0.33$

$L_1 = 1.25\ W_o$ $\quad$ $h_4 / y_1 = 0.67$

$L_2 = 2.1 W_o$ $\quad$ $h_5 / h_6 = 0.70$

$W_2 = 0.65 W_o$

Where:

$h_6 = 3.33\ y_e$ (for 1.5H:1V side slopes)

$h_6 = 2.69\ y_e$ (for 2H:1V 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 8-B-7.

$W_4 = 0.16W$

W_3 / W_4 = should be unity or greater $W_3 / W_4 \geq 1$

The height of hook (Figure 8-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 8-B-5

- Step 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.

- Step 2 . Compute the flow conditions in the downstream channel.

  a. Velocity, V_n
  b . Depth, y_n
  c . Ratio V_o/V_n

- Step 3. Select side slope:
  1.5H:1V or 2H:1V and compute L_B and W_6
  W_6/W_o can be as large as 2.0. See Figure 8-B-7.

  $L_B=3.0W_o$

- Step 4. Compute:

  . $L_1=1.25W_o$; 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:

$h_6 = 3.33\ y_e$ for 1.5H:1V side slopes
and
$h_6 = 2.69 y_e$ for 2H:1V side slopes

g. $W_4 = 0.16 W_o$; thickness of hooks

h. Check $W_3 / W_4 \geq 1$
where:
$W_3 = (W_2 - W_4)/2$

Step 5. Compute other hook dimension.
see Figure 8-B-6.

a. $h_2 = y_e$
b. $h_1 = 0.78 y_e$
c. $h_3 = 1.4 h_1$
d. $\beta = 135^o$
e. $r = 0.4 h_1$

Step 6. Assess downstream scour potential. Determine V_o / V_B from Figure 8-B-7 and compare with V_o/V_n from step 2c. See Chapter 2 for riprap recommendations.

Step 7. When large debris is expected, consider armoringupstream face of hooks.

Sample Design - Straight Trapezoidal Type

Values which remain the same as in the previous example are:

Steps1 and 2	Fr = 2.70	y_e = 1.829 m
	V_o =11.430 m/s	W_o = 3.658 m
	V_n =5.273 m/s	W_6 = 6.096 m
	V_o / V_n =2.17	y_2 = 1.676 m

Step 3. Proportion a hook-type energy dissipator in a uniform trapezoidal channel with bottom width of 6.096 meters and 1.5H:1V side slopes for the same inflow conditions as given in the former example.

$L_B = 3.0 W_o = 3.0(3.658) = 10.974$ m

Step 4. The other dimensions recommended for this dissipator are:

. $L_1 = 1.25\ W_o = 4.572$ m

b. $W_2 = 0.65\ W_o = 2.377$ m

c. $L_2 = 2.1\ W_o = 7.682$ m

d. $W_5 / W_6 = 0.33$, $W_5 = 0.33(3.658) = 1.207$ m

e. $h_4 / y_e = 0.67$, $h_4 = 0.67(1.829) = 1.225$ m

f. $h_6 = 3.33\ y_e = 6.091$ m
   $h_5 / h_6 = 0.70$, $h_5 = 0.70(6.091) = 4.264$ m

g. $W_4 = 0.16 W_o = 0.16(3.658) = 0.585$ m

h. $W_3 = (W_2 - W_4)/2 = (2.377 - 0.585)/2 = 0.896$ m,
   $W_3 / W_4 = 0.896/0.585 = 1.5$ OK

Step 5. Compute other hook dimensions.

a. $h_2 = y_e = 1.829$ m
b. $h_1 = 0.78(y_e) = 1.427$ m
c. $h_3 = 1.4(h_1) = 1.998$ m
d. $\beta = 135^o$
e. $r = 0.4(h_1) = 0.571$ m

A sketch of this energy dissipator with dimensions given in meters is shown in Figure 8-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 8-B-7, with a Froude number of 2.70 and $W_6/W_o = 6.096/3.658 = 1.67$, $V_o/V_B \cong 2.0$ making $V_B \cong 11.430/2 = 5.715$ m/s. 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 in table below.

Dimensions in Meters

Symbol	Warped Wingwall	Straight Trapezoidal
y_e	1.8	1.8
W_o	3.7	3.7
L_B	12.8	11.0
L_1	9.6	4.6

L_2 W_2 H_4 W_5 W_6	10.8 3.7 1.2 2.0 6.1	7.7 2.4 1.2 1.2 6.1
h_6 h_5 W_4 W_3 h_1	3.7 3.5 0.6 1.5 1.3	6.1 4.3 0.6 0.9 1.4
h_2 h_3 r	1.7 1.8 0.5	1.8 2.0 0.6

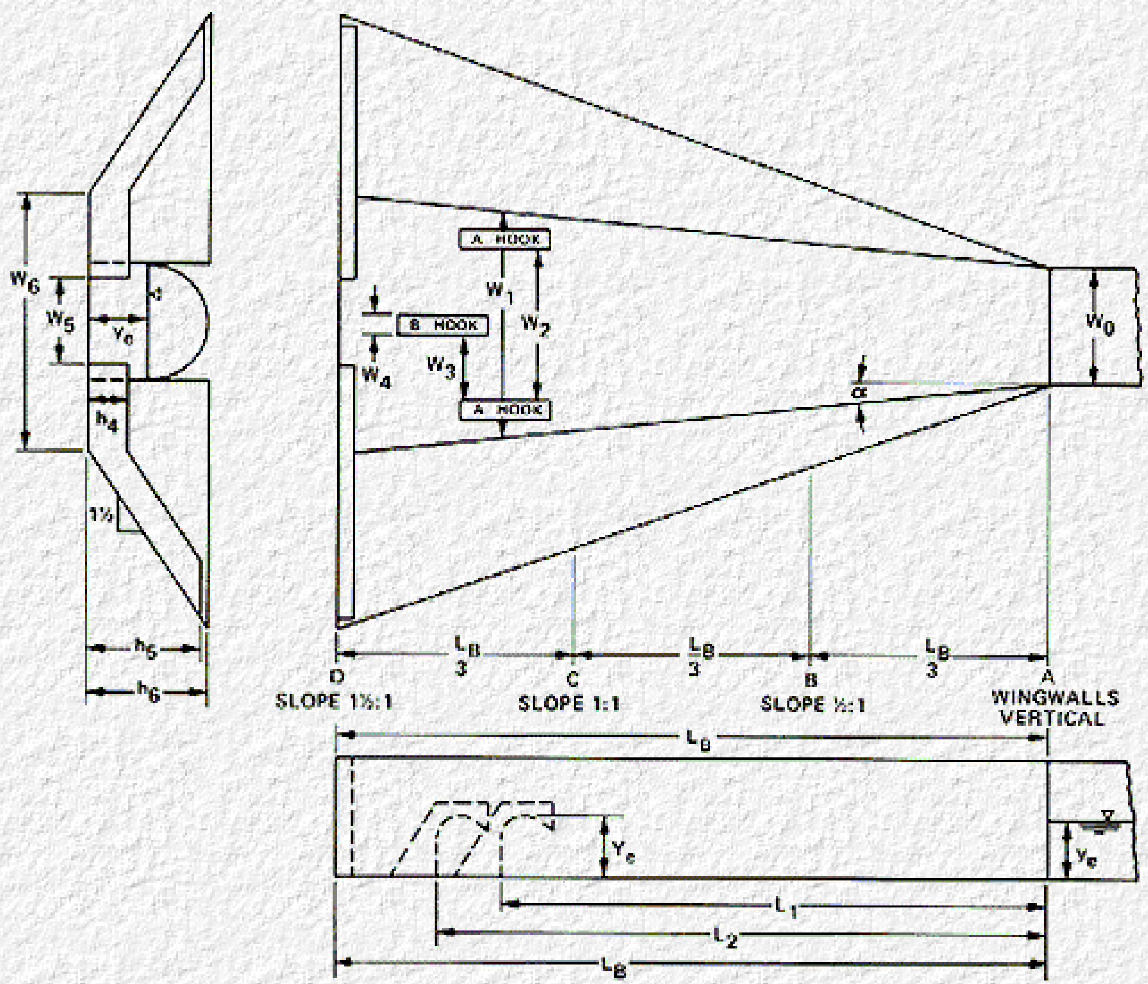
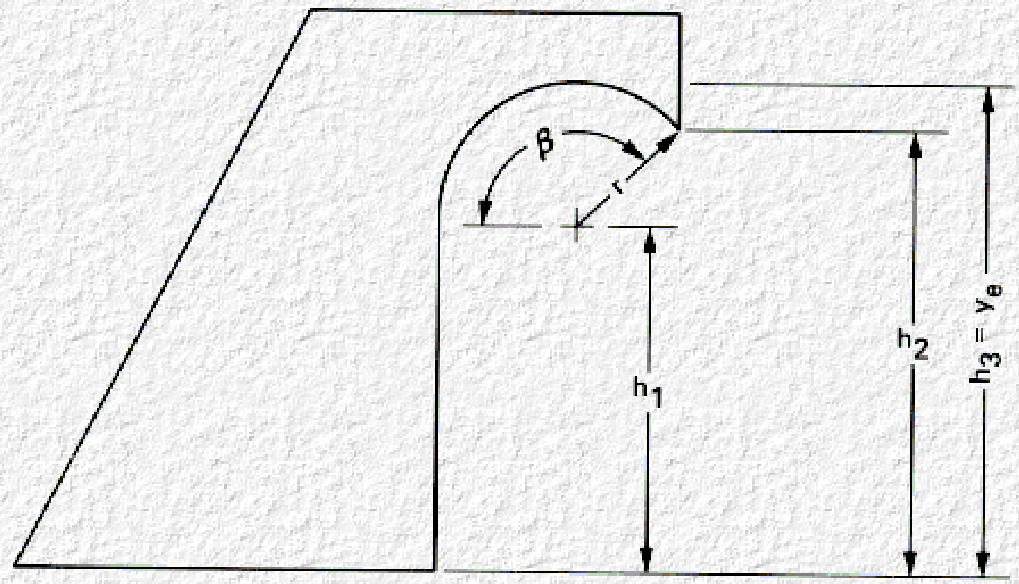


Figure 8-B-1. Warped Wingwall Type Basin



$h_3 = y_e$

$h_2 = 1.28\ h_1$

$h_1 = y_e / 1.4$

$\beta = 135^o$

$r = 0.4\ h_1$

Figure 8-B-2. Hook for Warped Wingwall Basin

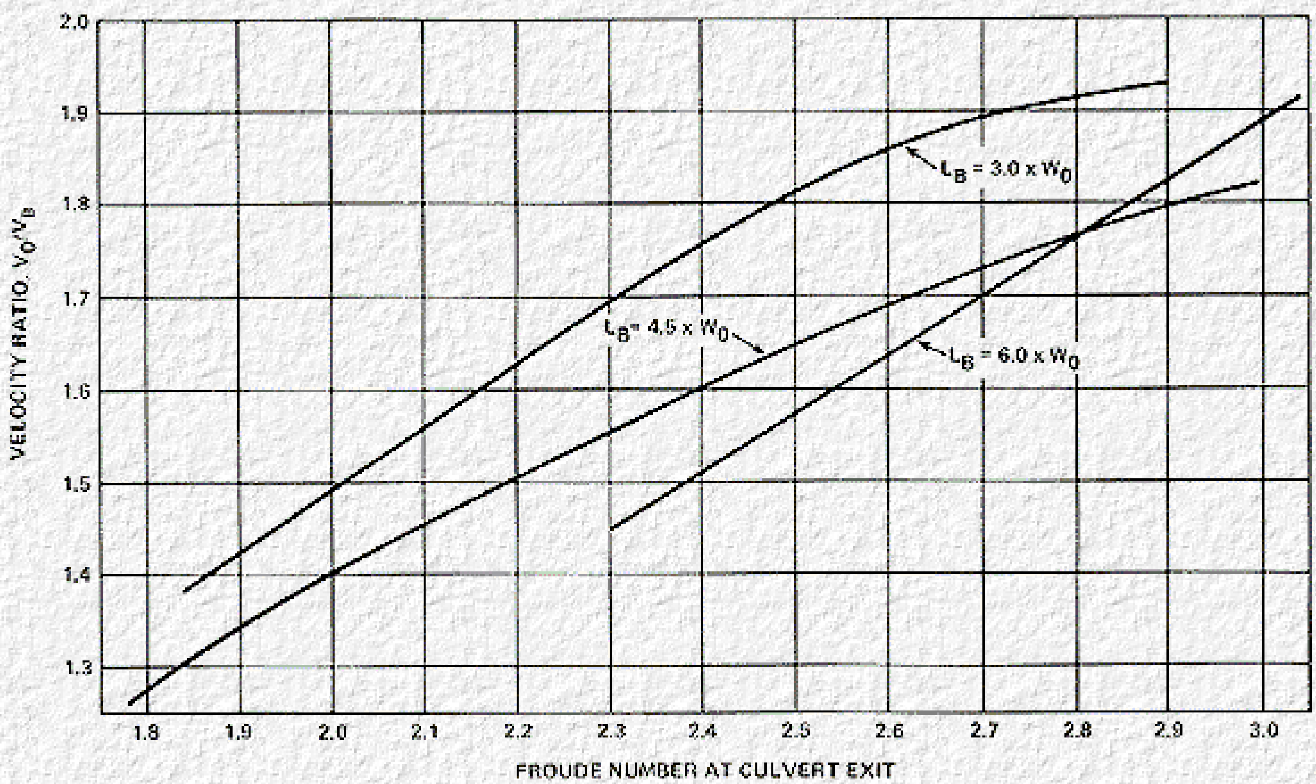


Figure 8-B-3. Froude Number vs. Velocity Ratio for Various Basin Lengths

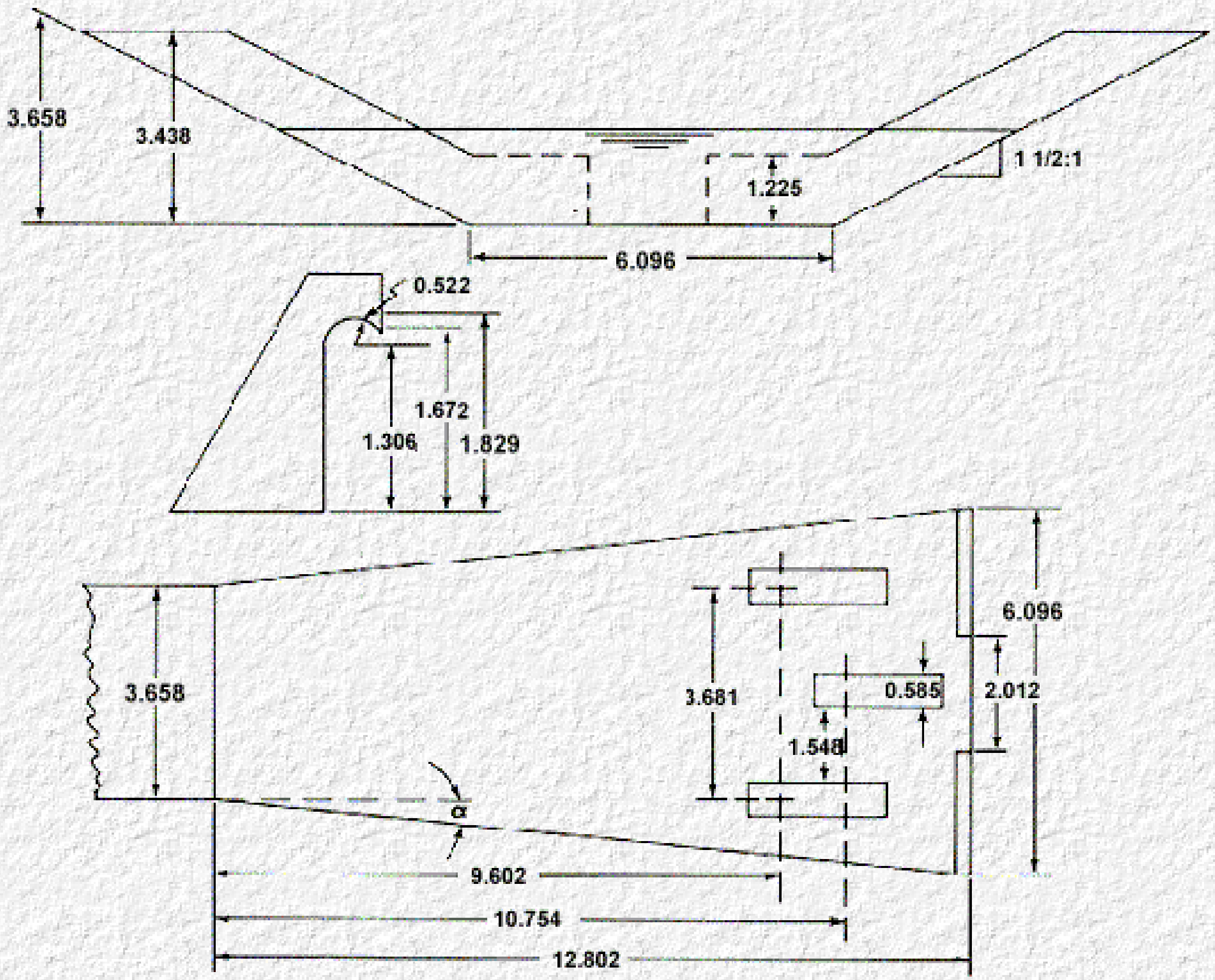


Figure 8-B-4. Sample Design - Warped Wingwall Basin

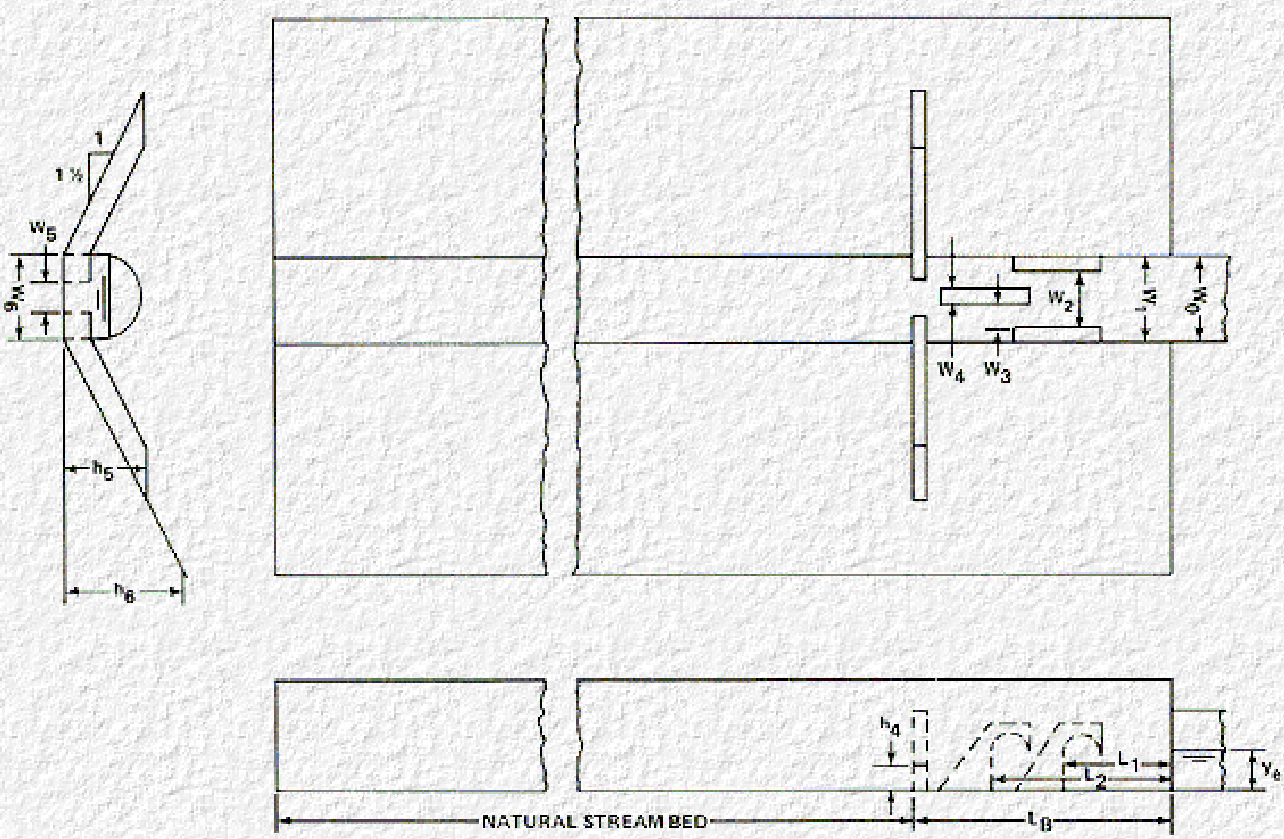
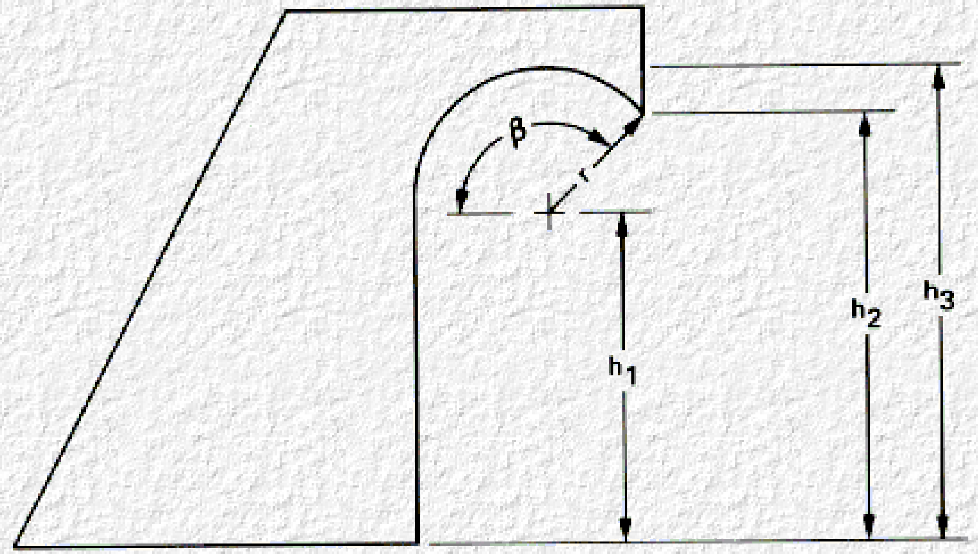


Figure 8-B-5. Hook Type Energy Dissipator in a Straight Trapezoidal Basin



$h_2 = y_e$

$h_1 = 0.78\, y_e$

$h_3 = 1.4\, h_1$

$r = 0.4\, h_1$

$\beta = 135^{o}$

Figure 8-B-6. Typical Shape of Hook Straight Trapezoidal Basin

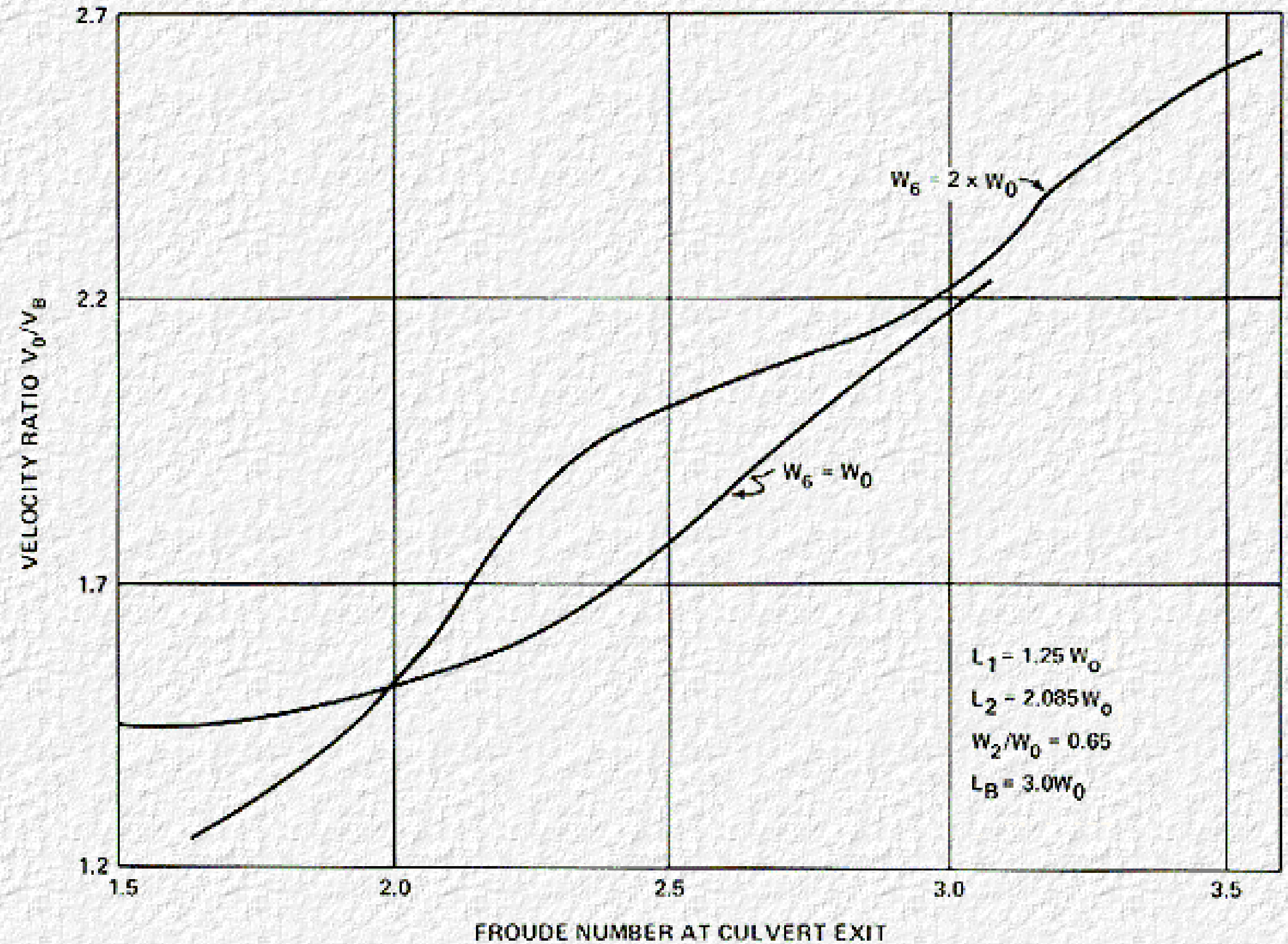


Figure 8-B-7. Velocity Reduction vs. Froude Number for Various Basin Widths

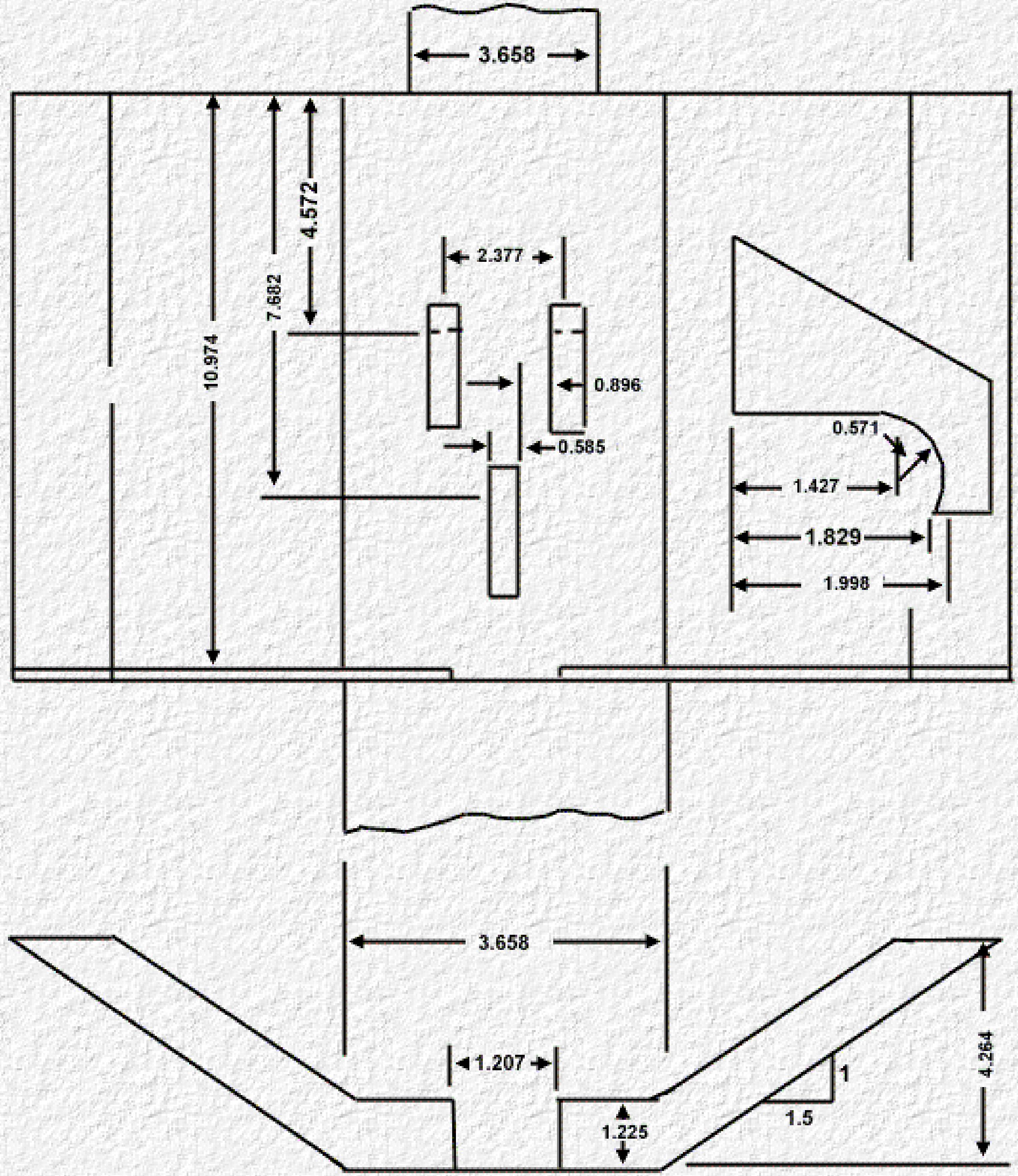


Figure 8-B-8. Sample Design - Straight Trapezoidal Basin

8-C Impact-Type Energy Dissipator

The impact-type energy dissipator developed by the Bureau of Reclamation (8C1) 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 8-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 8-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 8-C-2. This curve incorporates the original information contained in reference 21, 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:

$y_e = (A/2)^{1/2}$ or $(\pi D^2 /16)^{1/2}$

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 11.3 m^3/s and velocities as high as 15.2 m/s, 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 15.2 m/s 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 8-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 15º**.** 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 15º, 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 45º wingwall design as shown in Figure 8-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 2 .

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 15.2 meters per second and discharge is less than 11.3 m^3/s. This dissipator is not recommended where debris or ice buildup may cause substantial clogging.

Design Procedure

- Step 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, $y_e=(A/2)^{1/2}$.

- Step 2. Compute the Froude number Fr and the energy at the end of the pipe H_o. Enter the curve on Figure 8-C-2, and determine the required width of basin W.

- Step 3. With W known, obtain the remaining dimensions of the dissipator structure from Table 8-C-1.

Example Problem

Given:

D = 1.219 m
$S_o = 0.15$
Q = 8.496 m^3/s

Find:

USBR Impact Basin dimensions for use at the outlet of a concrete pipe.
n = 0.015
V_o =12.192 m/s
y_o =0.701 m

Solution:

Since Q is less than 11.3 m^3/s and V_o less than 15.2 m/s, the dissipator may be tried at this site.

- 1. Compute y_e.
  $y_e = (A/2)^{1/2}$
  $A=Q/V_o=8.496/12.192=0.697\ m^2$
  $y_e =(0.697/2)^{1/2} =0.590$ meter

- 2. Compute Fr and H_o and find W.
  $Fr=V_o /(gy_e)^{1/2} =12.192/(9.81x0.590)^{1/2} =5.07$
  $H_o =y_e +V_o^2 /2g=0.590+(12.192)^2 /19.620=8.166$ m

  From Figure 8-C-2.
  $H_o /W=1.68$

W=8.166 /1.68=4.861 m

3. From Table 8-C-1 select remaining dimensions.

Design a second baffle wall dissipator at the end of a long rectangular concrete channel 1.219 meters wide using the same depth of flow = 0.701meters,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 0.701 meters will be 10.612 m^3/s. The computations and comparison with the first example are tabulated below.

Channel			**Circular**	**Rectangular**
Depth of flow	y_o	m	0.701	0.701
Area of flow	A	m^2	0.697	0.855
Discharge	Q	m^3/s	8.496	10.612
Velocity	V_o	m/s	12.192	12.419
Flow depth (rect. section)	y_e	m	0.590	0.701
Velocity Head	$V_o^2/(2g)$	m	7.576	7.861
$H_o = y_e + V_o^2/2g$		m	8.166	8.562
$Fr = V_o/(gy_e)^{0.5}$		--	5.07	4.74
From Figure 8-C-2	H_o/W	--	1.68	1.55
Width of basin	W	m	4.861	5.524
H_L/H_o (100)-Low Tail-water		--	67%	65%

Entering Table 8-C-1 with W = 4.877 meters and W =5.486 m, the remaining dimensions of the two dissipators can be read directly in meters. The basin width is taken to the nearest 0.152 m (0.5 ft), while the other dimensions are read to the nearest 25 mm (1 inch). This degree of accuracy is sufficient.

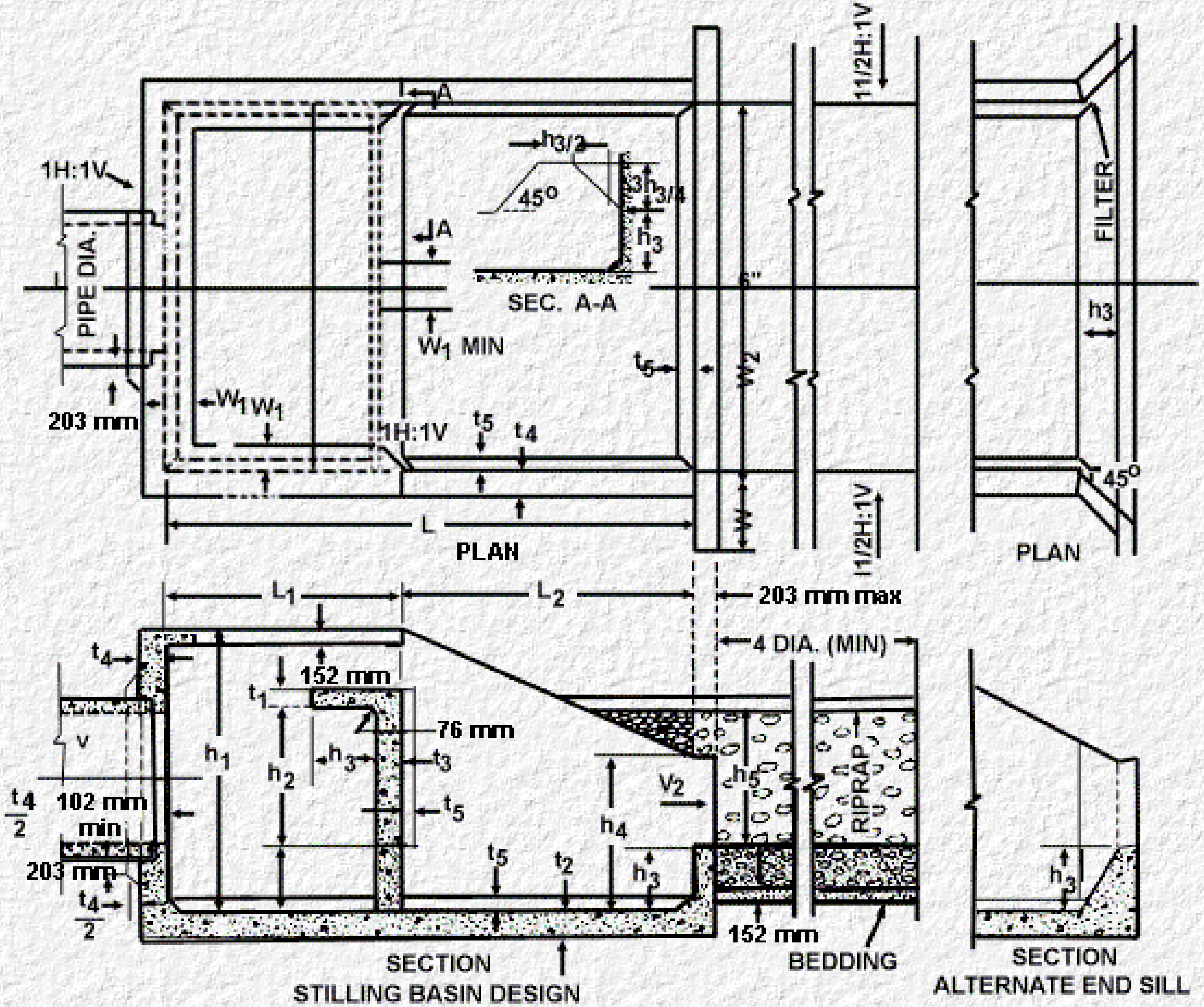


Figure 8-C-1. Baffle - Wall Energy Dissipator - USBR Type VI

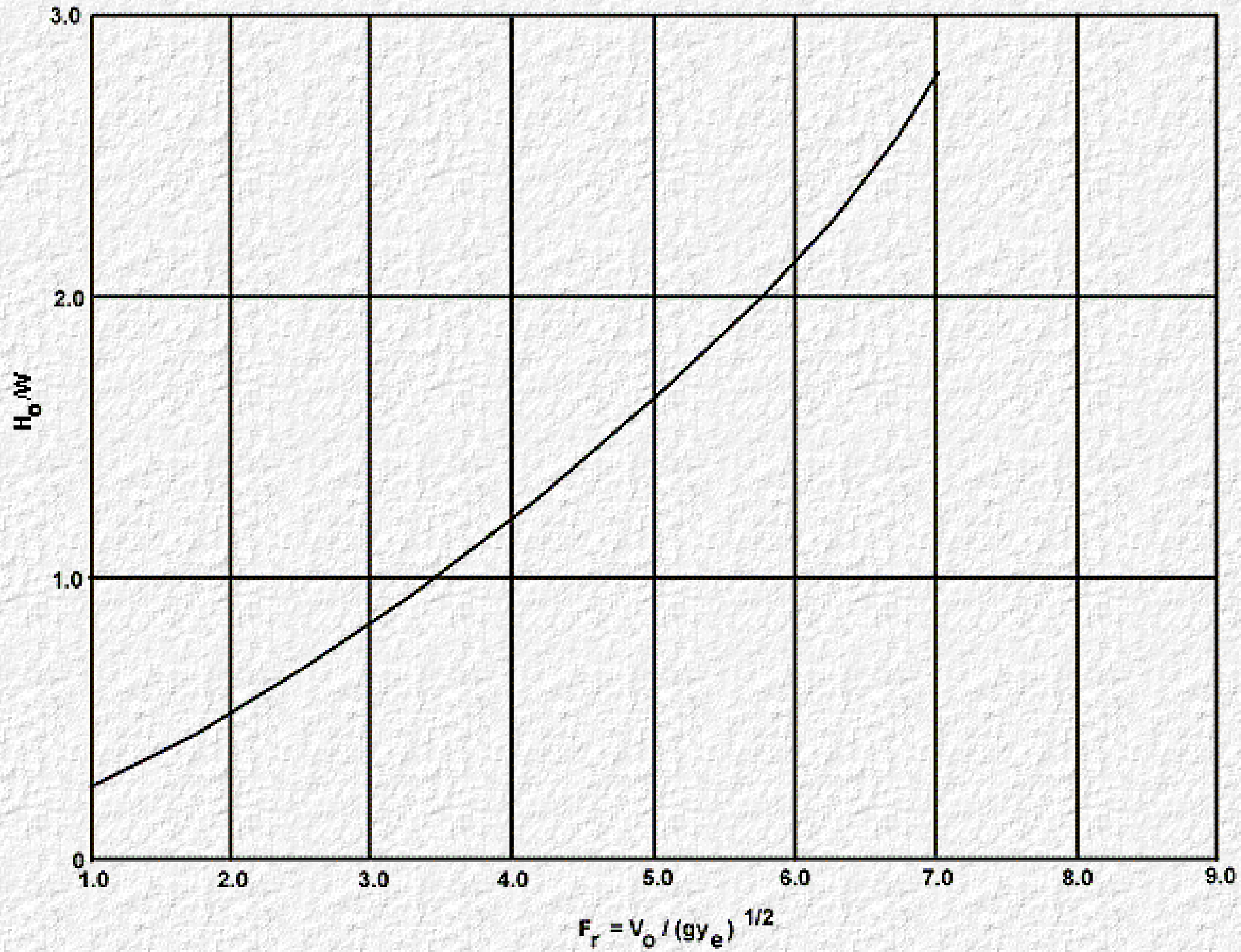


Figure 8-C-2. Design Curve - Baffle Wall Dissipator

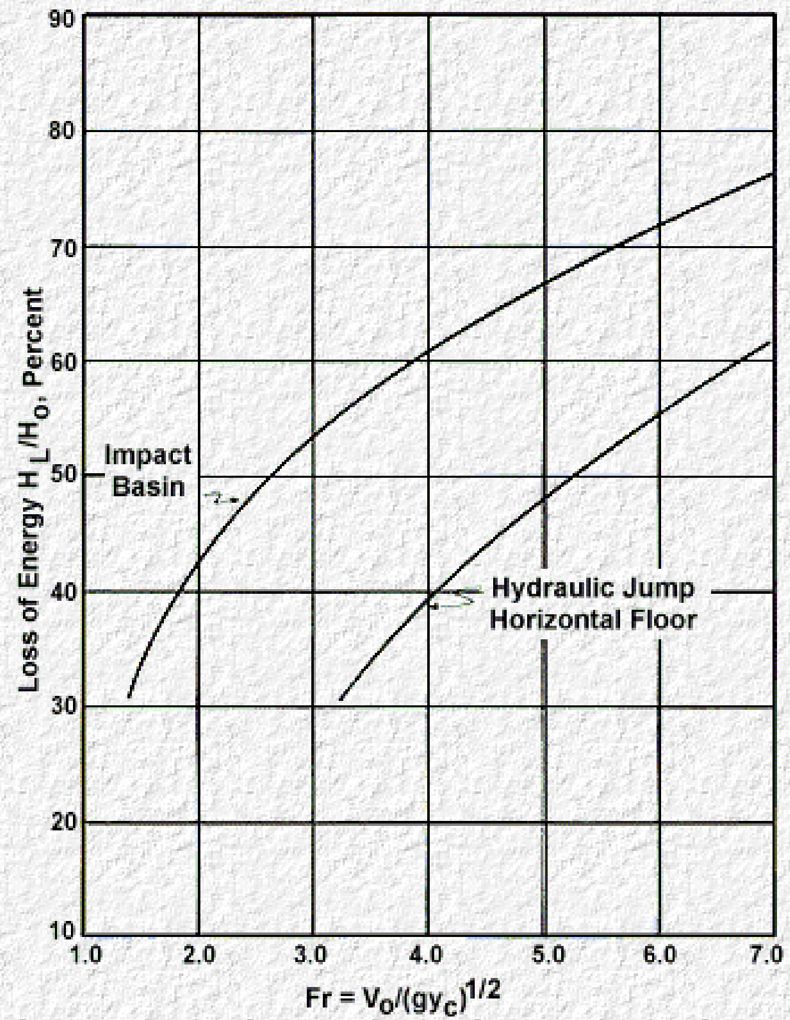


Figure 8-C-3. Energy Loss-Impact Basin - Hydraulic Jump

Table 8-C-1. Baffle Wall Dissipator

Dimensions of Basin in Meters														
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

1.22	0.94	1.65	0.46	0.20	0.71	0.94	0.51	0.10	0.33	0.15	0.15	0.15	0.15	0.08
1.52	1.17	2.03	0.58	0.25	0.99	1.17	0.64	0.13	0.43	0.15	0.15	0.15	0.15	0.08
1.83	1.40	2.44	0.69	0.30	1.04	1.40	0.76	0.15	0.51	0.15	0.15	0.15	0.15	0.08
2.13	1.65	2.87	0.79	0.36	1.22	1.65	0.89	0.15	0.58	0.15	0.15	0.15	0.15	0.08
2.44	1.88	3.25	0.91	0.41	1.40	1.88	1.02	0.18	0.66	0.18	0.18	0.15	0.15	0.08
2.74	2.11	3.66	1.04	0.46	1.57	2.10	1.14	0.20	0.76	0.20	0.18	0.18	0.18	0.08
3.05	2.34	4.09	1.14	0.50	1.75	2.34	1.27	0.23	0.84	0.23	0.20	0.20	0.20	0.08
3.35	2.57	4.45	1.27	0.56	1.93	2.57	1.40	0.25	0.91	0.23	0.23	0.20	0.20	0.10
3.66	2.79	4.88	1.37	0.61	2.08	2.79	1.52	0.28	0.91	0.25	0.25	0.20	0.23	0.10
3.96	3.05	5.28	1.50	0.66	2.26	3.05	1.65	0.30	0.91	0.28	0.28	0.20	0.25	0.10
4.27	3.28	5.67	1.60	0.71	2.44	3.28	1.78	0.33	0.91	0.28	0.30	0.20	0.28	0.13
4.57	3.51	6.10	1.70	0.76	2.59	3.51	1.91	0.36	0.91	0.3	0.30	0.20	0.3	0.13
4.88	3.73	6.50	1.83	0.81	2.77	3.73	2.03	0.38	0.91	0.3	0.30	0.23	0.30	0.15
5.18	3.96	6.86	1.93	0.86	2.95	3.96	2.16	0.41	0.91	0.33	0.33	0.23	0.30	0.15
5.49	4.19	7.29	2.03	0.91	3.12	4.19	2.29	0.41	0.91	0.33	0.33	0.25	0.33	0.18
5.79	4.45	7.72	2.16	0.97	3.30	4.45	2.41	0.43	0.91	0.33	0.36	0.25	0.33	0.18
6.10	4.67	8.10	2.29	1.02	3.48	4.67	2.54	0.46	0.91	0.36	0.36	0.25	0.36	0.20

8-D USFS Metal Impact Energy Dissipator

The metal impact energy dissipator was developed by the Bureau of Reclamation for the U. S. Forest Service (8D2). The Forest Service required a dissipator:

1. to use with either helical or corrugated-metal pipe, up to 900-mm (36 inches) or 1 m 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, and
5. with any parts subject to wear easily replaced.

The dissipator developed for an 450-mm (18 inch) pipe will operate with up to 3 meters 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 a 450-mm pipe. The basic design conditions and dimensions are modified to obtain designs for other pipe sizes up to 900-mm

The dissipator is independent of the incoming pipe, i.e., neither attached to nor supported by the pipe, Figure 8-D-2. This leaves an open area between the pipe and the dissipator back plate. To prevent backflow and scour behind the dissipator, a 102-mm wide splash guard is incorporated into the design.

For pipe slopes greater than 40 percent, the addition of two fillets, (Detail X, Figure 8-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 2 contains riprap recommendations.

Figure 8-D-1 provides a family of dimension factor curves for the design of dissipators for an 450-mm corrugated-metal pipe. The dimensions of the 450-mm dissipator are determined by applying the interpolated dimension factor, or the next higher factor, to all dimensions shown in Figure 8-D-2, and to the L_1 and h_i values for the location of the outfall end of the corrugated-metal pipe shown in Figure 8-D-3.

For other size pipes, the dimension factor obtained from Figure 8-D-1 is multiplied by the ratio "C" to determine the scale factor. C is D/STET where, D = nominal corrugated-metal pipe size in millimeters.

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

Figure 8-D-1, Figure 8-D-2, and Figure 8-D-3 are applicable for energy dissipators to be used with 450 cm corrugated-metal pipes. Revised curves, using the multiplier above, should be limited to corrugated-metal pipes not larger than 900 mm.

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:

- Step 1. 450-mm CMP's: With the slope of the CMP and the flow rate known, obtain a dimension factor from the dimension factor chart, Figure 8-D-1. Multiply all dimensions of basic 450-mm dissipator, Figure 8-D-2, by this dimension factor to obtain the dissipator dimensions for 450-mm pipe with the given slope and flow conditions.

- Step 2. Other sizes of CMP's.

  . Compute C and $C^{5/2}$ from C=D/450

  where:

  D is a nominal diameter of the CMP in millimeters.

  b. Compute Q_{450} from $Q_{450} = Q_D/C^{5/2}$

  where:

Q_D is the discharge for the given diameter pipe
Q_{450} is the discharge needed to enter Figure 8-D-1.

c. Enter Figure 8-D-1 with Q_{450} and for the given slope determine the Dimension Factor, DF. Slope is independent of the dimension factor and does not require adjustment.

4. Compute the Scale Factor, SF, from SF=(DF)(C).

5. Apply SF to all dimensions of Figure 8-D-2 to determine dimensions of required dissipator.

Step 3. To install the dissipator at an existing culvert, the L_1 and h_1 dimensions (Figure 8-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 8-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 8-D-4).

The elevation of the dissipator lip and the L_1, h_1 distances in Figure 8-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 8-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 under-cutting. See Chapter 2 for design recommendations.

Design Example No. 1

The necessary input is:

450-mm cmp
30% slope
Q=0.142 m^3/s

From Figure 8-D-1 with Q and S_o, the dimension factor is 0.56. Apply this to the dimension of Figure 8-D-2.

Compute L_1 and h_1, Figure 8-D-3. The distance from the baffle to the backplate is:

0.858(0.56)=480 mm

The splash guard dimension is:

102(.56)=57 mm

The L_1 distance is determined by placing the pipe inside the dissipator:

L_1 = (480 - (57 x 1.25)) = 409 mm

Convert this and enter Figure 8-D-3.

409/0.56=730 mm

This gives an initial h_1 distance of 610 mm, which is converted to 610(0.56) = 342 mm, the final h_1 dimension.

The remaining dimensions are obtained from Figure 8-D-2 and the final design is given in Figure 8-D-5.

Design Example No. 2

To apply the design to a 900-mm pipe with 0.990 m^3/s requires altering Figure 8-D-1 and Figure 8-D-2 using the "C" multipliers:

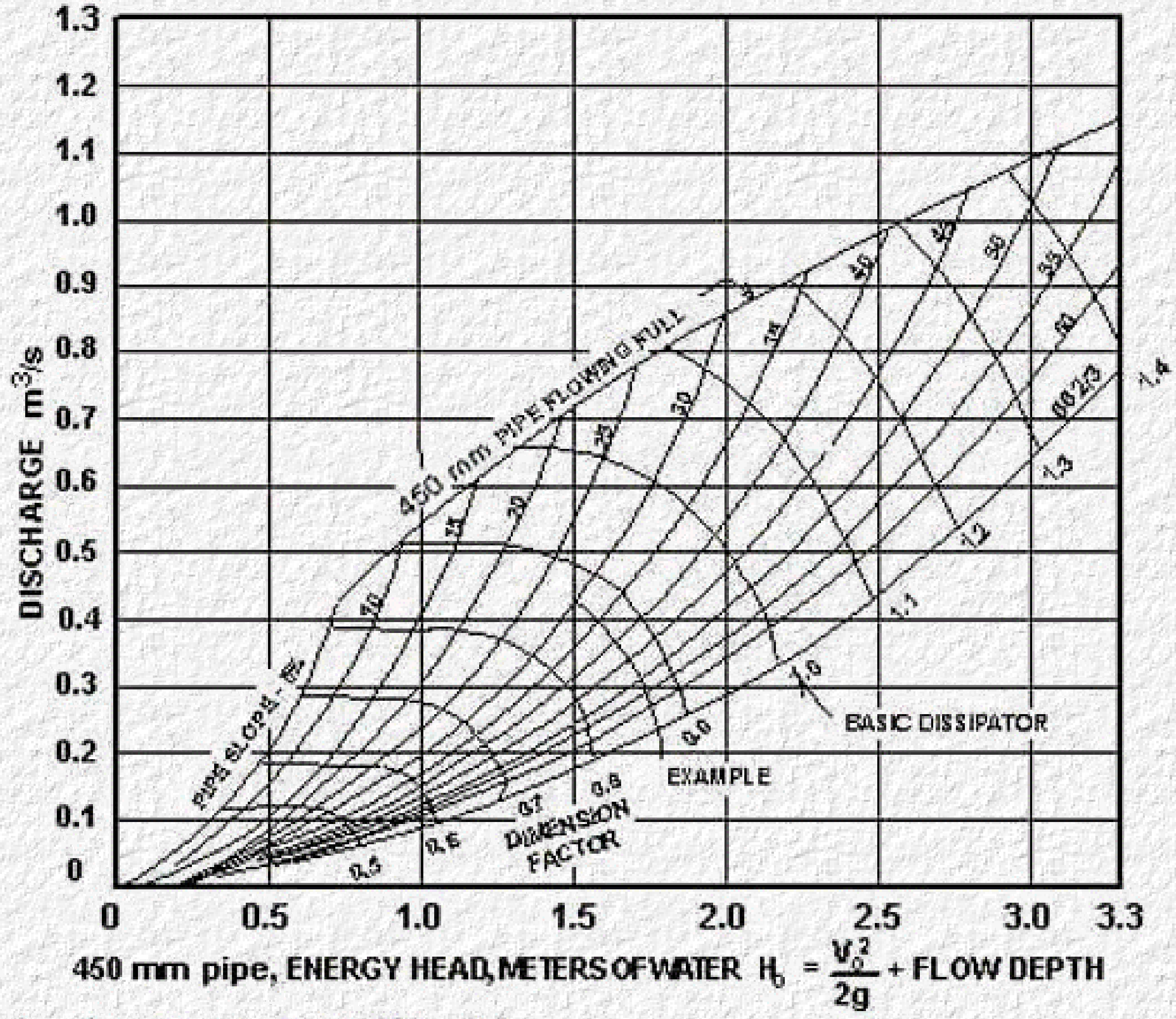
- 1. Start with step 2 for other CMP.
- 2. a. C=D/450=900/450=2
  $C^{5/2}$ =5.66

  b. Q=0.990 m^3/s = Design Discharge for 900-mm pipe.
  Q_{450} =0.990/5.66=0.175 m^3/s

  c. Entering Figure 8-D-2 with this discharge (0.175 m^3/s) gives a dimension factor (DF)=0.62.

  d. SF=DF(C)=.62(2)=1.24.

  e. The dimension in Figure 8-D-2 and Figure 8-D-3 are multiplied by 1.24 to obtain dimensions for the 900-mm pipe.
- 3. L_1 and h_1 can be determined as above.



For pipe slopes greater than 40%, deflector fillers must be used to achieve proper flow distribution and dissipator self cleaning capabilities. Detail Figure 8-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 a 450 mm corrugated pipe installed on a 30% slope and discharging a 0.425 cm, the dimension of the basic energy dissipator. Figure 8-D-2 should be modified by the factor 0.87.

Figure 8-D-1. Dimension Factor Chart for 450-mm Corrugated Pipe (from reference 8D1)

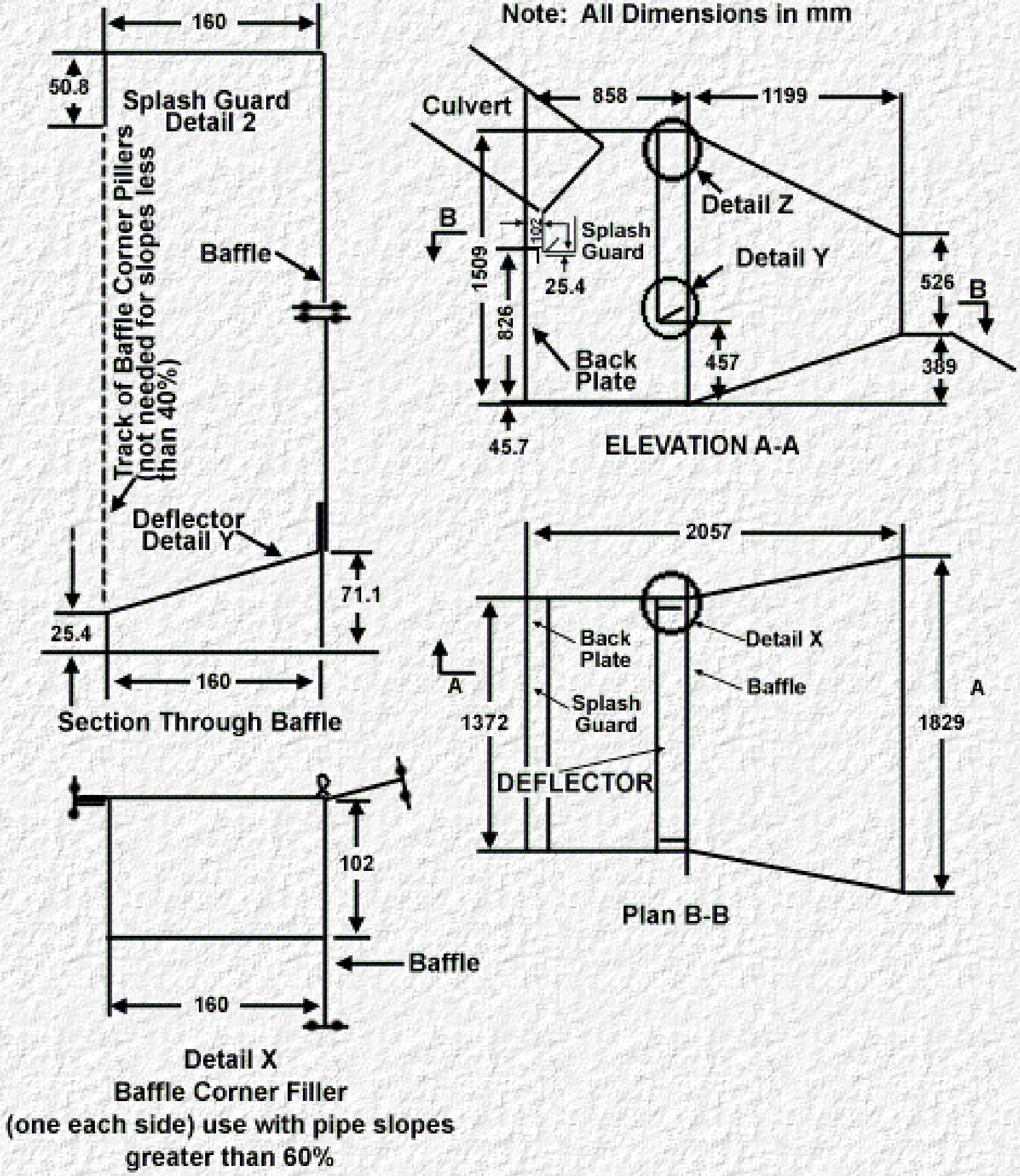
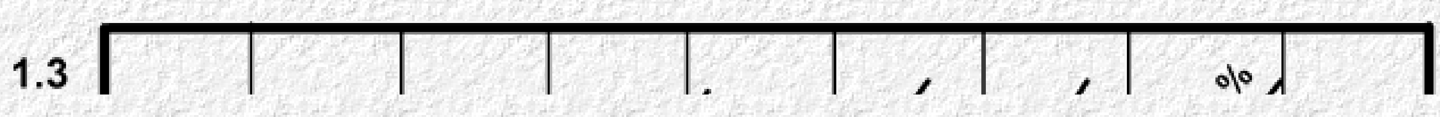
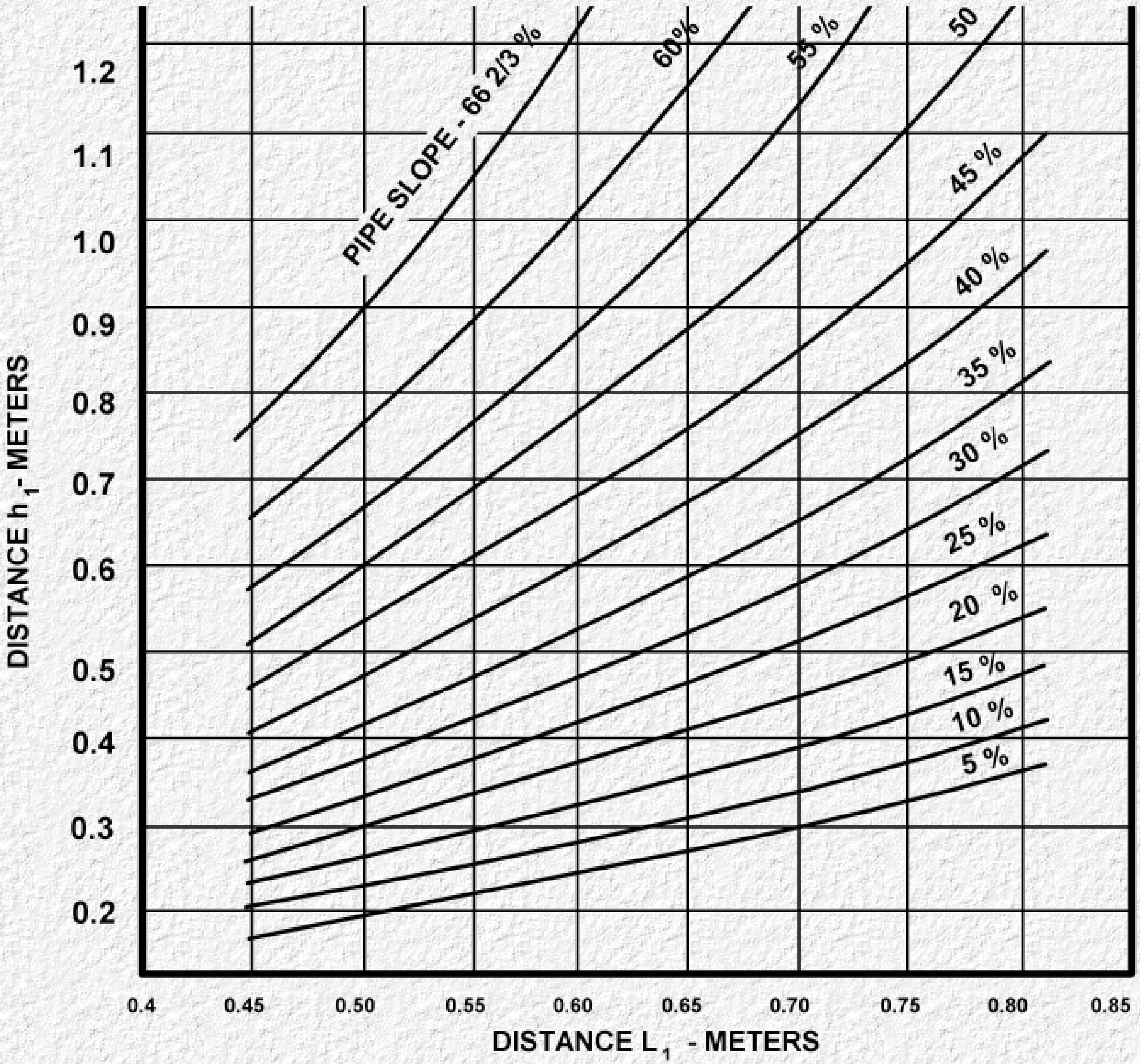
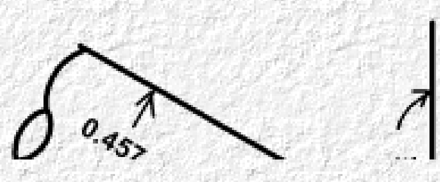


Figure 8-D-2. Basic Energy Dissipator Dimensions - 450-mm Corrugated Pipe (dimension factor = 1.0) (from reference 8D1)





PIPE SLOPE - 66 2/3 %
60%
55 %
50
45 %
40 %
35 %
30 %
25 %
20 %
15 %
10 %
5 %
DISTANCE h1 - METERS
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.4
0.45
0.50
0.55
0.60
0.65
0.70
0.75
0.80
0.85
DISTANCE L1 - METERS



0.457

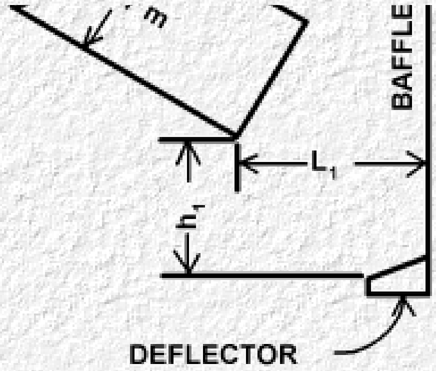


Figure 8-D-3. Location of Invert of Discharge End of 450-mm Corrugated - Metal Pipe for Various Pipe Slopes (from reference 8D1)

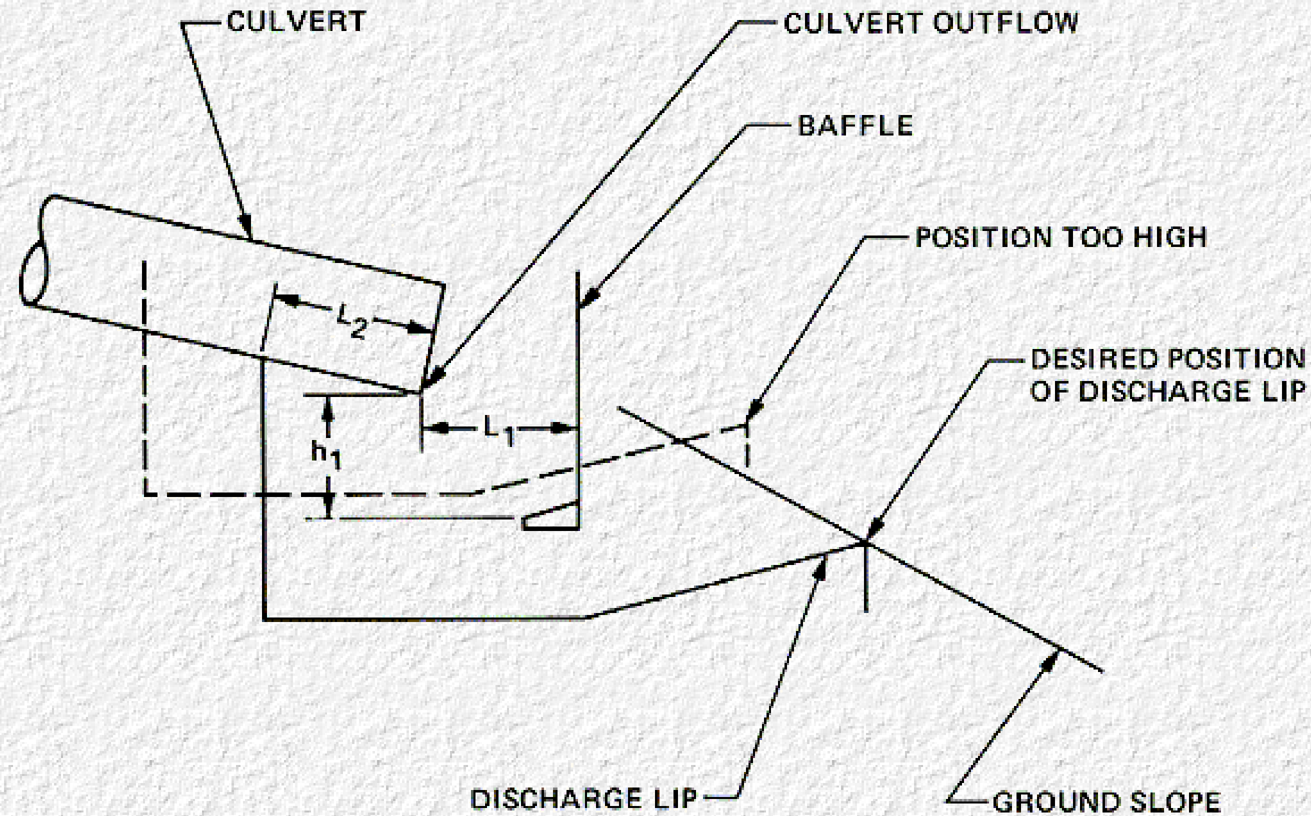


Figure 8-D-4. Installation Sketch

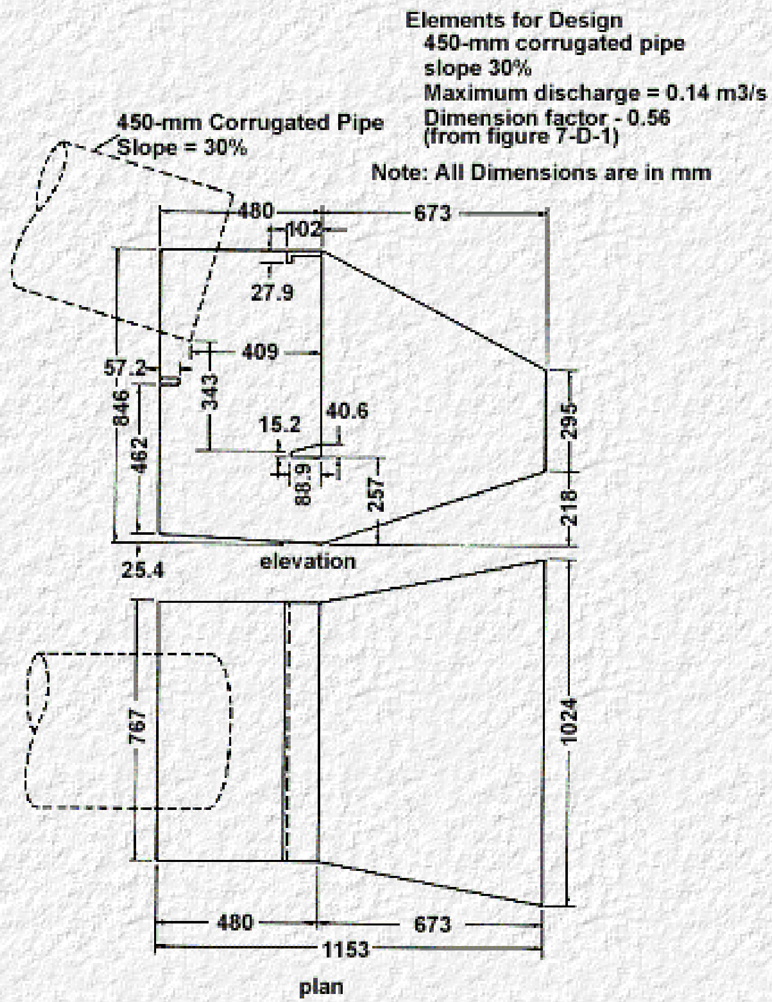


Figure 8-D-5. Energy Dissipator for Dimension Factor of 0.56

8A1. Keim, R. S.,
The Contra Costa Energy Dissipator
Journal of Hydraulic Division, A.S.C.E. Paper 3077,
March 1962, p. 109.

8B1. MacDonald, T. C.,
Model Studies Of Energy Dissipators For Large Culverts,
Hydraulic Engineering Laboratory Study HEL-13-5,
University of California, Berkeley, November 1967.

8C1. *"Hydraulic Design of Stilling Basins,"*
Journal of the Hydraulics Division, A.S.C.E.,
paper 1406, October 1957.

8D1 . Colgate, D.,

Hydraulic Model Studies of Corrugated Metal Pipe Under drain Energy Dissipators,
USBR, REC-ERC-71-10, January 1971.

Go to Chapter 9

Chapter 9 : HEC 14

Drop Structures

Go to Chapter 10

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 transfering 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 napes 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 9-1, can be described by the drop number, defined:

$N_d = q^2 / g h_o^3$

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:

$L_1 / h_o = 4.30\, N_d^{0.27}$
$y_1 / h_o = 1.00\, N_d^{0.22}$
$y_2 / h_o = 0.54\, N_d^{0.425}$
$y_3 / h_o = 1.66\, N_d^{0.27}$

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. See Figure 9-1.

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 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 L_j, is discussed in Chapter 6. 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 9-1), 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 4 Flow Transitions).

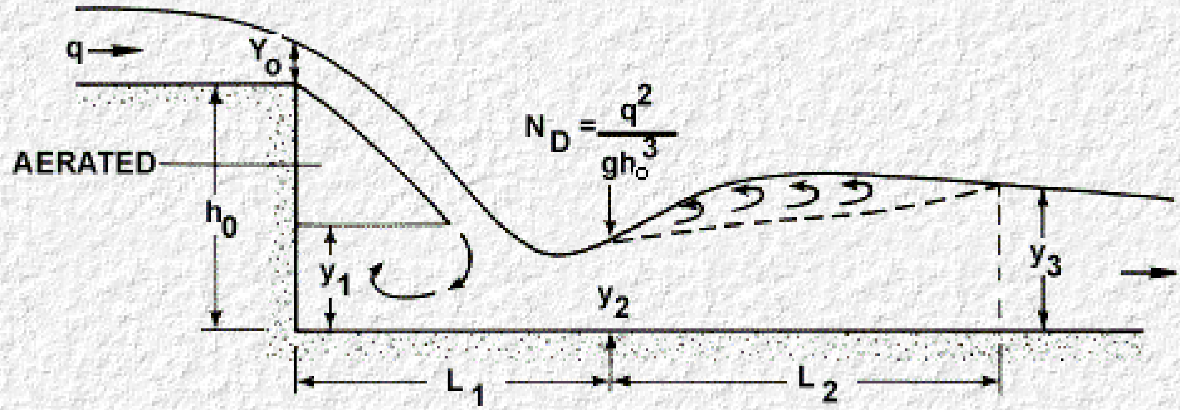


Figure 9-1. Flow Geometry of a Straight Drop Spillway

Grate Design

A grate or series of rails forming a "grizzly" may be used in conjunction with drop structures, Figure 9-2. 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 (reference 6- 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 meters, 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

$$L_G \cong Q/(y_o)^{1/2}$$

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.

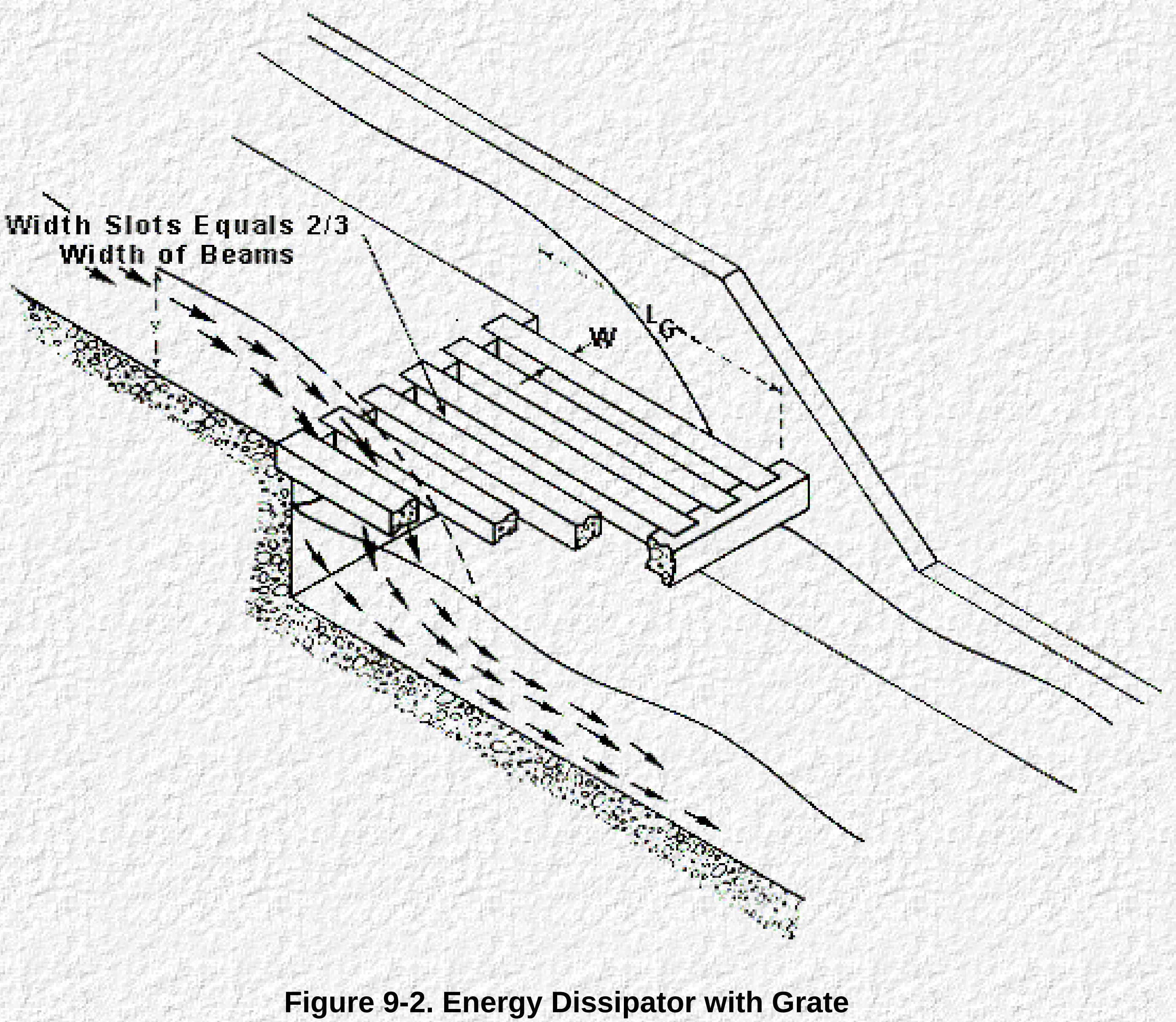


Figure 9-2. Energy Dissipator with Grate

9-A Straight Drop Structure

A general design for a stilling basin at the toe of a drop structure was developed by the Agricultural Research Service, St. Anthony Falls Hydraulic Laboratory, University of Minnesota (9A1). 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 4.6 meters 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 9-A-1 illustrates a straight drop structure which provides adequate protection from scour in the downstream channel.

Design Procedure

- Step 1.Calculate the specific head in approach channel.

$$H = y_o + \frac{V_o^2}{2g} \qquad \text{9-A-1}$$

- Step 2. Calculate critical depth.

$$y_c = \frac{2}{3}H \qquad \text{9-A-2}$$

- Step 3. Calculate the minimum height for tailwater surface above the floor of the basin.

$$y_3 = 2.15\, y_c \qquad \text{9-A-3}$$

- Step 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.

$$h_2 = -\,(h - y_o) \qquad \text{9-A-4}$$

- Step 5. Determine the location of the stilling basin floor relative to the crest.

$$h_o = h_2 - y_3 \qquad \text{9-A-5}$$

- Step 6. Determine the minimum length of the stilling basin, L_B. using:

$L_B = L_1 + L_2 + L_3 = L_1 + 2.55\ y_c$ 9-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:

$L_1 = (L_f + L_s)/2$ 9-A-7

where:

$$L_f = \left(-0.124 + \sqrt{3.195 - 4.368 h_o / y_c}\right) y_c$$

$$L_s = [-0.211 + 0.228(L_t / y_c)^2 - (h_o / y_c)] y_c / [0.185 + 0.456(L_t / y_c)]$$

$$L_t = \left(-0.124 + \sqrt{3.195 - 4.368 h_2 / y_c}\right) y_c$$

or L_1 can be found graphically from Figure 9-A-2.

L_2 is the distance from the point at which the surface of the upper nape strikes the stilling basin floor to the upstream face of the floor blocks, Figure 9-A-1. This distance can be determined by:

$L_2 = 0.8\ y_c$ 9-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:

$L_3 > 1.75\ y_c$ 9-A-9

- Step 7. Proportion the floor blocks as follows:

  . height is $0.8\ y_c$,

  b. width and spacing should be $0.4\ y_e$ with a variation of $\pm 0.15\ y_c$ permitted,

  c. blocks should be square in plan, and

  d. blocks should occupy between 50 percent and 60 percent of the stilling basin width.

- Step 8. Calculate the end sill height, $(0.4\ y_c)$.

- Step 9. Longitudinal sills, if used, should pass through,not between, the floor blocks. These sills are for structural purposes and are neither beneficial nor harmful hydraulically.

- Step 10. Calculate the sidewall height above the tailwater level, $(0.85\ y_c)$.

- Step 11. Wingwalls should be located at an angle of 45° with the outlet centerline and have a top slope of 1 to 1.

Step 12. Modify the approach channel as follows:

. 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 protect with riprap or paving for a distance upstream from the headwall equal to three times the critical depth, y_c. See Chapter 2 for recommendations on riprap design.

Step 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:

$Q = 7.075\ m^3/s$
$S_o = 0.002$ m/m, and the downstream channel has 3H:1V side slopes with a 3.048 meter bottom
$n = 0.03$
Normal depth of flow, $y_o = 1.024$ m
Normal velocity, $V_o = 1.128$ m/s.
Vertical drop, $h = 1.829$ m

Find:

Straight drop structure dimensions.

Solution:

1. $H = 1.024\text{ m} + \dfrac{(1.128\text{m}/\text{s})^2}{2\times 9.81\text{m}/\text{s}^2} = 1.089\text{ m}$

2. $y_c = (2/3)\ (1.089\text{ m}) = 0.726$ m

3. $y_3 = 2.15\ (0.726\text{ m}) = 1.561$ m

4. $h_2 = -\ (1.829 - 1.024) = -0.805$ m

5. $h_o = -0.805 - 1.561 = -2.366$ m
The floor of the stilling basin is, therefore, 0.537 m below the grade line of the downstream channel.

- 6. h_o / y_c = - 2.366/0.726 = -3.26
  h_2 / y_c = -0.805/.726 = -1.11

From Figure 9-A-2

$L_1 / y_c = 3.95$

$L_1 = 2.868$ m

$L_2 = 0.8\ y_c = (0.8)(0.726) = 0.581$ m

$L_3 \geq 1.75\ y_c = 1.75(0.726) = 1.271$ m or 1.28 m (4.20 ft)

$L_B = 2.868 + 0.581 + 1.28 = 4.73$ m

- 7. Proportion floor blocks
  . Height = 0.8 y_c = 0.8(0.726) = 0.581 m
  b. Width = 0.4 y_c = 0.4(0.726) = 0.290 m
  Spacing = 0.4 y_c = 0.4(0.726) = 0.290 m

- 8. Calculate end sill height = 0.4 y_c = 0.4(0.726) = 0.290 m

- 9. Use longitudinal sills passing through the floor blocks.

- 10. Calculate sidewall height above tailwater =
  0.85 y_c = 0.85(0.726) = 0.617 m

- 11. Locate wingwalls at 45° angle with outlet centerline.

- 12. Protect approach channel with riprap or paving for
  2.2 m (3 X 0.726) upstream of the headwall.

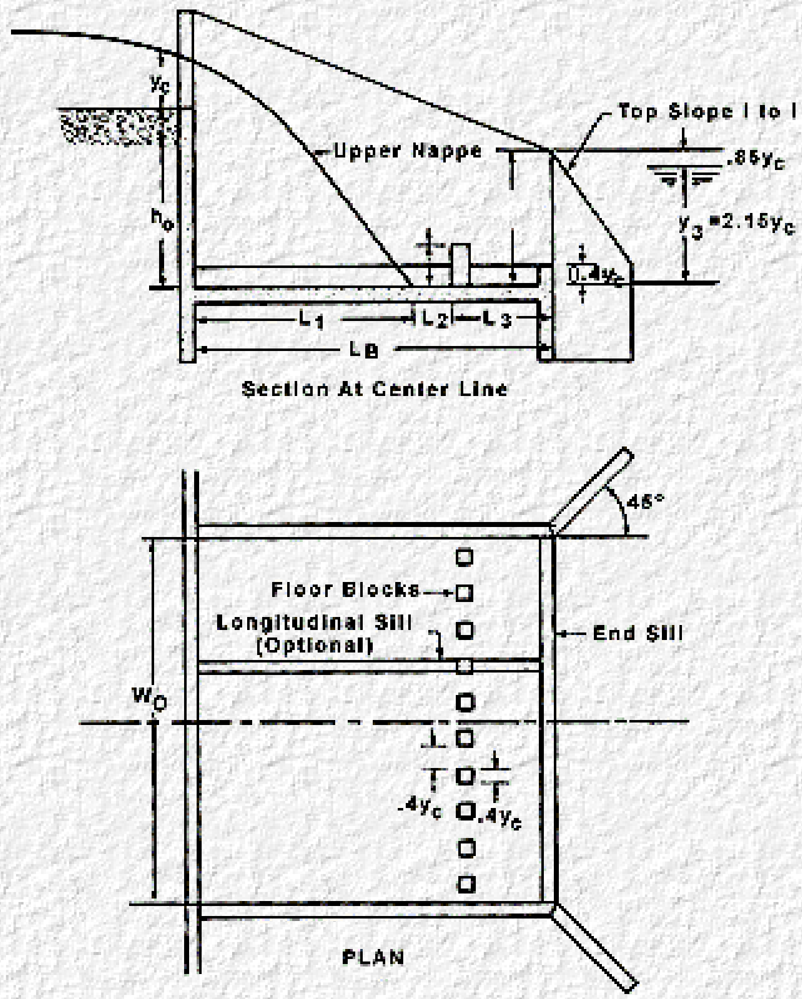


Figure 9-A-1. Straight Drop Spillway Stilling Basin (from reference 9-1)

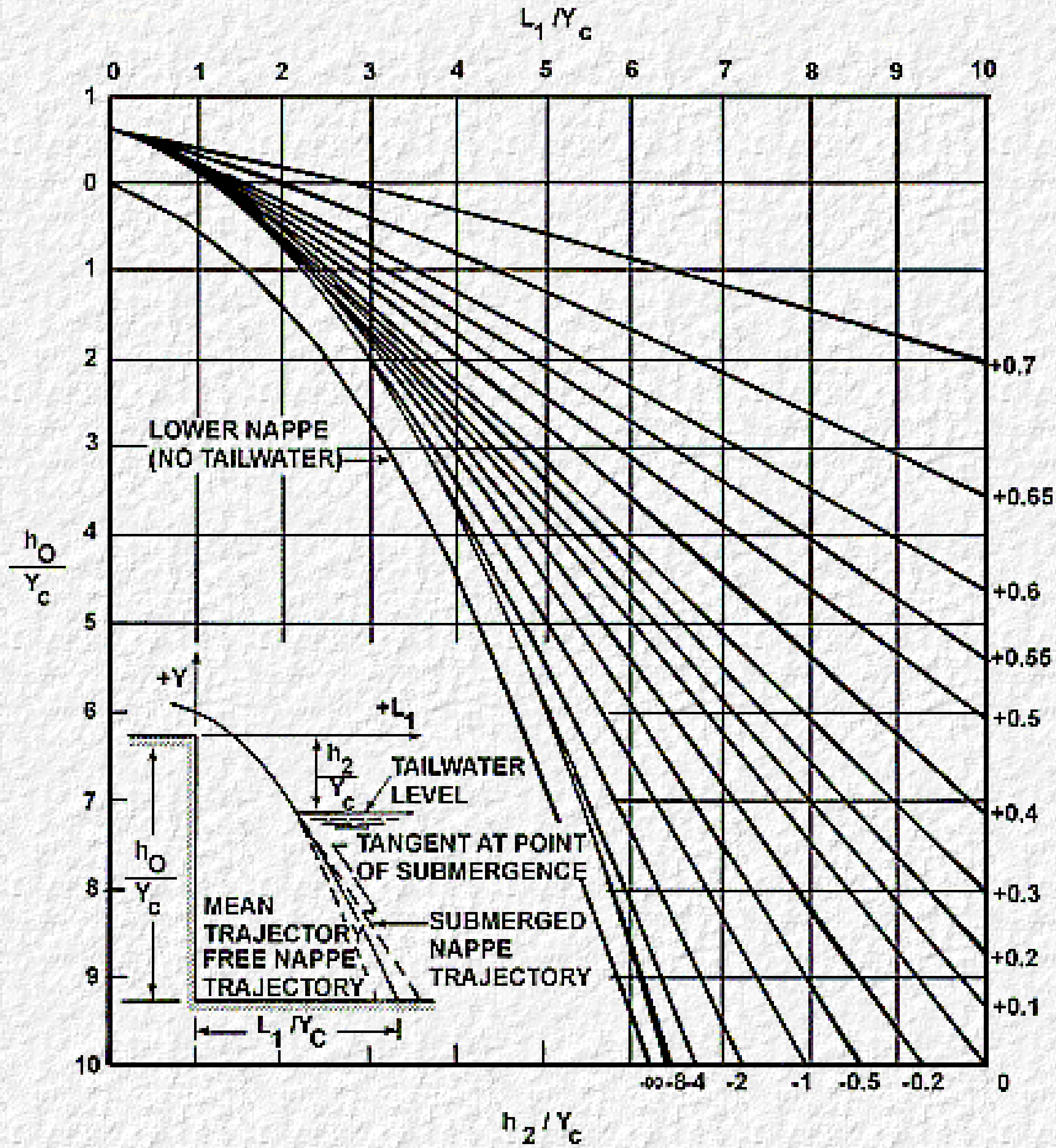


Figure 9-A-2. Design Chart for Determination of L_1

9-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 9-B-1). Water is directed to the crest of the box inlet by earth dikes and a headwalls. 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 0.6 meters to 3.7 meters.

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, (9B1).

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:

$$Q=1.89L_c\, y_o^{3/2}$$

Solving for head

$$y_o = [Q/1.89\, L_c]^{2/3} \qquad \text{9-B-1}$$

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 9-B-1 solved to obtain a head acceptable for the existing conditions.

The discharge coefficient (1.89) in Equation 9-B-1 must be multiplied by:

. The correction for head given in Figure 9-B-2.

b. The correction for box inlet shape given in Figure 9-B-3.

c. The correction for approach channel width givenin Figure 9-B-4.

d. The correction for dike proximity to the box inlet crest given in Table 9-B-1--these values have a low precision.

It is not necessary to make these corrections 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 $\pm$7 percent when there is no dike effect and $\pm$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:

$$Q=C_2 W_2 (2g)^{1/2} (y_o + C_H)^{3/2}$$

Solving for head

$$y_o = Q^{2/3} / [C_2 W_2 (2g)^{1/2}]^{2/3} - C_H \quad \text{9-B-2}$$

The discharge coefficient, C_2, is obtained from Figure 9-B-7.

The head correction C_h is given in Figure 9-B-6. If h_o/W_2 is between 1/4 and1, C_H may be more readily determined from Figure 9-B-7.

The precision of the design curves for headwall control is probably within $\pm$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" (3) 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:

$$y_c = [Q/W_2)^2 /g)]^{1/3} \quad \text{9-B-3}$$

Critical depth at the exit of the stilling basin is:

$$y_{c3} = [(Q/W_3)^2 /g]^{1/3} \quad \text{9-B-4}$$

The minimum length of the straight section is:

$$L_2 = y_c (.2/(L_1 /W_2) + 1) \quad \text{9-B-5}$$

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:

$$L_3 = L_c /(2L_1 /W_2) \quad \text{9-B-6a}$$

or

$$L_3 = z(W_3 - W_2)/2 \quad \text{9-B-6b}$$

which ever is larger. However, Equation 9-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:

$$y_3 = 1.6y_{c3} \quad \text{9-B-7}$$

When the stilling basin is more than 11.5 y_{c3} wide at the exit, the minimum tailwater depth over the basin floor is

$$y_3 = y_{c3} + 0.052W_3 \quad \text{9-B-8}$$

However, a stilling basin as wide as $11.5y_{c3}$ may make inefficient use of the outlet.

The height of the end sill is

$$h_4 = y_3 / 6 \qquad \text{9-B-9}$$

Longitudinal sills will improve the flow distribution in the outlet.
Considerations for their use are:

. 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:

$$h_3 = y_3 / 3 \qquad \text{9-B-10}$$

The sidewalls should extend above the tailwater surface under all conditions.

The wingwalls should be triangular in elevation and have 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 9-B-1. Correction for Dike Effect, C_E
(Control at Box-Inlet Crest)

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

- Step 1. Select h_o.
- Step 2. Select L_1, W_2, and L_c.

- Step 3. Calculate y_o for the crest, Equation 9-B-1.

- Step 4. Calculate h_o / W_2 and determine coefficient of discharge, C_2; Figure 9-B-5.

- Step 5. Calculate L_1 / h_o and determine relative head correction, C_H; Figure 9-B-6.

- Step 6. Calculate y_o for the headwall opening; Equation 9-B-2.

- Step 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 9-B-2.

  b. Calculate L_1 / W_2 and determine correction for box inlet shape, C_s; Figure 9-B-3

  c. Calculate W_1 / L_c and determine correction for approach channel width, C_A; Figure 9-B-4.

  d. Calculate W_4 / W_2 and determine correction for dike effect, C_E; Table 9-B-1.

  e. Determine adjusted y_o for crest from corrections found in steps a through e.

- Step 8. Calculate y_c; Equation 9-B-3

- Step 9. Calculate y_{c3}; Equation 9-B-4

- Step 10. Calculate L_2; Equation 9-B-5

- Step 11. Calculate L_3; Equation 9-B-6a or Equation 9-B-6b

- Step 12. Calculate y_3
  If $W_3 < 11.5y_{c3}$; use Equation 9-B-7
  If $W_3 > 11.5y_{c3}$; use Equation 9-B-8

- Step 13. Calculate h_4; Equation 9-B-9

- Step 14. Determine number of longitudinal sills,
  If $W_3 < 2.5W_2$, use two sills.
  If $W_3 > 2.5W_2$, use four sills.

- Step 15. Calculate h_3; Equation 9-B-10

Example Problem

Given:

$Q = 7.075\ m^3/s$
TW=0.853 meters
S_o =0.002 m/m, and the downstream channel has 2H:1V side slopes with a 6.096 meter bottom.
Find:

A box inlet drop structure dimensions.
Solution:

- 1. Select h_o =1.219 meters (4 ft)

- 2 . Select L_c =3.658 meters (12 ft)
  L_1 =1.219 meters
  W_2 =1.219 meters

- 3. Compute y_o for crest control.
  $y_o =[Q/ (1.89 L_c)]^{2/3} =(7.075 / 1.89 \times 3.658)^{2/3} = 1.016$ m

- 4. Calculate h_o /W_2 =1.219/1.219=1.0
  $C_2\sqrt{2g} = 1.905$ from Figure 9-B-5.
  C_2 =0.43

- 5. Calculate L_1 /h_o =1.219/1.219=1.0
  C_H /h_o =0.49, Figure 9-B-6.
  C_H =0.597

- 6. Compute y_o for headwall control.
  $y_o =Q/[c_2 W_2 (2g)^{1/2}]^{2/3} -C_H$

  $=7.075/[(0.43) (1.219) (19.620)^{1/2}]^{2/3} -0.597$

  =1.505 m

- 7. The head for the headwall controls, 1.505>1.016, so steps 7a through 7e are omitted.

- 8. $y_c =[(Q/W_2)^2 /g]^{1/3} =[(7.075/1.219)^2 /9.81]^{1/3}$

  =1.509 m

- 9 $y_{c3}=[(Q/W_3)^2/g]^{1/3}$

where W_3 =6.096 meters = downstream channel bottom width
$y_{c3} =[(7.075/6.096)^2 /9.81]^{1/3} = 0.516$ m

- 10. Calculate L_2.
$L_2 = y_c (0.2/(L_1 /W_2)+1) =1.509(0.2/(1.219/1.219)+1)$

$=1.811$ m

- 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_2=1.219/1.219=1 the first equation is valid:

L_3 =3.658/(2(1.219 /1.219))=1.829 m
L_3 =2.0(6.096 -1.219)/2=4.877 m

use L_3 =4.877 meters (16 ft)

- 12. $11.5(y_{c3})=11.5(0.516)=5.934< W_3$
so $y_3 = y_{c3} +0.052W_3$

y_3 =0.516+.052(6.096)=0.833 m

- 13. $h_4 = y_3 /6$=0.833/6=0.139 m

- 14. Longitudinal sills.
$2.5W_2$ =2.5(1.219)=3.048 m
W_3 >3.048 so use 4 sills

- 15. $h_3 = y_3 /3$=0.833/3=0.278 meters

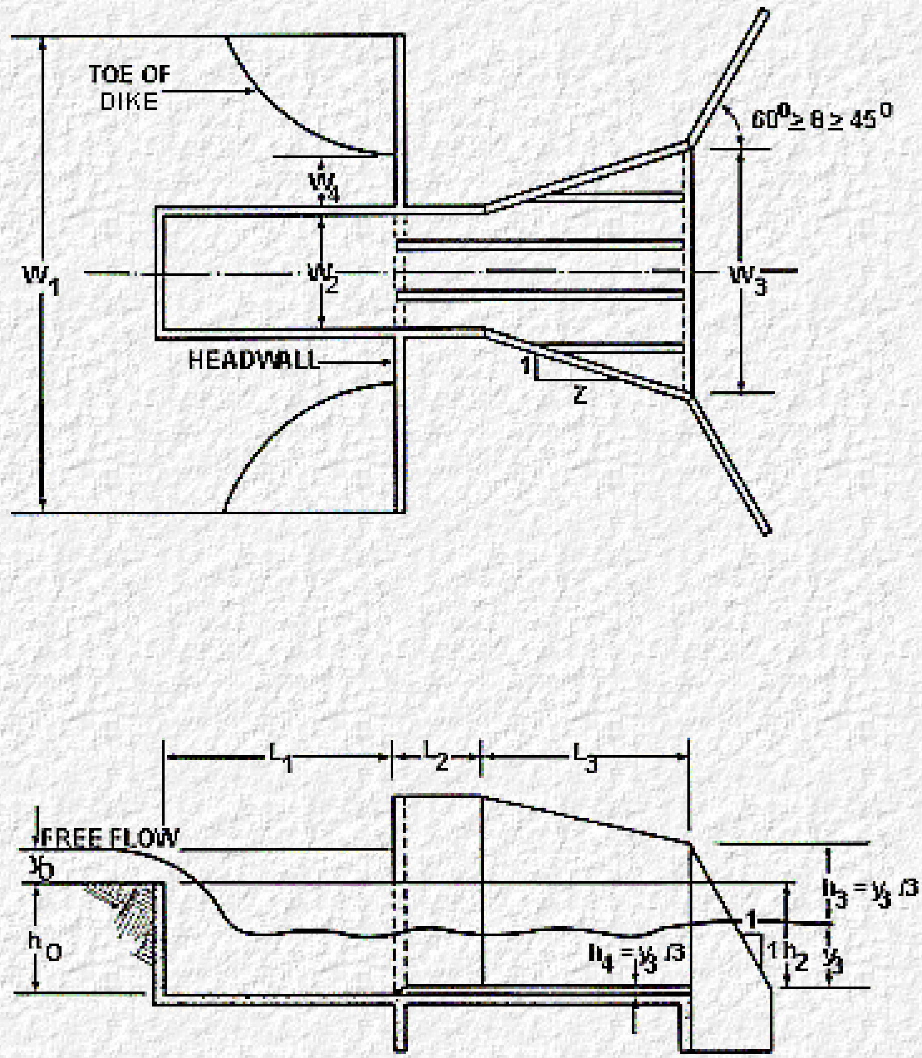


Figure 9-B-1. Box-Inlet Drop Structure

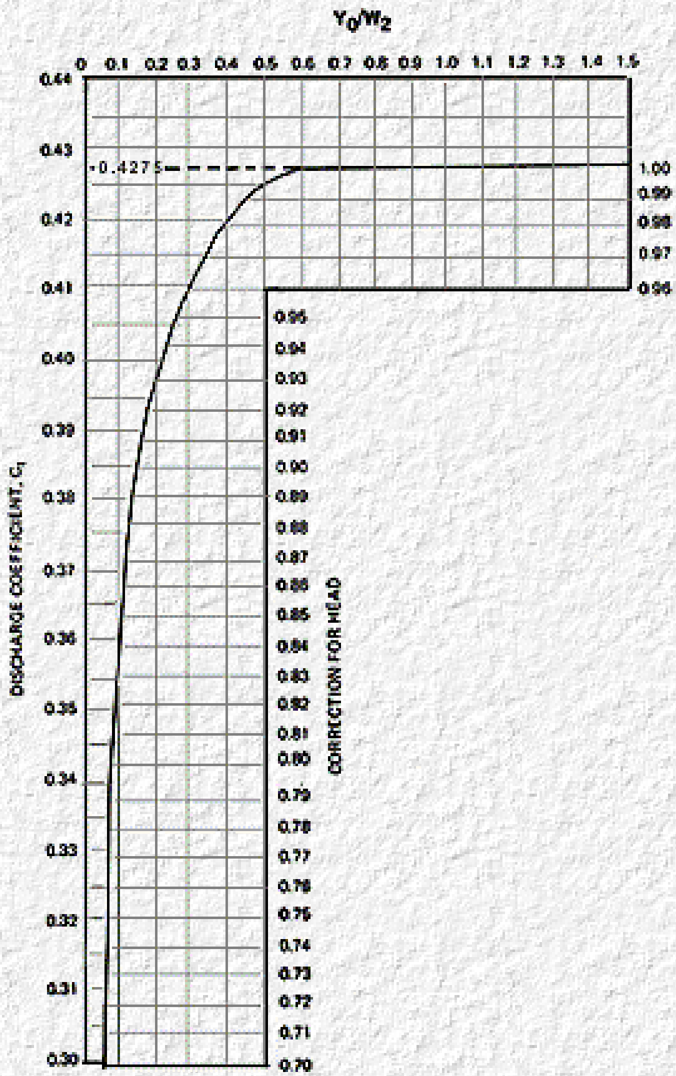


Figure 9-B-2. Discharge Coefficients and Correction for Head, with Control at Box-Inlet Crest

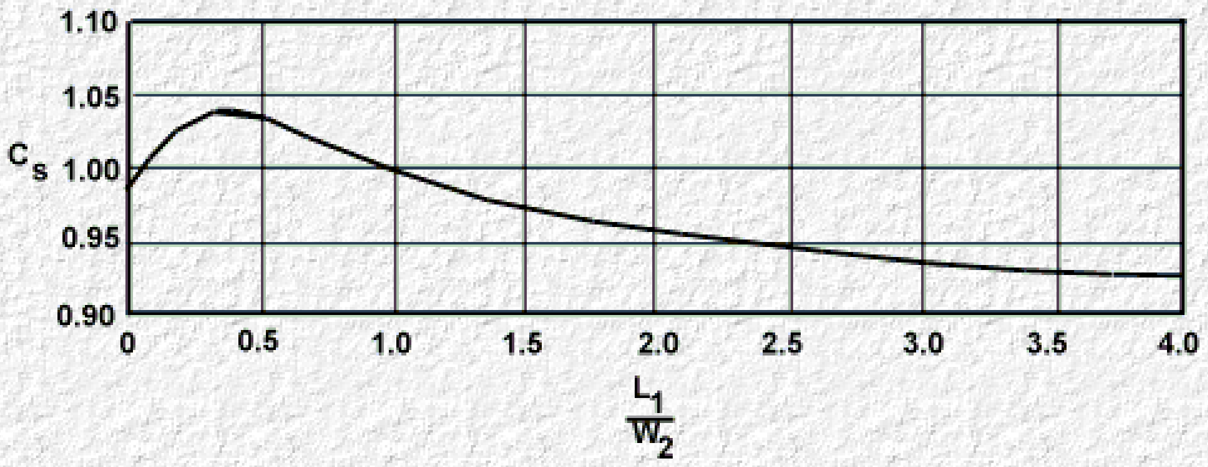


Figure 9-B-3. Correction for Box-Inlet Shape, with Control at Box-Inlet Crest $(\frac{W_1}{L_c} \geq 3)$

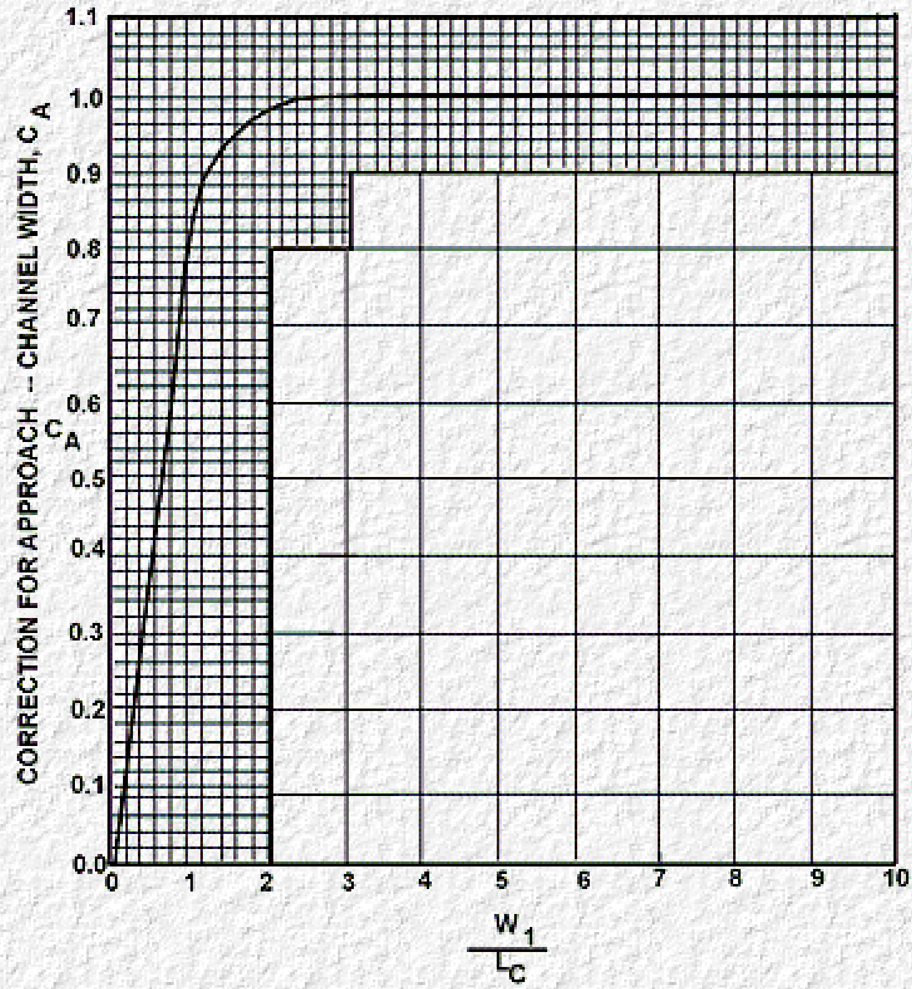


Figure 9-B-4. Correction for Approach Channel Width, with Control at Box-Inlet Crest

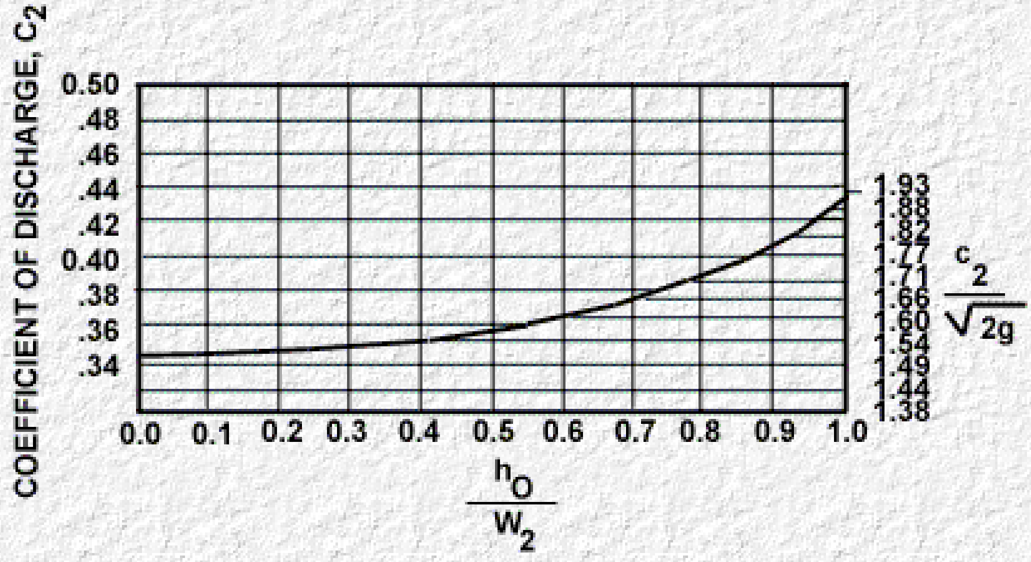


Figure 9-B-5. Coefficient of Discharge, with Control at Headwall Opening

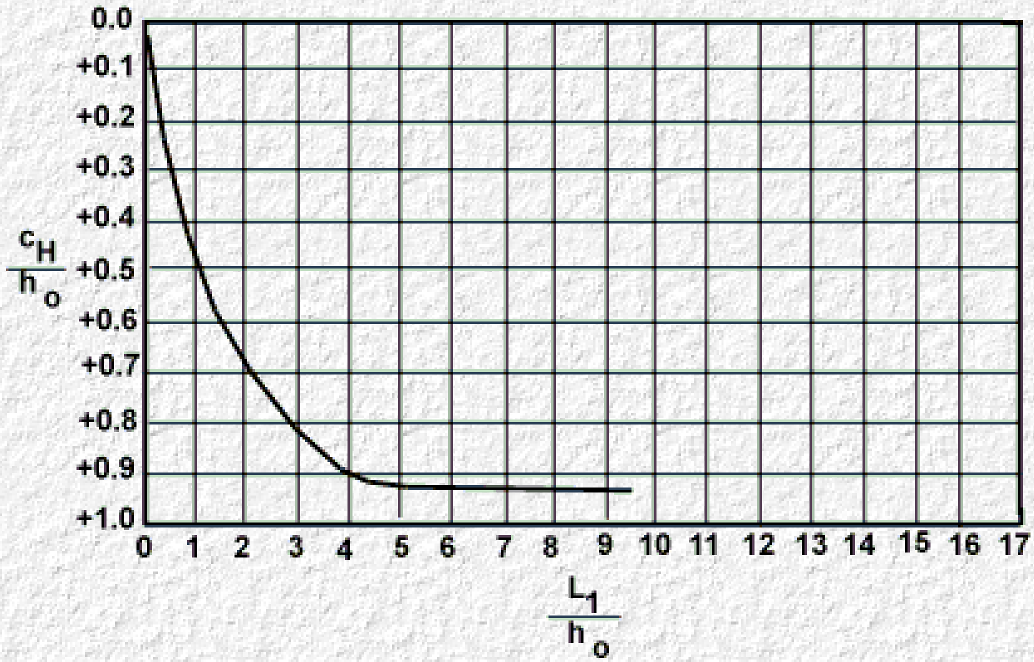


Figure 9-B-6. Relative Head Correction for $h_o/W_2 \geq 1/4$, with Control at Headwall Opening

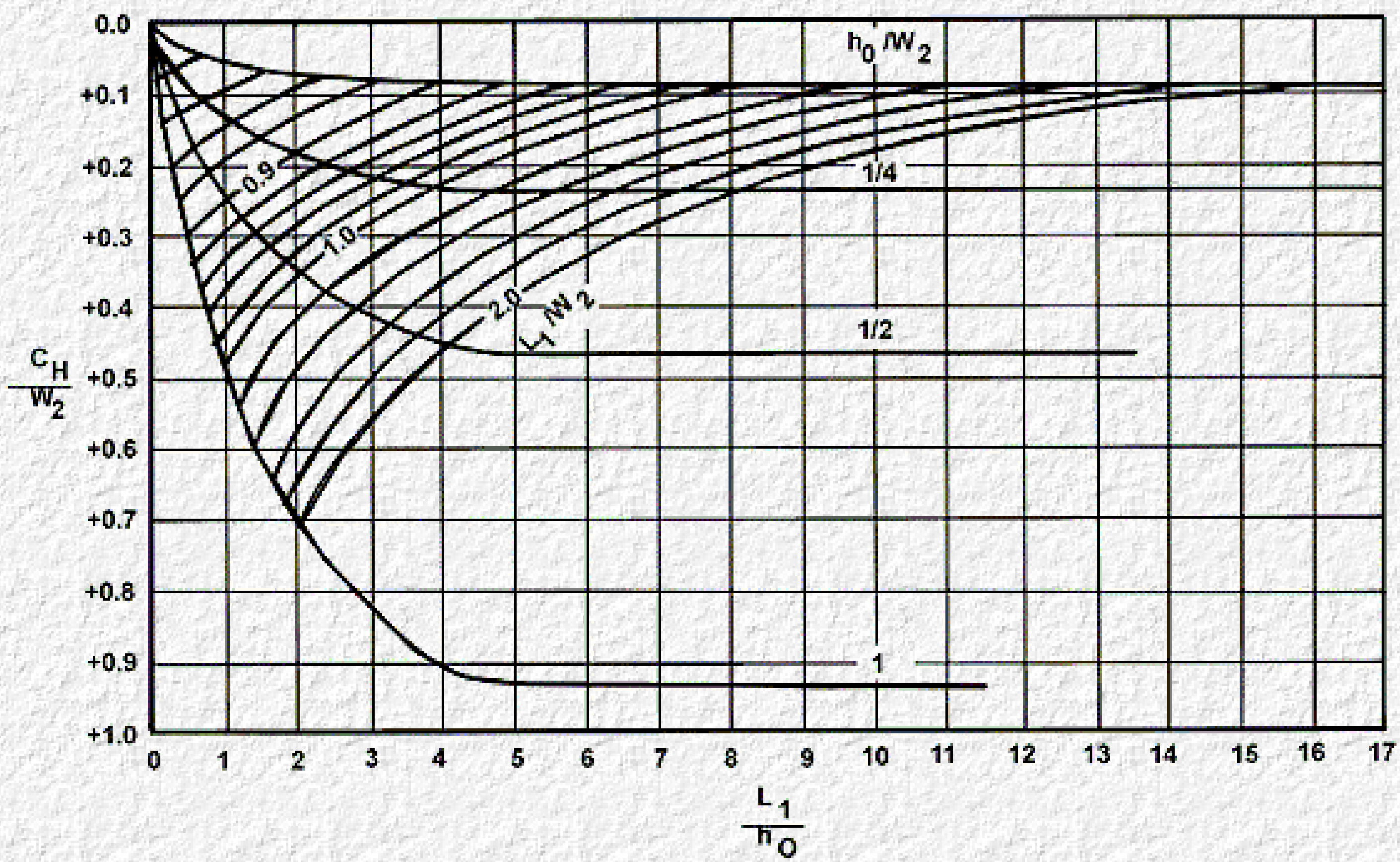


Figure 9-B-7. Relative Head Correction, with Control at Headwall Opening

9-1. Rand, Walter,
Flow Geometry At Straight Drop Spillways,
Paper 791, Proceedings, ASCE, Volume 81, pp. 1-13, September 1955.

9A1. 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.

9B1. Blaisdell, Fred W., and Donnelly, Charles A.,
The Box Inlet Drop Spillway And Its Outlet,
Transactions, ASCE, Vol. 121, pp. 955-986, 1956.

9-2, Chow, Ven Te,
Open-Channel Hydraulics,
McGraw-Hill Book Company, Inc., New York, 1959.

Go to Chapter 10

Chapter 10 : HEC 14

Stilling Wells

[Go to Chapter 11]

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.

10-A Manifold Stilling Basin

The manifold stilling basin (see Figure 10-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 10-A-1, it employs jets issuing upward into an overlying tail water. 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 10-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:

1. the manifold is an efficient energy dissipator.
2. it can be used where the total drop in elevation of the water surface is several tens of meters,
3. it has no high outlet velocity or concentrations of flow which can be directed against the bed or banks,
4. it can be used in either open channels or closed conduits, and
5. it is an economical structure to build.

On the other hand, its disadvantages are:

1. it needs a fairly deep tailwater to function efficiently,
2. it may be subject to clogging if debris is a problem and would require a debris control structure upstream of its entrance, and
3. 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 meter of length are approximately the same at all points. 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.

   $$L_B / \sqrt{A} < 10 \qquad \text{10-A-1}$$

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_2 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 10-A-1.

Table 10-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).

   $$L_3 = Q/(NV_1 W_B) \qquad \text{10-A-2}$$

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 10-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 10-A-4 gives the boil height (h_B) in the same manner that Figure 10-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 10-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 10-A-1.

5. Use Equation 10-A-2 to find L_3.

6. With the aid of Figure 10-A-3 find h_w. The amount of erosion protection required will depend on site conditions.

7. Figure 10-A-4 is used to find h_B .

Example Problem

The following example problem was taken from the paper by Fiala and Albertson (10A1).

Given:

$Q = 8.490\ m^3/s$
D of pipe = 1829 mm
tail-water depth (y) = 2.438 m.

Find:

Design a manifold stilling basin for the outlet of the pipe so that the wave height run-up on the 2H:1V side slope of the channel downstream does not exceed 0.457 m.

Solution:

1. To help provide the assumed uniform velocity distribution at the manifold entrance, use a transition of length 2D (2 x 1.829 = 3.658 m) from the circular pipe to the square entrance of the 1.829 x 1.829 entrance dimension of the manifold.

2. Assume an $L_B / \sqrt{A}$ ratio of 4. Then $L_B = 4\sqrt{3.345} = 4(1.829) = 7.316\ m$

3. Also assume, for ease of construction that $L_1 = L_2 = 305$ mm which makes the number of slots (N)=$L_B /(L_2 + L_1)$=7.316/0.610= 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=8.490 / [π(1.829)^2 /4]=3.231 m/s. The y/L_1 ratio = 2.438 /0.305 = 8. This lies within the experimental range of 5 to 20 and from Table 10-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. $V_1 = 1.24V_o = 1.24(3.231)=4.006$ m/s.

5. The effective jet width (L_3)=$Q/(NV_1W_B)$ or L_3 =8.490/12(4.006)1.829=0.096 m.

6. Obtain wave height (h_W) from Figure 10-A-3. For L_2 /L_1 =1.0 and y/L_3 =2.438/0.096=25.4, $h_W /V_1^2 /(2g)=0.42$ $h_W =0.42(4.006)^2 /19.62 =0.344$ m. Which is less than the 0.457 m. set up as a design condition.

7. If the boil height (h_B) is desired, see Figure 10-A-4. For y/L_3 of 25.4 and L_2/L_1 =1.0, $h_B/V_1^2 /(2g) = 0.3$; $h_B = 0.3(4.006)^2 /19.62 =0.245$ m.

Design Summary

L_B = 7.3 m	A=3.3 m^2	$L_1 = L_2$ = .3 m	N =12
V_o=3.2 m/s	V_1=4.0 m/s	y =2.4 m	
L_3 = 0.1 m	h_W=0.34 m	h_B = 0.25 m	

Prototype Installations

In the discussion and closure of the paper by Fiala and Albertson (10A1), 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 = 6.37 cubic meters per second, vertical drop 11 m. 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 = 5.66 cubic meters per second, vertical drop 10.4 m, minimum tailwater depth 1.83 m. Site No. 3 - Triple manifold installation for conduits through earth-storage dam. Longitudinal axis aligned with flow. Q = 14.1 cubic meters per second.

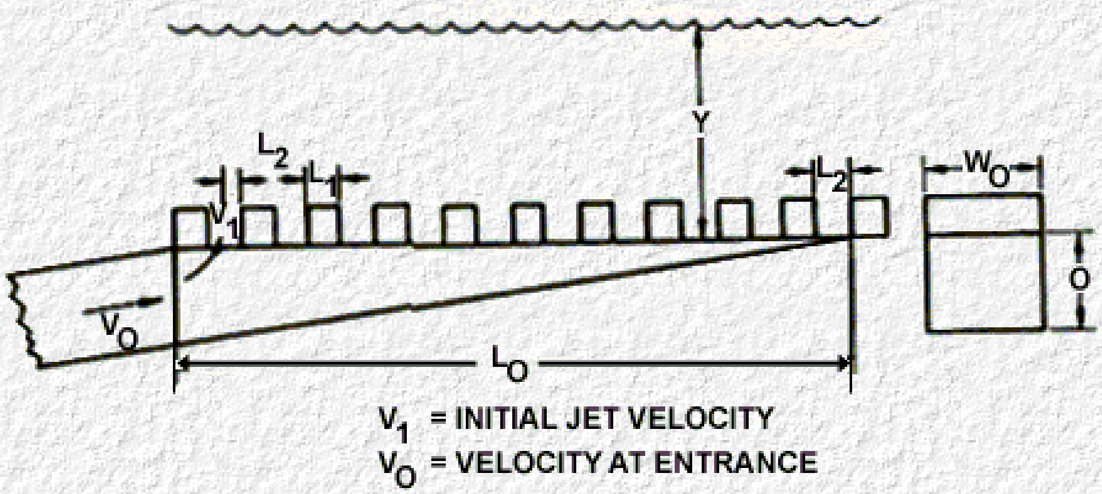


Figure 10-A-1. Manifold Stilling Basin Sketch

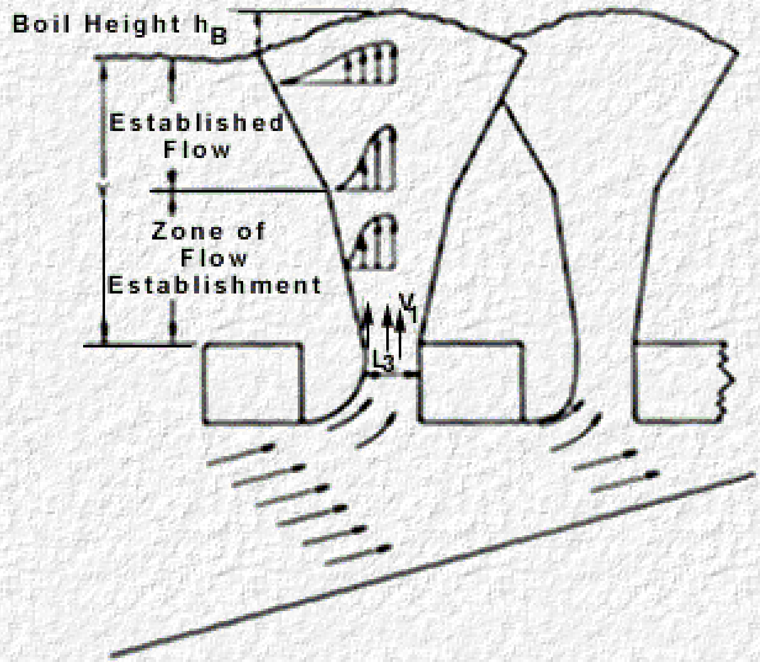


Figure 10-A-2. Jet Diagram for Manifold Stilling Basin (from reference 10A1)

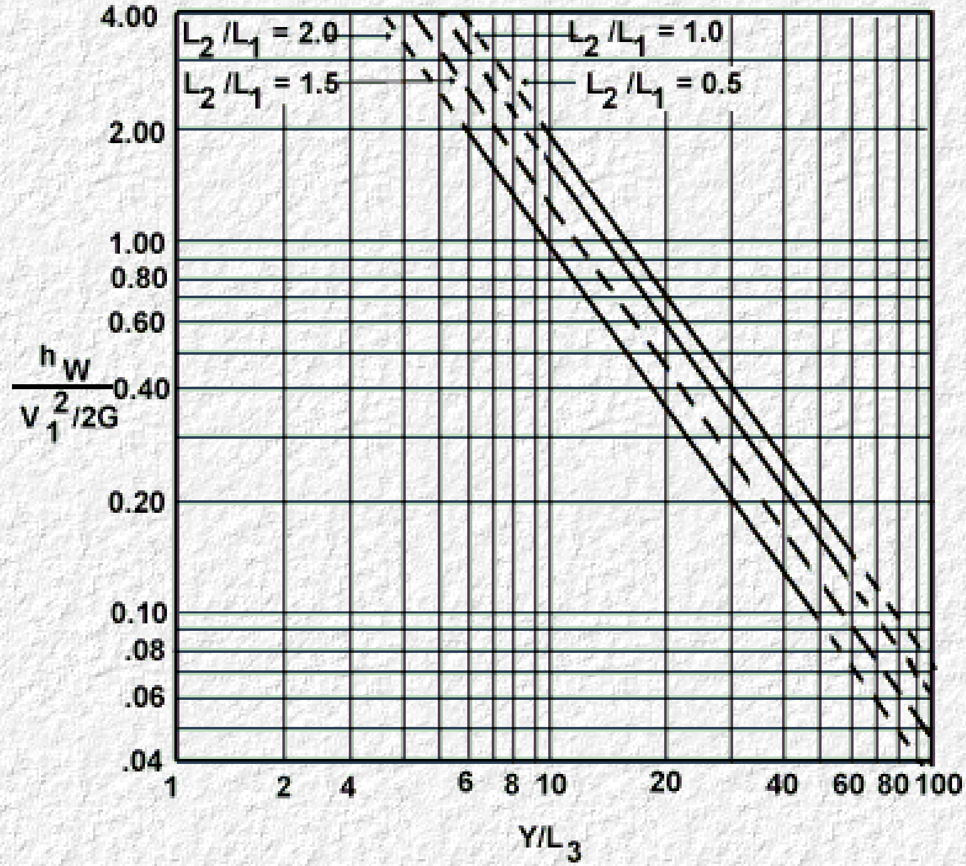


Figure 10-A-3. Wave Height (h_W) for Manifold Stilling Basin (from reference 10A1)

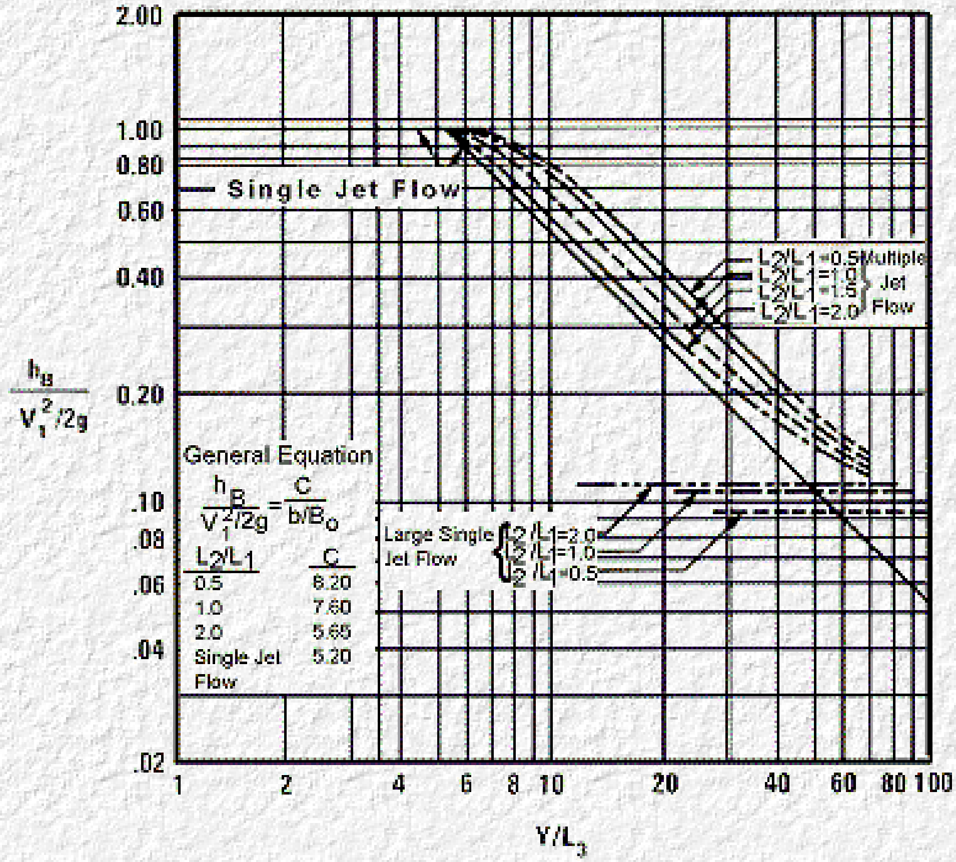


Figure 10-A-4. Boil Height (h_B) for Manifold Stilling Basin(from reference 10A1)

10A1. Fiala, G. R., and Albertson, M. L.,
Manifold Stilling Basin, Journal of Hydraulics Division, ASCE,
HY-4, July 1961, pp. 55-81.

10-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. (10B1 and 10B2).

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 10-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 5.5, 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 10-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 10-B-2 with the slope of the incoming pipe and using the stilling well diameter (D_W) previously obtained from Figure 10-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

- Step 1 Select approach pipe diameter (D) and discharge (Q).
- Step 2. Obtain well diameter (D_W) from Figure 10-B-1.
- Step 3. Calculate the culvert slope = (Vertical/horizontal distance). The depth of the well below the culvert invert (h_1) is determined from Figure 10-B-2.
- Step 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.

- Step 5. The total height of the well (h_W) = h_1 +h_2.

Example Problem

Given:

600 mm CMP down drain on a 2H:1V slope carrying a Q = 0.424 m^3/s

Find:

Stilling well dimensions.

Solution:

- 1. D=0.610 m., Q=0.424 m^3/s
- 2. From Figure 10-B-1. D_W =1.5 D=0.915 m.
- 3. Slope=1/2=0.5, h_1/D_W =0.42 from Figure 10-B-2.
  h_1 =0.42(0.915)=0.384 m., use h_1 = 0.396 m.
- 4. h_2 =2(D)=2(0.610)=1.220 m.
- 5. h_W = h_1 +h_2 =0.396+1.220=1.616 m.

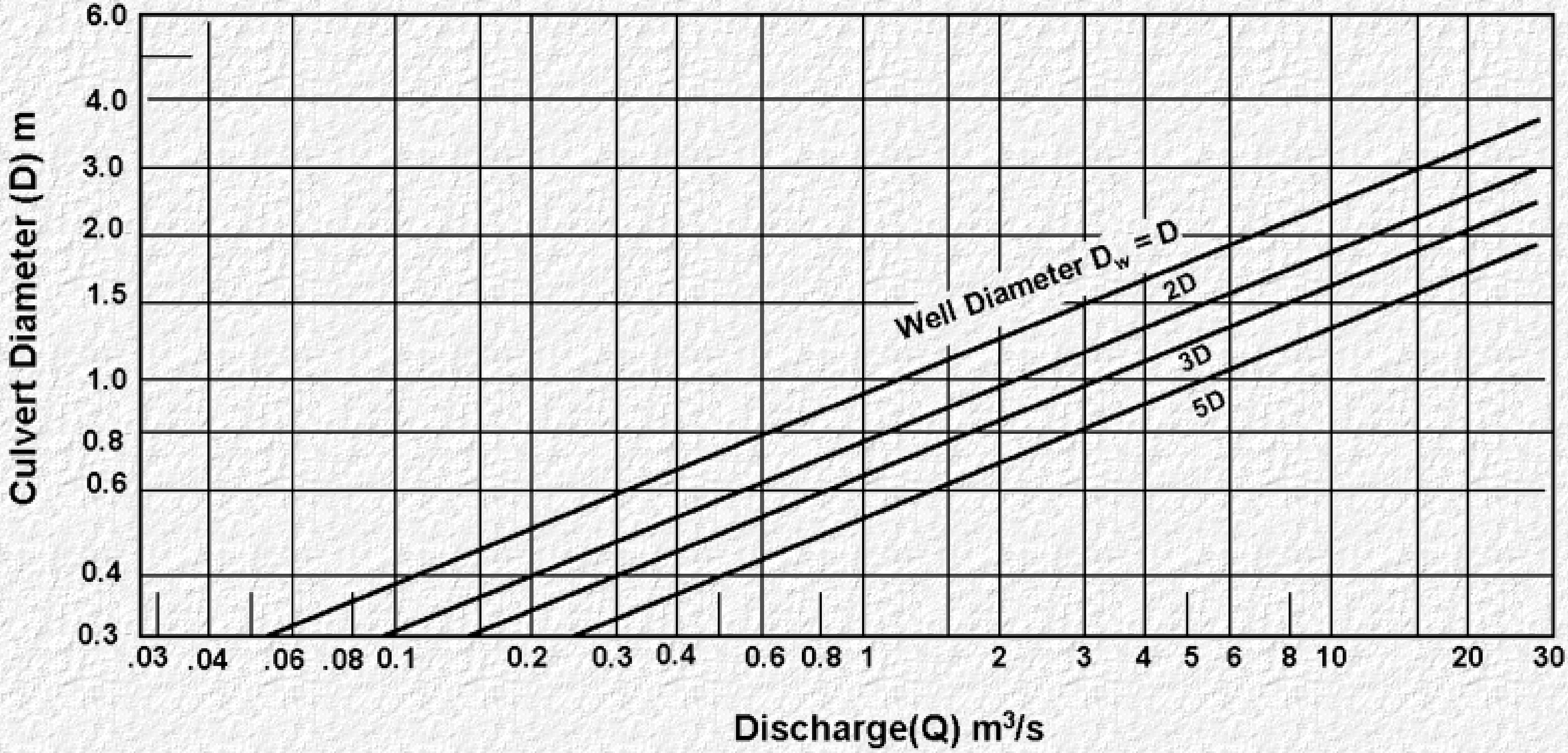


Figure 10-B-1 Stilling Well Diameter (Dw) (from reference 22)

Figure 10-B-1. Stilling Well Diameter (D_W) (from reference 10B1)

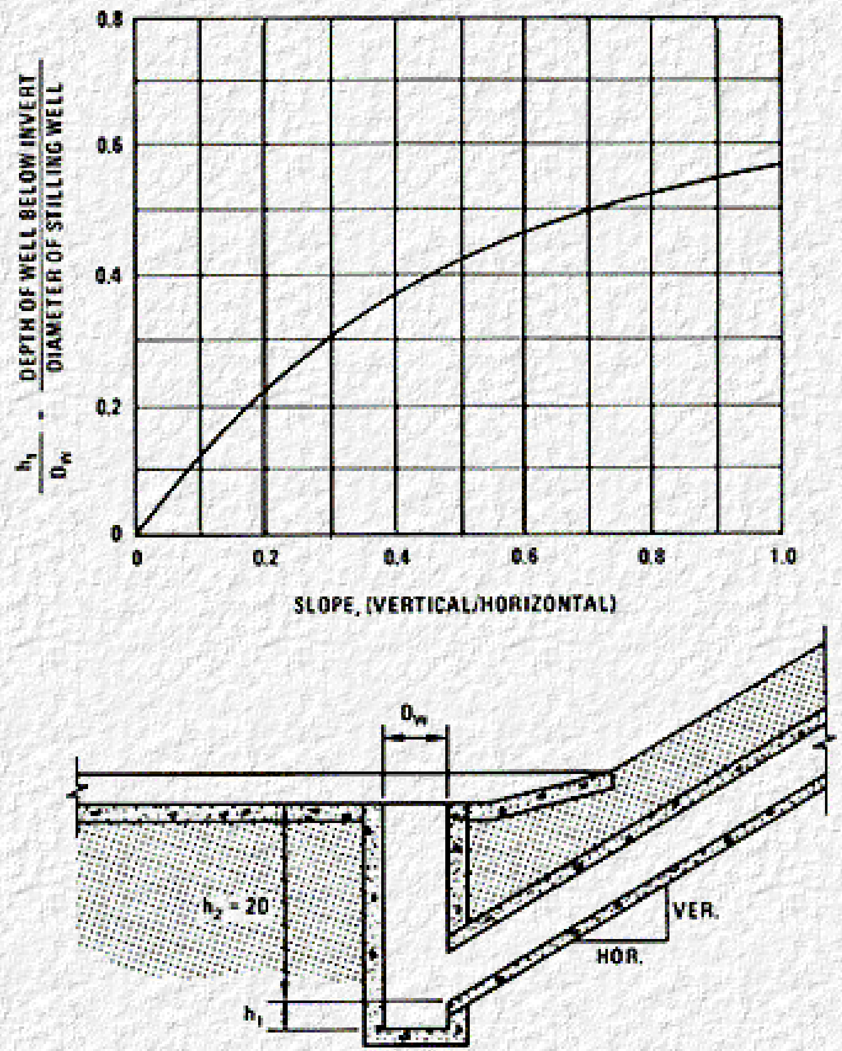


Figure 10-B-2. Stilling Well Height (from reference 10B1)

10B1.. *Impact Type Energy Dissipators For Storm-Drainage Outfalls Stilling Well Design,*
U.S. Army Corps of Engineers,
Technical Report No. 2-620 March 1963,
WES, Vicksburg, Mississippi.

10B2. Grace, J. L., Pickering, G. A.

Evaluation of Three Energy Dissipators For Storm Drain Outlets,
U.S. Army WES, HRB 1971, Washington, D.C.

Go to Chapter 11

Chapter 11 : HEC 14

Riprap Basins

[Go to Chapter 12](#)

The design procedure for riprap energy dissipators is based on data obtained during a study "Flood Protection at Culvert outfalls" (11-1 and 11-2) 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 152mm, 305-mm, 457-mm, and 914-mm pipes, and 152 by 305-mm, 152 by 457-mm, and 152 by 610-mm model box culverts with discharges ranging from 0.003 to 2.8 m^3/s. Both angular and rounded rock with an average size (d_{50}) ranging from 6mm to 177mm and gradation coefficients ranging from 1.05 to 2.66 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, 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 11-1](#). 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

Step 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.

Step 2. For subcritical flow conditions (culvert set on mild or horizontal slope) utilize Figure 3-9 or Figure 3-10 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.

Step 3. From site inspection and from field experience in the area, determine whether or not channel protection is required at the culvert outlet.

Step 4. If channel protection is required, compute the Froude number for brink conditions ($y_e =(A/2)^{1/2}$). Select d_{50} /y_e appropriated 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 11-2 , and check to see that $2<h_s /d_{50} <4$. Recycle computations if h_s /d_{50} falls out of this range.

Step 5. Size basin as shown in Figure 11-1 .

Step 6. Design procedures where allowable dissipator exit velocity is specified:

. 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, Figure 11-1

, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

Step 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.

Step 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 11-3. Shape downstream channel and size riprap using Figure 2-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(11-3) or similar highway department specifications.

Design Example No. 1

Given:

2438 mm by 1829 mm box culvert, Q=22.640 m^3/sec, supercritical flow in culvert, normal flow depth = brink depth y_o =1.219 m, tailwater depth TW=0.853 m.

Find:

Riprap basin dimensions for these conditions:

Solution:

- 1. $y_o = y_e$ for rectangular section, $y_e = 1.219$ m
- 2. $V_o = Q/A = 22.640/(2.438 \times 1.219) = 7.618$ m/s
- 3. $Fr = V_o / [(9.81)(y_e)]^{1/2} = 7.618/[(9.81)(1.219)]^{1/2} = 2.20$
- 4. $TW/y_e = 0.853/1.219 = 0.7$, $TW/y_e < 0.75$ O.K.
- 5.Try $d_{50}/y_e = 0.45$, $d_{50} = (0.45)(1.219) = 0.548$ m
  From Figure 11-2 $h_S/y_e = 1.6$
  $h_S = (1.219)(1.6) = 1.950$ m
  $h_S/d_{50} = 1.950/0.548 = 3.6$ m $2 < h_S/d_{50} < 4$ O.K.
- 6. $L_S = (10)(1.950) = 19.5$ m

L_S min=$(3)(W_o)=(3)(2.438)=7.314$ m, use $L_S = 19.5$ m
$L_B = 15(1.950) = 29.3$ m
L_B min=$(4)(W_o)=(4)(2.438)=9.752$, use $L_B = 29.3$ m

Other basin dimensions designed in accordance with details shown in Figure 11-1.

Design Example No. 2

Given:

2438 mm by 1829 mm box culvert, $Q = 22.640$ m^3/sec, supercritical flow in culvert, normal flow depth = brink depth $y_o = 1.219$ m, Tailwater depth TW=1.280 m, downstream channel can tolerate 2.134 m/s for design discharge.

Find:

Riprap basin dimensions for these conditions:

Solution:

Note: Highwater depth, $TW/y_o = 1.05 > 0.75$

1. Design riprap basin (Design Example 1) use steps1-7 d_{50} =0.549 m (1.8 ft), h_S =1.951 m (6.4 ft), L_S =19.507 m, L_B =29.261 m (96 ft)

8. Design riprap for downstream channel. Utilize Figure 11-3 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$A= \pi D_e^2 /4=(y_o)(W_o)=(1.219)(2.438)=2.972 \text{ m}^2$

$D_e =[2.972(4)/ \pi]^{1/2}$

$D_e =1.945$ m

$V_o =7.618$ m/sec (Design Example 1)

L/D_e	L (Compute) m	V_L/V_o (Figure 2-C-1)	V_L m/sec.	Rock size d_{50} (Figure 2-C-1) m
10	19.450	0.59	4.495	0.43
15	29.175	0.36	2.742	0.18
20	38.900	0.30	2.285	0.12
21	40.845	0.28	2.133	0.12

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 41.148 meters (135 ft) 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:

1829 mm diameter cmp, Q=3.823 m^3/sec., S_o =0.004, Manning's n=0.024 normal depth in pipe for Q=3.820 m^3/s is 1.372 m normal velocity is 1.798 m/s, flow is subcritical, tailwater depth (TW) is 0.610 m.

Find:

Riprap basin dimensions for these conditions:

Solution:

1. Determine y_o and V_o:

$1.81\ Q/D^{2.5} = 1.81\ (3.823)/1.829^{2.5} = 1.53$
$TW/D = 0.610/1.829 = 0.33$

From Figure 3-10, $y_o\ /D = 0.45$

$y_o = (0.45)(1.829) = 0.823$ m
TW/y_o 0.610/0.823=0.74 $TW/y_o < 0.75$ O.K.
Brink Area (A) for $y_o\ /D = 0.45$ is
$A = (0.343)(1.829)^2 = 1.147\ m^2$
(0.343 is from Table 3-2)

$V_o = Q/A = 3.823/1.147 = 3.333$ m/sec.

2. $y_e = (A/2)^{1/2} = (1.147/2)^{1/2} = 0.757$ m

3. $Fr = V_o\ /[(9.81)(y_e)]^{1/2} = 3.333/[(9.81)(0.757)]^{1/2}$
$= 1.22$

4. Try $d_{50}/y_e = 0.25$, $d_{50} = (0.25)(0.757) = 0.189$ m
From Figure 10-A-2, $h_S\ /y_e = 0.75$, $h_S = (0.75)(0.757) = 0.568$ m
check: $h_s/d_{50} = 0.568/0.189 = 3$, $2 < h_S\ /d_{50} < 4$ O.K.

5. $L_s = (10)(h_s) = (10)(0.568) = 5.680$ m
or $L_s = (3)(W_o) = (3)(1.829) = 5.487$ m, Use $L_S = 5.680$ m

$L_B = (15)(h_B) = (15)(0.568) = 8.520$ m
or $L_B = (4)(W_o) = (4)(1.829) = 7.316$ m, Use $L_B = 8.520$ m

$d_{50} = 0.189$ m use $d_{50} = 0.203$ m (8 in)

Other basin dimensions are designed in accordance with details shown on Figure 11-1.

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 53 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" (11-2). 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 (11-1) were utilized for developing design relationships.

Design Procedure Development

The design criteria specified in this report were developed in the following manner:

Step 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 152-mm, 305-mm, 457-mm, and 914-mm pipes and 152 by 305-mm, 152 by 457-mm, and 152 by 914-mm model box culverts. Two pipe slopes were considered, 0 and 3.75%. Discharges ranged from 0.003 to 2.83 m/s for basins constructed with angular and rounded rock with median rock size (d_{50}) ranging from 6.1 mm to 177 mm. 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 (11-3).

Step 2. Based upon an examination of CSU plots and data, the following significant dimensionless parameters were selected:

- 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 defiled 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.

2. 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 11-1).

3. $Fr = V_o /[(g)(y_e)]^{1/2}$ --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.

4. TW/y_e --the relative depth of tailwater. TW is the depth of tailwater referenced to the culvert invert.

Step 3. The dimensionless parameters cited above were computed for each set of data and were segregated into the following categories:

$0.10 < d_{50} /y_e < 0.2$
$0.21 < d_{50} /y_e < 0.3$
$0.31 < d_{50} /y_e < 0.4$
$0.41 < d_{50} /y_e < 0.5$
$0.51 < d_{50} /y_e < 0.6$
$0.61 < d_{50} /y_e < 0.7$

Step 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:

. model pipe culverts with a diameter equal to or less than 0.3 meter, basin constructed with rounded material,

b. model pipe culvert with a diameter equal to or less than 0.3 meter, basin constructed with angular material,

c. model pipe culvert with a diameter greater than 0.3 meter, basin constructed with rounded material,

d. model pipe culvert with a diameter greater than 0.3 meter, basin constructed with angular material, and

e. model box culverts with a height of 0.15 m, basin constructed with rounded material (the only material tested for box culverts).

Step 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).

Step 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 steps 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 0.15 meter with tailwater depths of 0 to 50-mm The scaled riprap was approximately 13 mm 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 human 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 11-2) 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, side slopes 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 0.03 meters 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 $1.50(d_{max})$, where 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 to3(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.

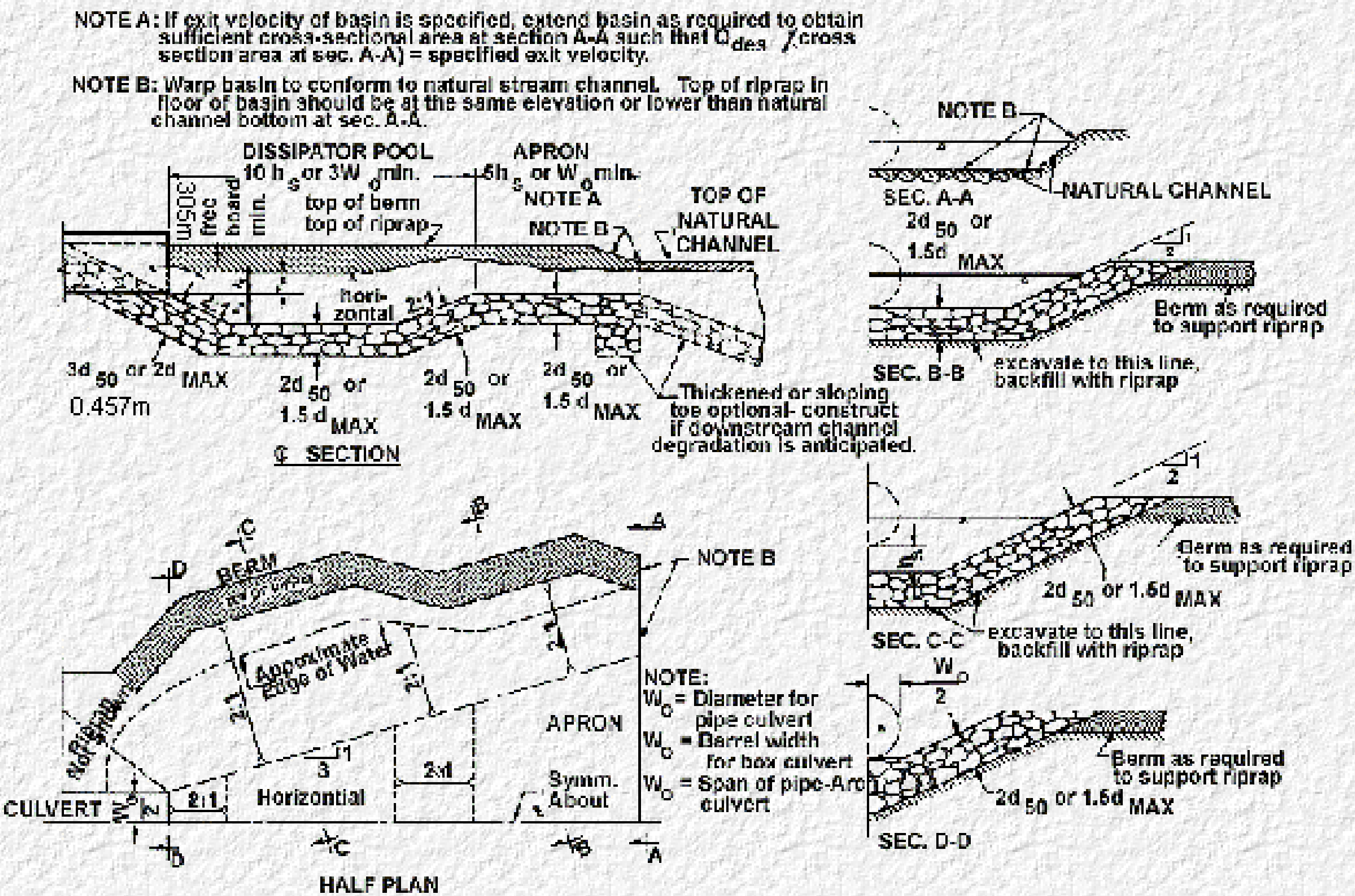


Figure 11-1. Details of Riprapped Culvert Energy Basin

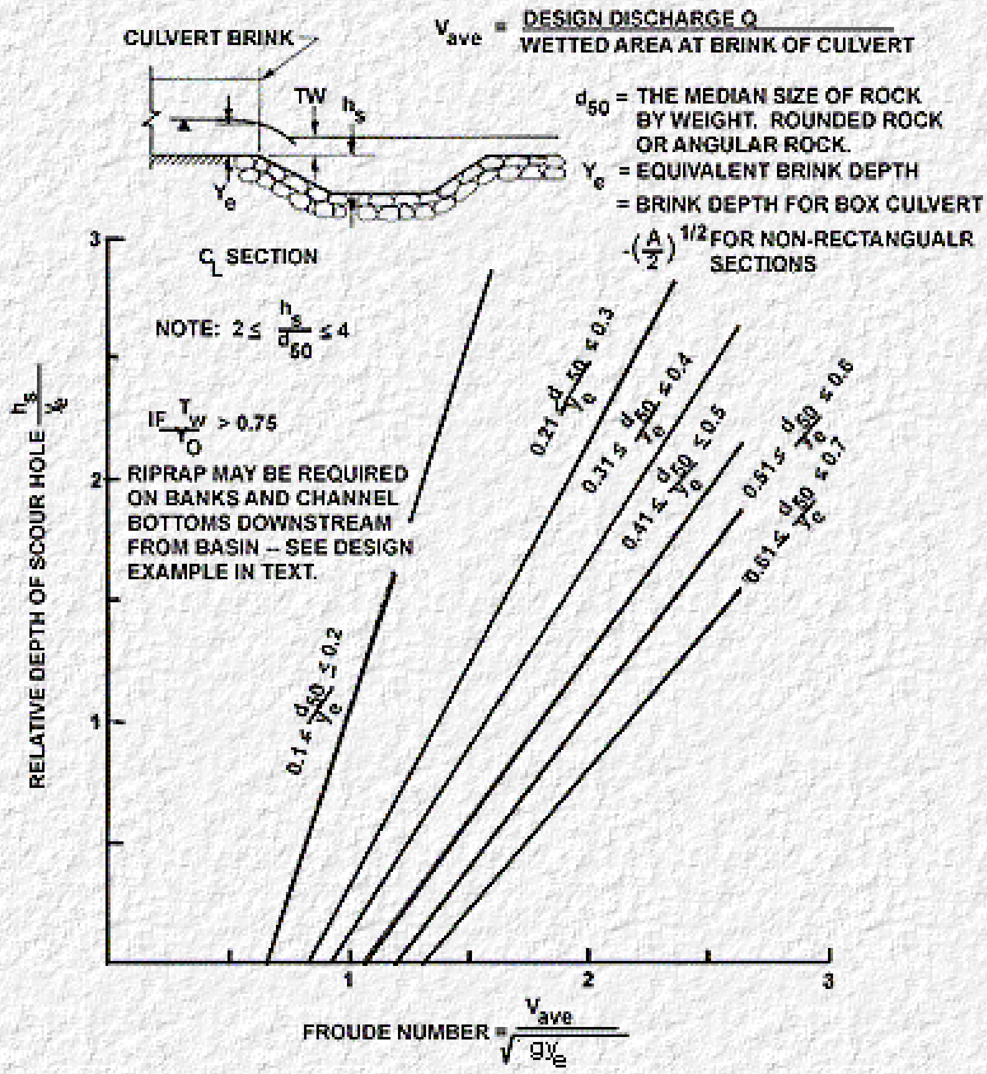


Figure 11-2. Relative Depth of Scour Hole versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

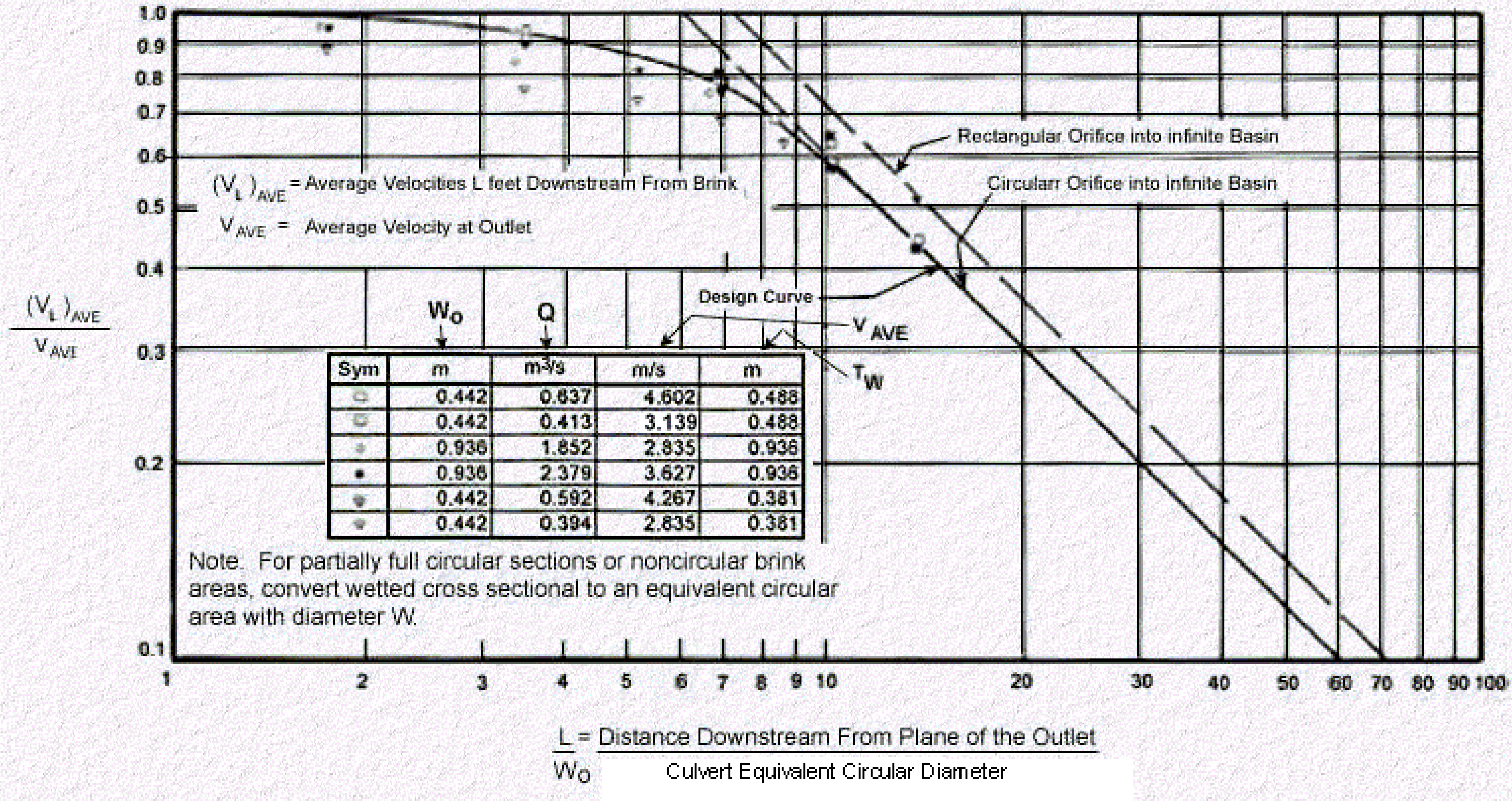


Figure 11-3. Distribution of Centerline Velocity for Flow from Submerged Outlets to be Used for Predicting Channel Velocities from Culvert Outlet where High Tailwater Prevails. Velocities Obtained from the Use of this Chart Can Be Used with HEC 11 for Sizing Riprap

11-1. 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.

11-2. 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.

11-3. U. S. Department of Transportation,
Use Of Riprap For Bank Protection,
Hydraulic Engineering Circular No. 11,
Government Printing Office, Washington, D.C., 1967.

11-4. 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.

Go to Chapter 12

Chapter 12 : HEC 14

Design Selection

Go to Appendix A

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 1-1, 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 3 y and V at end of apron, Chapter 4

   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 5 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, Section 7-C.

   For minimum velocity, V_c, design for tumbling flow, Section 7-B.

4. Energy Dissipator Design

   a. The dissipator types fall into two general groups based on Fr, see Table 12-1:

      1. Fr < 3, most designs are in this group

      2. 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: 3048 mm x 1829 mm (10' x 6') RCB

$Q = 11.801\ m^3/s$, $S_o = 6.5\%$

$y_n = 0.457$ m, $V_n = 8.473$ m/s, inlet control,

$L = 91.440$ m, $TW = 0.579$ m, $Fr_o = 4$

b . Standard Outlet: 45° wingwall--abrupt transition since culvert is in supercritical flow $y_o = y_n = 0.457$ m and $V_o = V_n = 8.473$ m

c . Channel: $Q = 11.801\ m^3$/sec., $S = 6.5\%$, trapezoidal, 2H:1V

$y_n = 0.579$ m, $V_n = 4.846$ m/s, graded gravel bed debris (no boulders, little floating)

d. Allowable Scour: Scour hole should be contained within channel $W_s = L_s = 3.048$ m and should be no deeper than 1.524 m. This allowable estimate can be obtained by observing scour holes in the vicinity.

2. Scour Estimate: Table 5-1

$y_e = 0.835$ m
$h_s = 2.530$ m
$W_s = 15.850$ m
$L_s = 21.640$ m

$V_s = 737\ m^3$

Scour appears to be a problem and consideration should be given to reducing the V_o = 8.473 m/s to the 4.846 m/s in the channel.

3. Velocity Modification in Culvert:

a . Increase resistance: Section 7-C

For h = 0.122 m, n_r = .053 for velocity check
n_r = .076 for Q check

The velocity at the outlet is 3.536 m/s. The elements are 1.006 meter (3.3 ft) apart for 9 rows or 29.779 m of the culvert barrel is needed for elements.

b. Tumbling Flow: Section 7-B

Five elements 0.579 m in height spaced 4.572 m apart are required to reduce the velocity to V_c =3.353 m/s and y_c =1.158 m. In order to accomplish this reduction, the last 30.480 meters of culvert must be increased in height to 2.042 m.

4. Energy Dissipator Design:

Since Fr>3 try USBR III, SAF, and USBR VI. The USBR III and SAF which were designed in Section 7-E and Section 7-G are summarized below:

USBR Type III:

L_B =7.924 m, total length including transitions=20.422 m, Elevation=26.670, V_2 =4.877 m/s

SAF:

L_B = 3.353 m, total length including transitions=12.192 m, elevation=27.889 m, V_2 = 4.877 m/s.

USBR Type VI (Impact):

W=3.353 m, h_1 = 2.560 m, L=4.450 m Exit Velocity= 3.353 m/s

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 =1.524 m
D = 1.524 m
Elevation of outlet invert = 30.480 m
Q_{Design} = 5.660 m^3/s
S_o = 0.03
y_o = 0.655 m
V_o = 5.791 m/s
L = 64.922 m
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 = 0.655 m
V_o = 5.791 m/s

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 0.914 meters deep is allowable at this site. Allowable outlet velocity should be about 3 m/s.

2. Scour Computation: Chapter 4

Convert to equivalent depth

y_e = 0.707 m
h_e = 1.707 m

W_S = 9.449 m
L_S = 14.935 m
V_S = 62 m^3

Since 1.707 meters is greater than the 0.914 meters allowable, an energy dissipator will be necessary.

3. Velocity Modification:

a . Try increased roughness, Section 7-C, with elements on bottom and sides of culvert.

For:

h =0.079 meters
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 1.829 meters. The length of roughness field and increased culvert diameter requested is 22.860 meters (75 ft).

This represents 25 rows spaced at 0.9 m/row.

b . Tumbling flow; Section 7-B

Five elements. 0.549 meters in height spaced 5.486 meters (18 ft) apart are required to reduce the velocity to 3.322m/s. The last 32.918 meters of the culvert must be increased in height to 1.981 m.

4. Energy Dissipator Designs:

Since Fr < 3 try the CSU rigid boundary basin, the USBR type VI, the Hook type and a Riprap basin.

CSU Basin, Chapter 7:
W_e = width of basin = 9.144 meters
L_B = length of basin = 8.534 meters
N_r = number of roughness rows = 4
N = number of elements = 17
U_e = divergence 1.9:1
W_1 = width of elements = 0.914 meters
h = height of elements = 0.229 meters

V_B = velocity at basin outlet 2.896 m/s
y_B = depth at basis outlet = 0.213 m

USBR type IV, Section 7-C:
W = width = 3.658 m
L = length = 4.877 m
h_1 = height = 2.804 meters
Exit velocity = 2.478 m/s

Hook Type Basin:
Assuming the downstream velocity, V_n, equals the allowable, 3.048 m/s, V_o/V_n = 1.9.

The dimensions for a straight trapezoidal basin are:
L_B = length = 4.572 meters
$W_6 = 2W_o$
Side Slope = 2:1
L_1 = length to first hook = 1.905 meters
L_2 = length to second hooks = 3.179 meters
h_3 = height of hook =0.716 meters
V_2 = 3.048 m/s

From Figure 8-B-7, V_o/V_B =2.0; V_B =5.8/2.0=2.896 m/s OK

Riprap Basin:
d_{50} = diameter of rock = 0.399 meters
h_s = depth of pool (scour) =0.820m/s
Length of pool = 8.199 m
Length of apron = 4.100 m
Length of basin = 12.299 m
Thickness of riprap on approach 3 d_{50} = 1.198 m
Remainder of basin $2d_{50}$ =0.799 m

5. Selecting Criteria

Right-of-way(ROW), 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 12-1. Dissipator Limitations

Dissipator Type	Froude Number Fr	ALLOWABLE DEBRIS			Tailwater TW	Special Considerations
		Silt/Sand	Boulders	Floating		
Free Hydraulic Jump	> 1	H	H	H	REQUIRED	
CSU Rigid Boundary Tumbling Flow Increased Resistance	< 3 > 1 --	M M M	L L L	M L L	-- -- --	$4<S_o<25$ CHECK OUTLET CONTROL HW
USBR Type II USBR Type III USBR Type IV SAF	4 to 14 4.5 to 17 2.5 to 4.5 1.7 to 17	M M M M	L L L L	M M M M	REQUIRED REQUIRED REQUIRED REQUIRED	
Contra Costa Hook USBR Type VI	< 3 1.8 to 3 --	H H M	M M L	M M L	<0.5D -- DESIRABLE	$Q<11 m^3/s$ $V<15$ m/s
Forest Service	--	M	L	L	DESIRABLE	D<900-mm
Drop Structure	< 1	H	L	M	REQUIRED	Drop< 4.57 m.
Manifold Corps Stilling Well Riprap	-- -- < 3	M M H	N L H	N N H	DESIRABLE DESIRABLE --	Note: N=none L=Low M=moderate H=heavy

---

[Go to Appendix A](#)

Appendix A : HEC 14
Computation Forms

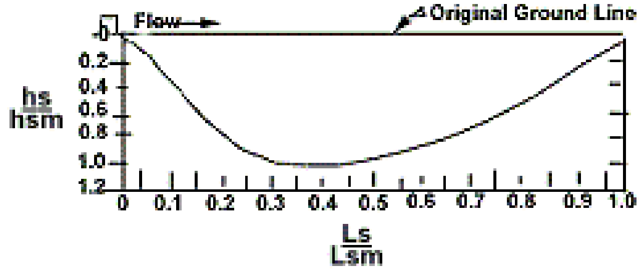
Go to Table of Contents

List of Forms Found In This Appendix

Go to Table of Contents

[Go to Form 2] [Go to Appendix A]

Culvert, Channel, Scour, and Other Site Data.



DIMENSIONLESS CENTER-LINE PROFILE

Project ____________
Station ____________
Designer ____________
Date ____________

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

	Type / Control	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													

	Type	Side Slope	"n"	Bottom Width	Slope	Discharge Q	Velocity	Flow Area	TW	Free Board	Fr	Type of Material
Channel												

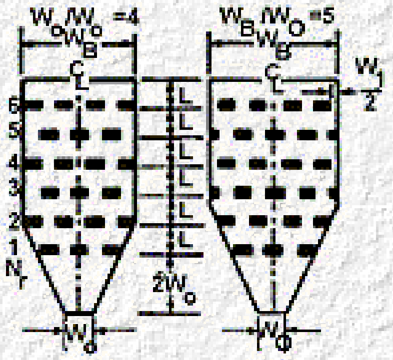
	Equivalent Depth y_e	Scour: Depth h_s	Scour: Width W_s	Scour: Length L_s	Scour: Volume V_s
Scour Computations					

	Velocity	Scour: Depth	Scour: Width	Scour: Length	Other Restrictions
Allowabe Conditions					

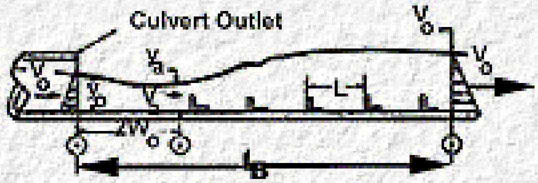
Other Site Constraints	

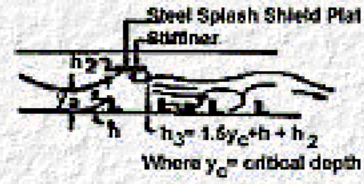
[Go to Form 2] [Go to Appendix A]

Go to Form 3 Go to Appendix A



CSU Rigid Boundary Basin





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.	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)													

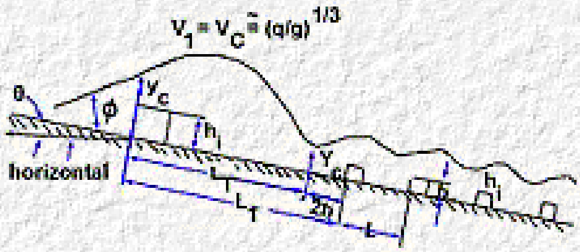
	Freeboard	y_c	h_2	h_3	L/2
Splash Shield					

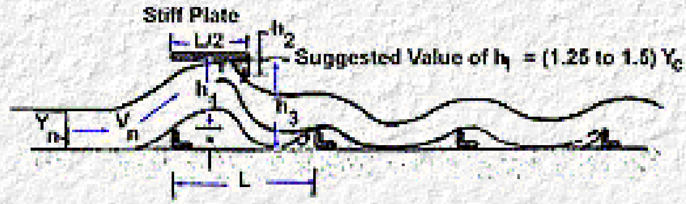
(1) From Values in this table
(2) From equation 51 or 52
(3) $2<W_1/h<8$

Go to Form 3 Go to Appendix A

[Go to Form 4](#) [Go to Appendix A](#)

Tumbling Flow--Rectangular Section

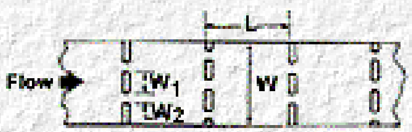




	q=Q/W	y_c	V_c^{+}	V_n	
Initial Conditions					

	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					



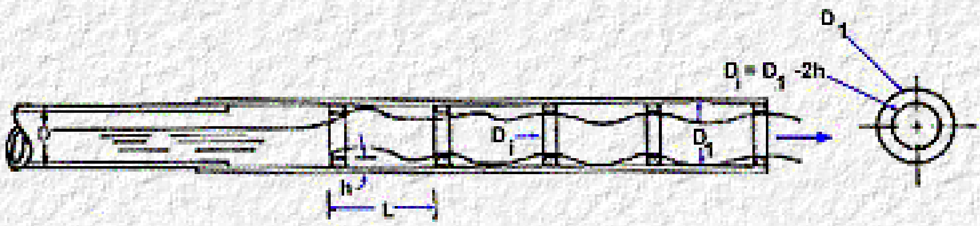
* For design using intial large element--Alternate design

+ V_c equals outlet velocity for dissipator

[Go to Form 4](#) [Go to Appendix A](#)

Go to Form 5 Go to Appendix A

Tumbling Flow - Circular Section



	$5.2(Q^2/g)^{1/5}$	$6.2(Q^2/g)^{1/5}$	D_1	
Initial Conditions				

	h	L	D_1	y_c	y_c/D_i	A_c/D_i^2	A_c	V_c	
Dissipator Conditions									

Go to Form 5 Go to Appendix A

Go to Form 6 Go to Appendix A

Increased Resistance--Box Culverts

L/h	h/R_i	L_r	P	L_r/P	$S_o^{1/2}$	R_1 full	
10							

Velocity Check	n_r/n	n_r	h	y_i	$K = \frac{1}{n_r} S_o^{1/2}$ (5)	A_i	R_i	$R_i^{2/3}$	$Q=KA_1R_1^{1/3}$	$V=\frac{Q}{A_i}$	No. of Rows

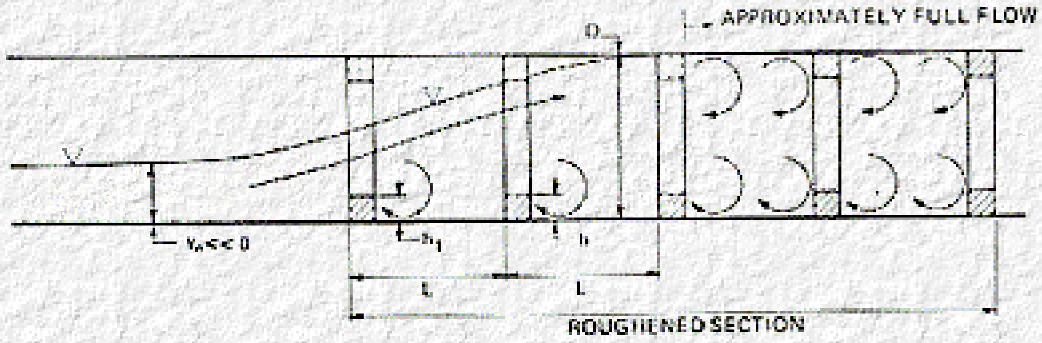
Capacity Check	n_r/n	n_r	h	y_i	$K= \frac{1}{n_r} S_o^{1/2}$ (5)	A_i	R_i	$R_i^{2/3}$	$Q=KA_iR_i^{2/3}$	$D_{(14)}$	

	h (2)	L = 10h	$h_i=2h$	V (3)	D_i	
Final Design						1. From Equation 7-C-10 2. From Capacity Check 3. From Velocity Check 4. New Diameter -- if necessary 5. Use 1.49 rather than 1 if in English units

Go to Form 6 Go to Appendix A

Go to Form 7 Go to Appendix A

Roughness Elements--Ciruclar Culverts



	Q (full) (n)	V (full) (n)	$\frac{Q(Design)}{Q(full)}$	$\frac{V\ (design)}{V\ (full)}$	n	Type of Flow Control
Initial conditions						

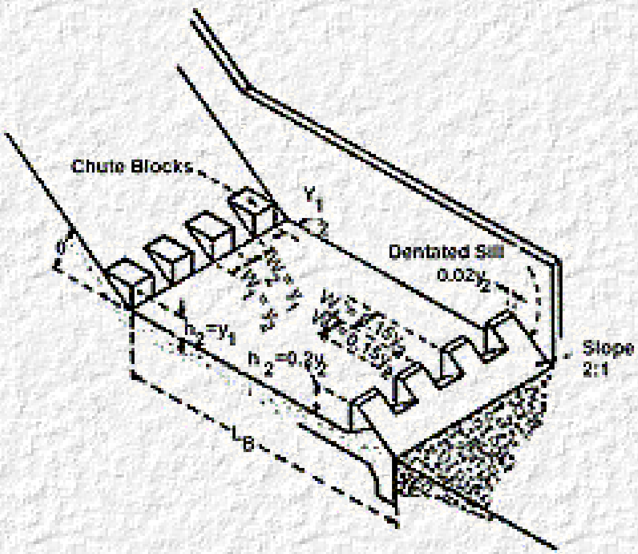
$n/D^{1/6}$	L/D_i	h/D_i	n_r/n	n_r	D_i	Q(full) (n_r)	V (full) (n_r)	$\frac{V\ (outlet)}{V\ (full)}$	V (Outlet)

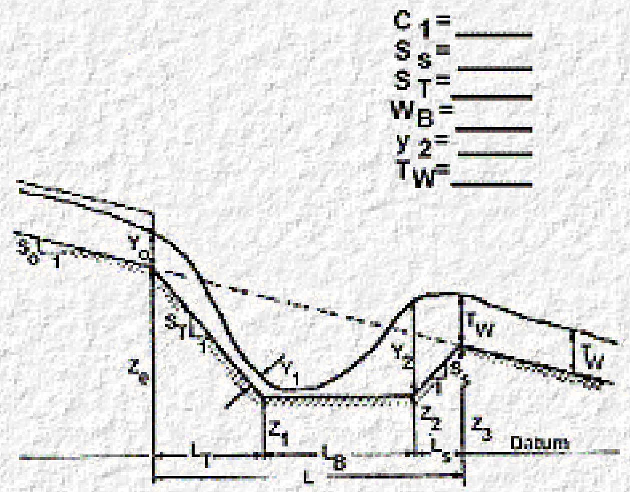
h	L	h_1

Go to Form 7 Go to Appendix A

Go to Form 8 Go to Appendix A

USBR TYPE II





If $y_2 > T_W$, Depress Basin

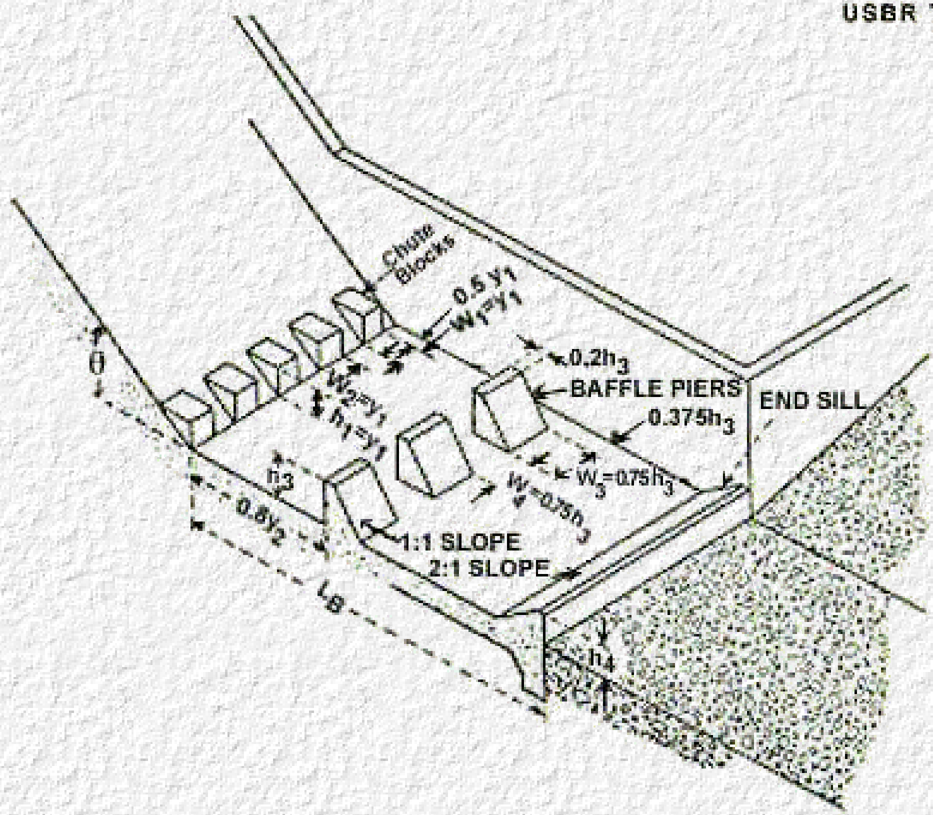
$Z_1 = Z_2$	y_1	V_1	Fr_1	y_2	L_B	L_T	Z_3	$y_2 + z_2$ (1)	$Z_3 + T_W$ (1)
L_S	L	W_1	h_1	N_c	h_2	W_3	W_4	N_3	

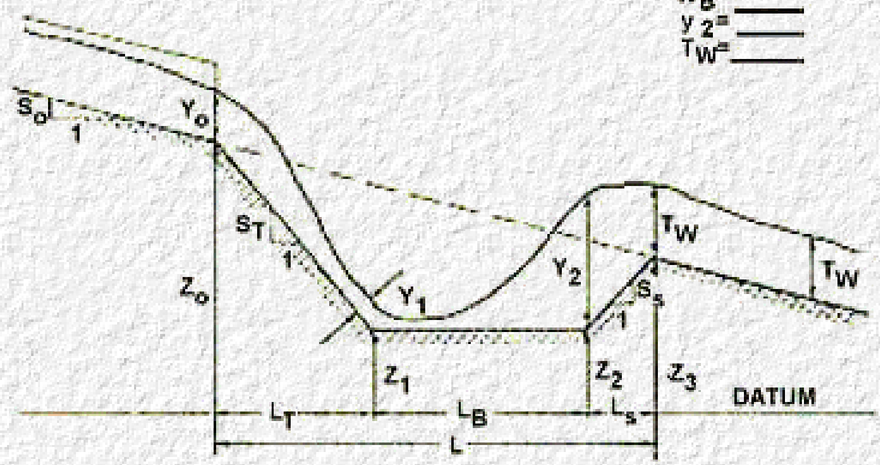
(1) $Z_3 + T_W > y_2 + Z_2$

Go to Form 8 Go to Appendix A

[Go to Form 9] [Go to Appendix A]

USBR Type III



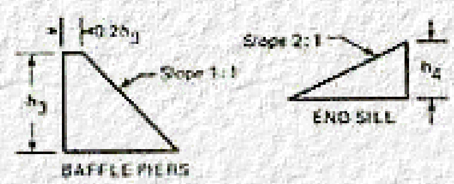


$C_1 =$ ______
$S_s =$ ______
$S_T =$ ______
$W_B =$ ______
$y_2 =$ ______
$T_W =$ ______

$Z_1 = Z_2$	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 Clocks	h_4

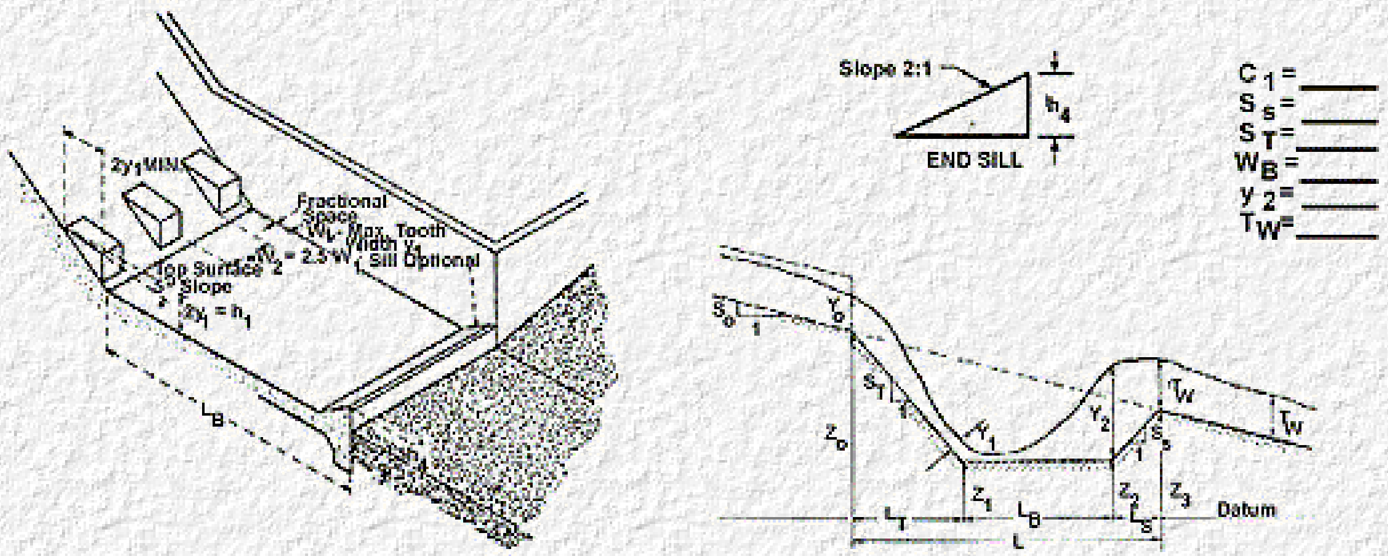
(1) $Z_3 + T_W > y_2 + Z_2$



Go to Form 9 Go to Appendix A

Go to Form 10 Go to Appendix A

USBR TYPE IV



If $y_2 > T_W$, Depress Basin

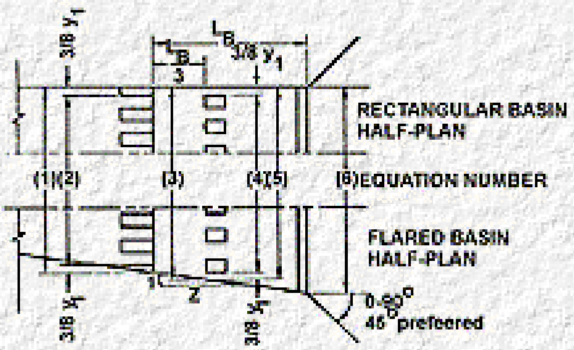
$Z_1 = Z_2$	y_1	V_1	Fr_1	y_2	L_B	L_T	Z_3	$y_2 + Z_2$ (1)	$Z_3 + T_W$ (1)
L_S	L	h_1	W_1	W_2	N_c	Wall Spaces	h_4		

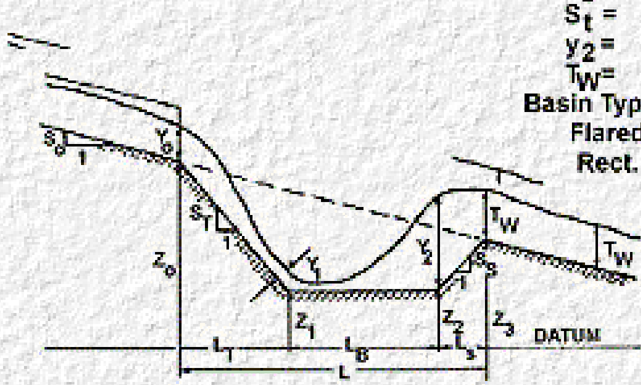
(1) $Z_3 + T_W > y_2 + Z_2$

Go to Form 10 Go to Appendix A

Go to Form 11 Go to Appendix A

SAF STILLING BASIN

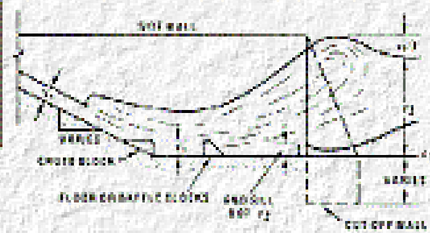




S_s =
S_t =
y_2 =
T_W =
Basin Type
Flared ______
Rect. ______

$Z_1 = Z_2$	y_1	V_1	Fr_1	y_2	L_B	L_T	Z_3	$y_2 \underset{(1)}{+} z_2$	$Z_3 \underset{(1)}{+} T_W$
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

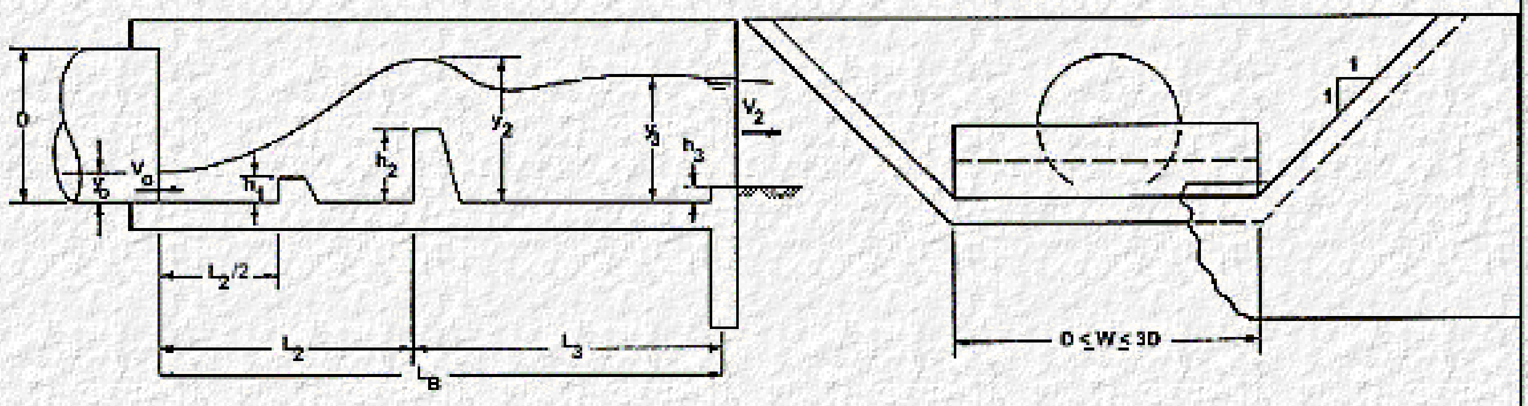


1. W_B = Basin Width Upstream
2. n Blocks at 3/4 y_1 *
3. $0.40W_{B2}$ < Aggregate Block Width < $0.55\ W_{B2}$
4. n Blocks at $3/4y_1$ $\frac{W_{B2}}{W_B}$ ±
5. $W_{B2} = W_B + 2L_B/3z$
6. $W_{B2} = W_B + 2l_B/Z$

Go to Form 11 Go to Appendix A

Go to Form 12 Go to Appendix A

CONTRA COSTA



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

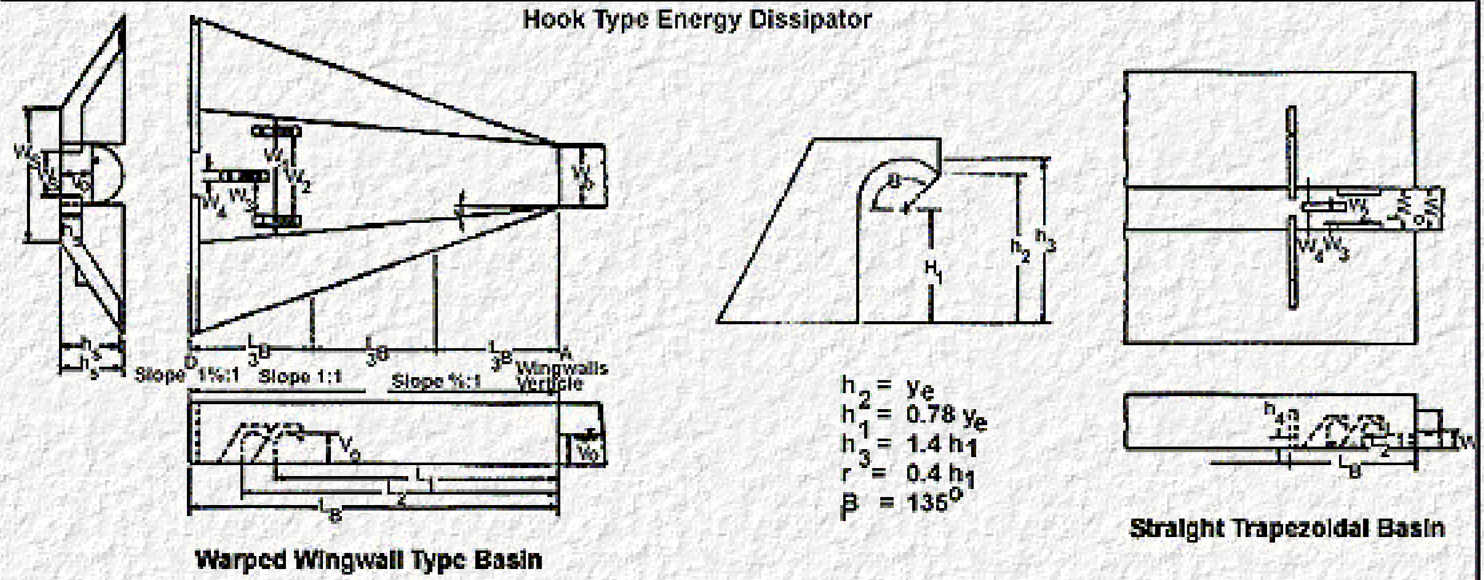
L_B	y_2	h_3

(1) y_o/D should be approx. 0.5

(2) $D<W \leq 3D$

Go to Form 12 Go to Appendix A

[Go to Form 13](#) [Go to Appendix A](#)



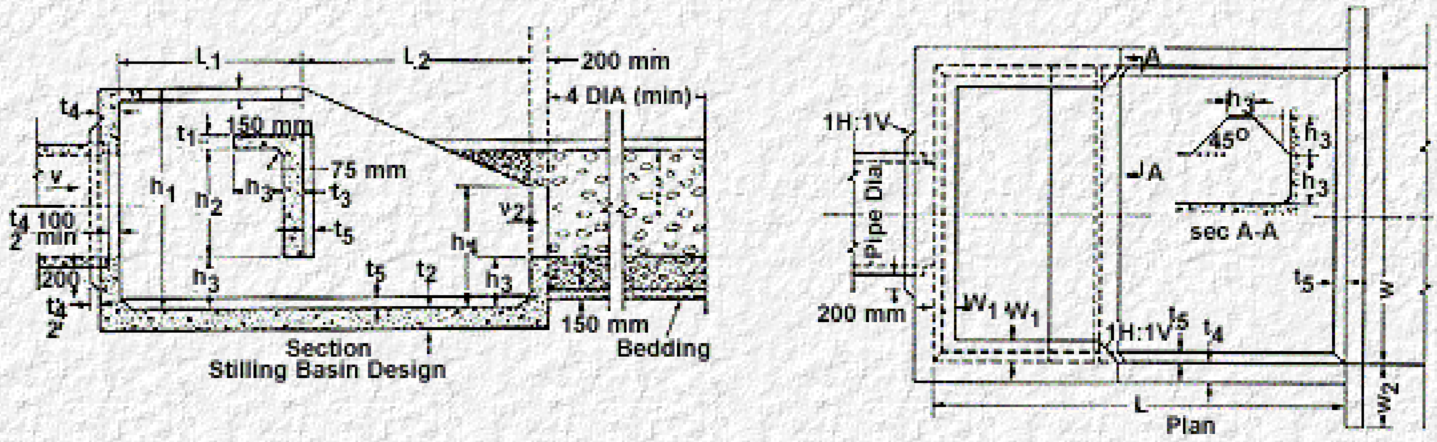
Type	V_o/V_n	W_6	L_B	L_1	W_1	W_2	L_2	W_5	h_4
Warped A = ______									
Straight Z = ______									

	h_5	W_4	h_6	W_3	h_1	h_2	h_3	r	V_0/V_B
Warped									
Straight									

[Go to Form 13](#) [Go to Appendix A](#)

Go to Form 14 Go to Appendix A

Hanging Baffle Type Energy Dissipator
USBR VI

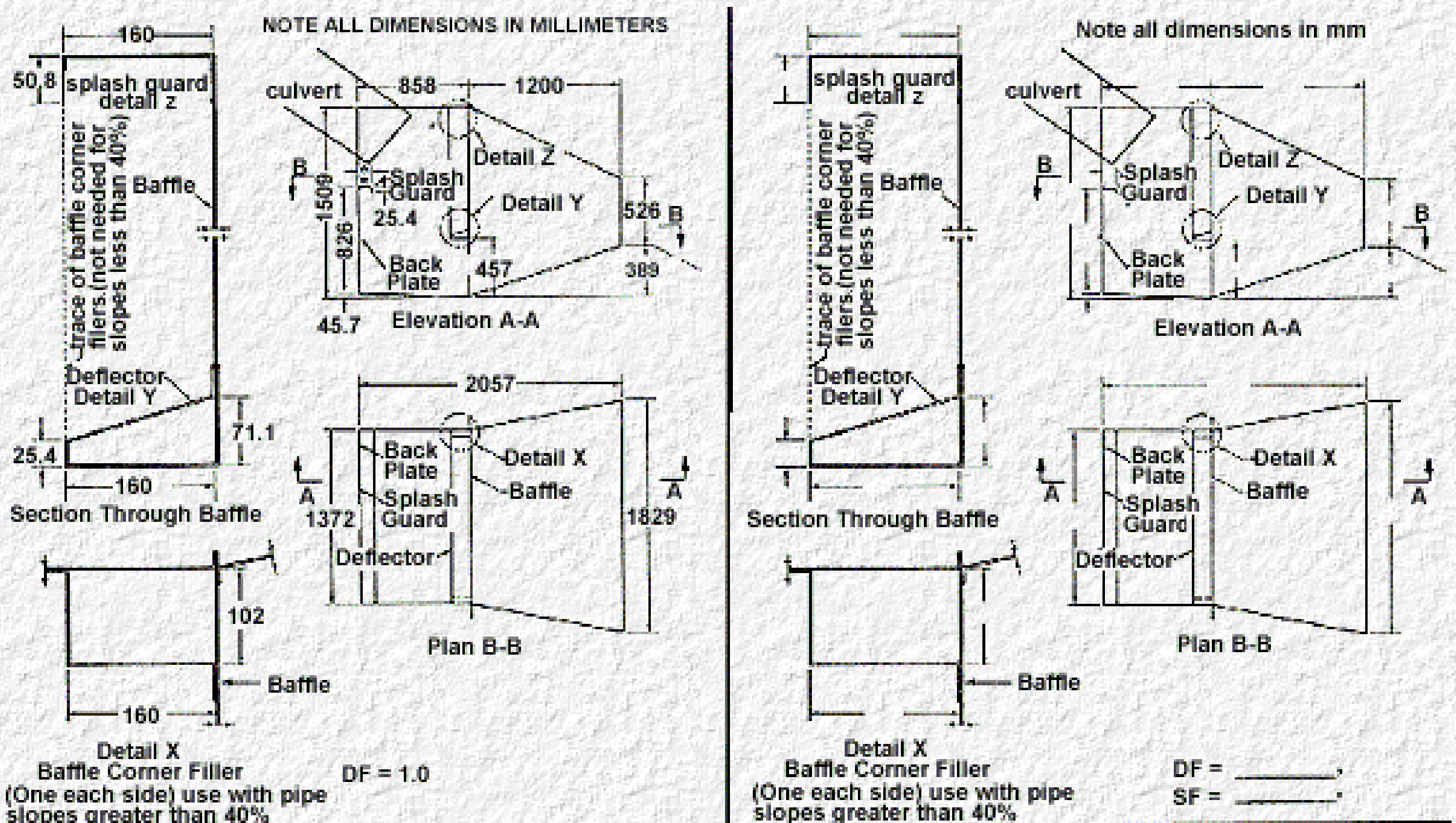


	H_o	H_o/W	W	h_1	L	h_2	h_3	L_1	L_2
	h_4	W	W_2	t_3	t_2	t_1	t_4	t_5	

Go to Form 14 Go to Appendix A

Go to Form 15 Go to Appendix A

USBR Baffle Wall Dissipator

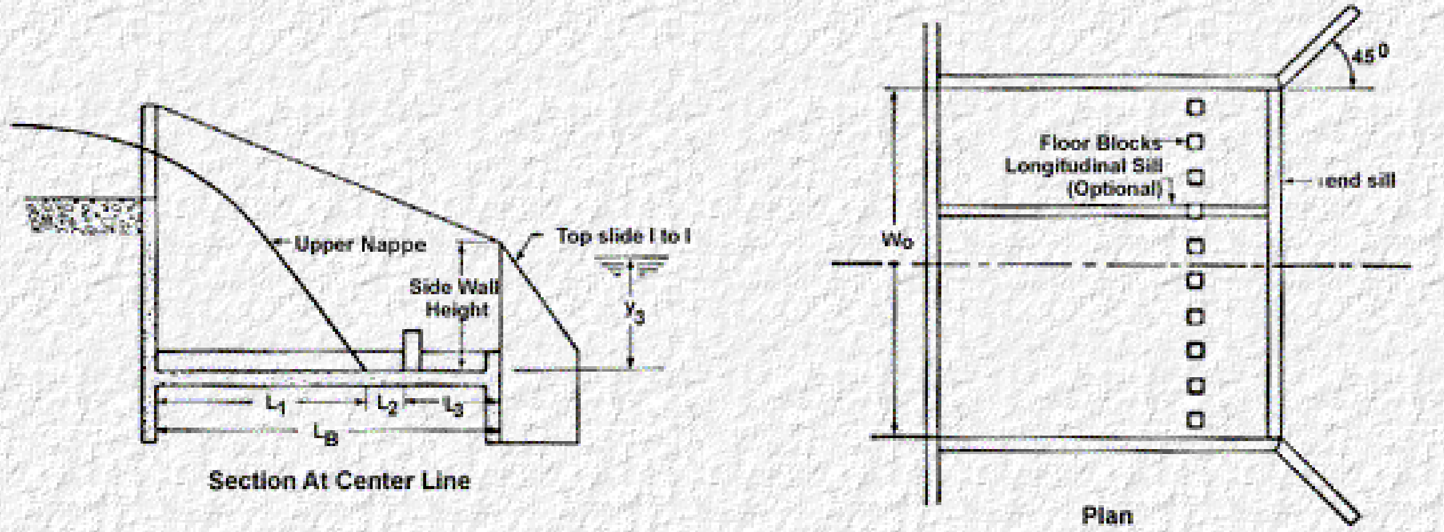


Pipe Size	c	$c^{5/2}$	450	DF	SF	L_1	h_1	
450 mm	1.0	1.0						
Other								

Go to Form 15 Go to Appendix A

Go to Form 16 Go to Appendix A

Straight Drop Structure



	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									

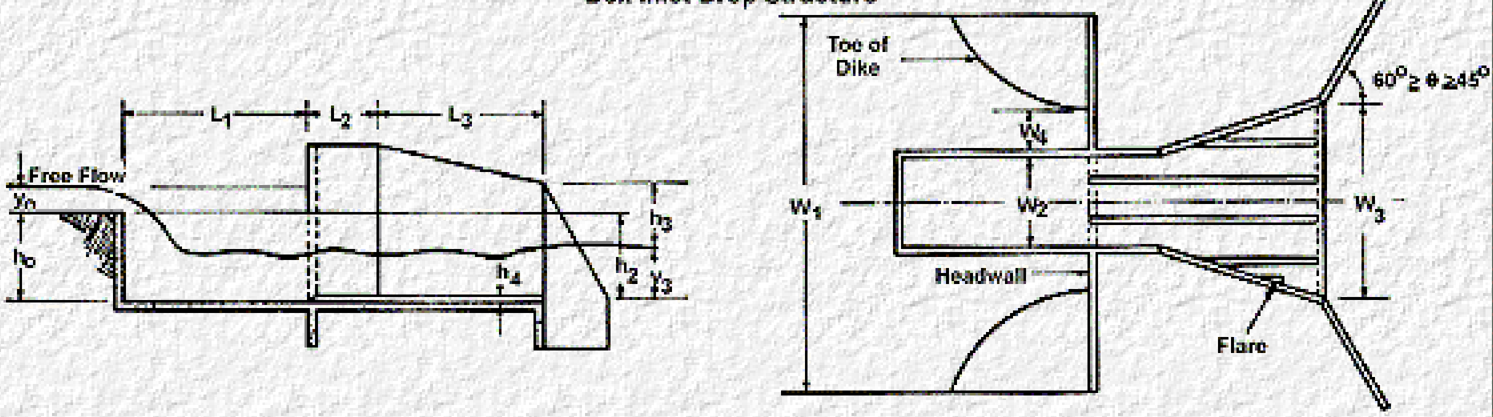
	$y_{3\ min}$

(1) Block should occupy 50 to 60 percent of basin width.

Go to Form 16 Go to Appendix A

Go to Form 17 Go to Appendix A

Box Inlet Drop Structure

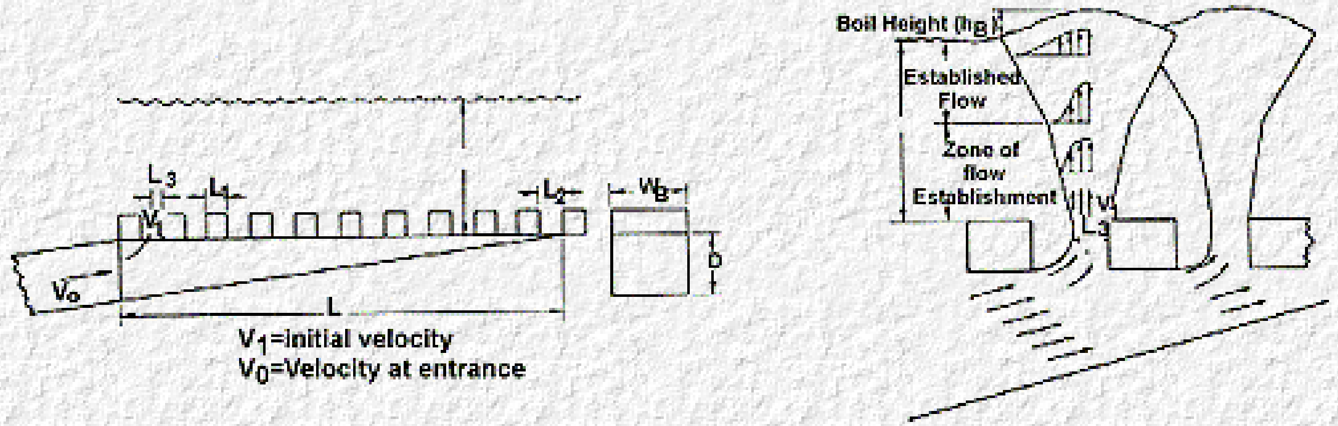


S_o	W_3	W_4	Normal Depth Upstream	Normal Depth Downstream	h_o	L_1	W_2	L_c	
Crest y_o	h_o/W_2	$C_2\sqrt{2g}$	C_2	L_1/h_o	C_H/h_o	C_H	Headwall y_o	Controlling y_o	
y_o/W_2	C_1	L_1/W_2	C_s	W_1/L_c	C_a	W_4/W_2	C_e	Adjusted Crest y_o	
y_c	y_{c3}	L_2	L_3	$11.5y_{c3}$	y_3	h_4	No. Sills		

Go to Form 17 Go to Appendix A

Go to Form 18 Go to Appendix A

Stilling Well--Manifold



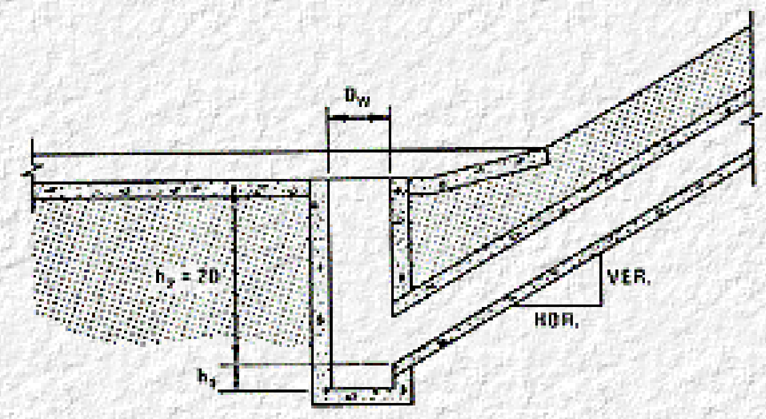
W	D	V_o	y	L_B	L_2/L_1	L_2	L_1	N	y/L_1 (1)
V_1/V_o	V_1	L_3	y/L_3	$V_1^2/2g$	h_W	h_B			

(1) $5 < y/L_1 < 20$

Go to Form 18 Go to Appendix A

Go to Form 19 Go to Appendix A

Stilling Well- Corp of Engineers

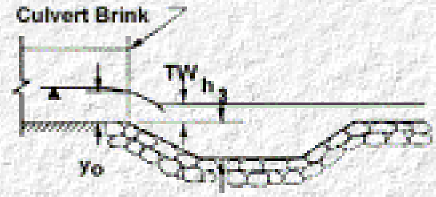


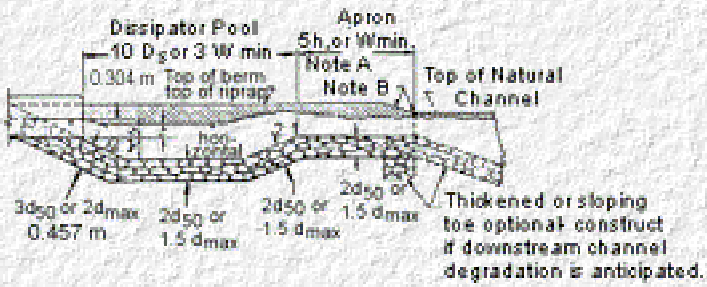
D	Q	Slope	DW	h_1/D_W	h_1	h_2	h_W		

Go to Form 19 Go to Appendix A

Go to Appendix A

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. $TW/y_e > 0.75$						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.			

Larger of

Length of Pool = 10 h or 3W = ______
Length of Apron = 4 h or W = ______
Thickness of Approach = 3d or 2d = ______
Thickness of Remainder of Basin = 2d or 1.5d = ______

(1) $TW/y_e < 0.75$ for low TW design
(2) $2 < hs/d_{50} < 4$
(3) $D_e = [4A/\pi]^{1/2}$

Go to Appendix A

Table 3-2. Uniform Flow in Circular Sections Flowing Partly Full (3-3)

d = depth of flow, m
D = diameter of pipe, m
A = area of flow, m^2
R= hydraulic radius, m

Q = discharge in m^3/s by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$
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	0.415
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	0.0294	0.0451	0.00455	5.47	0.57	0.1626	0.2703	0.287	1.286
0.08	0.0350	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0378	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	0.5405	0.29882	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	0.0139	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	0.3042	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3043	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3043	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3041	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$	$\frac{1.49Qn}{d^{8/3} s^{1/2}}$

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	0.453	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.458	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

Chapter 3

Table 3-3. Values of $BD^{3/2}$, $D^{3/2}$, and $D^{5/2}$

VALUES OF $BD^{3/2}$

B x D	$BD^{3/2}$	B x D	$BD^{3/2}$	B x D	$BD^{3/2}$
1.219 x 1.219	1.641	2.134 x 2.134	6.652	3.048 x 3.048	16.218
1.524 x 1.219	2.051	2.438 x 2.134	7.599	3.658 x 3.048	19.464
1.829 x 1.219	2.462	2.743 x 2.134	8.550	4.267 x 3.048	22.705
2.134 x 1.219	2.872	3.048 x 2.134	9.501	4.877 x 3.048	25.950
2.438 x 1.219	3.282	3.658 x 2.134	11.402		
		4.267 x 2.134	13.300	3.658 x 3.658	25.591
1.524 x 1.524	2.867			4.267 x 3.658	29.852
1.829 x 1.524	3.440	2.438 x 2.438	9.281	4.877 x 3.658	34.119
2.134 x 1.524	4.014	2.743 x 2.438	10.443	5.486 x 3.658	38.380
2.438 x 1.524	4.586	3.048 x 2.438	11.604		
2.743 x 1.524	5.160	3.658 x 2.438	13.926	4.267 x 4.267	37.609
3.048 x 1.524	5.733	4.267 x 2.438	16.244	4.877 x 4.267	42.986
				5.486 x 4.267	48.354
1.829 x 1.829	4.525	2.743 x 2.743	12.461		
2.134 x 1.829	5.280	3.048 x 2.743	13.847		
2.438 x 1.829	6.032	3.658 x 2.743	16.618		
2.743 x 1.829	6.786	4.267 x 2.743	19.385		
3.048 x 1.829	7.540				
3.658 x 1.829	9.050				

VALUES OF $D^{3/2}$

D	$D^{3/2}$	D	$D^{3/2}$	D	$D^{3/2}$
1.219	1.346	2.438	3.807	3.658	6.996
1.524	1.881	2.743	4.543	3.962	7.886
1.829	2.474	3.048	5.321	4.267	8.814
2.134	3.117	3.353	6.140	4.572	9.776

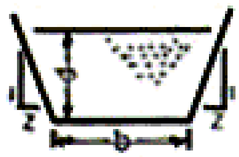
VALUES OF $D^{5/2}$

D	$D^{5/2}$	D	$D^{5/2}$	D	$D^{5/2}$
0.305	0.051	1.524	2.867	2.743	12.461
0.457	0.141	1.676	3.636	2.956	15.023
0.610	0.291	1.829	4.524	3.048	16.219
0.762	0.507	1.981	5.523	3.200	18.318
0.914	0.799	2.134	6.652	3.353	20.586
1.067	1.176	2.286	7.901	3.505	23.000
1.219	1.641	2.438	9.281	3.658	25.592
1.372	2.205	2.591	10.806	3.810	28.334

Table 3-2 Chapter 3

Table 3-1. Uniform Flow in Trapezoidal Channels by Manning's Formula (3-3)

Values of $\frac{1.49\ Qn}{b^{8/3} S^{1/2}}$



d/b[1]	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	.00429	.00433	.00434	.00437	.00443
.04	.00661	.00670	.00679	.00685	.00690	.00690	.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	.0312	.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
d/b[1]	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

.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
.45	.256	.303	.346	.385	.442	.455	.487	.519	.548	.665
.46	.263	.313	.359	.401	.439	.475	.509	.541	.547	.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
d/b^1	**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**

1.00	.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
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	25.8	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

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.

List of Tables for HEC 14-Hydraulic Design of Energy Dissipators for Culverts and Channels (Metric)

Back to Table of Contents

Back to Table of Contents

List of Charts & Forms for HEC 14-Hydraulic Design of Energy Dissipators for Culverts and Channels (Metric)

Back to Table of Contents

Back to Table of Contents

List of Equations for HEC 14-Hydraulic Design of Energy Dissipators for Culverts and Channels (Metric)

Back to Table of Contents

Back to Table of Contents

Introduction : HEC 14

Go to Table of Contents

Preface

The purpose of this circular is to provide design information for analyzing 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 Chapter 6, Chapter 7, Chapter 8, Chapter 9, Chapter 10, and Chapter 11 cover the general types of dissipaters: Hydraulic Jump, forced hydraulic jump, impact, drop structure, stilling well, and riprap. the design concept presented in Chapter 1 is illustrated in Chapter 12, Design Selection. In this chapter, the different dissipater 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 States' present practice and experience.

Acknowledgements

This circular was prepared as an integral part of Demonstration project no. 31, "Hydraulic Design of Energy Dissipators for Culverts and Channels," sponsored by Region 15. Mr. Philip L. Thompson of Region 15 and Mr. Murray L. Corry of the Hydraulics Branch wrote sections, coordinated, and edited the circular. Dr. F. J. Watts of the University of Idaho (on a year assignment with Hydraulics Branch), Mr. Dennis L. Richards of the Hydraulics Branch, Mr. J. Sterling Jones of the Office of Research, and Mr. Joseph N. Bradley, Consultant to the Hydraulics Branch, contributed substantially by writing sections of the circular. Mr. Frank L. Johnson, Chief, Hydraulics Branch and Mr. Gene Fiala, Region 10 Hydraulics Engineer, supported the authors by reviewing and discussing the drafts of the circular. Mr. John Morris, Region 4 Hydraulics Engineer, collected research results and assembled a preliminary manual which was used as an outline for the first draft. Mrs. Linda L. Gregory and Mrs. Silvia M. Rodriguez of Region 15 Word Processing center and Mrs. Willy Rudolph of the Hydraulics branch aided in manual preparation.

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

Go to Table of Contents

Go to Table of Contents

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

Go to Table of Contents

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
V	Volume
v	Velocity at a point
W	Transverse dimension; width
w	Weight
y	Depth of flow
y_e	Equivalent depth = $(A/2)^2$
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
±	Kinetic energy coefficient; inclination angle
²	Velocity (momentum) coefficient; wave front angle
³	Specific weight
”	Small increment
¸	Angle: inclination, contraction, central
¼	Dynamic viscosity
v	Kinetic viscosity
Á	Mass density of fluid (1.94 slugs/cu. ft. for water)
£	Summation symbol
Ä	Shear stress

References

2B2. AASHTO, Highway Drainage Guidelines,
Guidelines for the Hydraulic Design of Culverts,
Vol. IV, 1975, 45 pp.

5-11. AASHTO. Model Drainage Manual.
Task Force on Hydrology and Hydraulics
Chapter 11 (1991)

5-10. Abt, S.R., Donnell, C.A., Ruff, J.F., and Doehring, F.K.,
Culvert Slope and Shape Effects on Outlet Scour.
Transportation Research Record 1017, pp. 24-30, 1985.

5-11. Abt, S.R., Ruff, J.F., and Doehring, F.K.,
Culvert Slope Effects on Outlet Scour.
Journal of Hydraulic Engineering, ASCE, 111(10), pp. 1363-1367.October 1985b

5-12. Abt, S.R., Donnell, C.A., Ruff, J.F., and Doehring, F.K.,
Influence of Culvert Shape on Outlet Scour.
Journal of Hydraulic Engineering, ASCE, (113)(3), p. 393-400, March 1987.

5-13. Blaisdell, Fred W., and Anderson, C. L.
A Comprehensive Generalized Study of Scour at Cantilevered Pipe Outlets, I
IAHR, Journal of Hydraulic Resistance, 26(4), p. 357-376. (1988a)

5-14. Blaisdell, Fred W., and Anderson, C. L.
A Comprehensive Generalized Study of Scour at Cantilevered Pipe Outlets, II
IAHR, Journal of Hydraulic Resistance, 26(5), p. 509-524. (1988b)

4A2. Blaisdell, F. W.,
Flow Through Diverging Open Channel Transitions At Supercritical Velocities,
U.S. Department of Agriculture,
SCS Report-SCS-TP-76, April 1949

9B1. Blaisdell, Fred W., and Donnelly, Charles A.,
The Box Inlet Drop Spillway And Its Outlet,
Transactions, ASCE, Vol. 121, pp. 955-986, 1956.

7G1. Blaisdell, Fred W.,
The SAF Stilling Basin ,
U.S. Government Printing Office, 1959.

5-1. Bohan, J. P.,
"Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets."
U.S. Army Engineer Waterways Experiment Station,
Vicksburg, Mississippi.

6-1. 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.

9-2, 4A6 Chow, Ven Te,
Open-Channel Hydraulics,
McGraw-Hill Book Company, Inc., New York, 1959.

8D1 . Colgate, D.,
Hydraulic Model Studies of Corrugated Metal Pipe Under drain Energy Dissipators,
USBR, REC-ERC-71-10, January 1971.

5-15 . Corry, M. L., Thompson, P. L., Wyatts, F. J., Jones, J. S., and Richards, D.C.
Hydraulic Design of Energy Dissipators for Culverts and Channels.
Chapter 5, Hydraulic Engineering Circular No. 14,
Office of Engineering, FHWA Washington D.C. December 1975.

7B8. Culvert Velocity Reduction By Internal Energy Dissipators,
Concrete Pipe News, pp. 87-94,
American Concrete Pipe Association, Arlington, VA, Oct. 1972.

5-10. Doehring, F. K.
The influence of Drop Height on Outlet Scour
Master Thesis, Colorado State University, Fort Collins, Colorado, December, 1987.

5-16. Doehring F. K. and Abt, S. R.
Drop Height Influence on Outlet Scour,
Journal of Hydraulic Engineering, ASCE, 120(12), December 1994.

5-17. Donnell, C. A. and Abt, S. R.
Culvert Shape Effects on Localized Scour,
Hydrographic Engineering Publications CER82-83CAD-SRA42.
Colorado State University, Fort Collins, Co. 1983.

9A1. 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.

5-2 . Dunn, I. S.,
Tractive Resistance of Cohesive Channels.
Journal of the Soil Mechanics and Foundations,
Vol. 85, No. SM3, June 1959.

3-1. 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)

10A1. Fiala, G. R., and Albertson, M. L.,
Manifold Stilling Basin, Journal of Hydraulics Division, ASCE,
HY-4, July 1961, pp. 55-81.

5-3. Fletcher, B. P. and Grace, J. L.,
Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets."
Misc. Paper H-72-5, U.S. Army Waterways Experiment Station, 1972
Vicksburg, Mississippi.

10B2. Grace, J. L., Pickering, G. A.
Evaluation of Three Energy Dissipators For Storm Drain Outlets,
U.S. Army WES, HRB 1971, Washington, D.C.

2A3. Harrison, L. J., Morris, J. L., Norman, 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 Engineering Circular No. 13).

2A1. 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 p.
(Hydraulic Engineering Circular No. 5).

2A2. 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).

4A3. Hinds, J.,
The Hydraulic Design of Flume And Siphon Transitions, Transactions,
ASCE, vol. 92, pp. 1423-1459, 1928

7-1. Horner, S. F.,
Fluid Dynamic Drag
Published by author, 2 King Lane, Greenbriar, Bricktown, N.J. 08723, 1965.

8C1. *"Hydraulic Design of Stilling Basins,"*
Journal of the Hydraulics Division, A.S.C.E.,
paper 1406, October 1957.

10B1.. *Impact Type Energy Dissipators For Storm-Drainage Outfalls Stilling Well Design,*
U.S. Army Corps of Engineers,
Technical Report No. 2-620 March 1963,
WES, Vicksburg, Mississippi.

4A5. Ippen, A. T.,
Mechanics of Supercritical Flow, ASCE
Transactions Volume 116, pp. 268-295, 1951

7A3, 7B6. Jones. J. S.,
FHWA In-House research

8A1. Keim, R. S.,
The Contra Costa Energy Dissipator
Journal of Hydraulic Division, A.S.C.E. Paper 3077,
March 1962, p. 109.

5-18. Laursen E. M.
Observations on the Nature of Scour
Proceedings of the fifth Hyd. Conference Bulletin 34,
University of Iowa City, IA, P. 179-197. 1952.

5-4. Lamb, T. W. and Whitman, R. V.,
Soil Mechanics.
John Wiley and Sons, Inc., 1969.

8B1. MacDonald, T. C.,
Model Studies Of Energy Dissipators For Large Culverts,
Hydraulic Engineering Laboratory Study HEL-13-5,
University of California, Berkeley, November 1967.

5-5. McGowan, M. J.,
"Scour in Uniform Sand at Culvert Outlets."
Master Thesis Working Papers,
Colorado State University, Fort Collins,
Colorado, April 1980.

5-6. Mendoza, C.,
"Headwall Influence on Scour at Culvert Outlets."
Masters Thesis,
Colorado State University, Fort Collins,
Colorado, April 1980.

4B. Meshgin, K., Moore, W. L.,
Design Aspects and Performance Characteristics of Radical Flow Energy Dissipators,
University of Texas at Austin, August 1970.
Research Report 116-2F

7B4. Mohanty, P. K.,
The Dynamics Of Turbulent Flow In Steep, Rough, Open Channels,
Ph.D. Dissertation,
Utah State University, Logan, Utah, 1959.

7C4. Morris, H. M.,
Applied Hydraulics In Engineering,
The Ronald Press Company, New York, 1963.

7B5. Morris,
Bulletin 19,
pp. 50, 54 & 56.

7B3. 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.

7A2, 7B2. Morris, H. M.,
Hydraulics Of Energy Dissipation In Steep Rough Channels,
Virginia Polytechnical Institute Bulletin 19, VPI & SU, Blacksburg, VA., Nov. 1968.

2B1. Normann, J. M.,
Design of Stable Channels with Flexible Linings,
Federal Highway Administration, October 1975

7C1. Normann, J. M.,
Hydraulic Aspects of Fish Ladder Baffles in Box Culverts,
Preliminary FHWA Hydraulics Engineering Circular, Jan. 1974.

5-7. Opie, T. R.,
"Scour at Culvert Outlets."
Master Thesis,
Colorado State University, Fort Collins,
Colorado, December 1967.

7B1. Peterson, D. F. & P. K. Mohanty,
Flume Studies of Flow In Steep Rough Channels,
ASCE Hydraulics Journal, HY-9, Nov. 1960.

7C3. Powell, R. W.,
Flow in A Channel of Definite Roughness,

ASCE Transactions, Figure 10, p. 544, 1946.

9-1. Rand, Walter,
Flow Geometry At Straight Drop Spillways,
Paper 791, Proceedings, ASCE, Volume 81, pp. 1-13, September 1955.

5-19. Robinson, A. R.
"Model Study of Scour From Cantilevered Outlets
Transitions, ASAE 14, p. 571-581. 1971.

5-20. Rouse H.
Experiments on the Mechanics of Sediment Suspension."
Proceedings of fifth International Congress For Applied Mechanics.
John Wiley and Sons, Inc. New York, N.Y. 1938.

5-8. Ruff, J. F. and Abt, S. R., "
Scour at Culvert Outlets in Cohesive Bed Materials."
Prepared for the U.S. Department of Transportation, Federal Highway Administration,
CER79-80JFR-SRA61, May 1980.

5-21. Ruff, J. F., Abt, S. R., Mendoza, C., Shaikh, A. and Kloberdanz, R. (1982).
Scour at Culvert Outlets in Mixed Bed Materials,
Rep. No. FHWA/RD-82/011,
Offices of Research and Development, Washington D.C. 20590, September 1982.

2C1. Searcy, James K.,
Use of Riprap for Bank Protection
Federal Highway Administrations,
Washington, D.C., 1967, pp. 43.

5-9. Shaikh, A.,
"Scour in Uniform and Graded Gravel at Culvert Outlets."
Master Thesis,
Colorado State University, Fort Collins,
Colorado, May 1980.

7C2. Shoemaker, R. F.
Hydraulics of Box Culverts With Fish Ladder Baffles,
Proceedings of the 35th Ann. Meeting of the HRB,
Washington, D.C., 1956, pp.196-209

3-2, 7A1, 11-2. 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.

11-1. 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.

6-2. Sylvester, R.,
Hydraulic Jump In All Shapes Of Horizontal Channels,
Jourl. Hyd. Div. ASCE, Vol. 90, HY1. Jan 1964, pp. 23-55.

4A4. U.S. Army, Corps of Engineers,
Hydraulic Design of Flood Control Channels,
Engineering and Design Manual EM 1110-2-1601, July 1970, pp. 20-26

11-4. 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.

6-1. U.S. Bureau of Reclamation,
Design of Small Dams, Second Edition
1973, GPO

3-3. U. S. Department of the Interior, Bureau of Reclamation
Design of small Canal Structures,
1974, pp. 127-130.

11-3. U. S. Department of Transportation,
Use Of Riprap For Bank Protection,
Hydraulic Engineering Circular No. 11,
Government Printing Office, Washington, D.C., 1967.

4A1, 7B9. Watts, F. J.,
Hydraulics of Rigid Boundary Basins,
Ph.D. Dissertation,
Colorado State University, Fort Collins,
Colorado, August 1968

7B7. 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.