DESIGN OF

URBAN DRAINAGE SYSTEMS

Notes for an Educational Course

presented by

Stormwater Consultants

PART A

SC

R-202

DESIGN OF URBAN DRAINAGE SYSTEMS

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Presented and Published by

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[1] Inlets

CHAPTER IV*

4.0 STORM SEWER DESIGN

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* Virginia Department of Highways and Transportation, "Drainage Manual", Richmond, Virginia, 1980. from Dept Highway Admin -

4.0 Storm Sewer Systems

This chapter addresses the planning and design of roadway drainage systems. The complete system will be referred to as a storm sewer system, and may consist of curbs and gutters, inlet structures, laterals and trunk lines, junction chambers and manholes.

The purpose of a storm sewer system is to collect and transport storm water runoff from the highway to an outfall.

4.1 Definition

For purposes of interpretation of the policies and procedures of the Virginia Department of Highways and Transportation, a storm sewer system is defined as follows:

A STORM SEWER SYSTEM IS A DRAINAGE SYSTEM INSTALLED TO CARRY STORM WATER RUNOFF, CONSISTING OF ONE OR MORE PIPES CONNECTING TWO OR MORE DROP INLETS. An exception to this general rule is; one or more cross drain pipes connecting two or more drop inlets, "hydraulically designed" to function as a culvert or culverts and not connected to a storm sewer system.

4.2 Policy

can vary

1. In general, all storm sewers will be designed for storms having a 10-year recurrence frequency. Exceptions to this will be based on local conditions where potential damage to contiguous property is excessive and justifies employment of a design storm of less frequency (greater intensity) and storm sewers that drain depressed roadway and interstate roadways.

2. Concrete pipe only will be permitted for storm sewer installations on roadway projects constructed by the Department. Exceptions to this rule are outlined in current "Instructional and Information Memorandum LD 81(D)121. N = .012 (Smooth) → .022 (Rough)

3. Cost participation in the construction of Storm Sewers- See current "Instructional and Information Memorandum" LD78 (R)-9.1 entitled "Commission Policies on Participation by towns, Cities and Others."

4. Access Points - Generally the distance between points of access in storm sewer trunk lines shall be limited to 50 feet where the pipe diameter is 12"; 300 feet where the pipe diameter is between 12" and 42" and 800 feet where the pipe diameter is 42" or larger. The distance between access points may be increased 100 feet if the pipe diameter is greater than 12" and the velocity of flow will exceed 5 feet per second when the depth of flow is equal to 25 per cent of the pipe diameter.

5. A storm sewer system is to be designed using an "n" factor of 0.013.

6. A hydraulic grade line or potential water surface profile should be determined for the entire system.

4.3 Criteria

← Spread — 8-10' Fed Hwy or ½ Traffic Lane

1. Curb Inlets - For curb or curb and gutter sections, the inlet spacing shall be sufficient to limit the spread to 8' to 10' from the curb. A rainfall intensity of 3 inches per hour will be used for the design spacing of inlets on non-federal aid highways. On Federal Aid highways, the roadway inlets should be spaced so that not more than half of a through lane will be flooded during a 10-year frequency storm runoff except that a 50-year frequency shall be used for underpasses or other depressed roadways where ponded water can be removed only through the storm sewer system.

2. Storm Sewers - Storm sewers will generally be designed to accommodate the 10-year storm runoff without surcharge except the storm sewers will be designed to accommodate the 50-year storm where necessary to prevent flooding of an interstate roadway and at underpasses or other depressed roadways on Federal-Aid projects where ponded water can be removed only through the storm sewer system. The acceptable practice of matching crowns (not inverts) at junctions will be employed where practical, and where this cannot be accomplished, due consideration to the hydraulic operation will be given so as to prevent the hydraulic grade line of the system from rising above the top of junction facilities such as manholes, etc. Where a 10-year or prescribed greater design is not feasible, a lower design will have to be given special consideration and approval by the Central Office.

- Cole Hydr Gradeline thru-out system

4.4 Surveys

When a storm sewer system is in place, invert elevations should be determined on all pipes, drop inlets, catch basins, manholes, etc. It is essential that all utilities in the area of of the existing or proposed storm sewer system be located and their elevations given in order to avoid conflict between the existing utilities and the future storm sewer system. This information should be extended well beyond the limits of the proposed project, both laterally and longitudinally, at least to the next access structure. The invert elevation of each pipe entering and leaving a drop inlet and/or manhole must be acquired.

For more detailed information relative to survey data required see the Virginia Department of Highways and Transportation Survey Instruction Manual.

4.5 Hydrology (NOT yet Exact, If ever) 2. If rain duration => than time of Concentration 3. weight C Values.

The RATIONAL METHOD to determine discharge is used exclusively by the Department in the design of storm sewer systems. This method is discussed in detail in Chapter 1 of this manual. 1. For small watershed up to 50 or pos. 100 Ac. 2. Break into pieces 3. then design routing 4. log time

4.6 Hydraulic Design Methods and Charts

This section contains design charts, nomographs and design methods . Certain of the design methods are limited to specific conditions and have limitations as to their application and accuracy.

It is recommended that the user of this material become familiar with the various explanations and detailed instructions which precede each design method.

(regression Equation) C = (~~Value~~ function land Use) SCS Soil + land use (TR 55 Manual)

4.6.1 Shallow Triangular Channels

Shallow triangular channels as used here include gutters along curbs having either vertical or sloping faces, depressed medians that approximate a triangular cross section, and other triangular ditches having side slopes flatter than about 10:1.

Flow in shallow triangular channels is based on the equation $Q = 0.56 \frac{z}{n} S^{1/2} y^{8/3}$. This is a variation of the Manning formula, solved graphically by means of the nomograph in Figure 4.6.1.1.

The friction along the face of a vertical curb is ignored as being negligible. This assumption also facilitates the use of the formula in paragraph 2 above and Figure 4.6.1.1 with triangular shaped swales and ditches.

The depth of water at the curb "y" and the spread on the pavement, "Sp", in a curbed section, depend upon discharge, "Q", longitudinal gutter slope, "S", pavement cross slope $S_x = 1/Z = y/S_p$ and gutter roughness coefficient "n". Given Q, S, S_x and n, depth and spread can be determined by use of Figure 4.6.1.1. These charts are all based on the equation in paragraph 2 above.

Z = Cross slope

Qx = Q into grated inlet

CHART 29

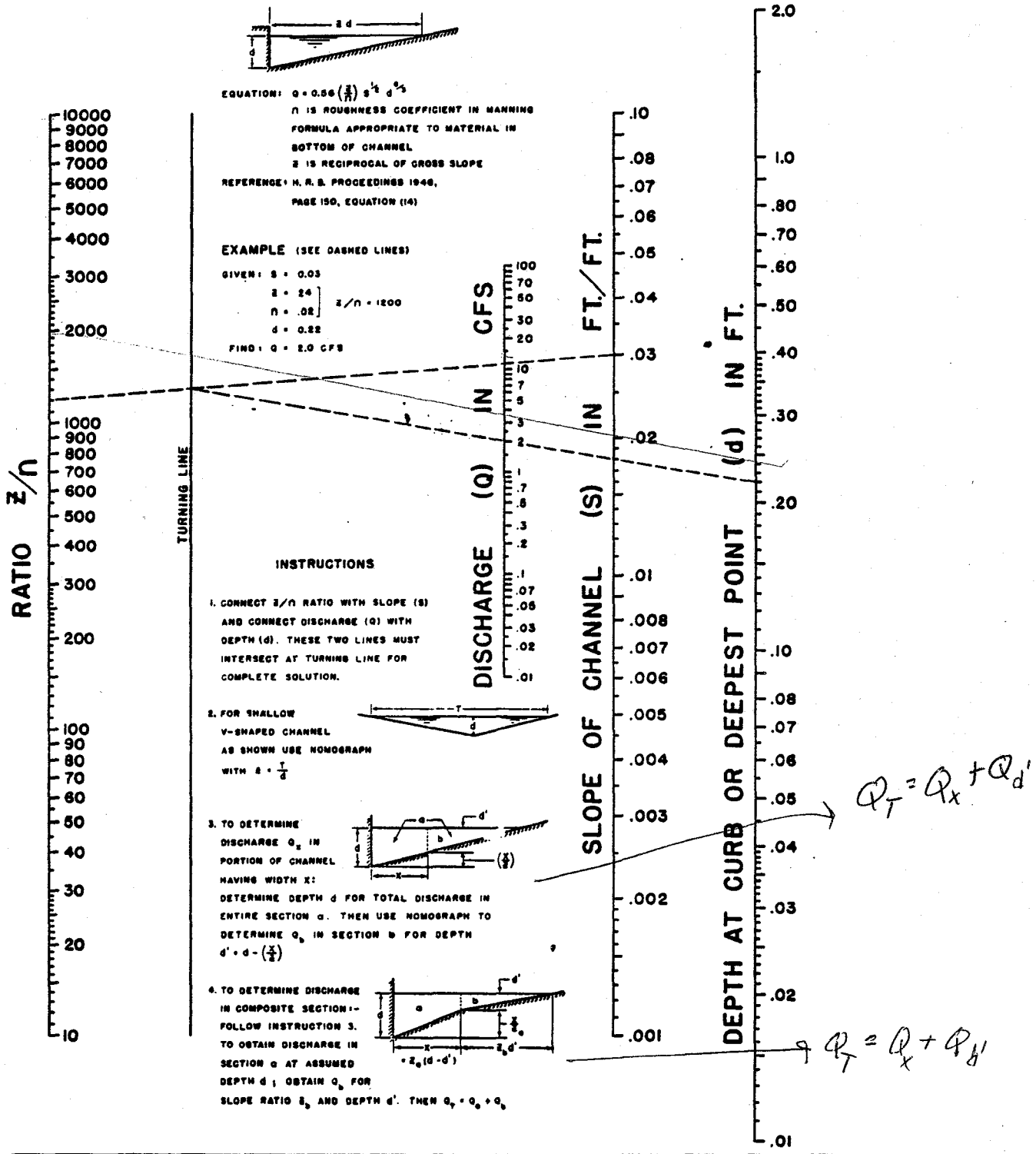


Fig. 4.6.1.1. NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

Appendix B - TABLES

*Table 8 - Roughness coefficients (Manning's n)

Highway Channels and Swales with Maintained Vegetation

Type of Grass	Manning n			
	Depth 0.7 ft.		Depth 0.7 - 1.5 ft.	
Velocity in fps	2	6	2	6
a. Bermuda, Kentucky bluegrass, buffalo				
(1) Mowed to 2"	0.07	0.045	0.05	0.035
(2) Length 4" - 6"	0.09	0.05	0.06	0.04
b. Good stand, any grass				
(1) Length 12" +	0.18	0.09	0.12	0.07
(2) Length 24" +	0.30	0.15	0.20	0.10
c. Fair stand, any grass				
(1) Length 12" +	0.14	0.08	0.10	0.06
(2) Length 24" +	0.25	0.13	0.17	0.09

Street and Expressway Gutters

a. Concrete gutter troweled finish	0.012
b. Asphalt pavement	
(1) Smooth texture	0.013
(2) Rough texture	0.016
c. Concrete gutter with asphalt pavement	
(1) Smooth	0.013
(2) Rough	0.015
d. Concrete pavement	
(1) Float finish	0.014
(2) Broom finish	0.016
e. Brick	0.016

For gutters with small slope where sediment may accumulate, increase all above values of "n" by 0.002.

* Searcy, J. K., "Drainage of Highway Pavements", Federal Highway Administration, U.S. Dept. of Transportation, Washington, D.C., Hydraulic Engineering Circular No. 12, 1969.

4.6.2 Inlet Spacing

Inlets should be so spaced as to limit the spread of water on the pavement to a predetermined amount that will not interfere with traffic for a frequent design storm. The allowable spread depends upon the width of gutter and location and width of travel lanes. Suggested design storm is noted in Section 4.3.

The design discharge for use in determining gutter flow in the spacing of storm water inlets is determined by means of the Rational formula.

Having selected the design storm under the first paragraph, the spacing of inlets will be effected by the capacity of the individual inlet. See Criteria - 4.3 - Paragraph 1. - Curb Inlets.

Where an inlet is located on a continuous grade, flow bypassing that inlet must be included in the total gutter flow contributing to the next inlet downstream unless it is carried off on a side street or otherwise intercepted.

Where an inlet is located at the bottom of a sag vertical curve (referred to as a sump or low point) all of the flow must go into the inlet unless it overtops the curb or crown of the pavement. The problem in an analysis is to determine how deep the water must be at the inlet in order for it to intercept the peak flow.

To properly drain sag vertical curves, it is usually a good practice to place three inlets in each curve; one at the low point and one on each side of the low point. A check of the width-spread approaching the low point from each direction, where the gutter grade is one tenth per cent, will determine the need for and proper location of the two inlets. This removes the flow before the spread becomes excessive and will reduce deposits of sediment on the pavement.

Where the pavement on a continuous grade is warped in transitions between superelevated and normal sections, water should normally be picked up before the cross slope of the pavement begins to change. This is particularly important in areas where icing occurs.

Large quantities of runoff from areas off the project that would normally enter the project from side streets should, wherever possible, be picked up on the side street before it reaches the project.

Street inlets are, at best, inefficient devices for intercepting water. Where curbs are used, runoff from cut slopes and areas off the right-of-way should, wherever possible, be intercepted by ditches at the top of slopes or in a swale along the shoulder. This reduces the amount of water that has to be picked up by gutter inlets and prevents mud and debris from being carried onto the pavement. This is particularly important at the high side of superelevated curves on divided

highways using a continuous raised median.

Grate-type inlets are more effective if designed and spaced to pick up 75 to 85 per cent of the flow in the gutter. Curb opening inlets should be designed and spaced to pick up 85 to 90 per cent of the flow.

Having located the inlets for the design storm described in paragraph 1, it is desirable to check the design for a 10-year frequency storm. If excessive flooding is indicated the spacing should then be corrected.

When bridges are located adjacent to curb and gutter sections, it will be necessary to coordinate the drainage design with the bridge designer. For bridges without scuppers, the flow from the entire bridge plus any flow crossing the bridge will need to be picked up in the storm sewer system. For bridges with scuppers, it will be necessary for the drainage designer to review the scupper size, type and location. The flow capacity as well as the potential for scupper clogging should be fully considered and the storm sewer system designed accordingly.

4.6.3 Grated Inlets

Where the gutter or other approach channel is on a continuous grade, grate inlets with efficient openings can be expected to intercept, over the end, all the water

Curb above Grate & 2 Systems
low pt. (Grate - Curb - Grate) best —

flowing in the part of the gutter cross section above the grating plus an amount that will flow in along the exposed edges. For grates less than 2 feet long this side inflow is very small.

To estimate the inflow along the side of the grate use Fig. 4.6.10.1 to 4.6.10.7. Consider the edge of the grate the same as a curb opening. Ignore crossbars in determining the length of opening. Use the depth at the outer edge of the grate and take into account any local depression at the outer edge. In a rigid analysis the length of the outer edge of the grate should be reduced by the length of the jet falling in over the end of the grate.

A grating has <u>efficient</u> openings if:

a. The bars run parallel to the direction of flow. (Bicycle problem)

b. The openings cover at least 50 per cent of the width of the grate.

c. The unobstructed opening is long enough (generally at least 18 inches) parallel to the direction of flow to allow the water entering the end of the grate to fall clear of the downstream end of the slot.

The capacity of any other type of grating must be determined experimentally.

<u>Efficient openings</u> (as defined herein) also appear to have the best nonclogging characteristics.

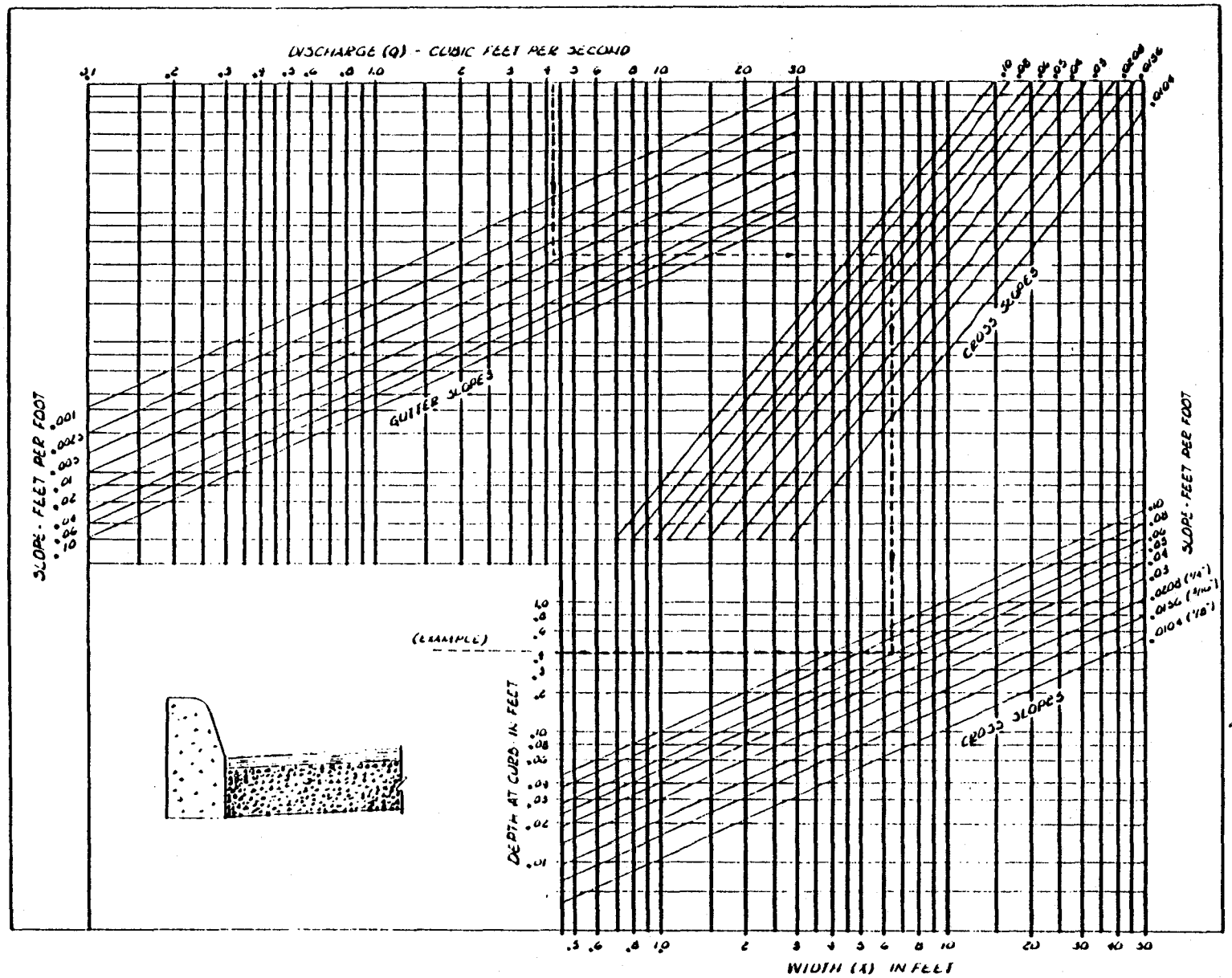


FIG. 4.6.10.1

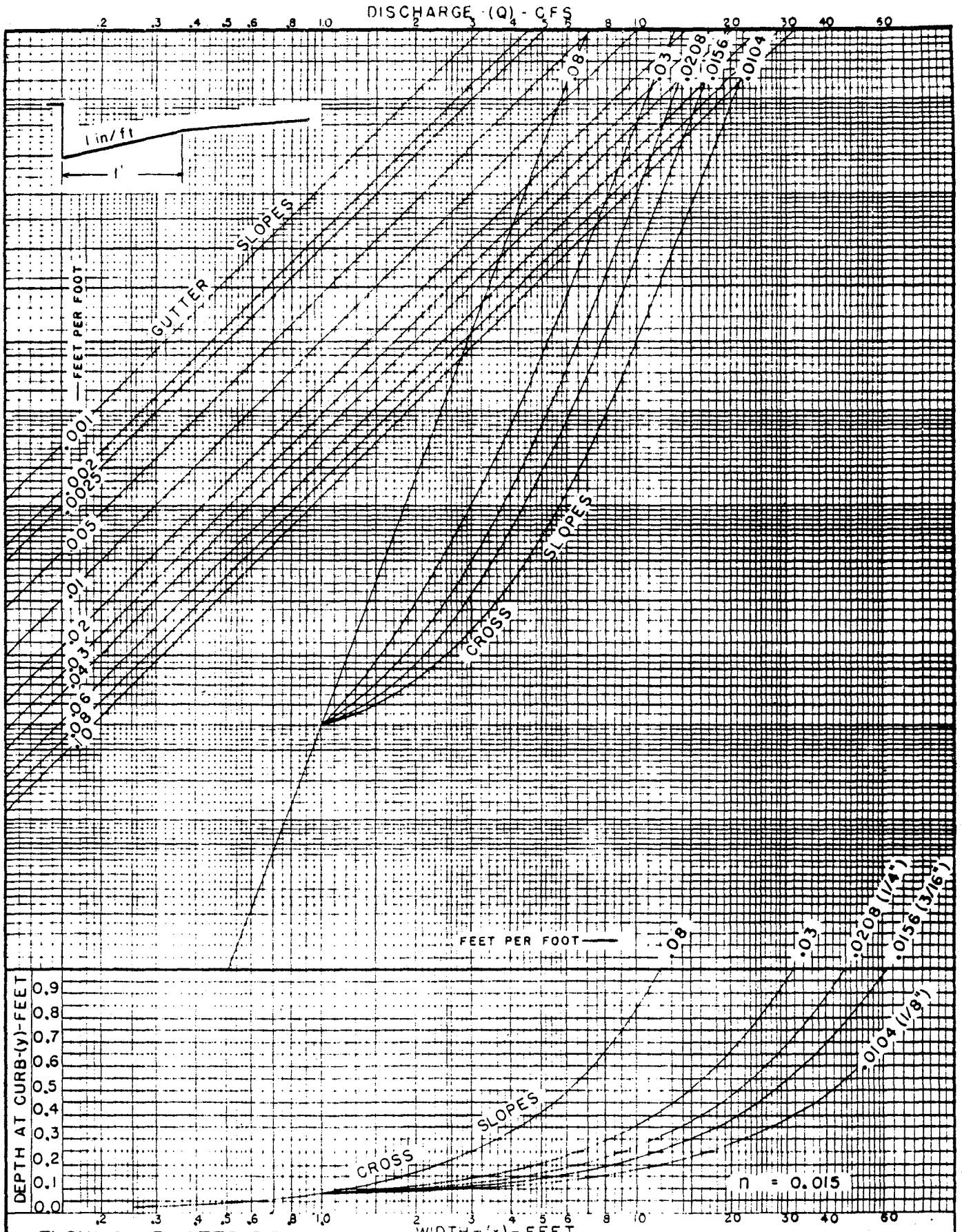
1. Grate on Continuous grade
2. Gutter at low pt.
   a. Given Q what is depth impound.
   b. Set Height + Set Q - no over flow

FLOW CHARACTERISTIC CURVES

STRAIGHT CROSS SLOPES WITH CURB

n=0.015

FIG. 4.6.10.3



FLOW CHARACTERISTIC CURVES

STRAIGHT CROSS SLOPES
CURB, 1 FT GUTTER-1 IN/FT.

The capacity of a grate inlet, where the gutter is on a continuous grade, increases rapidly when part of the flow is allowed to go past the inlet. This is due to the increased depth in the cross section of flow over the grating.

Uniformly spaced inlets in series on a uniform continuous grade tend to adjust automatically to intercept the runoff tributary to each inlet by the time the third or fourth inlet is reached, provided the carryover from the first inlets does not exceed about 25 per cent.

The capacity of inlets with diagonal bars is increased substantially by rounding the top surface of each bar to the approximate form of a free overfall.

The capacity of an efficient grate inlet increases with the slope up to the point where the falling jet hits the downstream end of the slot.

Capacity of a grate inlet in a sump or low point (water ponded) may be estimated as follows:

a. For depths of water up to 0.4 feet use the weir formula $Q = CPH^{3/2}$

Weir =

Where P = perimeter of the grate opening ignoring the bars

H = depth of water

C = may be taken as 3.0

Where one side of grate is against a curb, this side must be omitted in computing the perimeter.

Submerged Inlet

b. For depths of water above 1.4 feet use orifice formula $Q = CA\sqrt{2\ gH}$

Where A = total area of clear opening

H = depth of water

C = may be taken as 0.67

c. For depths between 0.4 and 1.4 feet operation is indefinite due to vortices and other disturbances. Capacity will be somewhere between that given by the above formulas.

d. Problems involving the above criteria may be solved graphically by use of Figure 4.6.10.11.

The minimum length of clear opening, parallel to the direction of flow, required to allow the jet of water falling through the opening to clear the downstream end of the slot depends upon the depth and velocity of flow in the approach gutter and the thickness of the grate at the end of the slot. The minimum length may be estimated by the following formula:

$$L = \frac{V}{2}\sqrt{y+d_b}$$ 100% of flow

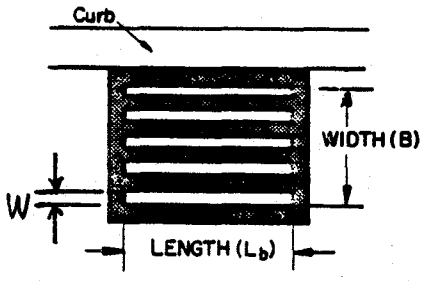
Where L = minimum length of slot in feet

V = mean velocity of flow in the approach gutter, fps

y = depth of water at curb in approach gutter, ft

d_b = thickness of grate at downstream end of the slot. (depth of the bar) diam (HT) Sq.

This equation is based on trajectory of a free falling body $L = Vt$ $S = y + d_b = 1/2\ gt^2$ with a numerical constant modified on the basis of experimental data.

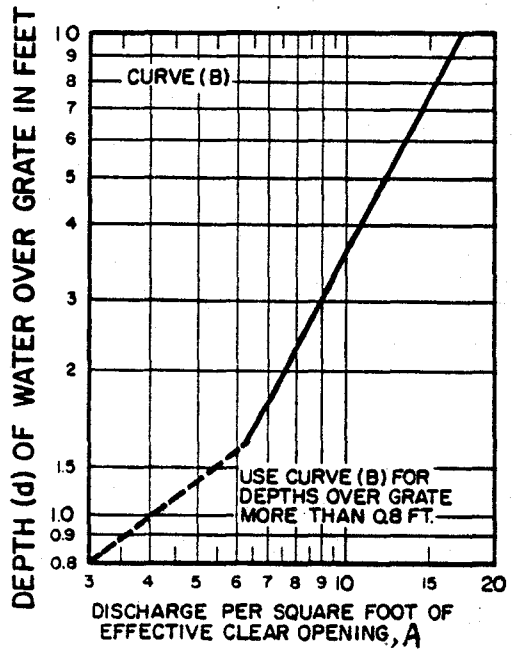


$P = 2B + L_b$

A = AREA OF CLEAR OPENING IN GRATE TO ALLOW FOR CLOGGING DIVIDE P OR A BY 2 BEFORE OBTAINING d, water depth

WITHOUT CURB $P = 2(B + L_b)$

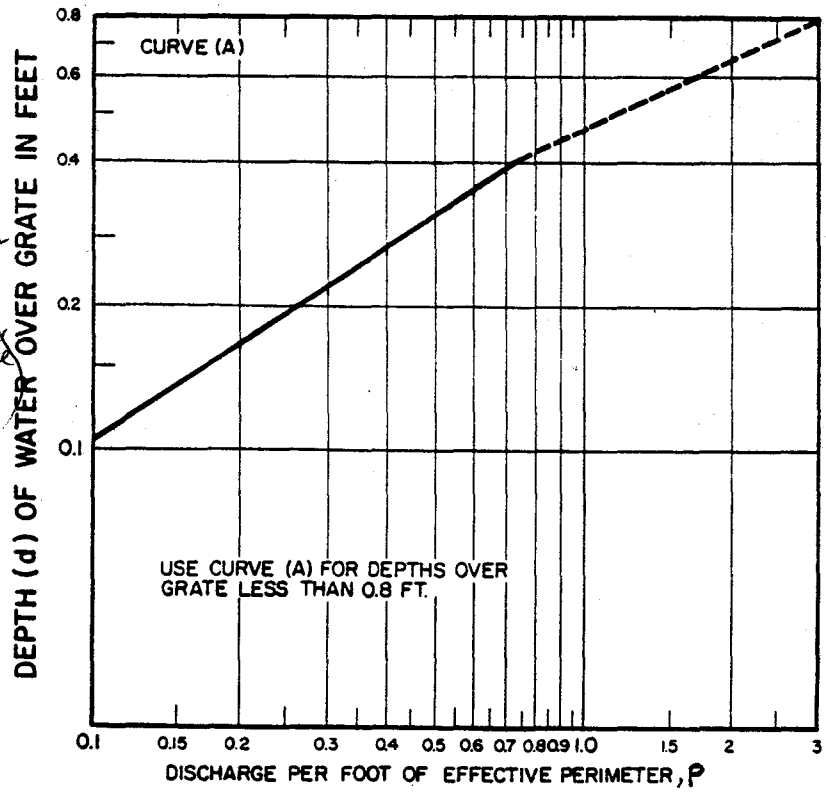
$A = 5 \cdot L_b \cdot W$



$= \frac{Q}{A}$

$d = < .4\,FT) =$ weir flow

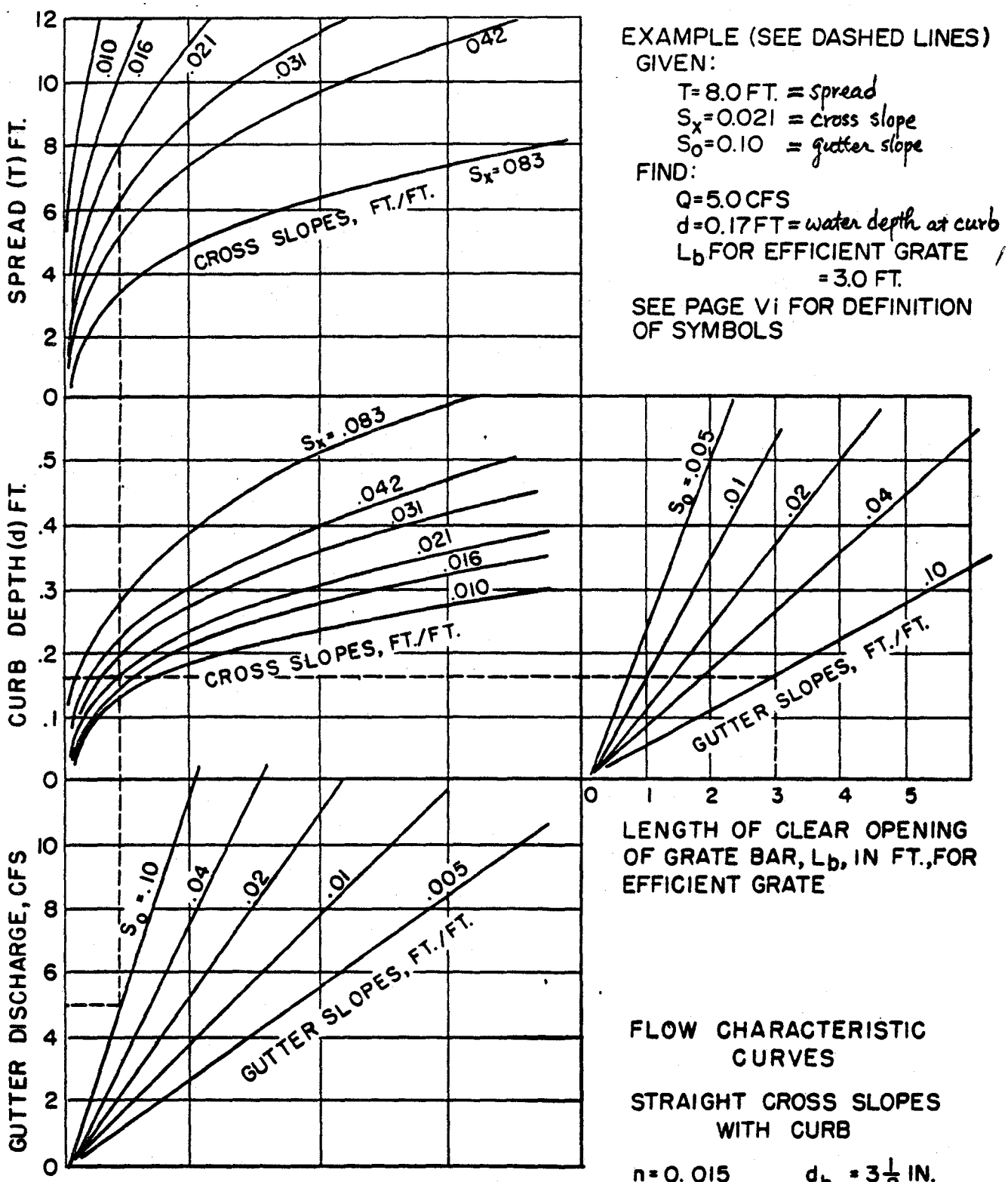
$d = > 1.4$ (orifice flow)



BUREAU OF PUBLIC ROADS
REV. AUG. 1968

HYDRAULIC CAPACITY OF GRATE INLET IN SUMP

Fig. 4.6.10.11



EXAMPLE (SEE DASHED LINES)

GIVEN:

$T = 8.0$ FT. = spread

$S_x = 0.021$ = cross slope

$S_o = 0.10$ = gutter slope

FIND:

$Q = 5.0$ CFS

$d = 0.17$ FT = water depth at curb

L_b FOR EFFICIENT GRATE 100%

$= 3.0$ FT.

SEE PAGE Vi FOR DEFINITION OF SYMBOLS

LENGTH OF CLEAR OPENING OF GRATE BAR, L_b, IN FT., FOR EFFICIENT GRATE

FLOW CHARACTERISTIC CURVES

STRAIGHT CROSS SLOPES WITH CURB

$n = 0.015$ $d_b = 3\frac{1}{2}$ IN. = depth of grate bar

BUREAU OF PUBLIC ROADS MAY 1968.

In order to develop the full capacity of an inlet consistant of two or more grates separated by short sections of paved gutter on a continuous grade, the grates should be spaced far enough apart so the carryover from the first grate will move into the curb before it reaches the second grate. This distance increases with the longitudinal slope and decreases with an increase in the cross slope. To determine the capacity of a given spacing, compute the flow over the end and side of the first grate based on total gutter flow. Compute capacity of the second grate based on carryover from the first grate by two methods: (1) Flow over the end and side assuming the carryover to have reached normal gutter depth. (2) Consider distance from lower end of first grate to lower end of second grate as the length of a curb opening inlet and the depth of flow as the depth at the outer downstream corner of the first grate. Compute interception as a curb opening. Take the lower of the two values found in (1) and (2) as the interception of the second grate.

4.6.4 Curb Opening Inlets

The capacity of a curb opening inlet on a continuous grade varies directly with:

a. Depth of water at the inlet entrance

b. Length of clear opening

The depth of water at the inlet entrance for a given discharge varies directly with:

a. Cross slope of the pavement at the curb

b. Amount of warping or depression of the gutter flow line at the inlet

c. Roughness of the flow line

Inversely with:

a. Longitudinal slope of the gutter

The capacity of a curb opening inlet when intercepting 100 per cent of the flow in the gutter is given by the formula:

$$Q = 0.7\ L\ (a + y)^{3/2}$$

Where y = depth of flow in approach gutter

a = depth of depression of flow line at inlet (usually 1 or 2")

L = length of clear opening

The capacity of a curb opening inlet is increased somewhat by allowing part of flow to go by but not as much as with a grate inlet.

The capacity of a curb opening inlet in a sump or low point varies directly with the length of the inlet and the depth of water at the entrance. The inlet will operate as a weir until

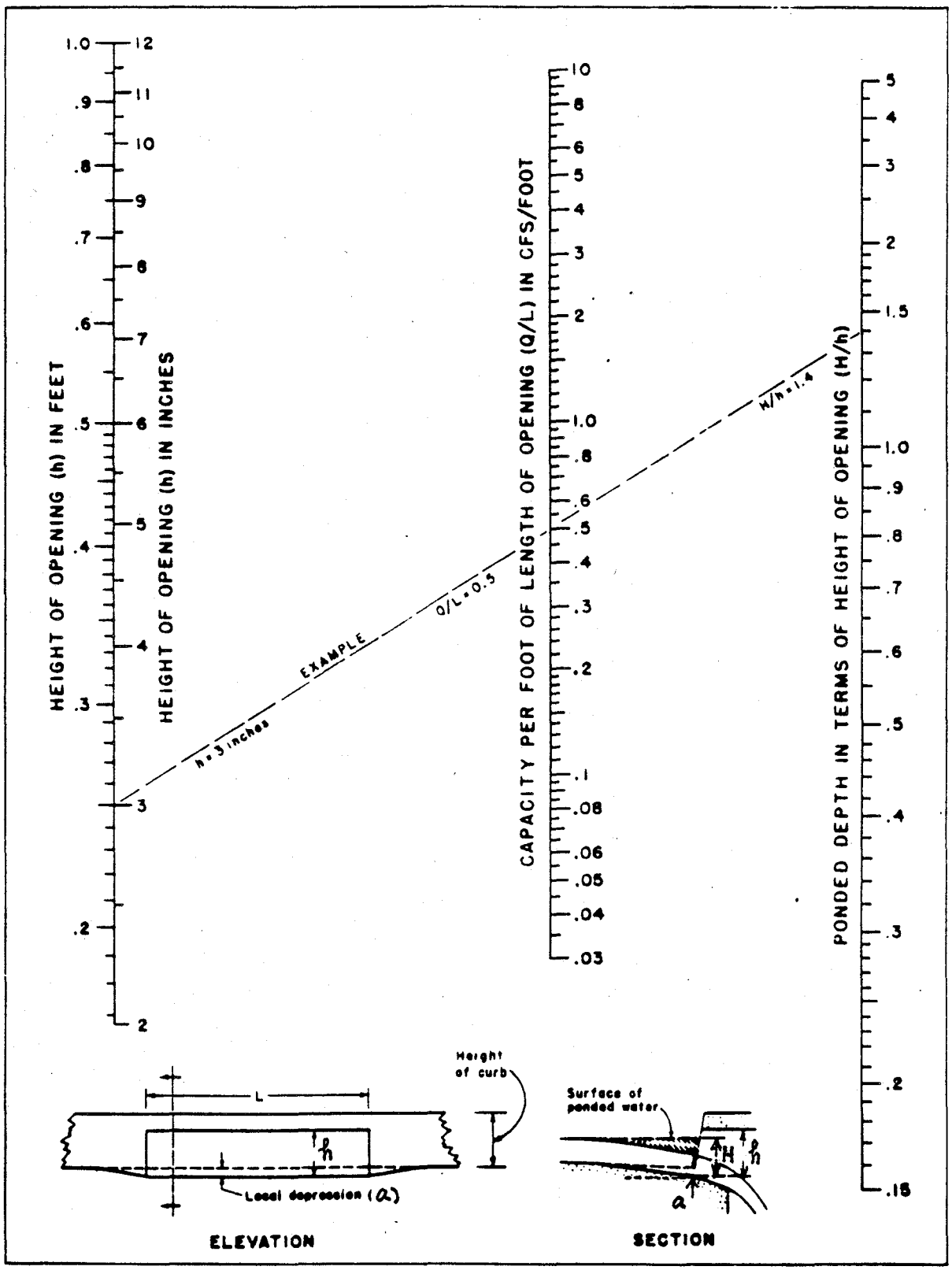
the water submerges the entrance. When the depth of water is about 1.4 h (where h is the height of the curb opening entrance) or more, it will operate as an orifice. Between these two depths it will operate somewhere between the two. The capacity may be computed graphically by use of Figure 4.6.10.9.

When using a double curb opening inlet with a center support flush with the face at the curb the effective area of the second opening should be reduced by one half to compensate for turbulence caused by the support. If the support is set back several inches from the gutter flow line the effectiveness of the second opening is much improved.

The capacity of multiple curb openings is often reduced to the capacity of a single curb opening by debris caught on the center supports.

A curb opening inlet constructed with a cantilevered top slab having no intermediate supports, and which can be built in variable lengths with a local depression of at least 2 inches is a desirable type of curb opening inlet design. This type of curb opening inlet can be varied in length according to the amount of water to be intercepted and has no supports to catch debris or deflect the flow.

FIG. 4.6.10.9



CAPACITY OF
CURB OPENING INLET
AT LOW POINT IN GRADE

4.6.5 Combination Grate and Curb Opening Inlets

The capacity of a combination inlet on a continuous grade using an "efficient" grate is not appreciably greater than with the grate alone. In computing the capacity ignore the curb opening and compute as a grate inlet.

A combination inlet on a continuous grade with the curb opening placed upstream from the grate is considered to operate as two separate inlets. Compute the capacity of the curb opening based on the toal gutter flow and the capacity of the grate based on the carry-over from the curb opening. This arrangement is desirable as in addition to increasing the capacity, the curb opening will tend to intercept the debris brought down by the first flush of water and thereby reduce the chances of clogging the grate.

A combination inlet in a sump or low point is very desirable. The curb opening should be considered as a relief opening in event the grate becomes clogged by debris. In estimating capacity, consider the inlet as a grate only but do not divide the perimeter and area by 2 as with the grate alone. See 4.6.7 - Procedure Step 2.

For a combination inlet on a continuous grade with the curb opening placed downstream from the grate to operate effectively as two separate inlets, the curb opening must be a sufficient distance below the grate to allow the carry-over from the grate to move in against the curb before it reaches the curb opening. The minimum distance will vary with both the cross slope and the longitudinal slope. It will be relatively small for a steep cross slope and flat longitudinal slope and long for a flat cross slope and steep longitudinal slope.

4.6.6 "Capacity of Curb Opening Inlets on Continuous Grade"

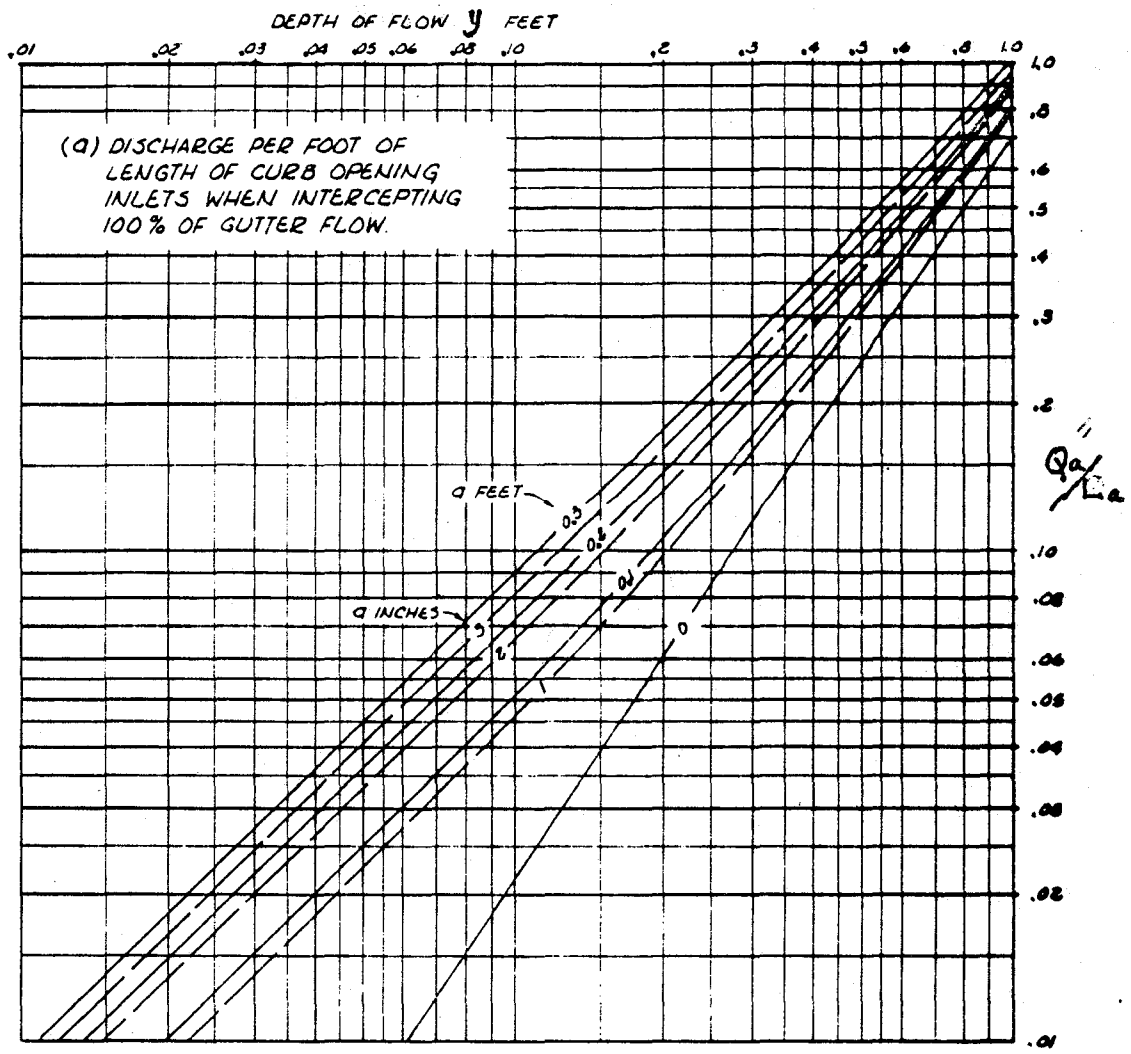
Figure 4.6.10.8 applies to curb or side opening inlets on continuous grades. It may also be used to determine inflow at the side parallel to the direction of flow (outer edge) of grate inlets on continuous grades. For interception along his edge, the grate operates like a curb opening except for possible interference from the grate bars.

The capacity of the inlet depends upon the length of opening and the depth of flow at the upper end of the opening. This depth in turn depends upon the amount of depression of the flow line at the inlet and the cross slope, longitudinal slope, and the roughness of the gutter.

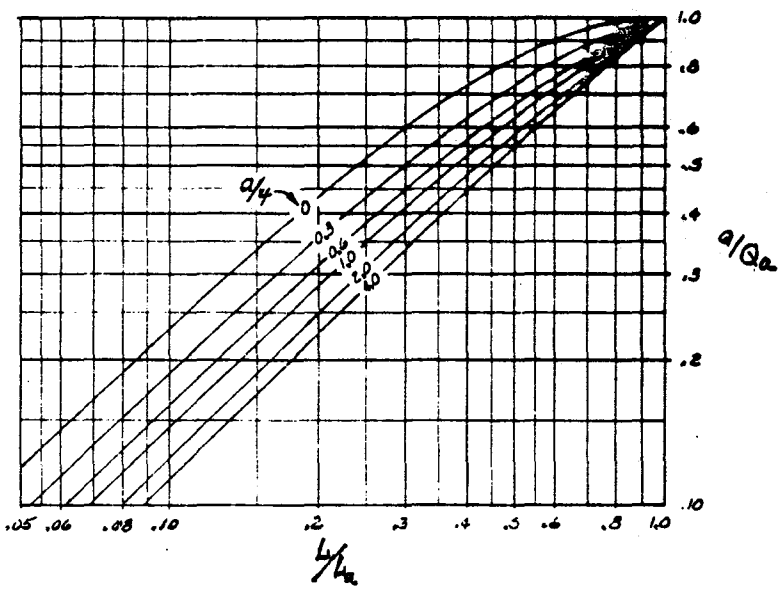
To use Figure 4.6.10.8 (a) and (b) for curb opening inlets the following information must be known:

1. Length (L) of the inlet opening.
2. Depth (a) of local flow line depression, if any, at the inlet.
3. Design discharge (Q_a) in the gutter or information as to drainage area, rainfall intensity, and runoff coefficients from which a design discharge can be estimated. Any carry-over from a previous inlet must be included.
4. Depth of flow in normal gutter for the particular longitudinal and cross slopes at the inlet in question. This may be determined from one of the Figures 4.6.10.1 thru 4.6.10.7

To use Figure 4.6.10.8 (a) and (b) for determining inflow over th outer edge of an <u>efficient type grate</u>, the following



(b)
PARTIAL INTERCEPTION FOR RATIO FOR INLETS OF LENGTH LESS THAN L_a



CAPACITY OF CURB OPENING INLETS ON CONTINUOUS GRADE

information must be known or determined:

1. Length (L) of total grate opening less an amount estimated to be occupied by the flow entering over the end of the grate.

2. Depth (a') of the local pavement depression at the outer edge of the grate.

3. The portion of the total gutter flow (Q_a) that is flowing on the pavement beyond the outer edge of the grate, i.e., total flow less the inflow over the end of the grate.

4. Depth of flow (y') on the pavement at a point in line with the outer edge of the grate.

Procedure

1. Enter Figure 4.6.10.8 (a) with depth of flow y or y' and local depression, a or a', and determine Q_a/L_a, the interception per foot of inlet opening if the inlet were intercepting 100% of the flow.

2. Determine length of inlet L_a required to intercept 100% of the flow. L_a = total flow Q_a divided by the factor Q_a/L_a.

3. Compute ratio L/L_a where L = actual length of inlet in question.

4. Enter Figure 4.6.10.8 (b) with L/L_a and the ratio a/y or a'/y' and determine ratio Q/Q_a, the proportion of the toal flow intercepted by the inlet in question.

5. Flow intercepted, Q, is this ratio Q/Q_a times the total flow Q_a.

6. Flow carried over to next inlet is $Q_a - Q$.

The partial interception over the end of an efficient type grate where the pavement has a straight cross slope may be closely approximated by use of the a/y = 0 curve on Figure 4.6.10.8(b).

4.6.7 - Capacity of Grate Inlet In a Sump

Figure 4.6.10.11 applies only when the grate is located in a low point or sump where the water will pond at the grate.

The capacity of the grate depends upon either the exposed perimeter or the area of the openings and the depth of water at the grate. Recent experiments (1) have determined that a grate will act as a weir and follow the weir formula for depths (heads) on the grate up to 0.4 ft. It will act as an orifice and follow the orifice formula for heads of 1.4 ft. and over. For heads between 0.4 and 1.4 the operation is indefinite because of vortices and eddies over the grate.

In the usual problem, the following are given:

1. A particular design of grate with dimensions
2. A design discharge (Q) or information as to drainage area, rainfall intensities and runoff coefficients from which a design discharge can be estimated.

Procedure

1. Compute the perimeter of the grate opening (P) ignoring the bars and omitting any side over which the water does not enter, such as when one side is against the face of a curb. Divide the result by 2. This allows for partial clogging of the grate by assuming that only half of the perimeter will be effective.
2. Compute the Q/P ratio, using effective perimeter after allowing for clogging.
3. Compute the total area of clear opening (A), excluding area taken up by bars, and divide by 2. This allows for partial clogging of the grate by assuming that only half of the area will be effective.

4. Compute the Q/A ratio, using effective area after allowing for clogging.

5. Enter the chart at the bottom scale, using line (a) with the Q/P value and line (b) with the Q/A value, and read the required head in feet at the left margin.

6. If the required head falls below 0.4, (a) only will apply. This is the usual case.

7. If the required head falls above 1.4 ft., (b) only will apply.

8. If the required head falls between 0.4 ft. and 1.4 ft., the actual head may be anywhere between (a) and (b). Use the value that gives the most conservative result, being sure to use line (a) with Q/P and line (b) with Q/A.

Vortices and eddies over the grate may cause the actual head to differ from the indicated head in this case (See Second Paragraph).

9. If the inlet is a combination type with grate and curb opening the recommended procedure is the same as with a grate alone except the perimeter and area are not divided by 2. The reason for this is that the curb opening will serve as a relief in event the grate becomes clogged. With the grate operating freely it is questionable whether much water will get to the curb opening until the discharge is sufficient to submerge the entire grate.

10. If the grate has an appreciable cross slope so the side away from the curb is higher than that next to the curb, the inflow over the side should be determined separately from that over the ends. Use the depth at the middle of the grate for end inflow and depth at edge away from the curb for side inflow.

---

(1) Airfield Drainage Structure Investigation (Final Report)
Hydraulic Laboratory Report No. 54
U.S. Corps of Engineers, St.Paul District Suboffice, April 1949

4.6.8 Capacity of Curb Opening Inlet at a Low Point in Grade

1. Figure **4.6.10.9 is used to solve inlet capacity problems under the following conditions:**

   (a) The curb opening inlet (no grate) is located at a low point in the grade.

   (b) All flow coming to the inlet must eventually enter the inlet and will pond until sufficient head is built up so the outflow through the inlet will equal the peak inflow from the gutters.

2. The definition of the symbols and the terminology used are as follows:

   Q = total flow to the inlet in cfs

   h = vertical height of inlet opening in feet at the face of curb

   L = length of opening in feet

   a = depth of local depression in feet

   W = width of local depression in feet

   P = perimeter of local depression in feet (= 3.14 W + L)

   S_x = normal gutter cross slope in vicinity of inlet (do not include local depression)

   H_1 = total head at face of inlet opening

   H_2 = total head at outer edge of local depression

   If a local depression is used ($a > 0$) the depth that determines the spread of water on the pavement may be at the outer edge of the local depression.

3. To determine the spread of water on the pavement proceed as follows:

(a) Enter nomograph with values of h and Q/L. Read H_1/h and compute H_1.

(b) If a = 0 then spread of water on the pavement $Sp = H_1/S_x$ (feet from curb).

(c) If $a \neq 0$, enter the nomograph with a convenient value of h, designated h (e.g., 0.4) and Q/P. Read H_2/h and compute H_2.

(d) Then if $H_1 > H_2 + WS_x + a$, it will submerge H_2 and the spread of water equals $(H_1 - a)/S_x$.

(e) If $H_1 < H_2 + WS_x + a$, the spread will be determined by H_2 and will equal $(H_2/S_x) + W$.

4. The spread in each gutter approaching the low point inlet should be checked in the same way as an inlet on a continuous grade since it may be greater than the spread caused by the inlet. Since the longitudinal gutter slope approaches zero at the inlet, it is suggested that a value of 0.001 ft/ft be used in checking the spread in the approach gutters.

1-22. CLOSED CONDUIT SYSTEM*

1. DESIGN FLOW:

The closed conduit system shall be designed for a 10-year rainfall frequency when its intended use is to function as the minor drainage system. Design flows will be determined by methods discussed in Sections 1-21., and pipes will be sized by the amount of run-off actually entering the system.

2. STORM SEWER PIPE:

Size of storm sewer pipe may be determined by the Manning Formula which is expressed as:

$$Q = va = \frac{1.49}{n} r^{2/3} S^{1/2} a$$

Q = Quantity of flow in cubic feet per second

v = Velocity of flow in feet per second

a = Required area in square feet

n = Coefficient of roughness

r = Hydraulic radius in feet

$$= \frac{\text{cross sectional area of flow}}{\text{wetted perimeter}}$$

S = Slope of energy gradient in feet per foot

Adjustments of pipe sizes as determined by the Manning Formula may be necessary due to hydraulic gradient considerations.

The Manning Formula is shown on nomograph form on page 44.

Other guidelines related to size and configuration of storm sewer pipe are as follows:

A. Minimum size of pipe to be used outside of the Virginia Department of Highways and Transportation right of way will be 12"

* County of Fairfax, Virginia, "Public Facilities Manual", Volume I and II, 1976.

diameter where the distance between access openings is 50 feet or less and 15" diameter where access openings exceed 50 feet. The minimum size of pipe permitted within the Virginia Department of Highways and Transportation is 15".

B. Pipes will be designed for flows intercepted by the inlets.

C. Pipes 18 inches in diameter and larger may be constructed on horizontal curves. Public Facilities Manual, Vol. II, provides for the geometric limitation and information to assist in design of concrete pipes on horizontal curves.

D. Except where noted differently under Item A, the maximum length between access openings shall not exceed 400 feet for pipes less than 36 inches in diameter or 800 feet for pipes 36 inches in diameter or greater. Access opening may be in the form of an inlet, manhole, junction box or other approved appurtenance.

E. Prefabricated tee and wye sections may be used under the conditions stated in the Tee and Wye Standard, Public Facilities Manual, Vol. II, when approved by the Director.

F. Prefabricated bend sections may be used under the conditions stated in the Bend Standard, Public Facilities Manual, Vol. II, when approved by the Director.

G. In general there may not be a reduction in pipe size along the direction of flow.

H. Minimum cover for storm sewer pipe shall be two feet from finish grade to the outside top of pipe, except where approved structural correction is provided when cover requirements cannot be met.

I. Minimum easement widths shall be determined as follows:

Pipe Size	Easement Width
15" - 18"	10'
21" - 33"	15'
36" - 48"	20'
54" - 72"	24'

Where multiple pipes are installed, edge of easement shall be five feet clear of outside of pipe. Where easements do not generally follow established lot lines, add five feet to the easement width on side toward the building. Storm sewers to be maintained by Fairfax County Department of Public Works shall be within dedicated "Storm Drainage Easements".

J. Storm sewers shall be designed to provide an average velocity when running full of not less than two and one-half feet per second.

K. The need for concrete anchors must be investigated on storm sewer lines with slopes of 20% or greater. If anchors are required, the design engineer will show a detail on the plans with spacing requirements.

L. Plain concrete pipe shall conform to the requirements of ASTM Designation C-14 Extra Strength; reinforced concrete pipe shall conform to ASTM Designation C-76 Classes II, III, and IV; Asbestos Cement pipe shall conform to ASTM C663-73a, Type II and AASHTO M-217-75.

3. PIPE MATERIALS:

Pipe materials acceptable for storm drain construction with the accompanying roughness coefficients are shown below:

Material	Manning "n"
Plain Concrete Culvert Pipe (PCCP)*	.013
Non-Reinforced Concrete Sewer Pipe (NRCSP)*	.013
Reinforced Concrete Culvert Pipe (RCCP)	.013
Reinforced Concrete Sewer Pipe (RCSP)	.013
(1) Vitrified Clay Pipe, Extra Strength (VCPX)	.013
(1) Cast Iron Pipe (CIP)	.013
Asbestos Cement Pipe (ACP)**	.013
(2) Corrugated Metal Pipe (CMP)	
(1) Plain	.024
(1) 25% Paved	.021
(1) 50% Paved	.018
(1) 100% Paved	.013

(1) Until specifications have been developed and approved by the State and County, use of this pipe material must be approved by the Director.

(2) Except for residential driveways, and temporary installations, this type of pipe is to be used only when approved by the Director. When using corrugated metal pipe, the designer should be aware that the coating will probably not last the life of the pipe. The velocity of flow through the pipe should therefore be computed using the smoothest pipe lining (lowest value of 'n') while the capacity of the pipe should be computed using an uncoated or plain pipe (n=0.024).

Note *: Plain Concrete Culvert Pipe (PCCP) and Non-Reinforced concrete sewer pipe (NRCSP) shall conform to the Virginia Department of Highways and Transportation Road and Bridge Specifications. Pipe sizes 12" through 24" are permitted.

4. ENERGY AND HYDRAULIC GRADIENTS

The Hydraulic Gradient for a storm sewer system is a line connecting points to which water will rise in manholes and inlets throughout the system during the design flow. The energy gradient is a line drawn a distance $\frac{V^2}{2g}$ above the hydraulic gradient of the pipes.

The total energy losses at a junction, H_L, is assumed to be made up of one or more of the following losses:

1. Expansion loss, h_i, when storm water enters the junction.
2. Contraction loss, h_o, when storm water leaves the junction.
3. Bend loss, h_Δ, due to the change in horizontal direction of storm water velocity.

These losses may be estimated as follows:

$$H_L = h_i + h_o + h_\Delta = 0.1 \frac{Vi^2}{2g} + 0.05 \frac{Vo^2}{2g} + K_\Delta \frac{Vi^2}{2g}$$

H_L = Total Energy Loss

h_i = Expansion Loss (flow <u>in</u> to junction)

h_o = Contraction Loss (flow <u>out</u> of junction)

h_Δ = Bend Loss

Vi = Velocity in feet per second, Q/A, of upstream pipe

V_o = Velocity in feet per second, Q/A, of downstream pipe

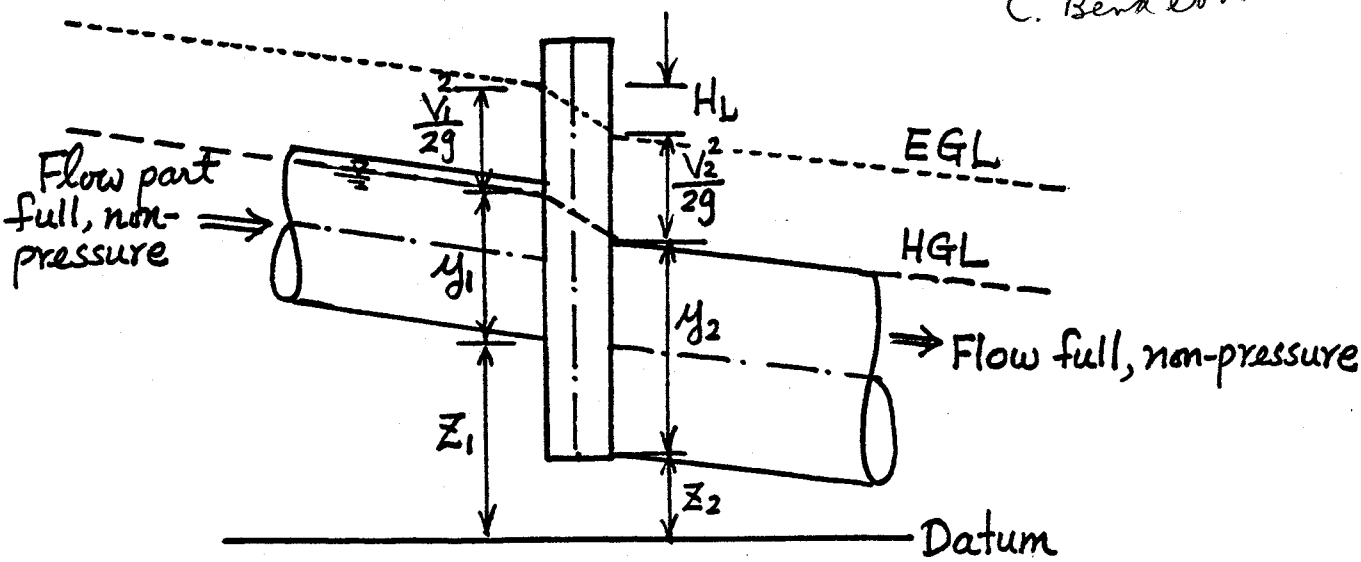
Δ = Horizontal angle in degrees between the direction of flow of incoming and outgoing pipes

K_Δ = Bend loss coefficient

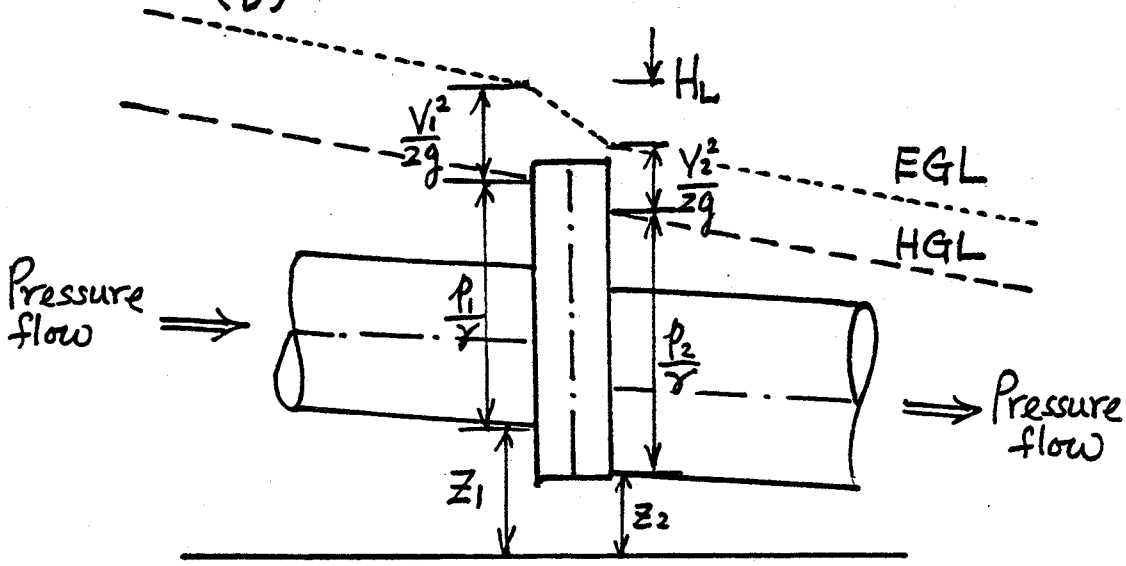
1. Select pipe size
2. Hyd. gradeline

Junction loss -
a. Expansion
b. Contraction
c. Bend loss

(a)



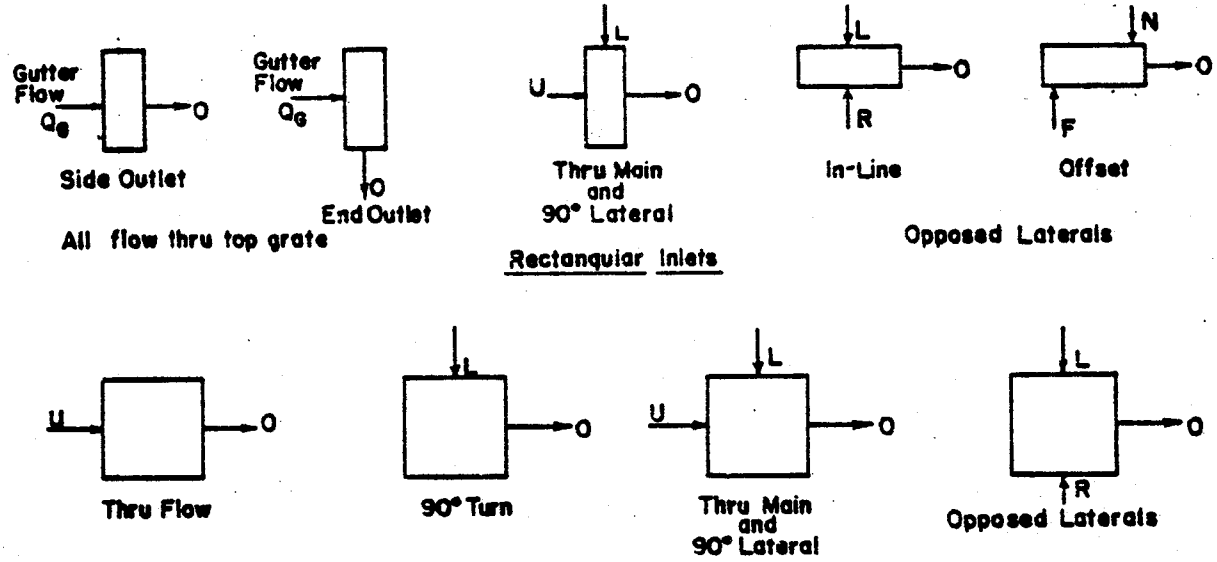
(b)

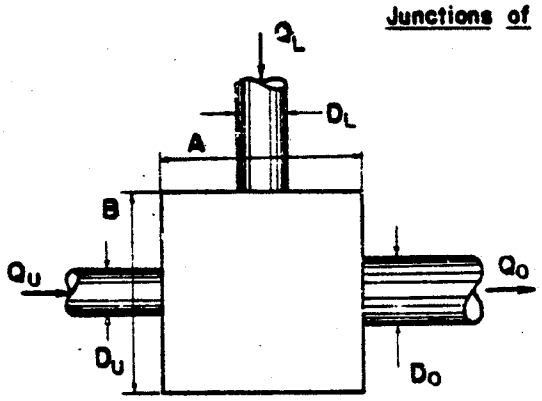


(a) $Z_1 + \frac{V_1^2}{2g} + y_1 = Z_2 + \frac{V_2^2}{2g} + y_2 + H_L$ (Non-pressure flow)

(b) $Z_1 + \frac{V_1^2}{2g} + \frac{P_1}{\gamma} = Z_2 + \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + H_L$ (Pressure flow)

H_L = energy losses = $H_{exit} + H_{entrance} + H_{bend}$





Junction Dimensions

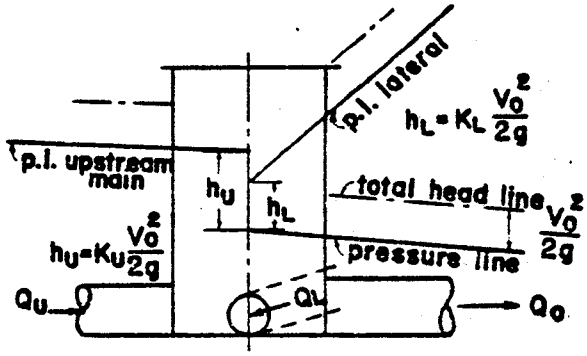


Diagram of Pressure Line Elevations at Junction of A Lateral with A Thru Main

Nomenclature

Q = rate of flow
D = pipe diam.

Q rate of flow
D diameter of pipe
A dimension of junction in direction of outfall pipe
B dimension of junction at right angles to outfall pipe
d depth of water in inlet
S slope of pipe
S_f friction slope
Q_G flow into inlet thru top grate
D_O Q_O dia. and flow in outfall
D_U Q_U dia. and flow in upstream main
D_L Q_L dia. and flow in left lateral
D_R Q_R dia. and flow in right lateral
D_N Q_N dia. and flow in near lateral } of opposed laterals; designation with reference
D_F Q_F dia. and flow in far lateral } to position relative to outfall end of junction.
$D_{h.v.}$ $Q_{h.v.}$ dia. and flow in lateral with higher-velocity flow } for in-line opposed laterals only
$D_{l.v.}$ $Q_{l.v.}$ dia. and flow in lateral with lower-velocity flow }

K_G water depth with all flow thru grate
K_U upstream main pressure
K_R or K_L lateral pipe pressure
K_N near lateral pipe pressure } offset opposed
K_F far lateral pipe pressure } laterals
$\overline{K_U}$, $\overline{K_L}$ pressure coefficient at $Q_L = Q_O$
M_U, M_L multipliers for $\overline{K_U}$ or $\overline{K_L}$ to obtain K_U or K_L

FIGURE 8-3. MANHOLE JUNCTION TYPES & NOMENCLATURE (15)

Source: Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual", Denver Regional Council of Governments, Denver, Colorado, 1969.

4.7 Hydraulic Grade Line*

The final step in the design of a storm sewer system is to check the Hydraulic Grade Line. Computing the H.G.L. will determine the elevation, under design conditions, to which the water will rise in the various inlets, manholes, junctions, etc.

4.7.1 The Hydraulic Grade Line will be computed for all storm sewer systems and may be tabulated on Form LD-347 using the following procedure:

Step 1 Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. H.G.L. computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Col. 2 the outlet water surface elevation if the outlet will be submerged during the design storm or .8 D + invert out elevation of the outflow pipe whichever is greater.

Step 3 Enter in Col. 3 the diameter (D_o) of the outflow pipe

Step 4 Enter in Col. 4 the design discharge (Q_o) for the outflow pipe.

Step 5 Enter in Col. 5 the length (L_o) of the outflow pipe.

Step 6 Enter in Col. 6 the friction slope (Sf_o) in Ft./Ft. of the outflow pipe. This can be determined by entering the pipe flow charts or similar pipe flow calculators with the discharge (Q_o), diameter (D_o), and "n" factor (0.013) and reading the friction slope directly.

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* Virginia Department of Highways and Transportation, "Drainage Manual", Richmond, Virginia, 1980.

Step 7 Multiply the friction slope (Sf_o) in Col. 6 by the length (L_o) in Col. 5 and enter the friction loss (Hf) in Col. 7.

Step 8 Enter in Col. 8 the velocity of flow (V_o) of the outflow pipe.

Step 9 Enter in Col. 9 the Contraction Loss (H_o) by using the formula $H_0 = 0.25(\frac{V_o^2}{2g})$, where $g = 32.2$ ft./sec.^{2}

Step. 10 Enter in Col 10 the design discharge (Q_i) for each pipe flowing into the junction. (For exception, See 4.7.2 No. 3)

Step 11 Enter in Col. 11 the velocity of flow (V_i) for each pipe flowing into the junction. (For exception, See 4.7.2 No. 3)

Step 12 Enter in Col. 12 the product of $Q_i V_i$ for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest $Q_i V_i$ product is the line which will produce the greatest Expansion Loss (H_i). (For exception See 4.7.2 No. 3)

Step 13 Enter in Col. 13 the controlling Expansion Loss (H_i) using the formula $H_i = 0.35\ (\frac{V_i^2}{2g})$.

Step 14 Enter in Col. 14 the angle of skew of each inflowing pipe to the outflow pipe.

Step 15 Enter in Col. 15 the greatest Bend Loss (H_Δ) calculated by using the formula $H_\Delta = K \frac{V_i^2}{2g}$, where K = the Bend Loss coefficient corresponding to the various angles of skew of the inflowing pipes.

Step 16 Enter in Col. 16 the Total Head Loss (H_t) by summing the values in Col. 9 (H_o), Col. 13 (H_i), and Col. 15 (H_Δ).

Step 17 If the junction incorporates surface inflow, i.e. drop inlet, increase H_t by 30% and enter the adjusted H_t in Col. 17.

Step 18 If the junction incorporates full diameter inlet shaping, reduce the value of H_t by 50% and enter that adjusted value in Col. 18.

Step 19 Enter in Col. 19 the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of H_t.

Step 20 Enter in Col. 20 the sum of the elevation in Col. 2 and the Final H in Col. 19. This elevation is the potential water surface elevation for the junction under design conditions.

Step 21 Enter in Col. 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration and compare it with the potential water surface elevation in Col. 20. If the potential water surface elevation exceeds the rim elevation or the gutter flow line, which-

ever is lowest, adjustments are needed in the system to reduce the elevation of the H.G.L.

Step 22 Repeat the procedure starting with Step 1 for the next junction upstream.

4.7.2 The normal design of a storm sewer system will, whenever possible, incorporate the following:

1. When a trunk line passes through a junction, the pipe crown lines will be at the same elevation. An exception is that when the inflow and outflow pipes are of the same diameter, the outflow pipe invert elevation will be 0.1' lower than the inflow pipe invert.
2. The angle of an inflow trunk line pipe to the outflow trunkline pipe at a junction will not exceed 90°.
3. If a lateral pipe line enters a junction with its invert elevation above the crownline elevation of the outflow trunkline pipe, the lateral discharge will be considered as drop inlet inflow in adjusting H_t for Step 17. (4.7.1)

Drop Inlet increase jet loss by 50%.

LD-347

HYDRAULIC GRADE LINE

PROJECT

INLET STATION	Outlet Water Surface Elev.	D_o	Q_o	L_o	S_{f_o}	H_f	JUNCTION LOSS												Final H	Inlet Water Surface Elev.	Rim Elev.
							V_o	H_b	Q_i	V_i	Q_iV_i	$\frac{V_i^2}{2g}$	H_i	Angle	H_Δ	H_t	1.3 H_t	0.5 H_t			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)		(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

$H_i = 0.35 \frac{V_i^2}{2g}$ $H_o = 0.25 \frac{V_o^2}{2g}$ $H_\Delta = K \frac{V_i^2}{2g}$

FINAL $H = H_f + H_t$

$H_t = H_o + H_i + H_\Delta$

SEE LD-72(D)67

90° K = 0.70	50° K = 0.47	20° K = 0.16
80° K = 0.66	40° K = 0.38	15° K = 0.10
70° K = 0.61	30° K = 0.28	
60° K = 0.55	25° K = 0.22	

Column	Comment
1	Line being investigated
2, 3	Inlet or manhole being investigated
4	Length of the line
5	Subarea for the inlet
6	Accumulated subareas
7	Value of the concentration time for the area draining into the inlet
8	Travel time in the pipe line
9	Weighted C for the area being drained
10	Rainfall intensity based on time of concentration and a 5-yr frequency curve
11	Unit runoff $q = CI$
12	Accumulated runoff that must be carried by line
13	Slope of line
14	Size of pipe
15	Pipe capacity
16	Velocity in full pipe
17	Actual velocity in pipe

Typical Storm Sewer Computations for the Rational Method for a System

	Manhole No.			Area (acres)		Flow Time (min)																				
Line	From	To	Length (ft)	Increment	Total	To Upper End	In Section	Average Runoff Coefficient	Rainfall (in./hr)	Runoff (cfs/acre)	Total Runoff (cfs)	Slope of Sewer (%)	Diameter (in.)	Capacity, Full	Velocity, Full	Design Flow: Velocity (fps)	Design Flow: Velocity Head (ft)	Design Flow: Depth of Flow (in.)	Design Flow: Total Energy (ft)	Manhole Losses (ft)	Manhole Invert Drop (ft)	Fall in Sewer (ft)	Sewer Invert: Upper End	Sewer Invert: Lower End	Ground Elevation: Upper End	Ground Elevation: Lower End
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
1	1–6	1–5	400	2.64	2.64	20.0	1.4	0.30	3.7	1.11	2.93	0.85	12	3.3	4.0	4.6		9			...	3.40	93.00	89.60	98.4	94.9
1	1–5	1–4	400	3.61	6.25	21.4	1.2	0.30	3.6	1.08	6.75	0.75	18	9.2	5.1	5.6		11			0.40	3.00	89.20	86.20	94.9	91.8
1	1–4	1–3	400	3.88	10.13	22.6	1.2	0.42	3.4	1.43	14.50	0.45	24	15.2	4.8	5.6		18			0.40	1.80	85.80	84.00	91.8	89.7
3	3–2	3–1	400	5.55	5.55	20	1.1	0.30	3.7	1.11	6.16	1.00	15	6.4	5.1	5.9		12			...	4.00	91.00	87.00	96.2	92.3
3	3–1	1–3	400	6.43	11.98	21.1	1.1	0.30	3.6	1.08	12.92	0.60	24	17.5	5.5	6.1		15			0.60	2.40	86.40	84.00	92.3	89.7
1	1–3	1–2	400	3.92	26.03	23.8	1.1	0.39	3.3	1.29	33.60	0.30	36	37.0	5.1	5.9		26			0.80	1.20	83.20	82.00	89.7	89.5
2	2–1	1–2	400	2.52	2.52	20	1.4	0.30	3.7	1.11	2.80	0.90	12	3.2	4.1	4.7		9			...	3.60	87.50	83.90	92.7	89.5
1	1–2	1–1	400	3.86	32.41	24.9	1.1	0.41	3.2	1.31	42.50	0.24	42	50.0	5.2	5.9		29			0.40	0.96	81.60	80.64	89.5	88.5
1	1–1	Outfall	125	5.44	37.85	26.0	...	0.44	3.2	1.41	53.20	0.30	42	56.0	5.7	6.6		33			0.10	0.38	80.54	80.16	88.5	...

Source: Design and Construction of Sanitary Storm Sewers, ASCE Manuals and Reports on Engineering Practice, No. 37, 1970.

Storm Sewer System

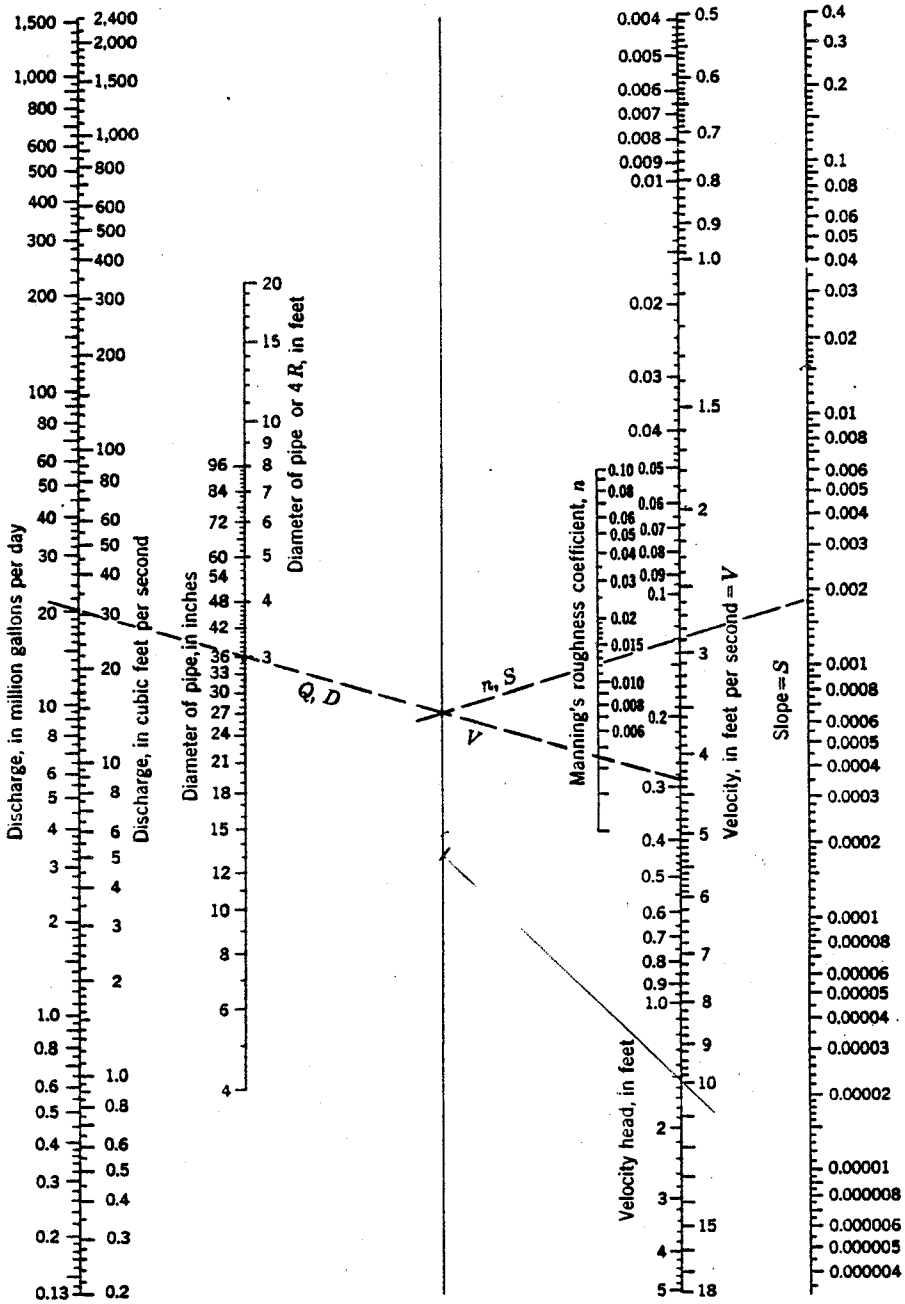
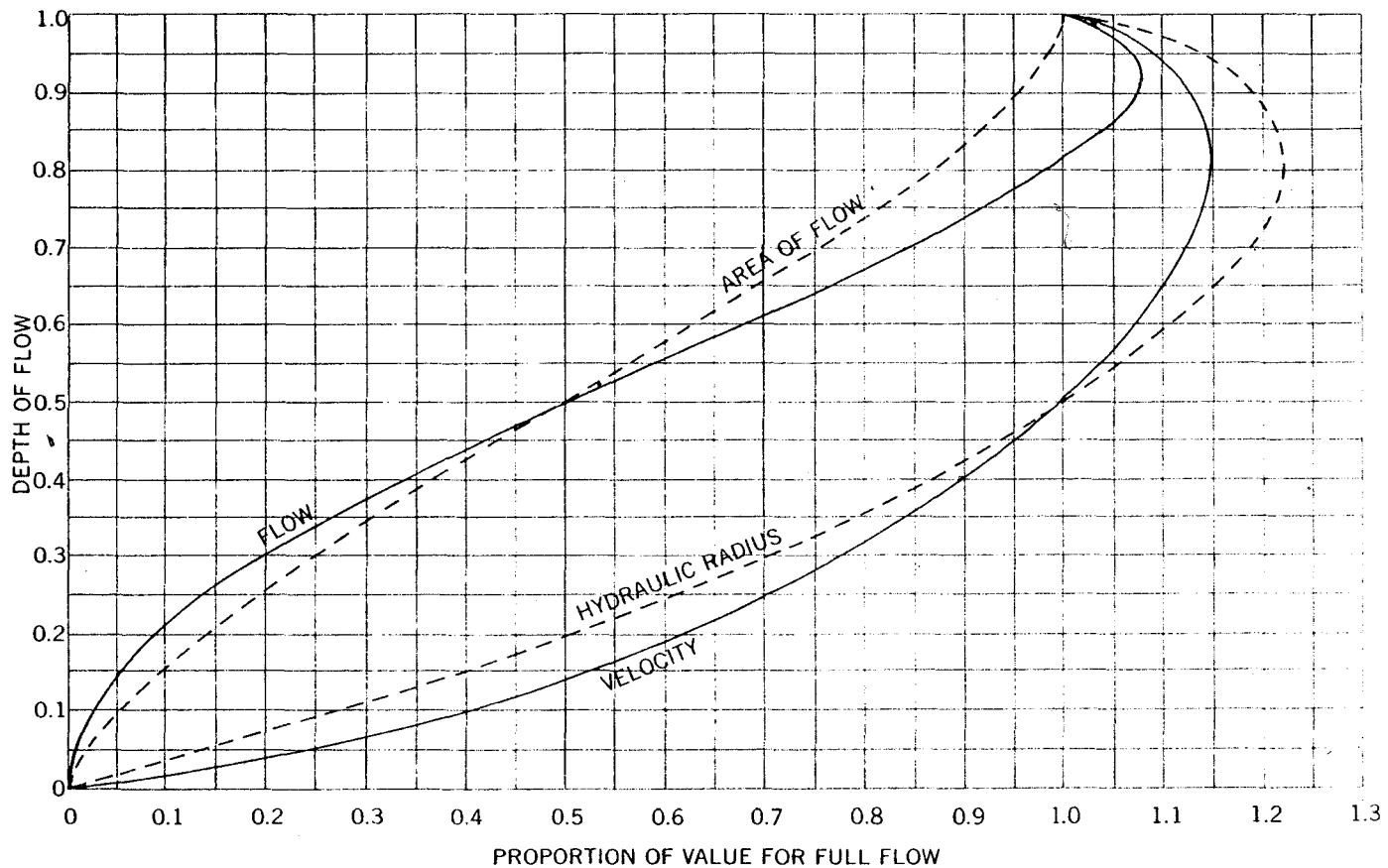


FIGURE 22.—Alignment chart for Manning formula for pipe flow.

Flowing full

Source: American Society of Civil Engineers, Design and Construction of Sanitary and Storm Sewers, ASCE Manuals and Reports in Engineering Practice - No. 37, 1976.

RELATIVE VELOCITY AND FLOW IN CIRCULAR PIPE FOR ANY DEPTH OF FLOW



$\frac{Q_P}{Q_F}$ $\frac{V_P}{V_F}$

$\frac{d}{D}$

American Concrete Pipe Association, Concrete Pipe Design Manual, Arlington, VA, 1974.

[3] Open Channel Flow

PART III

OPEN CHANNEL FLOW*

This section deals with the design of new stormwater conveyance channels in accordance with Std. & Spec. 1.35, and the determination of an adequate channel as required by General Criteria GC-7. It is assumed that the reader has some basic knowledge of hydraulic engineering principles and terms.

Calculation of Flow in Open Channels

To calculate the flow in an open channel, it must be assumed that the quantity of flow in the channel does not change with time and that the cross-sectional area and slope of the channel remain constant. While in reality these conditions are seldom met, the channel can be divided into reaches which have similar cross-sections and slope, and the flow can be considered at one point in time, such as the peak flow, when the quantity of flow will be more or less constant.

Two equations are used to calculate the flow in open channels. They are:

Manning's Equation,

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

where,

V = the average velocity in the channel, (ft./sec.)
n = Manning's roughness coefficient, based upon the lining of the channel
R = the hydraulic radius (feet)

S = the slope of the channel, (Feet/foot)

and The Continuity Equation,

$$Q = AV$$

where,

Q = flow in the channel, ($ft.^3$/sec.)
A = cross-sectional area of the channel, ($ft.^2$)
V = average velocity in the channel, (ft./sec.)

Manning's Equation and the Continuity Equation are used simultaneously to determine flow capacity and velocity in the channel. A nomograph for solving Manning's Equation is given on plate 5-32.

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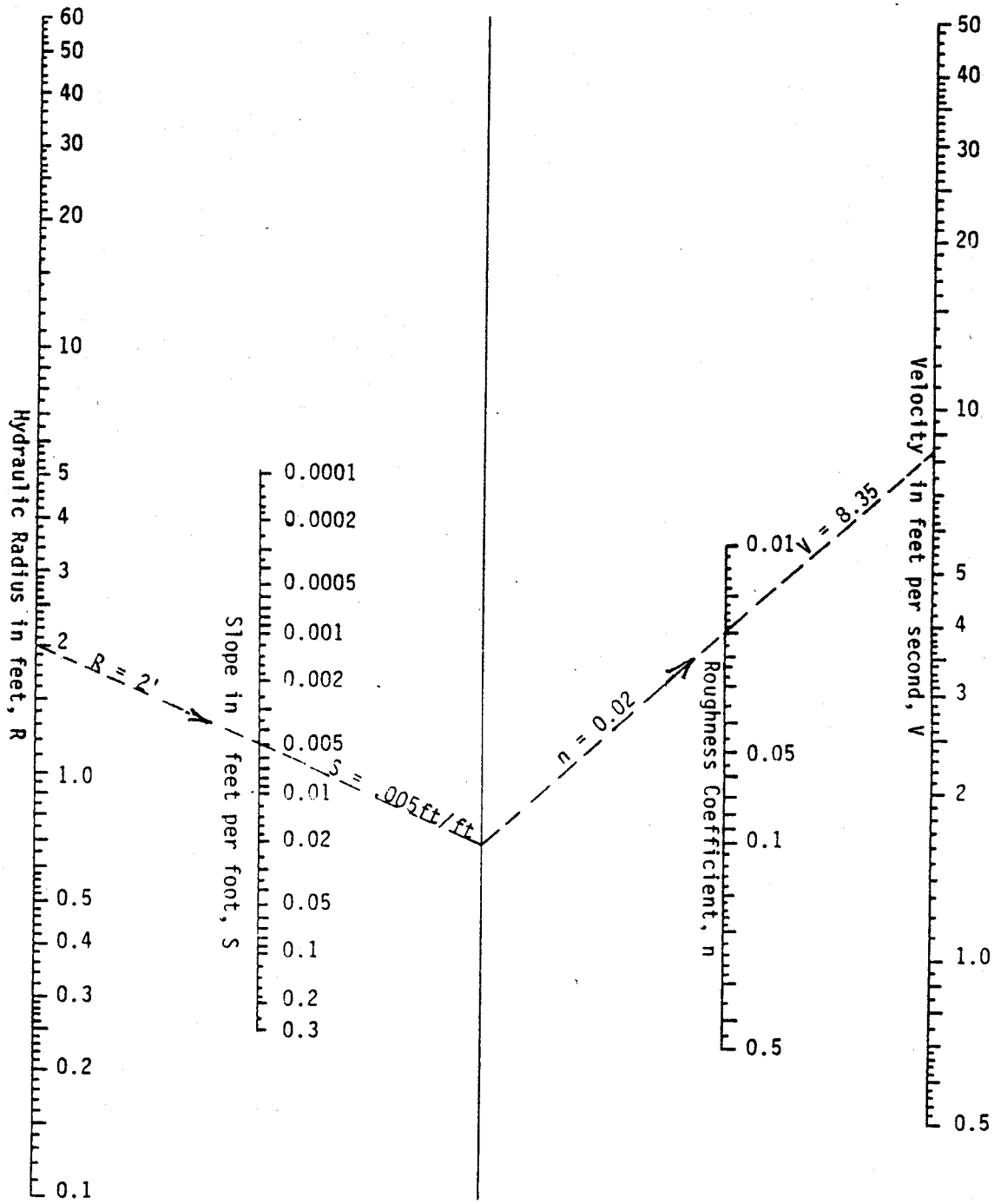
* Virginia Soil and Water Conservation Commission," Virginia Erosion and Sediment Control Handbook", Richmond, Virginia, 1980.

NOMOGRAPH FOR SOLUTION OF MANNING EQUATION



Source: VDH&T

Plate 5-32

Design of Stormwater Conveyance Channels

The design criteria for STORMWATER CONVEYANCE CHANNELS (Std. & Spec. 1.35) requires that two primary conditions be satisfied. First, the channel must have sufficient capacity to pass the peak flow expected from a 2-year frequency storm or greater (depending upon other local or State drainage criteria), and second, the channel lining must be resistant to erosion for the velocity of flow expected in the channel from the 2-year storm.

Both the capacity of the channel and the velocity of flow are functions of the channel lining, its cross-sectional area and its slope. The task of the designer is to determine a channel section and lining which will have sufficient capacity and be stable for a given slope.

The design procedure, which follows, is for determining a cross-section and lining for straight, uniform channel reaches on a constant slope. The procedure does not account for channel constrictions (i.e. culverts), changes in cross-section or slope, or curved alignments. Other engineering procedures are available to take such factors into account.

Selecting a Channel Lining

For design purposes, erosion resistance is a function of flow velocity in the channel. There are a number of possible linings to choose from. Commonly used channel linings include grass, riprap and concrete.

Concrete and similar structural linings generally do not erode and their design is not restricted by maximum permissible flow velocities. However, riprap and grass-lined channels do have maximum permissible velocities beyond which they will erode.

Table 5-7 lists maximum permissible velocities for various grass linings. Permissible velocities for riprap linings is dependant upon the stone size and can be determined by procedures outlined in other references.

Determining Manning's "n"

Manning's Roughness Coefficient, "n", is determined by the type of channel lining.

Ranges of "n" factors for various structural linings are listed in Table 5-8. Generally the lower values should be used to calculate velocity and the higher values should be used to calculate capacity of the channel.

For riprap lined channels "n" can be determined from the following equation:

$$n = 0.0395\ D_{50}^{\ 1/6}$$

Critical Shear Stress = τ_c → $\tau_o = \gamma R S_o$ (slope)
(gamma)
$\cong \gamma (y) S_o$ (wide channel)
y_{max} = max depth

Table 5-7

PERMISSIBLE VELOCITIES FOR GRASS-LINED CHANNELS

Channel Slope	Lining	Permissible Velocity
0-5%	Bermudagrass	6 ft/sec
	Reed canarygrass Tall fescue Kentucky bluegrass	5 ft/sec
	Grass-legume mixture	4 ft/sec
	Red fescue Redtop Sericea lespedeza Annual lespedeza Small grains (temporary)	2.5 ft/sec
5-10%	Bermudagrass	5 ft/sec
	Reed canarygrass Tall fescue Kentucky bluegrass	4 ft/sec
	Grass-legume mixture	3 ft/sec
Greater than 10%	Bermudagrass	4 ft/sec
	Reed canarygrass Tall fescue Kentucky bluegrass	3 ft/sec

Source: USDA Soil Conservation Service

where,

n = Manning's roughness coefficient
D_{50} = the median size stone in the mixture of riprap, (feet)

For grass-lined channels, "n" can be determined by the following procedures

1. Determine the maximum permissible velocity (V) for the grass to be used. (See Table 5-7.)

2. Calculate the hydraulic radius (R) of the channel (cross-sectional area divided by the wetted perimeter - Plate 5-31).

3. From Table 5-9, determine the retardance class of the grass to be used. When calculating channel capacity, the highest retardance class of the grass should be used (i.e. long condition). When calculating velocity, the lowest retardance class should be used (i.e. mowed condition).

4. Enter Plate 5-30 with the product of V times R. Move vertically until the correct retardance curve is intersected. Read "n" on the left axis.

Selecting a Channel Cross-Section

The three most commonly used channel cross-sections are vee, parabolic, and trapezoidal shapes. Plate 5-31 gives mathematical formulas for determining the area, wetted perimeter, hydraulic radius and top width of these shapes. St. & Spec. 1.35 contains guidelines for selecting an appropriate shape based upon size, intended use, and lining of the channel.

Initial estimates of the cross-sectional area of the channel can be made using the Continuity Equation as follows:

$$A = Q/V$$

where,

A = cross-sectional channel area, (feet)
Q = required flow capacity, (ft^3/sec.)
V = permissible or desired flow velocity, (ft./sec.)

Channel Slope

The slope of the channel is generally fixed by the topography and proposed route of the channel. There is little a designer can do to alter this factor.

Table 5-8

MANNINGS "n" FOR STRUCTURAL CHANNEL LININGS

Channel Lining	n Values Min.	Design	Max.
Asphaltic concrete, machine placed		0.014	
Asphalt, exposed prefabricated		0.015	
Concrete	0.012	0.015	0.018
Concrete, rubble	0.017		0.030
Metal, corrugated	0.021	0.024	0.026
Plastic	0.012		0.014
Shotcrete	0.016		0.017

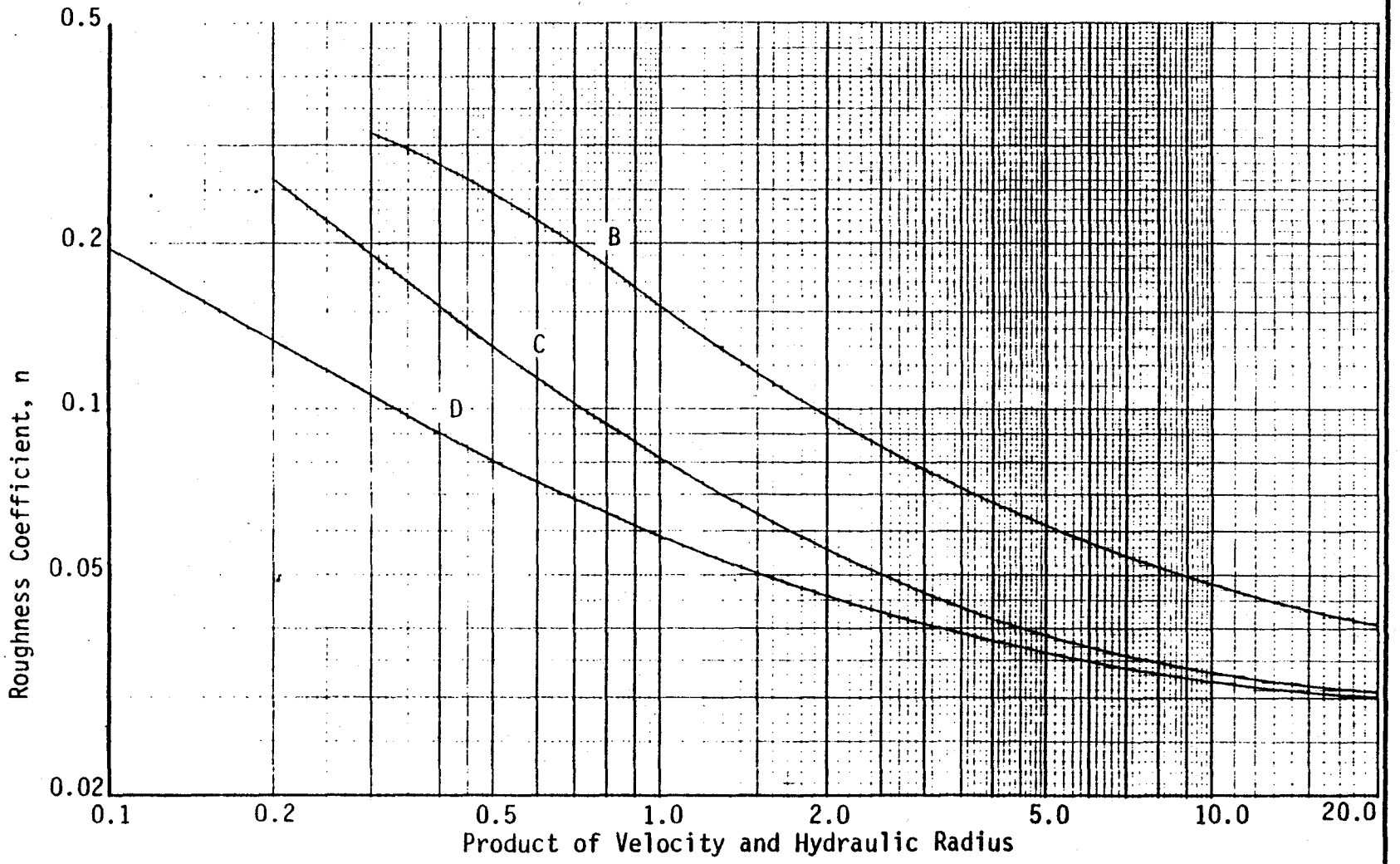
Source: American Society of Civil Engineers

Table 5-9

RETARDANCE CLASSIFICATIONS

Retardance		Stand	Condition
B	Tall Fescue	Good	Unmowed - 18 inches
	Sericea Lespedeza	Good	Unmowed - 19 inches
	Grass-Legume Mixture	Good	Unmowed - 20 inches
	Small grains, mature	Good	Uncut - 19 inches
	Bermudagrass	Good	Tall - 12 inches
	Reed Canarygrass	Good	Mowed - 14 inches
C	Bermudagrass	Good	Mowed - 6 inches
	Redtop	Good	Headed - 18 inches
	Grass-Legume Mixture, summer	Good	Unmowed - 7 inches
	Kentucky Bluegrass	Good	Headed - 9 inches
	Small grains, mature	Poor	Uncut - 19 inches
	Tall Fescue	Good	Mowed - 6 inches
D	Bermudagrass	Good	Mowed - 25 inches
	Red Fescue	Good	Headed - 15 inches
	Grass-Legume Mixture, spring and fall	Good	Unmowed - 5 inches
	Sericea Lespedeza	Good	Mowed - 2 inches

Source: USDA-Soil Conservation Service



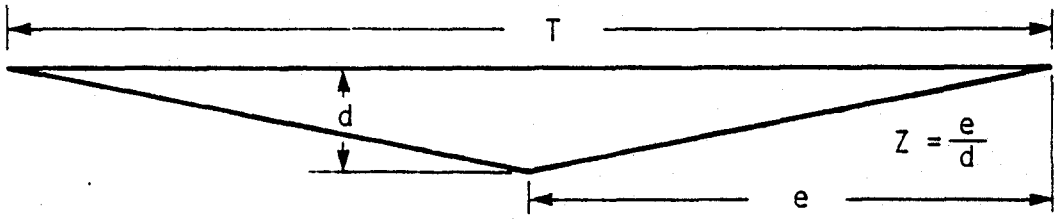
ROUGHNESS COEFFICIENT AS A FUNCTION OF v x R FOR VARIOUS RETARDANCE CLASSES

Source: USDA-Soil Conservation Service

Plate 5-30

CHANNEL GEOMETRY

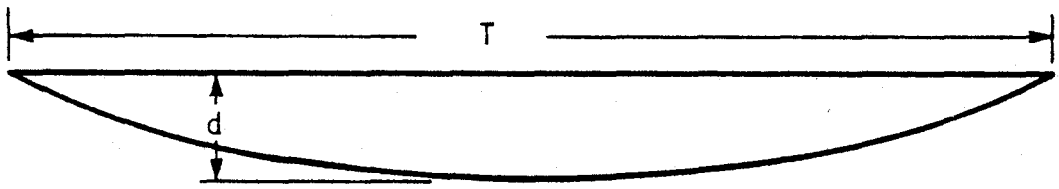
V - Shape



Cross-Sectional Area (A) = Zd^2
Top Width (T) = $2dZ$

Hydraulic Radius (R) = $\dfrac{Zd}{2\sqrt{Z^2 + 1}}$

Parabolic Shape

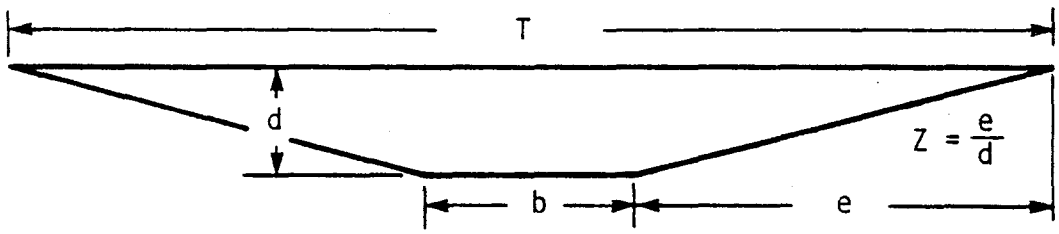


Cross-Sectional Area (A) = $\frac{2}{3}$ Td

Top Width (T) = $\dfrac{1.5\ A}{d}$

Hydraulic Radius = $\dfrac{T^2 d}{1.5T^2 + 4d^2}$

Trapezoidal Shape



Cross-Sectional Area (A) = $bd + Zd^2$
Top Width (T) = $b + 2dZ$

Hydraulic Radius = $\dfrac{bd + 2d^2}{b + 2d\sqrt{V^2 + 1}}$

Source: USDA-SCS

Design Procedure

Following is a step-by-step procedure for designing a stormwater conveyance channel using the Manning and Continuity equations:

Step 1 - Determine the required flow capacity (Q). (Use methods in Part I of this chapter to calculate runoff for the required design storm.)

Step 2 - Select a channel cross-section shape and lining, and determine the slope.

Step 3 - Determine the permissible velocity (if grass lined), or the desired velocity (if structurally lined).

Step 4 - Determine an initial channel size (i.e. area, top width, depth). (See Plate 5-31) pp. 53

Step 5 - Calculate the hydraulic radius (R). (See Plate 5-31) pp. 53

Step 6 - Determine the roughness coefficient "n".

Step 7 - Calculate the channel velocity (V) using the Manning Equation (or Plate 5-32) and calculate the channel capacity (Q) using the Continuity equation. (Plate on pp. 47)

Check design velocity against permissible velocity and design capacity to determine if design is acceptable.

Step 8 - If design is not acceptable, return to step 4 (try new size), or step 2 (try new cross-section or lining).

Example 5-6

Design a parabolic channel to be lined with bermudagrass, which can convey 75 cfs on a 2% slope.

Solution

Step 1 - Q = 75 cfs (given)

Step 2 - A parabolic channel with bermudagrass lining on a 2% slope are design conditions.

Step 3 - The permissible velocity is 6 ft./sec. (Table 5-7).

Step 4 - An initial estimate of required cross-sectional area is determined from the required capacity and permissible velocity using the Continuity Equation.

$$A = \frac{Q}{V} = \frac{75\ ft^3/sec}{6\ ft/sec} = 12.5\ ft^3$$

Try 16 ft^3*

*An area larger than 12.5 ft^2 is assumed initially since the capacity and velocity will be calculated with different roughness coefficients.

Assume an initial top width of 12 feet. The depth can be determined from equations on Plate 5-31, pp. 53

$$d = \frac{3}{2}\frac{A}{T} = \frac{3}{2}\left(\frac{16}{12}\right) = 2ft.$$

Step 5 - Hydraulic Radius $R = \frac{T^2 d}{1.5\,T^2 + 4d^2}$ (from Plate 5-31)

$$R = \frac{(12)^2\ (2)}{1.5(12)^2+4d^2} = 1.24\ ft.$$

Step 6 - Use Table 5-9 and Plate 5-30 to determine n. Assume a retardance class B for calculating capacity and a retardance class C for checking velocity.

$V \times R = 6 \times 1.24 = 7.44$

n = .053 (for retardance class B)
n = .035 (for retardance class C)

Step 7 - Check capacity and velocity

$$Q = \frac{1.49}{.053}\ (1.24)^{2/3}\ (.02)^{1/2}\ (16)$$

Q = 73.01 cfs (slightly less than required 75 cfs)

$$V = \frac{1.49}{.035}\ (1.24)^{2/3}\ (.02)^{1/2}$$

V = 6.9 ft/sec (more than permissible 6 ft/sec.)

Step 8 - Return to step 4 and try larger, wider cross-section.

Step 4 - Try new parabolic cross-section with 20 ft^2 area and 20 ft. top width.

$$d = \frac{3}{2}\left(\frac{A}{T}\right) = \frac{3}{2}\left(\frac{20}{20}\right) = 1.5\ ft$$

Step 5 - $R = \frac{(20)^2 (1.5)}{1.5(20)^2 + 4(1.5)^2} = .99$

Step 6 - $V \times R = 6 \times .99 = 5.94$

$n = .055$ (for retardance class B)
$n = .037$ (for retardance class C)

Step 7 - check Q and V

$$Q = \frac{1.49}{.055} (.99)^{2/3}(.02)^{1/2}(20)$$

$Q = 75.85$ ft^3/sec (OK)

$$V = \frac{1.49}{.037} (.99)^{2/3}(.02)^{1/2}$$

$V = 5.64$ ft/sec (OK)

<u>Final Channel Design</u>: Parabolic channel, lined with bermudagrass; 20 ft. top width; 1.5 ft. depth.

[4] Design of Culverts

Given: 1. Q of a given frequency
2. approximate L
3. approximate slope, S_o
4. allowable head water $(H.W.)_a$
5. mean and max. stream velocity

Solution:

1. Select a size, shape, material, entrance type for a culvert as a first trial
2. Assume inlet control:
   given Q, D, entrance type, from chart $\frac{H.W.}{D}$ is found.
   Revise D until $H.W. < (H.W)_a$ is satisfied.
3. Assume outlet control:
   Compute or estimate T.W. by means of either Manning's equation for uniform flow or water surface profile computation.
   If T.W. $\geq$ top of the culvert at outlet, the following equation holds.

   $$H.W. = H + h_o - LS_o$$

   where
   h_o = T.W.
   H is found from chart with given data L, Ke, D, Q, n

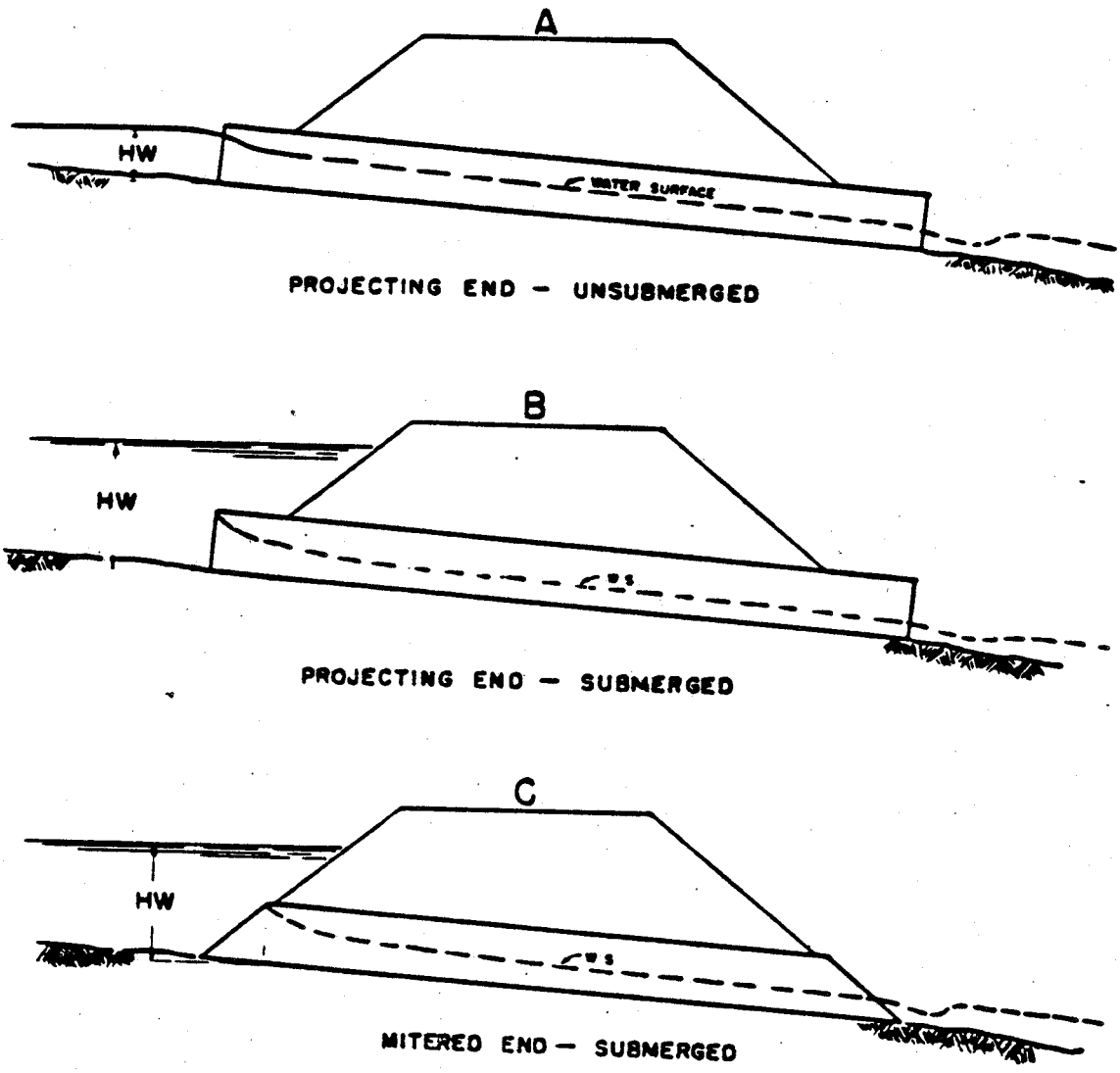
   If T.W. < top of the culvert, h_o is expressed as

   $$h_o = \frac{d_c + D}{2} \text{ or T.W. whichever is the greater.}$$

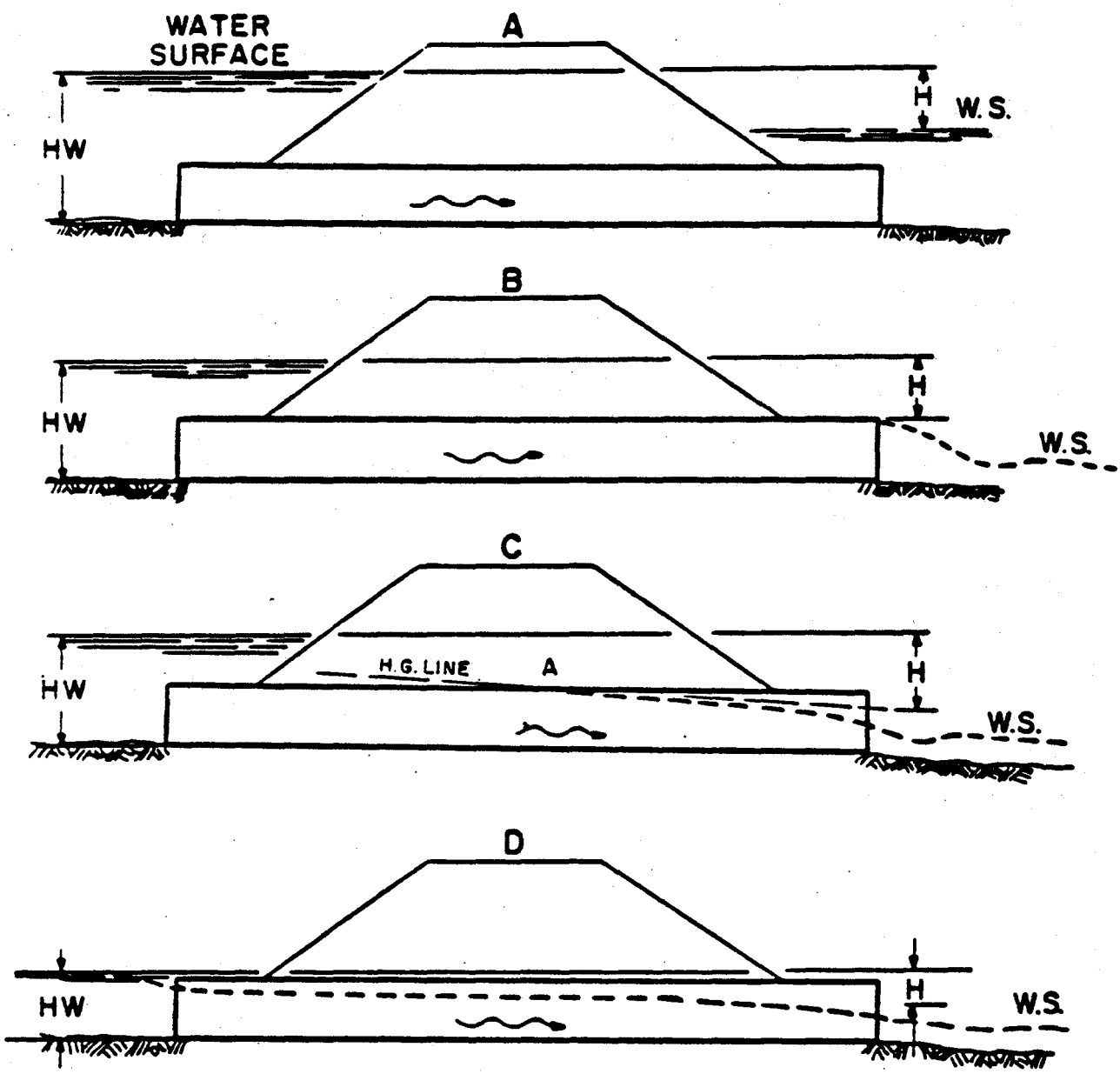
   where
   d_c = critical depth
4. Compare H.W. from the results of step 3 and step 4. Bigger value of H.W. controls.
5. Revise D if necessary.
6. Example

---

* Herr, L. A. and Bossy, H. G., "Hydraulic Charts for the Selection of Highway Culverts", Federal Highway Administration, U.S. Dept. of Transportation, Washington, D.C., Hydraulic Engineering Circular No. 5, 1965, reprinted 1980.

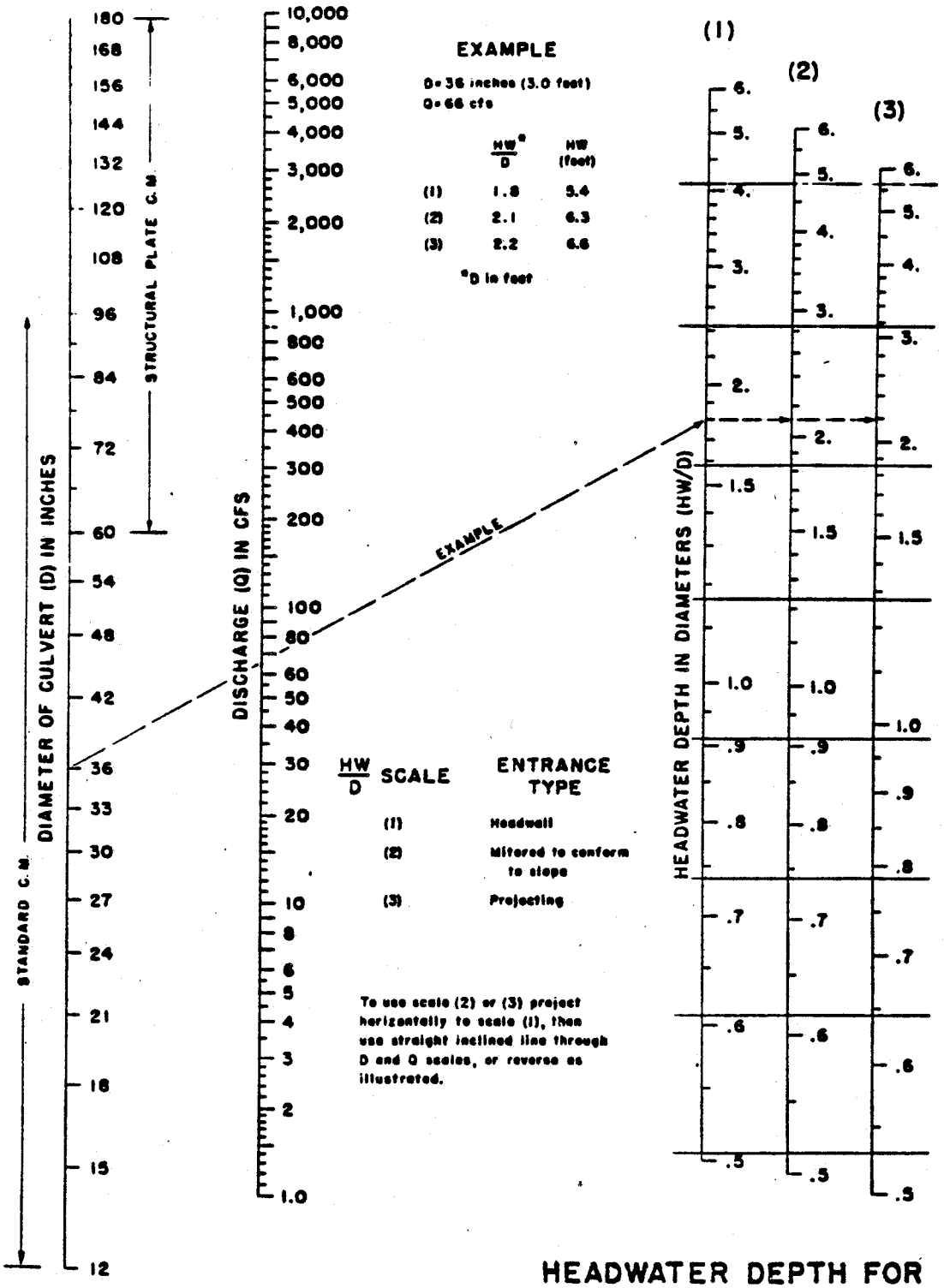


INLET CONTROL



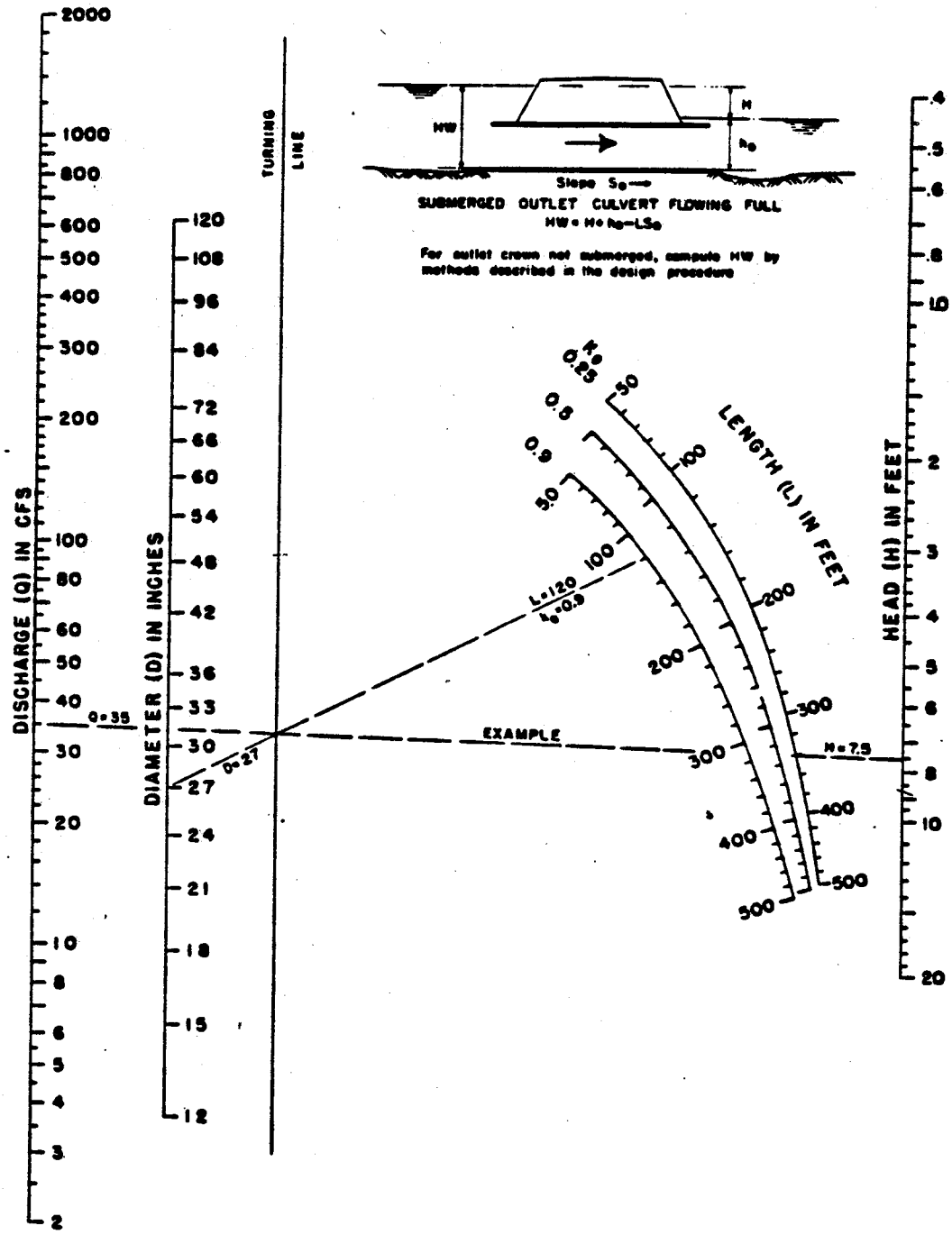
OUTLET CONTROL

$\frac{HW}{D} = 2.3 \times 4$



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963



HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

BUREAU OF PUBLIC ROADS JAN. 1963

PROJECT: I-46 (1)

DESIGNER: J.A.F.

DATE: 2-18-64

HYDROLOGIC AND CHANNEL INFORMATION

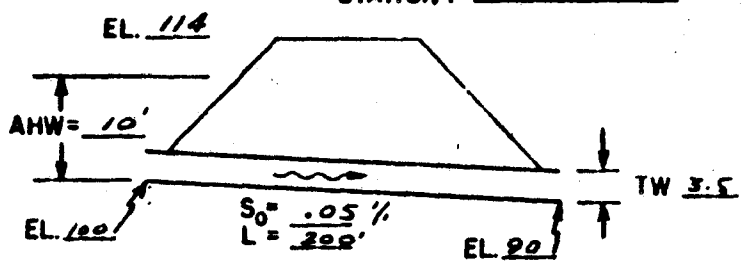
Q_1 = 180 cfs. = Q_{25} TW_1 = 3.5

Q_2 = 225 cfs. = Q_{50} TW_2 = 4.0

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 6+21



MEAN STREAM VELOCITY = 10'/sec
MAX. STREAM VELOCITY = 14'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW=H+h_0-LS_0$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_0	LS_0	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CIRCULAR CMP PROJ. ENT.	180		ASSUME 1.5	7.5	D=	60"	or	Try	Smaller	Size		HW	=10'			
"	180	54"	2.2	9.9	.9	9.7	3.9	4.2	3.5	4.2	10.0	3.9	9.9	16.5		
"	225	54"	3.15	14.2	.9	15.3	4.2	4.4	4.0	4.4	10.0	9.7	14.2	17.0		HW high for Q_{50} - Try 60"
"	180	60"	1.51	7.55	.9	5.9	3.9	4.4	3.5	4.4	10.0	0.3	7.55	16.7		
"	225	60"	2.1	10.5	.9	9.3	4.2	4.6	4.0	4.6	10.0	3.9	10.5	17.5		

SUMMARY & RECOMMENDATIONS: VELOCITIES READ FROM CHART 46, 47 - "DESIGN CHARTS FOR OPEN CHANNEL FLOW". (SEE P. 5-14). OUTLET VELOCITIES ARE ABOUT THE SAME FOR EACH SIZE, INDICATING CHANGE IN SIZE HAS LITTLE EFFECT. SIZE SELECTED (60 OR 54-INCH) DEPENDS ON DESIGNER'S CONFIDENCE IN FLOOD ESTIMATE AND DAMAGE INCURRED IF A LARGER FLOOD SHOULD OCCUR. NOTE THAT TW MUST BE GREATER THAN 10.' FOR OUTLET CONTROL TO GOVERN FOR THE 54" PIPE FLOWING 180 CFS. ACCURATE DETERMINATION OF TW DEPTHS IS UNNECESSARY IN MOST CASES.

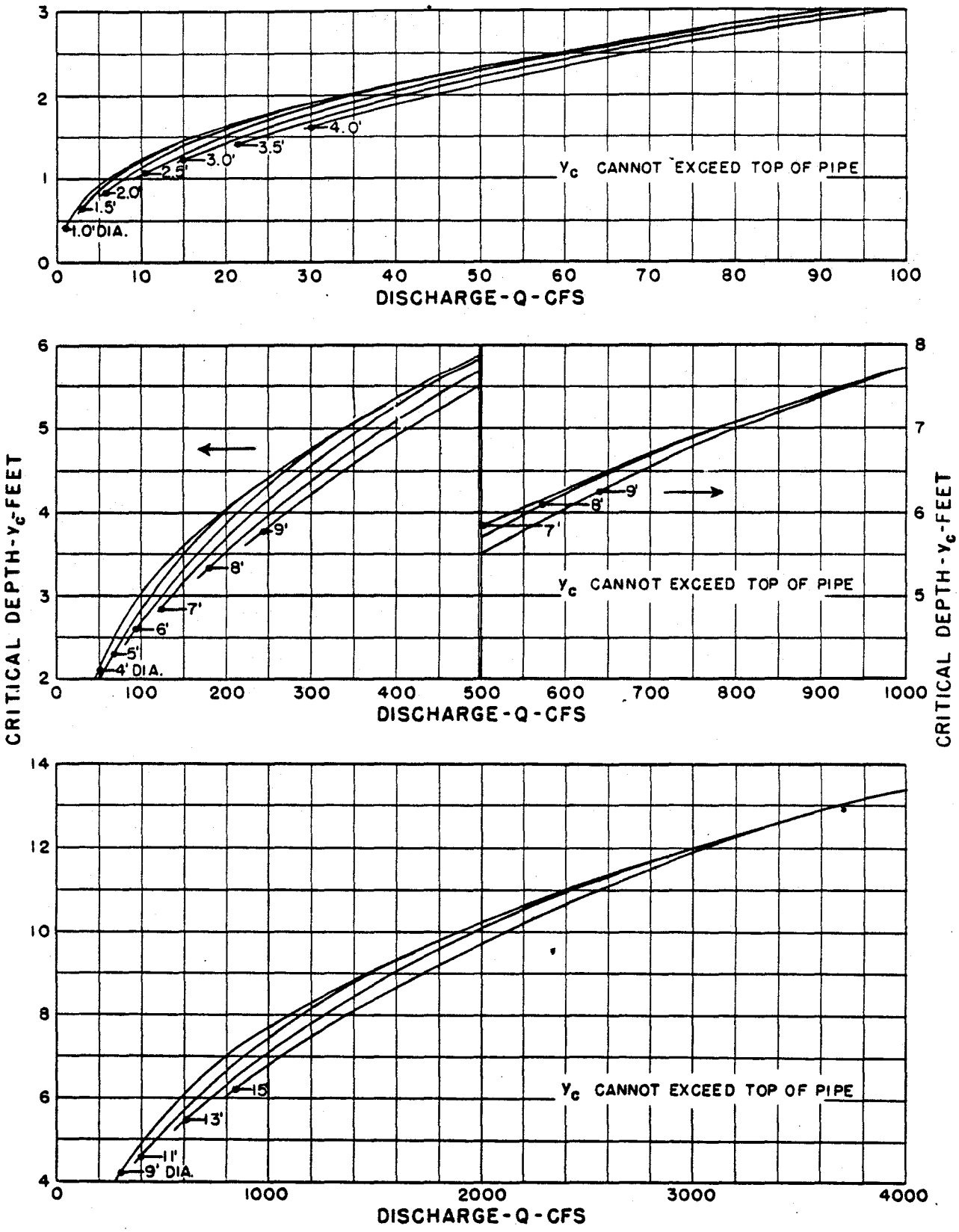


FIGURE III-4

CRITICAL DEPTH
CIRCULAR PIPE

TABLE 1 - ENTRANCE LOSS COEFFICIENTS

Outlet Control, Full or Partly Full

Entrance head loss $H_e = k_e \frac{V^2}{2g}$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end) . . .	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge . .	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides . . .	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet, p. 5-13.

[5] Culvert Inlet Improvement

Bevel-Edged Inlets

The first degree of inlet improvement is a beveled edge. The bevel is proportioned based on the culvert barrel or face dimension and operates by decreasing the flow contraction at the inlet. A bevel is similar to a chamfer except that a chamfer is smaller and is generally used to prevent damage to sharp concrete edges during construction.

Adding bevels to a conventional culvert design with a square-edged inlet increases culvert capacity by 5 to 20 percent. The higher increase results from comparing a bevel-edged inlet with a square-edged inlet at high headwaters. The lower increase is the result of comparing inlets with bevels with structures having wingwalls of 30 to 45 degrees.

Although the bevels used herein are plane surfaces, rounded edges which approximate the bevels are also acceptable.

As a minimum, bevels should be used on all culverts which operate in inlet control, both conventional and improved inlet types. The exception to this is circular concrete pipes where the socket end performs much the same as a beveled edge. Examples of bevels used in conjunction with other improved inlets are shown in Figures 5 and 6. Culverts flowing in outlet control cannot be improved as much as those in inlet control, but the entrance loss coefficient, k_e, is reduced from 0.5 for a square edge to 0.2 for beveled edges. Therefore, it is recommended that bevels be used on all culvert entrances if little additional cost is involved.

Side-Tapered Inlets

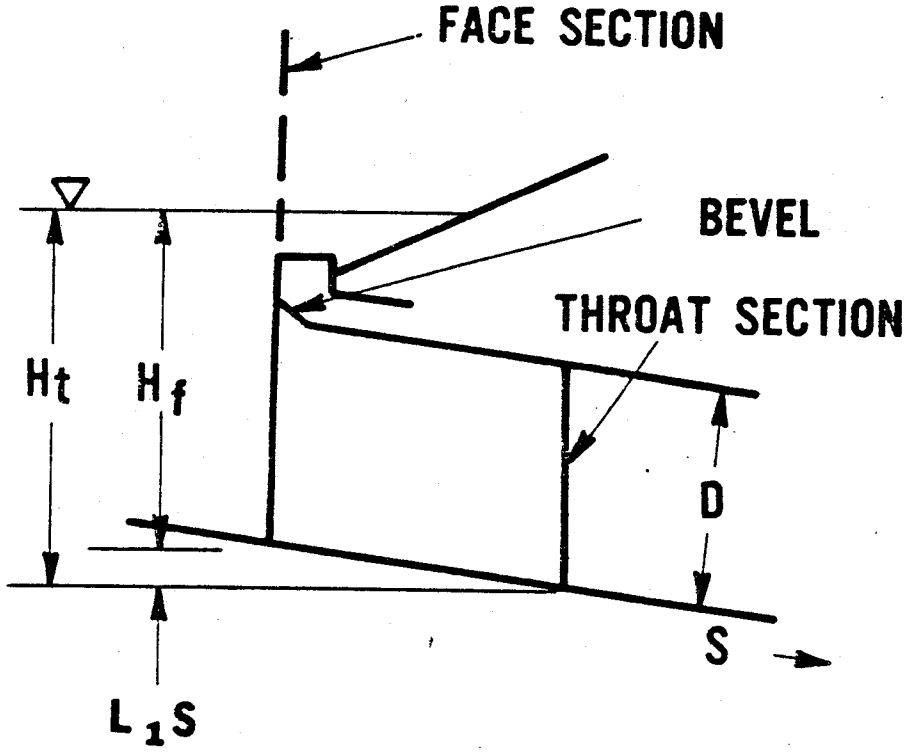
The second degree of improvement is a side-tapered inlet (Figure 5). It provides an increase in flow capacity of 25 to 40 percent over that of a conventional culvert with a square-edged inlet. This inlet has an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. The intersection of the sidewall tapers and barrel is defined as the throat section.

Side-tapered inlets of other configurations were tested, some with tops tapered upward but with sidewalls remaining an extension of the barrel walls, and others with various combinations of side and top tapers. Each showed some improvement over conventional culverts, but the geometry shown in Figure 5 produced superior performance.

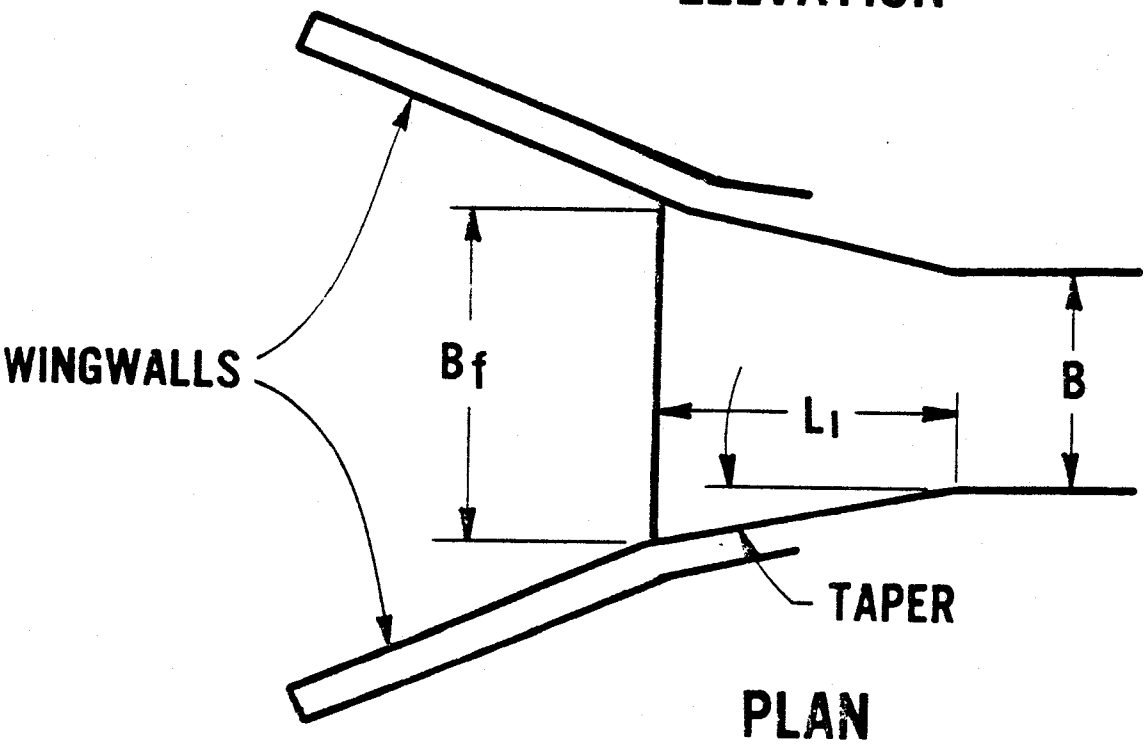
Harrison, L. J., et. al., "Hydraulic Design of Improved Inlets for Culverts", Federal Highway Administration, U.S. Dept. of Transportation, Washington, D.C., Hydraulic Engineering Circular, No. 13, 1972.

Figure 5

SIDE - TAPERED INLET



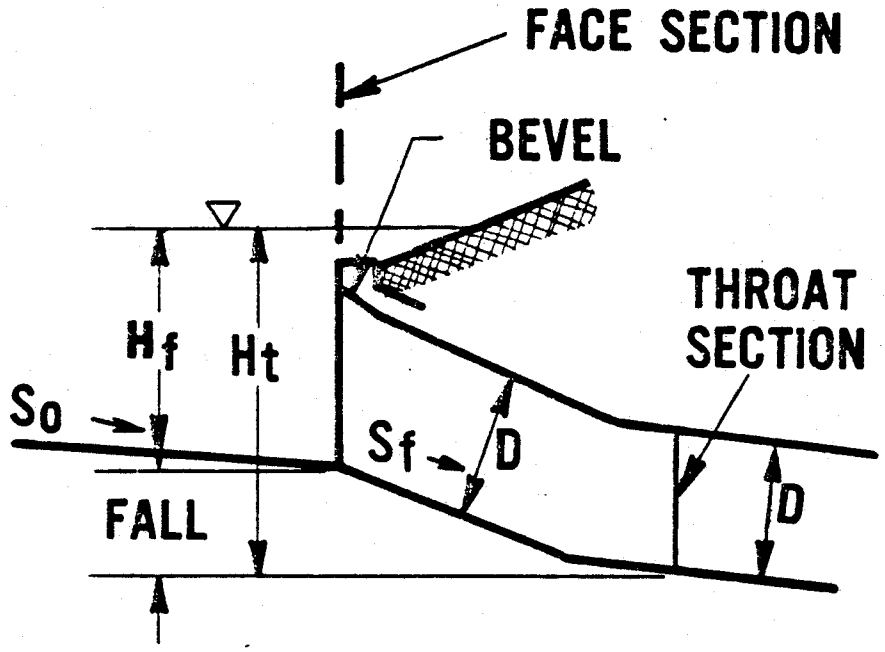
ELEVATION



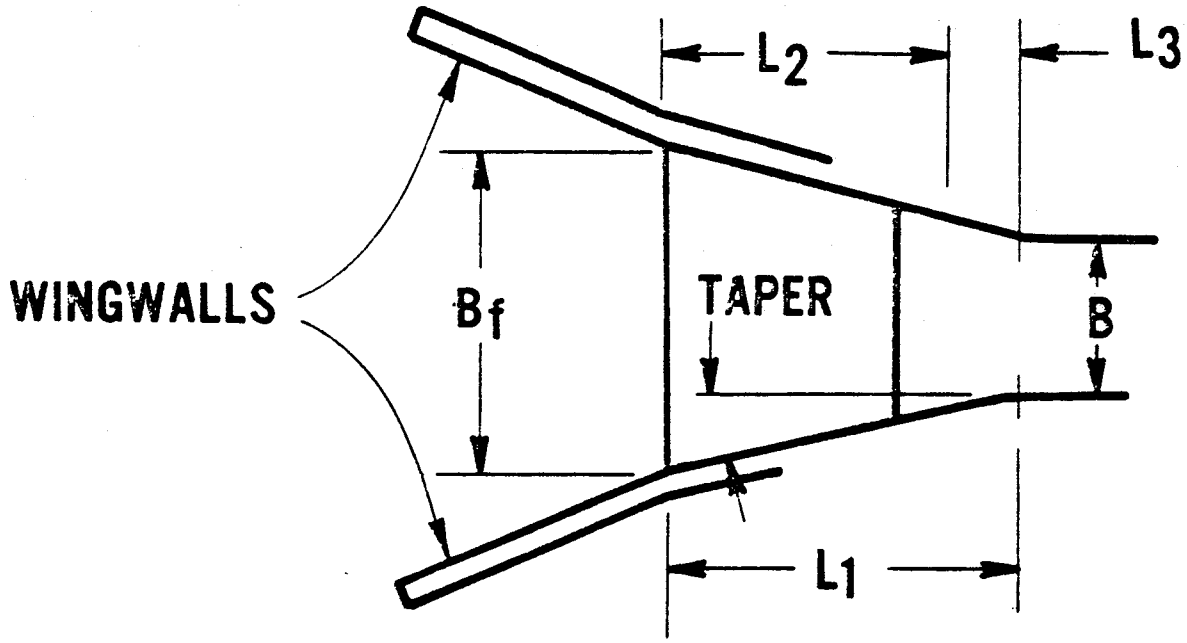
PLAN

Figure 6

SLOPE - TAPERED INLET



ELEVATION



PLAN

For the side-tapered inlet, there are two possible control sections: the face and the throat. H_f, as shown in Figure 5, is the headwater depth based upon face control. H_t is the headwater depth based upon throat control.

The advantages of a side-tapered inlet operating in throat control are: The flow contraction at the throat is reduced; and, for a given pool elevation, more head is applied at the throat control section. The latter advantage is increased by utilizing a slope-tapered inlet or a depression in front of the side-tapered inlet.

Slope-Tapered Inlets

A slope-tapered inlet is the third degree of improvement. Its advantage over the side-tapered inlet without a depression is that more head is available at the control (throat) section. This is accomplished by incorporating a FALL in the enclosed entrance section (Figure 6).

This inlet can have over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends largely upon the amount of FALL available between the invert at the face and the invert at the throat section. Since this FALL may vary, a range of increased capacities is possible.

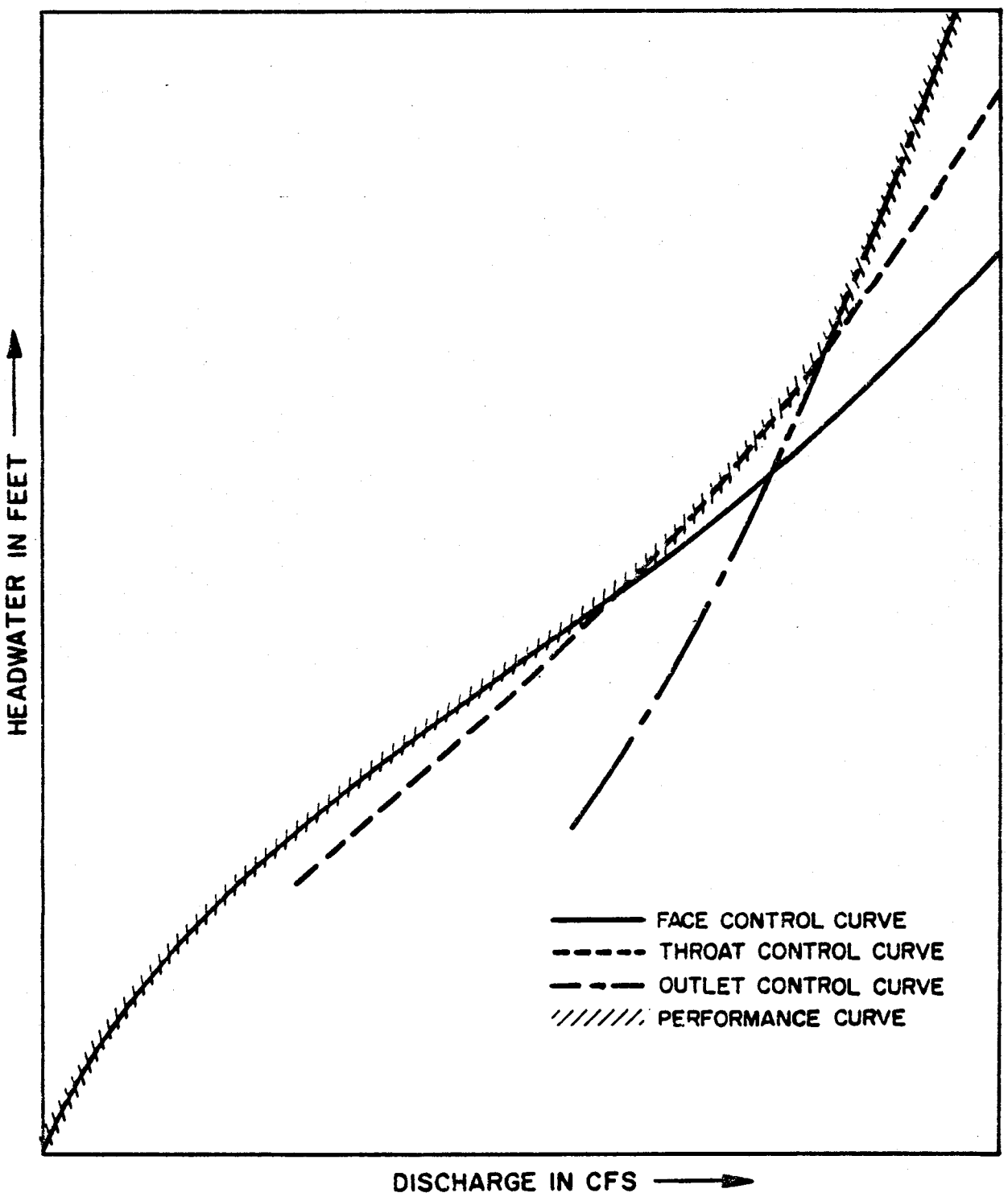
Slope-tapered inlets of alternate designs were considered and tested during the research. The inlet shown in Figure 6 is recommended on the basis of its hydraulic performance and ease of construction. As a result of the FALL concentrated between the face and the throat of this inlet, the barrel slope is flatter than the barrel slope of a conventional or side-tapered structure at the same site.

Both the face and throat are possible control sections in a slope-tapered inlet culvert. However, since the major cost of a culvert is in the barrel portion and not the inlet structure, the inlet face should be designed with a greater capacity at the allowable headwater elevation than the throat. This insures that flow control will be at the throat and more of the potential capacity of the barrel will be utilized.

Performance Curves

To understand how a culvert at a particular site will function over a range of discharges, a performance curve, which is a plot of discharge versus headwater depth or elevation, must be drawn. Figure 7 is a schematic performance curve for a culvert with either a side-tapered or slope-tapered inlet.

Figure 7



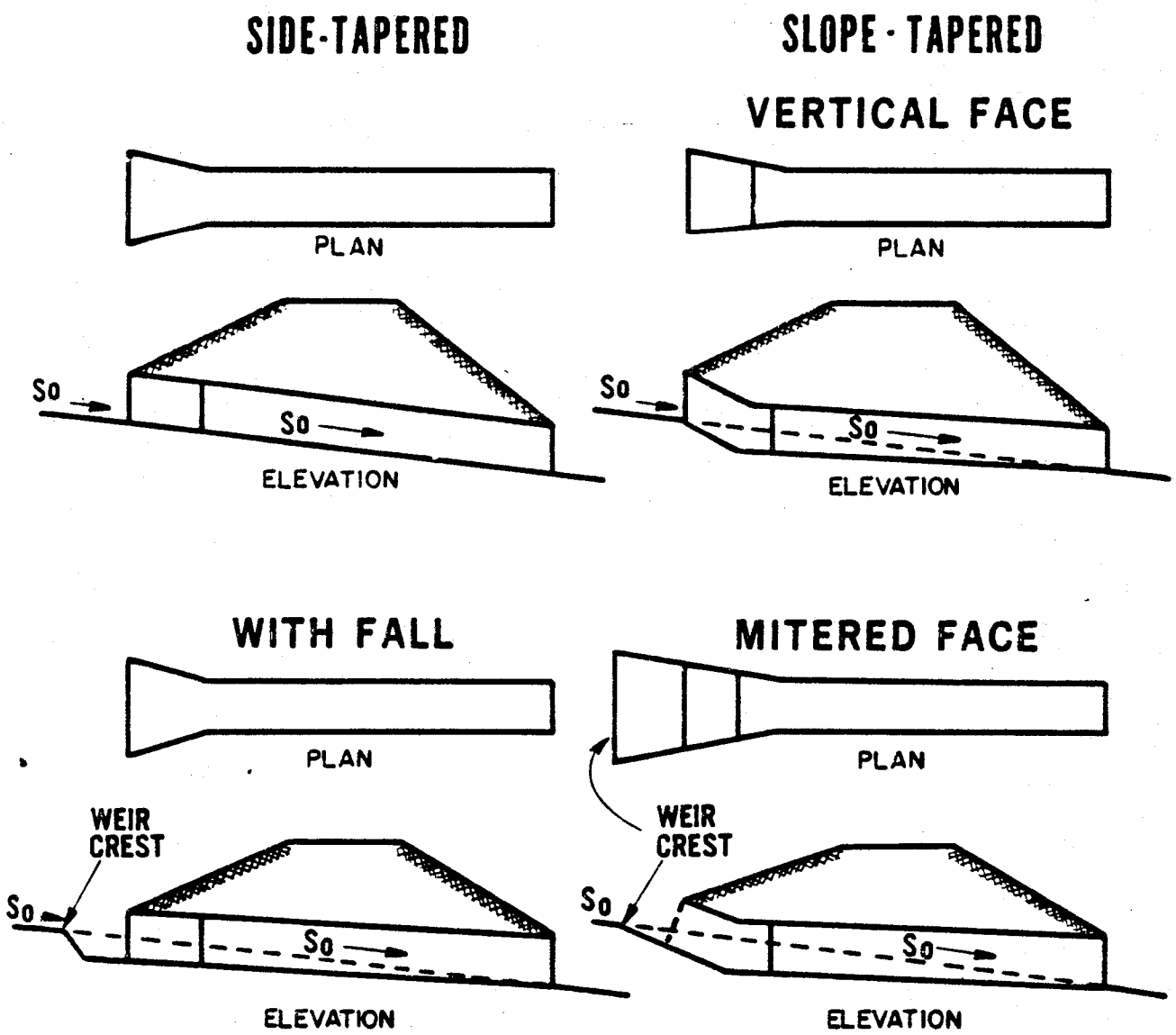
SCHEMATIC PERFORMANCE CURVE

For these inlets, it is necessary to compute the performance of the face section (face control curve), the throat section (throat control curve), and the barrel (outlet control curve), in order to develop the culvert performance curve for a range of discharges. The actual culvert performance curve, the hatched line of Figure 7, represents the performance of the face, throat and barrel sections in the ranges where their individual performance determines the required headwater. In the lower discharge range, face control governs; in the intermediate range, throat control governs; and in the higher discharge range, outlet control governs.

Performance curves should always be developed for culverts with side-tapered or slope-tapered inlets to insure that the designer is aware of how the culvert will function over a range of discharges, especially those exceeding the design discharge. It is important to emphasize that outlet control may govern for the larger discharges, and, as shown in Figure 7, the outlet control curve has a much steeper slope - a more rapidly rising headwater requirement for increasing discharges - than either the face or throat control curve. It should be recognized that there are uncertainties in the various methods of estimating flood peaks and that there is a chance that the design frequency flood will be exceeded during the life of the project. Culvert designs should be evaluated in terms of the potential for damage to the highway and adjacent property from floods greater than the design discharge.

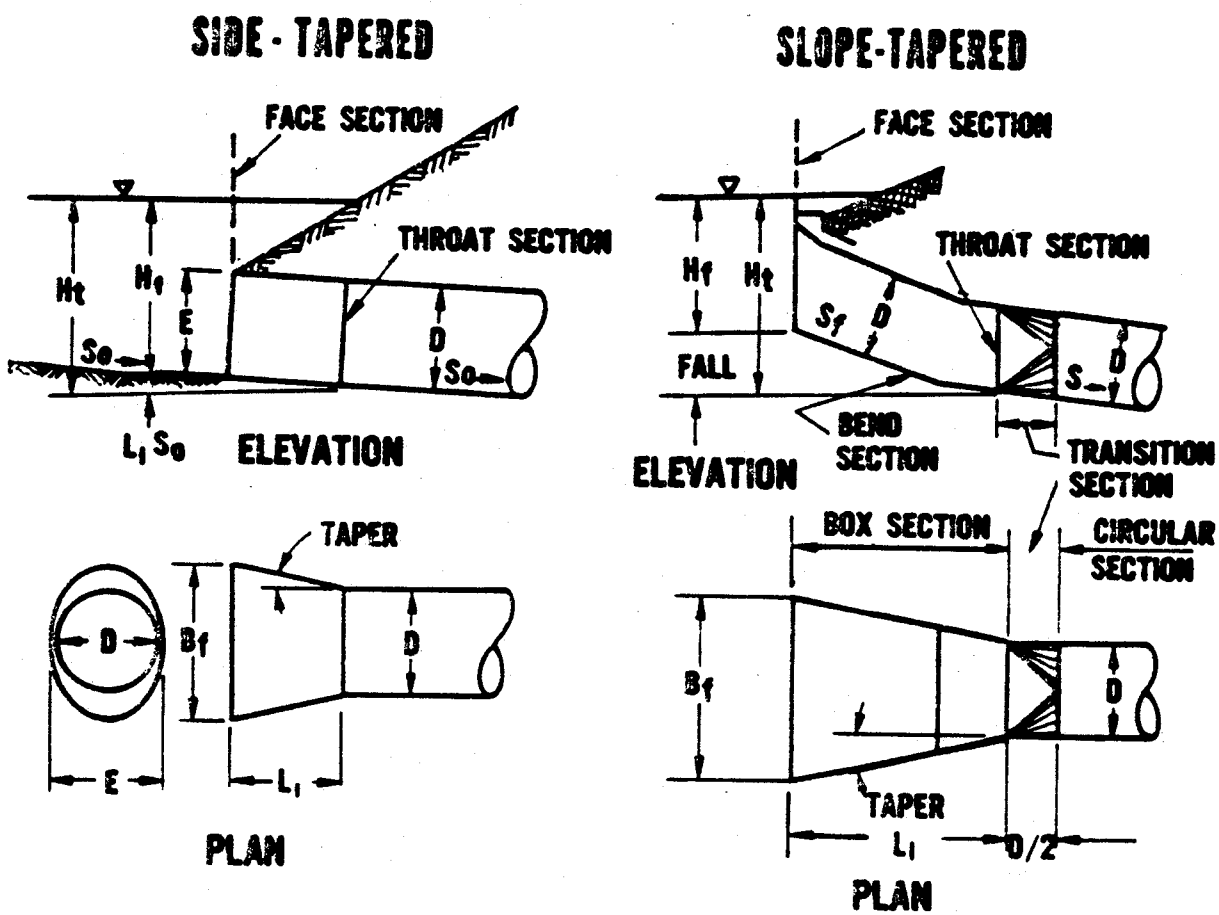
As alternate culverts are possible using improved inlet design, a performance curve should be plotted for each alternate considered. The performance curve will provide a basis for selection of the most appropriate design.

Figure 9



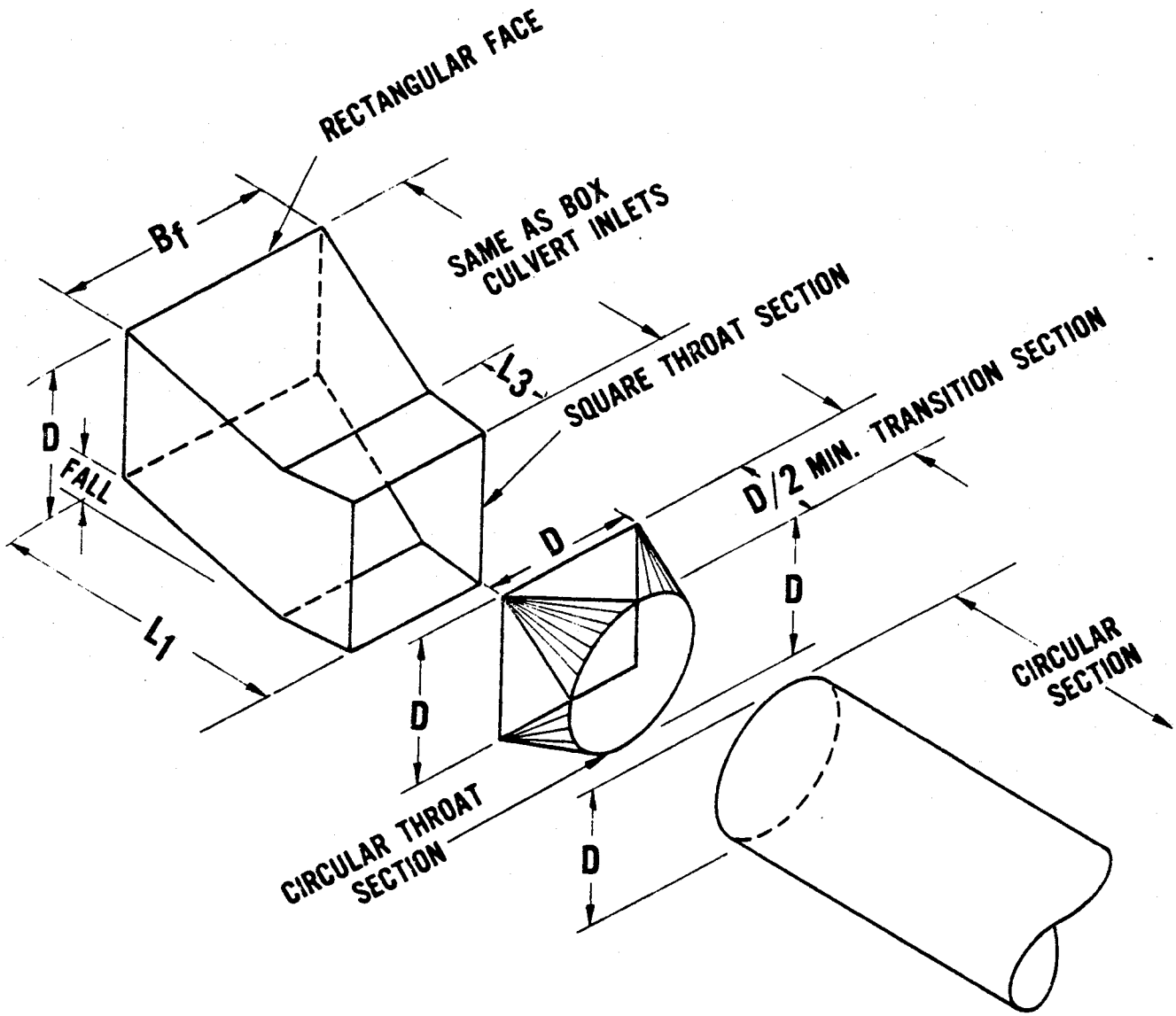
TYPES OF IMPROVED INLETS FOR BOX CULVERTS

Figure 15



TYPES OF IMPROVED INLETS FOR PIPE CULVERTS

Figure 17



SLOPE-TAPERED INLET
APPLIED TO CIRCULAR PIPE

[6] Outlet Protection

OUTLET PROTECTION *
(Std. & Spec. 1.36)

Definition

Structurally lined aprons or other acceptable energy dissipating devices placed at the outlets of pipes or paved channel sections.

Purpose

To prevent scour at stormwater outlets and to minimize the potential for downstream erosion by reducing the velocity of concentrated stormwater flows.

Conditions Where Practice Applies

Applicable to the outlets of all pipes and paved channel sections where the velocity of flow at design capacity of the outlet will exceed the permissible velocity of the receiving channel or area.

* Virginia Soil and Water Conservation Commission," Virginia Erosion and Sediment Control Handbook", Richmond, Virginia, 1980.

Planning Considerations

The outlets of pipes and structurally lined channels are points of critical erosion potential. Stormwater which is transported through man-made conveyance systems at design capacity generally reaches a velocity which exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at stormwater outlets, a flow transition structure is needed which will absorb the initial impact of the flow and reduce the flow velocity to a level which will not erode the receiving channel or area.

The most commonly used device for outlet protection is a structurally lined apron. These aprons are generally lined with riprap, grouted riprap or concrete. They are constructed at a zero grade for a distance which is related to the outlet flow rate and the tailwater level. Criteria for designing such an apron are contained in this practice. Sample problems of outlet protection design are provided.

Where flow is excessive for the economical use of an apron, excavated stilling basins may be used. Acceptable designs for stilling basins may be found in the following sources:

1. *Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No.14, U. S. Department of Transportation Federal Highway Administration, Washington, D.C. (August, 1975).*

2. *Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25, U. S. Department of the Interior - Bureau of Reclamation, Washington, D.C. (March, 1974).*

   (Both of the above are available from the U. S. Government Printing Office).

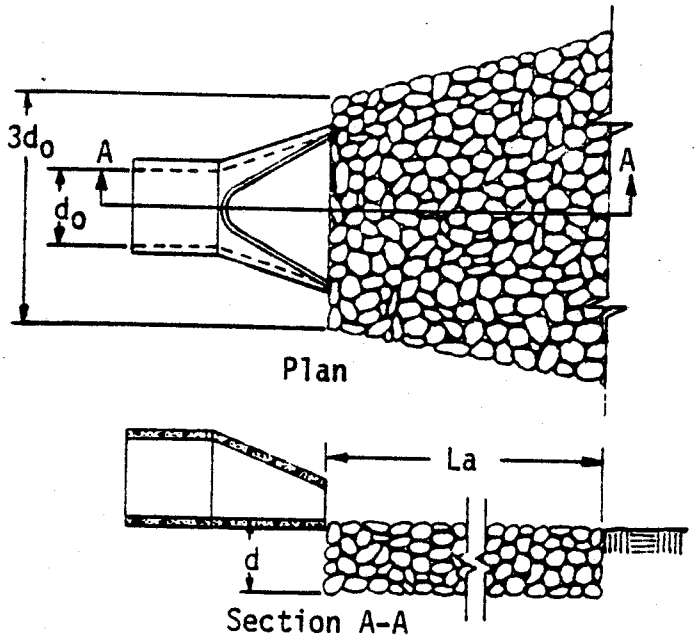
3. *Road Designs and Standards, Location and Design Division, Virginia Department of Highways and Transportation, Richmond, VA (1978).*

Design Criteria

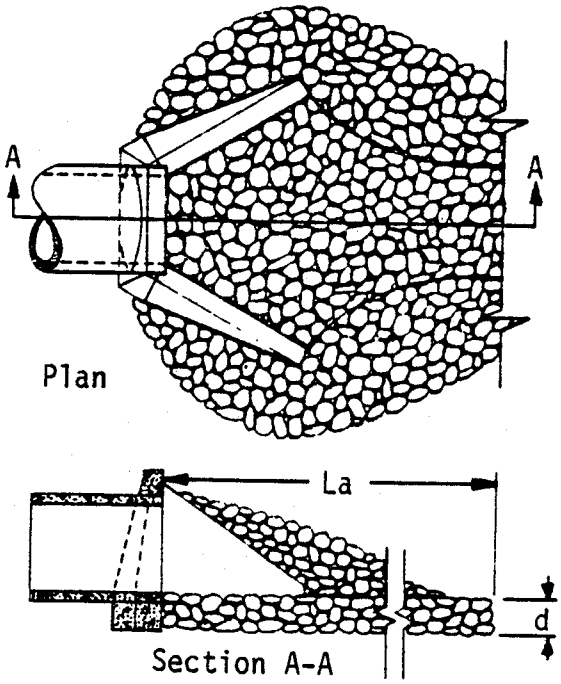
Structurally lined aprons at the outlets of pipes and paved channel sections shall be designed according to the following criteria:

PIPE OUTLETS
(See Plate 1.36b)

1. Tailwater depth: The depth of tailwater immediately below the pipe outlet must be determined for the design capacity of the pipe. Manning's Equation may be used to determine tailwater depth (see 1980 State Handbook - Chapter 5). If the tailwater depth is less than half the diameter of the outlet pipe, it shall be classified as a Minimum Tailwater Condition. If the tailwater depth is greater than half the pipe diameter, it shall be classified as a Maximum Tailwater Condition. Pipes which outlet onto flat areas with no defined channel



Pipe Outlet To Flat Area With No Defined Channel



Pipe Outlet To Well-Defined Channel

Notes

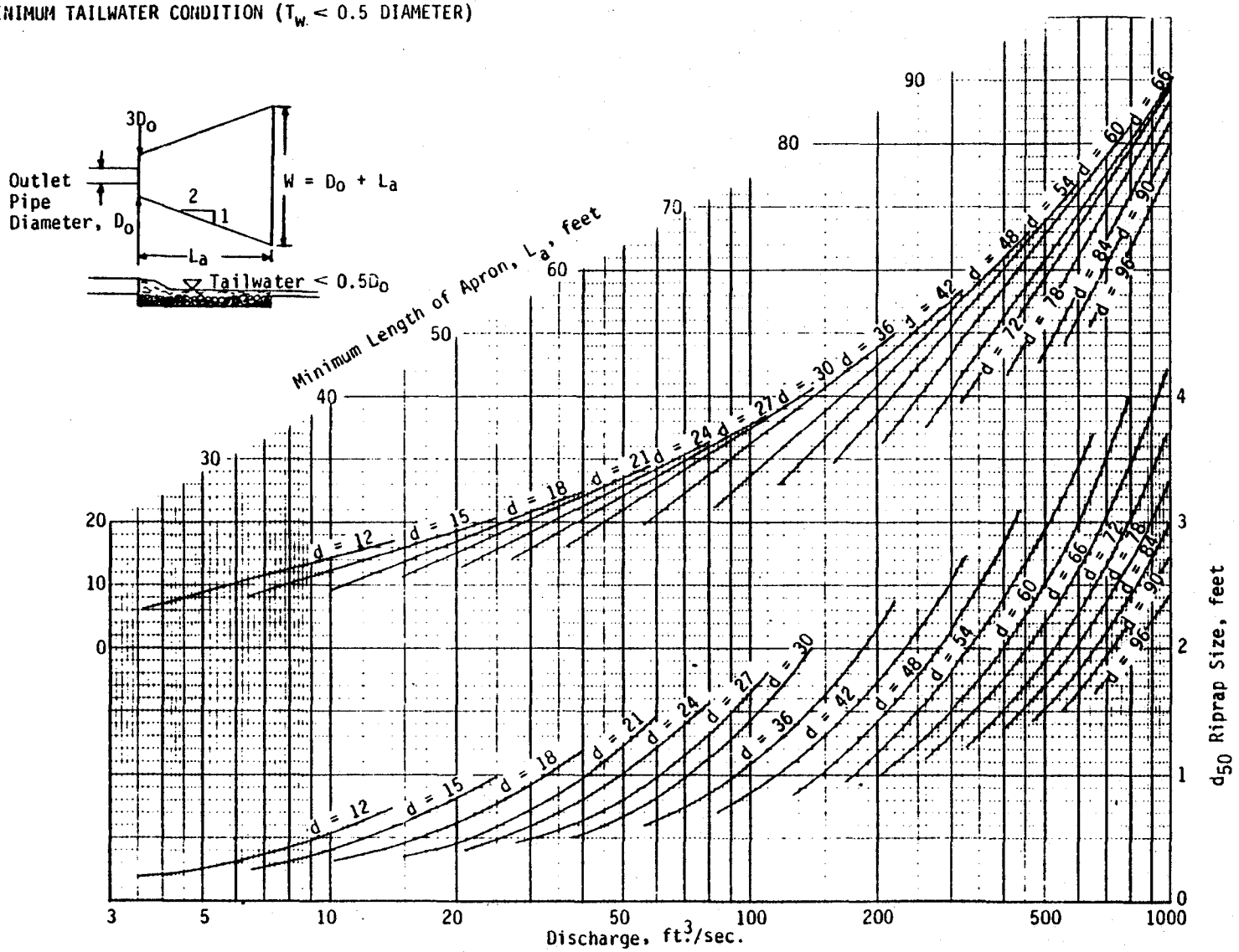
1. Apron lining may be riprap, grouted riprap, or concrete.
2. La is the length of the riprap apron as calculated using Plates 1.36d and 1.36e.
3. d = 1.5 times the maximum stone diameter but not less than 6 inches.

PIPE OUTLET CONDITIONS

Source: VaSCSS

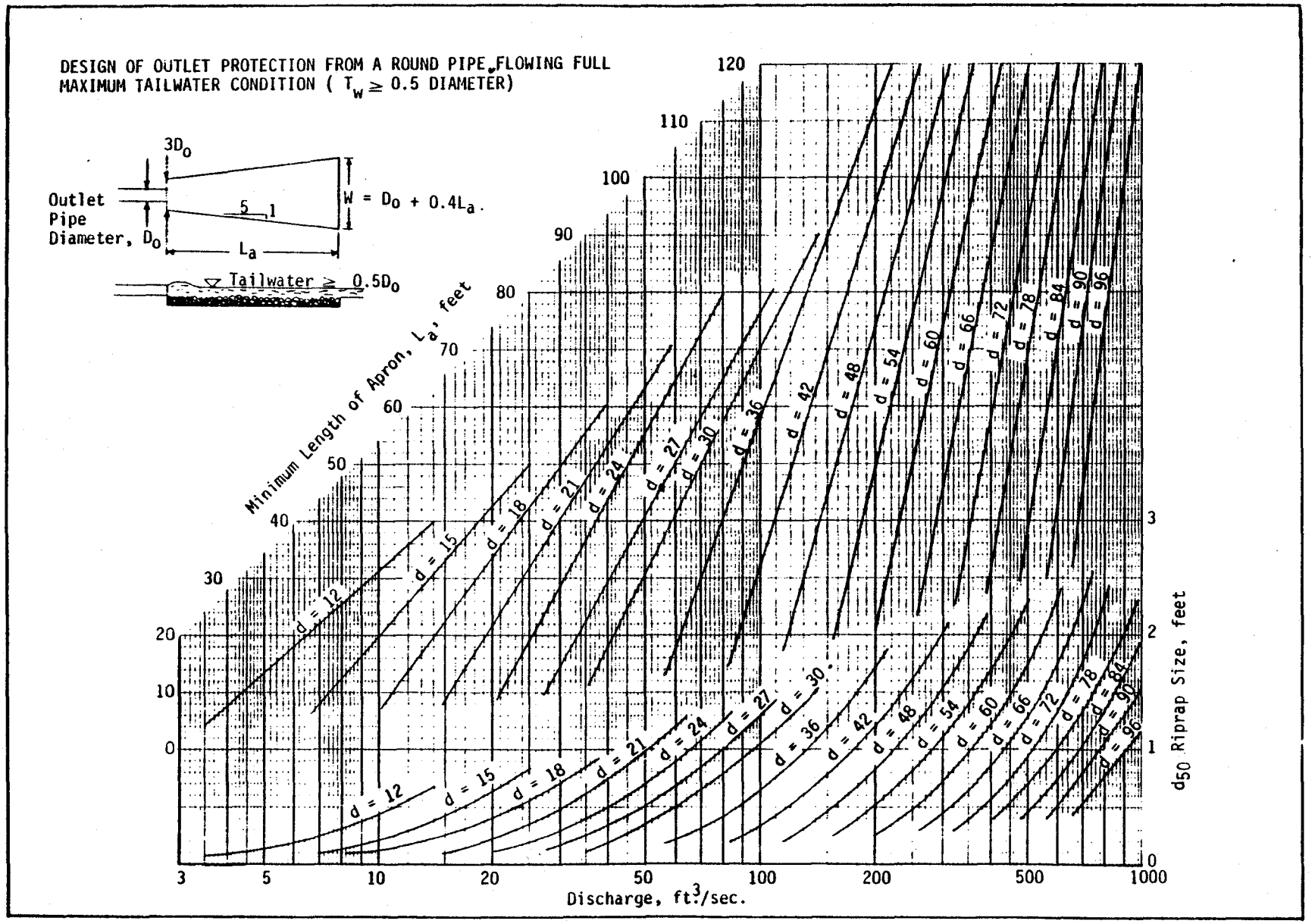
Plate 1.36b

DESIGN OF OUTLET PROTECTION FROM A ROUND PIPE FLOWING FULL
MINIMUM TAILWATER CONDITION ($T_w < 0.5$ DIAMETER)



Source: USDA-Soil Conservation Service

Plate 1.36d



Source: USDA-Soil Conservation Service

Plate 1.36e

may be assumed to have a Minimum Tailwater Conditions.

2. Apron length: The apron length shall be determined from the curves according to the tailwater condition:

   Minimum Tailwater - Use Plate 1.36d.

   Maximum Tailwater - Use Plate 1.36e.

3. Apron width: If the pipe discharges directly into a well-defined channel, the apron shall extend across the channel bottom and up the channel banks to an elevation one foot above the maximum tailwater depth or to the top of the bank (whichever is less).

   If the pipe discharges onto a flat area with no defined channel, the width of the apron shall be determined as follows:

   a. The upstream end of the apron, adjacent to the pipe, shall have a width three times the diameter of the outlet pipe.

   b. For a Minimum Tailwater Condition, the downstream end of the apron shall have a width equal to the pipe diameter plus the length of the apron.

   c. For a Maximum Tailwater Condition, the downstream end shall have a width equal to the pipe diameter plus 0.4 times the length of the apron.

4. Bottom grade: The apron shall be constructed with no slope along its length (0.0% grade). The invert elevation of the downstream end of the apron shall be equal to the elevation of the invert of the receiving channel. There shall be no overfall at the end of the apron.

5. Side slopes: If the pipe discharges into a well-defined channel, the side slopes of the channel shall not be steeper than 2:1 (Horizontal: Vertical).

6. Alignment: The apron shall be located so that there are no bends in the horizontal alignment.

7. Materials: The apron may be lined with riprap, grouted riprap, or concrete. The median sized stone for riprap shall be determined from the curves in Appendix (Plates 1.36d and 1.36e) according to the tailwater condition. The gradation, quality and placement of riprap shall conform to Std. & Spec. 1.37 - RIPRAP. (pp. 84)

PAVED CHANNEL OUTLETS
(See Plate 1.36c)

1. The flow velocity at the outlet of paved channels flowing at design capacity must not exceed the permissible velocity of the receiving channel. (See Plate 1.36a)

2. The end of the paved channel shall merge smoothly with the receiving channel section. There shall be no overfall at the end of the paved section. Where the bottom width of the paved channel is narrower than the bottom width of the receiving channel, a transition section shall be provided (see Plate 1.36c). The maximum side divergence of the transition shall be 1 in 3F where;

$$F = \frac{V}{\sqrt{gd}}, \text{ and}$$

F = Froude number
V = Velocity at beginning of transition (ft./sec.)
d = depth of flow at beginning of transition (ft.)
g = 32.2 ft./sec.

3. Bends or curves in the horizontal alignment of the transition is not allowed unless the Froude number (F) is 1.0 or less, or the section is specifically designed for turbulent flow.

PERMISSIBLE VELOCITIES: GRASS AND EARTH-LINED CHANNELS

GRASS LININGS

Channel Slope	Lining	Permissible* Velocity
0-5%	Bermudagrass	6 ft/sec
	Reed canarygrass Tall fescue Kentucky bluegrass	5 ft/sec
	Grass-legume mixture	4 ft/sec
	Red fescue Redtop Sericea lespedeza Annual lespedeza Small grains (temporary)	2.5 ft/sec
5-10%	Bermudagrass	5 ft/sec
	Reed canarygrass Tall fescue Kentucky bluegrass	4 ft/sec
	Grass-legume mixture	3 ft/sec
Greater than 10%	Bermudagrass	4 ft/sec
	Reed canarygrass Tall fescue Kentucky bluegrass	3 ft/sec

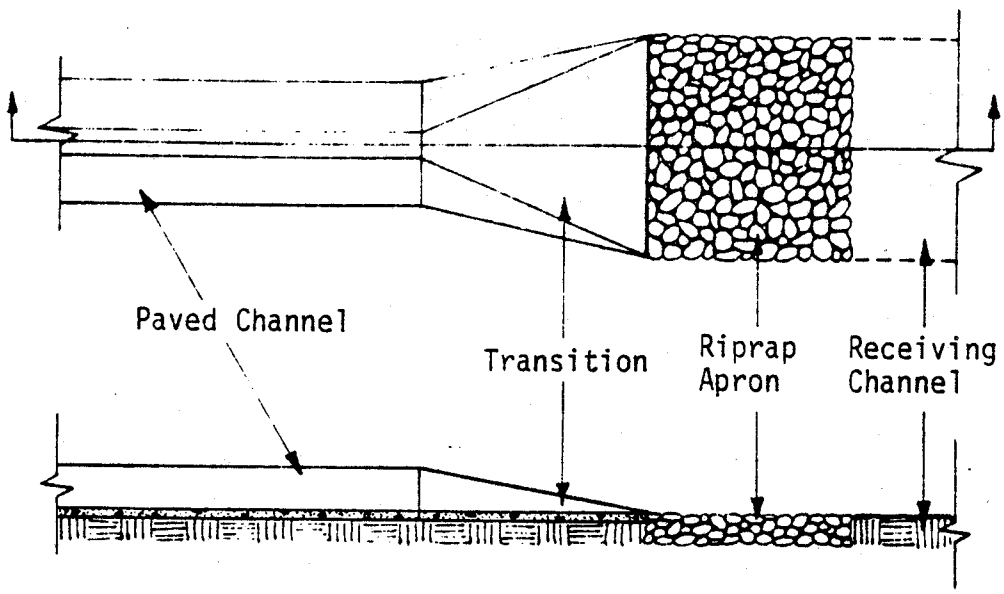
* For highly erodible soils, decrease permissible velocities by 25%.

EARTH LININGS

Soil Types	Permissible Velocity
Fine Sand (noncolloidal)	2.5 ft/sec
Sandy Loam (noncolloidal)	2.5 ft/sec
Silt Loam (noncolloidal)	3.0 ft/sec
Ordinary Firm Loam	3.5 ft/sec
Fine Gravel	5.0 ft/sec
Stiff Clay (very colloidal)	5.0 ft/sec
Graded, Loam to Cobbles (noncolloidal)	5.0 ft/sec
Graded, Silt to Cobbles (colloidal)	5.5 ft/sec
Alluvial Silts (noncolloidal)	3.5 ft/sec
Alluvial Silts (colloidal)	5.0 ft/sec
Coarse Gravel (noncolloidal)	6.0 ft/sec
Cobbles and Shingles	5.5 ft/sec
Shales and Hard Pans	6.0 ft/sec

Source: USDA-Soil Conservation Service and American Society of Civil Engineers

Plate 1.36a



Notes

1. Riprap apron reduces the flow velocity below the permissible velocity of the natural receiving channel.

2. Transition side divergence is 1 in 3F, where

$$F = \text{Froude Number} = \frac{V}{\sqrt{gd}}, \text{ where}$$

V = Velocity at the beginning of the transition

d = Depth of flow at the beginning of the transition

g = 32.2 ft./sec.^{2}

PAVED CHANNEL OUTLET

Source: VaSWCC

Plate 1.36c

Sample Problems
Outlet Protection Design

Example 1

Given: An 18-inch pipe discharges 24 ft.^{3}/sec. at design capacity onto a grassy slope (no defined channel).

Find: The required length, width and median stone size (d_{50}) for a riprap lined apron.

Solution:

1. Since the pipe discharges onto a grassy slope with no defined channel, a Minimum Tailwater Condition may be assumed.

2. From Plate 1.36d, an apron length (L_a) of 20 feet and a median stone size (d_{50}) of 0.8 ft. is determined.

3. The upstream apron width equals three times the pipe diameter; 3 x 1.5 ft. = 4.5 ft.

4. The downstream apron width equals the apron length plus the pipe diameter; 20 ft. + 1.5 ft. = 21.5 ft.

Example 2

Given: The pipe in example No.1 discharges into a channel with a triangular cross-section, 2 feet deep and 2:1 side slopes. The channel has a 2% slope and an "n" factor of .045.

Find: The required length, width and the median stone size (d_{50}) for a riprap lining.

Solution:

1. Determine the tailwater depth using Manning's Equation.

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} A$$

$$24 = \frac{1.49}{.045} \left(\frac{2d}{2\sqrt{2^2 + 1}}\right)^{2/3} (.02)^{1/2} (2d^2)$$

where,

d = depth of tailwater
d = 1.74 ft.*

*Since d is greater than half the pipe diameter, a Maximum Tailwater Condition exists.

2. From Plate 1.36e, a median stone size (d_{50}) of 0.5 ft. and an apron length (L_a) of 41 ft. is determined.

3. The entire channel cross-section should be lined, since the maximum tailwater depth is within one foot of the top of the channel.

[7] Riprap, Riprap Basins, and Channel Linings

RIPRAP*
(Std. & Spec. 1.37)

Definition

A permanent, erosion resistant ground cover of large, loose, angular stone.

Purposes

1. To protect the soil surface from the erosive forces of concentrated runoff.

2. To slow the velocity of concentrated runoff while enhancing the potential for infiltration.

3. To stabilize slopes with seepage problems and/or non-cohesive soils.

Conditions Where Practice Applies

To soil-water interfaces where the soil conditions, water turbulence and velocity, expected vegetative cover, etc., are such that the soil may erode under the design flow conditions. Riprap may be used, as appropriate, at stormdrain outlets, channel banks and/or bottoms, roadside ditches, drop structures, at the toe of slopes, etc.

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* Virginia Soil and Water Conservation Commission," Virginia Erosion and Sediment Control Handbook", Richmond, Virginia, 1980.

Planning Considerations

Graded vs. Uniform Riprap

Riprap is classified as either graded or uniform. A sample of graded riprap would contain a mixture of stones which vary in size from small to large. A sample of uniform riprap would contain stones which are all fairly close in size.

For most applications, graded riprap is preferred to uniform riprap. Graded riprap forms a flexible self-healing cover, while uniform riprap is more rigid and cannot withstand movement of the stones. Graded riprap is cheaper to install, requiring only that the stones be dumped such that they remain in a well graded mass. Hand or mechanical placement of individual stones is limited to that necessary to achieve the proper thickness and line. Uniform riprap requires placement in a more or less uniform pattern, requiring more hand or mechanical labor.

Riprap sizes can be designated by either the diameter or the weight of the stones. It is often misleading to think of riprap in terms of diameter, since the stones should be rectangular instead of spherical. However, it is simpler to specify the diameter of an equivalent size of spherical stone. Table 1.37a lists some typical stones by weight, spherical diameter and the corresponding rectangular dimensions. These stone sizes are based upon an assumed specific weight of 165 lbs./ft.^{3}.

TABLE 1.37a

SIZE OF RIPRAP STONES

Weight (lbs.)	Mean Spherical Diameter (ft.)	Rectangular Shape Length (ft.)	Rectangular Shape Width, Height(ft.)
50	0.8	1.4	0.5
100	1.1	1.75	0.6
150	1.3	2.0	0.67
300	1.6	2.6	0.9
500	1.9	3.0	1.0
1000	2.2	3.7	1.25
1500	2.6	4.7	1.5
2000	2.75	5.4	1.8
4000	3.6	6.0	2.0
6000	4.0	6.9	2.3
8000	4.5	7.6	2.5
20000	6.1	10.0	3.3

Since graded riprap consists of a variety of stone sizes, a method is needed to specify the size range of the mixture of stone. This is done by specifying a diameter of stone in the mixture for which some percentage, by weight, will be smaller. For example, d_{85} refers to a mixture of stones in which 85% of the stone by weight would be smaller than the diameter specified. Most designs are based on d_{50}. In other words, the design is based on the average size of stone in the mixture. Table

1.37b lists Virginia Department of Highways and Transportation (VDH&T) standard graded riprap sizes by diameter and weight of the stone.

TABLE 1.37b

GRADED RIPRAP

Riprap Class	*D_{15} Weight (lbs.)*	*D_{15} Mean Spherical(ft.) Diameter*	*D_{50} Mean Spherical(ft.) Diameter*
Class I	*50*	*0.8*	*1.1*
Class II	*150*	*1.3*	*1.6*
Class III	*500*	*1.9*	*2.2*
Type I	*1500*	*2.6*	*2.8*
Type II	*6000*	*4.0*	*4.5*

Sequence of Construction

Since riprap is used where erosion potential is high, construction must be sequenced so that the riprap is put in place with the minimum possible delay. Disturbance of areas where riprap is to be placed should only be undertaken when final preparation and placement of the riprap can follow immediately behind the initial disturbance. Where riprap is used for outlet protection, the riprap should be placed before or in conjunction with the construction of the pipe or channel so that it is in place when the pipe or channel begins to operate.

Design Criteria

Gradation

The riprap shall be composed of a well-graded mixture down to the one-inch size particle such that 50% of the mixture by weight shall be larger than the d_{50} size as determined from the design procedure. A well-graded mixture as used herein is defined as a mixture composed primarily of the larger stone sizes but with a sufficient mixture of other sizes to fill the progressively smaller voids between the stones. The diameter of the largest stone size in such a mixture shall be 1.5 times the d_{50} size.

The designer, after determining the riprap size that will be stable under the flow conditions, shall consider that size to be a minimum size and then, based on riprap gradations actually available in the area, select the size or sizes that equal or exceed the minimum size. The possibility of damage by children shall be considered in selecting a riprap size, especially if there is nearby water to toss the stones into.

Thickness

The minimum thickness of the riprap layer shall be 1.5 times the maximum stone diameter but not less than 6 inches.

Quality of Stone

Stone for riprap shall consist of field stone or rough unhewn quarry stone of approximately rectangular shape. The stone shall be hard and angular and of such quality that it will not disintegrate on exposure to water or weathering and it shall be suitable in all other respects for the purpose intended. The specific gravity of the individual stones shall be at least 2.5.

Rubble concrete may be used provided it has a density of at least 150 pounds per cubic foot, and otherwise meets the requirements of this Standard and Specification.

Riprap at Outlets

Design criteria for sizing the stone and determining the dimensions of riprap pads used at the outlet of drainage structures are contained in STORMDRAIN OUTLET PROTECTION (Std. & Spec. 1.36).

Riprap for Channel Stabilization

Riprap for channel stabilization shall be designed to be stable for the condition of bank-full flow in the reach of channel being stabilized. The design procedure in Appendix 1.37a which is extracted from the Hydraulic Engineering, Circular No. 15 of the Federal Highway Administration shall be used.

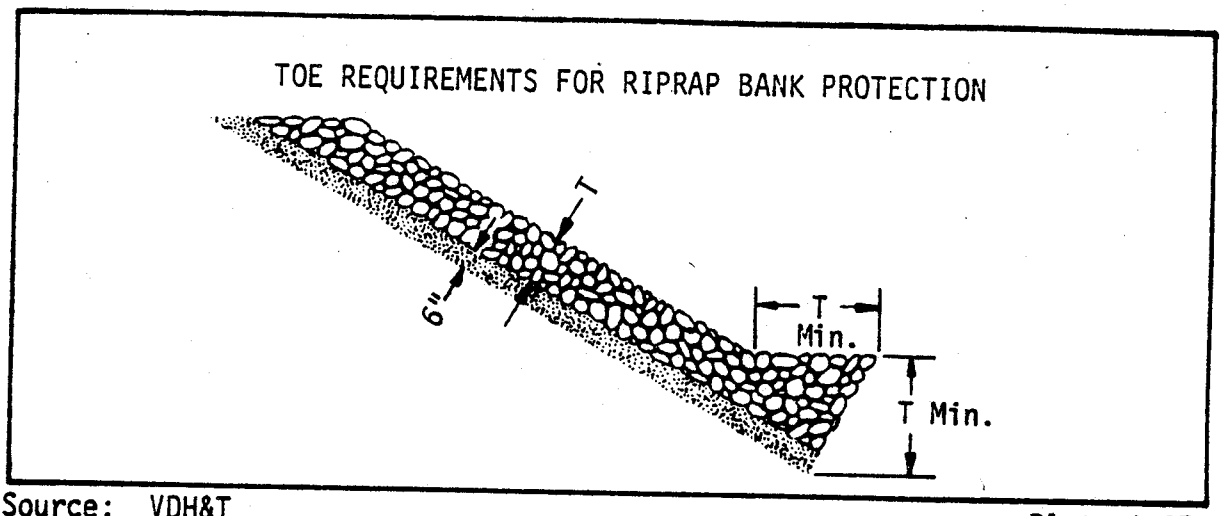
Riprap shall extend up the banks of the channel to a height equal to the maximum depth of flow or to a point where vegetation can be established to adequately protect the channel.

The riprap size to be used in a channel bend shall extend upstream from the point of curvature and downstream from the point of tangency a distance of at least 5 times the channel bottom width. The riprap shall extend across the bottom and up both sides of the channel.

Where riprap is used only for bank protection and does not extend across the bottom of the channel, riprap shall be keyed into the bottom of the channel to a minimum depth equal to the thickness of the blanket and shall extend across the bottom of the channel the same distance. (See Plate 1.37a).

Riprap for Slope Stabilization

Riprap for slope stabilization shall be designed so that the natural angle of repose of the stone mixture is greater than the gradient of the slope being stabilized (see Plate 1.37d).



Source: VDH&T

Plate 1.37a

Filter Blankets

A filter blanket is a layer of material placed between the riprap and the underlying soil surface to prevent soil movement into or through the riprap.

A filter blanket can be of two general forms; a gravel layer or a plastic filter cloth. A determination of the need for a filter blanket is made by comparing particle sizes of the overlying material and the base material in accordance with the criteria below.

Gravel filter blanket: The following relationships must exist:

$$\frac{d_{15}\text{filter}}{d_{85}\text{ base}} < 5 < \frac{d_{15}\text{ filter}}{d_{15}\text{ base}} < 40$$

and,

$$\frac{d_{50}\text{filter}}{d_{50}\text{ base}} < 40$$

In these relationships, filter refers to the overlying material and base refers to the underlying material. The relationships must hold between the filter material and the base meterial and between the riprap and the filter material. In some cases, more than one layer of filter material may be needed. Each layer of filter material should be approximately 6 inches thick.

Plastic filter cloth: Plastic filter cloth may be used in place of or in conjunction with gravel filters. The following particle size relationships must exist:

1. For filter cloth adjacent to granular materials containing 50 percent or less (by weight), fine particles (less than 0.074mm)

   a. $\frac{d_{85} \text{ base (mm)}}{\text{EOS* filter cloth (mm)}} > 1$

   b. Total open area of filter is less than 36 percent

2. For filter cloth adjacent to all other soils:

   a. EOS* less than U. S. Standard Sieve No.70.
   b. Total open area of filter is less than 10 percent.

No filter cloth should be used with less than 4 percent open area or an EOS* less than U. S. Standard Sieve No.100.

* EOS - Equivalent Opening Size to a U. S. Standard Sieve Size.

Filter blankets should always be provided where seepage from underground sources threatens the stability of the riprap. No filter blanket is required for riprap used for storm drain outlet protection.

Riprap for Shoreline Erosion Control

Refer to VDH&T Drainage Manual

Riprap for Abrupt Channel Contractions

Refer to VDH&T Drainage Manual

Construction Specifications

Subgrade Preparation: The subgrade for the riprap or filter shall be prepared to the required lines and grades. Any fill required in the subgrade shall be compacted to a density approximating that of the surrounding undisturbed material. Brush, trees, stumps and other objectionable material shall be removed.

Filter Blanket: Placement of the filter blanket should be done immediately after slope preparation. For granular filters the stone should be spread in a uniform layer to the specified depth. Where more than one layer of filter material is used, the layers should be spread so that there is minimal mixing of the layers.

For plastic filter cloths, the cloth should be place directly on the prepared slope. The edges of the sheets should overlap by at least 12 inches. Anchor pins, 15 inches long, should be spaced every 3 feet along the overlap. The upper and lower ends of the cloth should be buried a minimum of 12 inches deep. Care should be taken not to damage the cloth when placing the riprap. If damage occurs, that sheet should be

removed and replaced. For large stone (12 inches or greater), a 4-inch layer of gravel may be necessary to prevent damage to the cloth.

Stone Placement: Placement of riprap should follow immediately after placement of the filter. The riprap should be placed so that it produces a dense well-graded mass of stone with a minimum of voids. The desired distribution of stones throughout the mass may be obtained by selective loading at the quarry, controlled dumping of successive loads during final placing, or by a combination of these methods. The riprap should be placed to its full thickness in one operation. The riprap should not be placed in layers. The riprap should not be placed by dumping into chutes or similar methods which are likely to cause segregation of the various stone sizes. Care should be taken not to dislodge the underlying material when placing the stones.

The finished slope should be free of pockets of small stone or clusters of large stones. Hand placing may be necessary to achieve the required grades and a good distribution of stone sizes. Final thickness of the riprap blanket should be within plus or minus 1/4 of the specified thickness.

Maintenance

Once a riprap installation has been completed, it should require very little maintenance. It should, however, be inspected periodically to determine if high flows have caused scour beneath the riprap or dislodged any of the stone. If repairs are needed, they should be accomplished immediately.

XI. RIPRAP BASINS*

The design procedure for riprap energy dissipators is based on data obtained during a study "Flood Protection at Culvert Outfalls" (XI-1, XI-2) sponsored by the Wyoming Highway Department and conducted at Colorado State University. The purpose of the experimental program was to establish relationships between flow properties and the dimensions of riprapped basins at culvert outfalls.

Tests were conducted with 6-inch, 12-inch, 18-inch, and 36-inch pipes, and 6 by 12-inch, 6 by 18-inch, and 6 by 24-inch model box culverts with discharges ranging from 0.1 to 100 cfs. Both angular and rounded rock with an average size (d_{50}) ranging from 1.05 to 4.29 were tested. Two pipe slopes were considered, 0 and 3.75%. In all, 459 model basins were studied.

The following conclusions were drawn from an analysis of the experimental data and observed operating characteristics.

The depth (h_s), length (L_s), and width (W_s) of the scour hole were related to the characteristic size of riprap (d_m), 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_m) was greater than 2.0, and the scour hole functioned very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunged into the hole, a jump formed against the downstream extremity of the scour hole, and flow was generally well dispersed as it left the basin.

The mound of material which formed on the bed downstream of the scour hole contributed to the dissipation of energy and reduced the size of the scour hole; i.e., if the mound from a stable scoured basin was removed and the basin was again subjected to design flow, the scour hole enlarged somewhat.

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* Corry, M. L., et. al., "Hydraulic Design of Energy Dissipators for Culverts and Channels", Federal Highway Administration. U.S. Dept. of Transportation, Washington, D.C., Hydraulic Engineering Circular No. 14, 1975.

For high tailwater basins (TW/y_o greater than 0.75) the high velocity core of water emerging from the culvert retained its jetlike character as it passed through the basin, and diffused in a manner very similar to that of a concentrated jet diffusing in a large body of water. As a result, the scour hole was much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

General details of the basin recommended in this report are shown on figure XI-1. Principal features of the basin are:

1. The basin is preshaped and lined with riprap.

2. The surface of the riprapped floor of the energy dissipating pool is constructed at an elevation h_s below the culvert invert. h_s is the approximate depth of scour that would occur in a thick pad of riprap, constructed at the outfall of the culvert, if subjected to the design discharge. The ratio of h_s to d_{50} of the material should be greater than 2 and less than 4.

3. The length of the energy dissipating pool is $10(h_s)$ or $3W_o$ which ever is larger. The overall length of the basin is $15(h_s)$ or $4W_o$ which ever is larger. W_o = culvert diameter or width of box culvert.

DESIGN PROCEDURE

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that $TW/y_o \leq 0.75$ for design discharge.

2. For subcritical flow conditions (culvert set on mild or horizontal slope) utilize figures III-9 or III-10 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

3. From site inspection and from field experience in the area, determine whether or not channel protection is required at the culvert outlet.

NOTE A – IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS–SECTIONAL AREA AT SECTION A–A SUCH THAT Q_{des}/(CROSS SECTION AREA AT SEC. A–A) = SPECIFIED EXIT VELOCITY.

NOTE B – WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A–A.

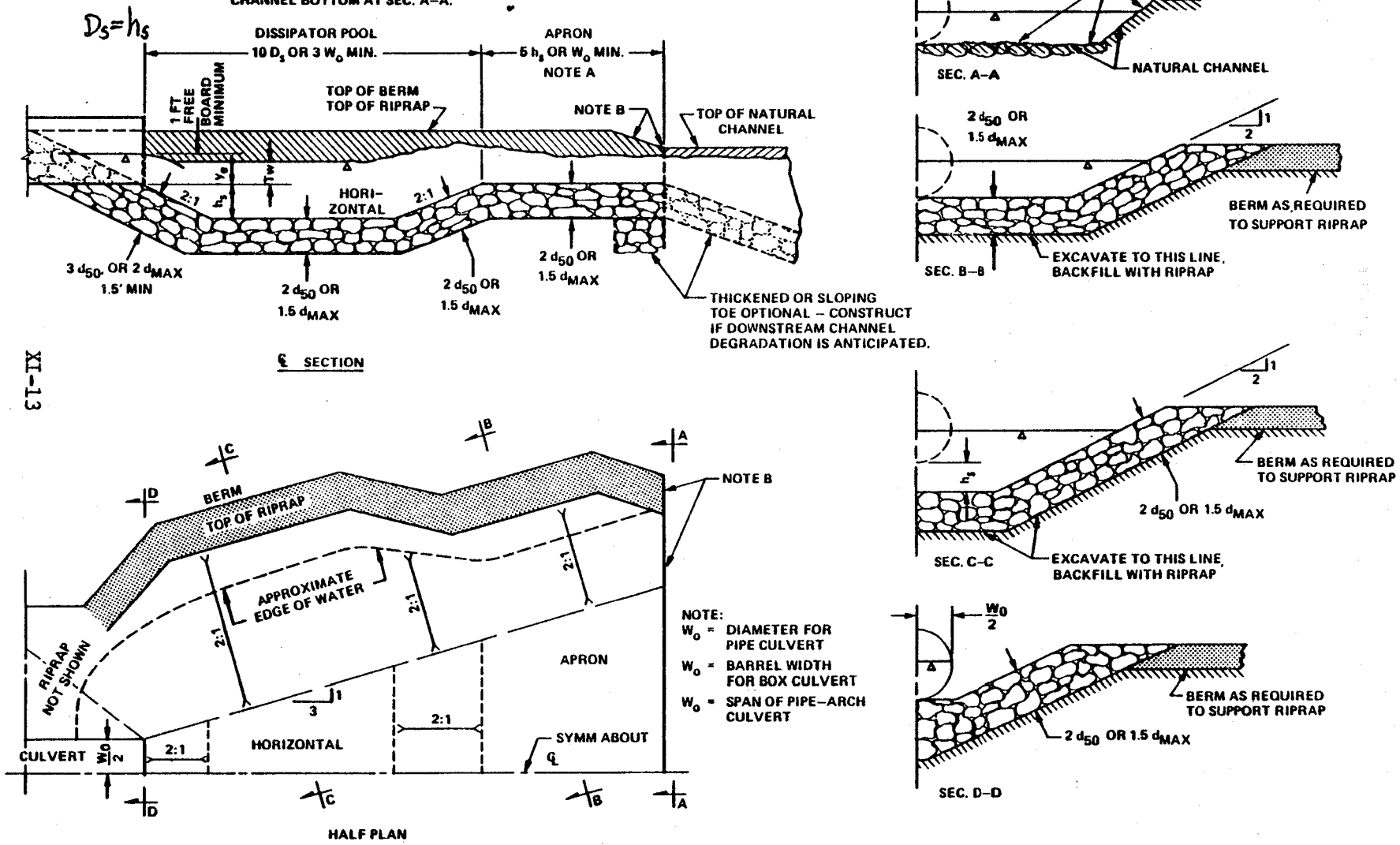


FIGURE X1–1. DETAILS OF RIPRAPPED CULVERT ENERGY BASIN

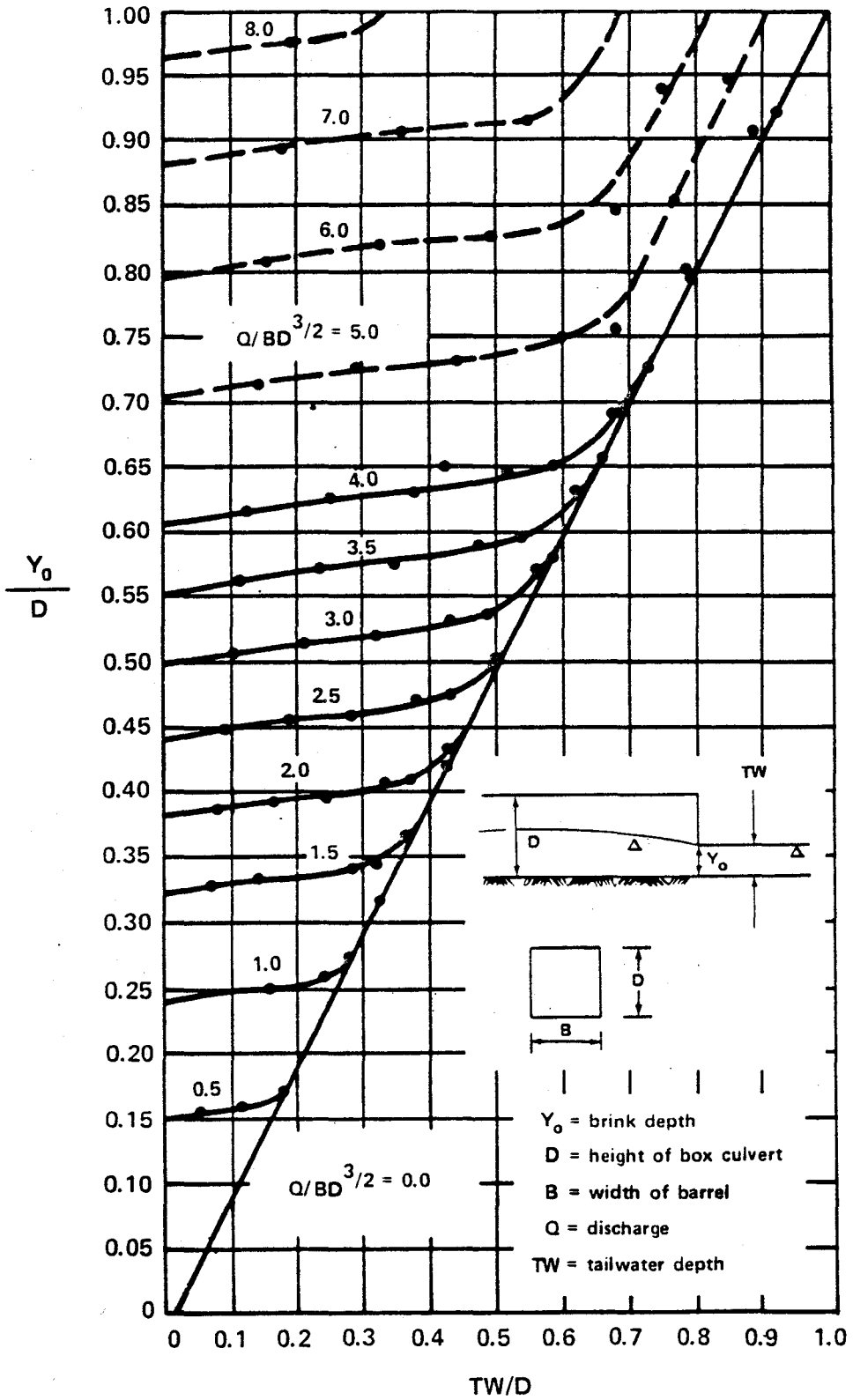
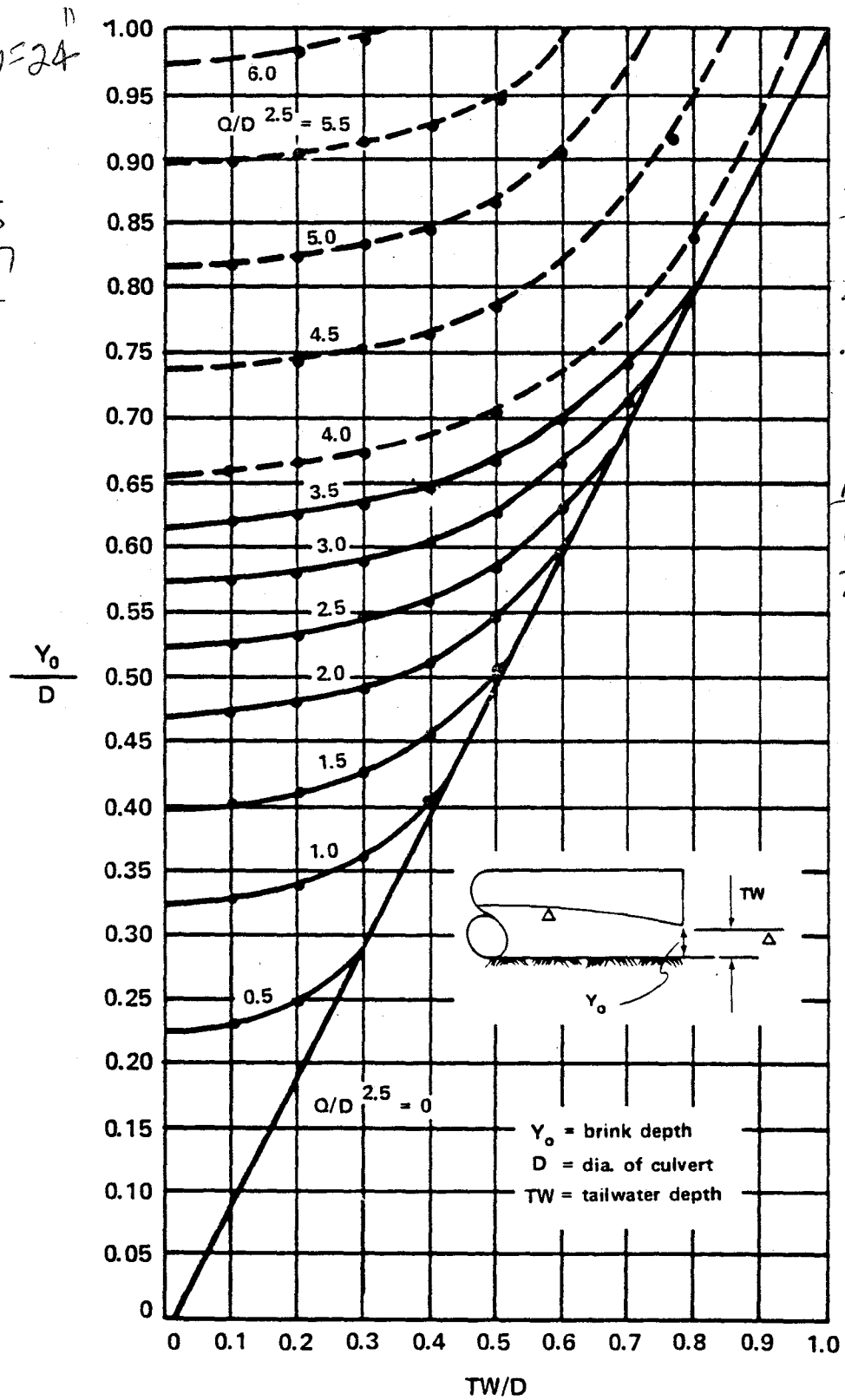


Figure III-9 Dimensionless Rating Curves for the Outlets of Rectangulars Culverts on Horizontal and Mild Slopes from Reference III-2

Q=60cfs; TW=24"

D	$\frac{Q}{D^{2.5}}$	$\frac{TW}{D}$
42"	2.6	.57
60"	1.5	.4



$\frac{Y^0}{D}$	Y^0
.63	2.2
.41	2.05

A	EXIT V
6.4	9.4
7.5	8.0

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

4. If channel protection is required, compute the Froude number for brink conditions ($y_e=(A/2)^{1/2}$). Select d_{50}/y_e appropriation for locally available riprap (usually the most satisfactory results will be obtained if $0.25<d_{50}/y_e<0.45$). Obtain h_s/y_e from figure XI-2, and check to see that $2<h_s/d_{50}<4$. Recycle computations if h_s/d_{50} falls out of this range.

5. Size basin as shown in figure XI-1.

6. Design procedures where allowable dissipator exit velocity is specified:

   a. Determine the average normal flow depth in the natural channel for the design discharge.

   b. Extended the length of the energy basin (if necessary) so that the width of the energy basin at section A-A, figures XI-1, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

7. In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

1. y_o, A, V_o
2. y_e
3. F
4. h_s
5. Dimensions

Steps

Design Example No. 3

Given: 6 feet diameter cmp, Q=135 cfs, S_o=0.004, Mannings n=0.024 normal depth in pipe for Q=135 cfs is 4.5 feet, normal velocity is 5.9 fps, flow is subcritical, tailwater depth (TW) is 2.0 feet.

Find: Riprap basin dimensions for these conditions:

find h_s = Key

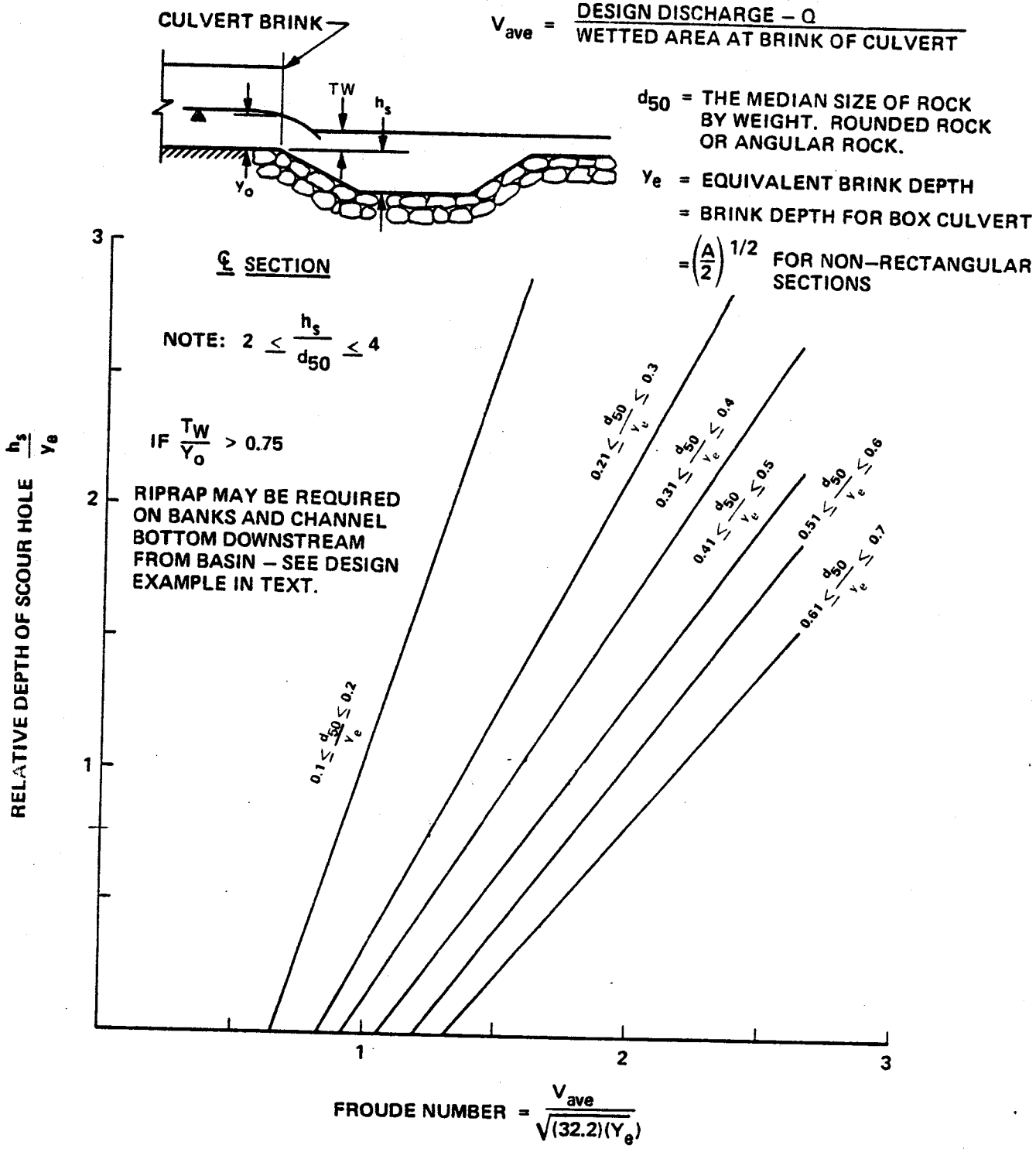


FIGURE X1–2. RELATIVE DEPTH OF SCOUR HOLE VERSUS FROUDE NUMBER AT BRINK OF CULVERT WITH RELATIVE SIZE OF RIPRAP AS A THIRD VARIABLE

Solution:

1. Determine y_o and V_o :

   $Q/D^{2.5}=135/(6)^{2.5}=1.53$
   $TW/D=2.0/6=0.33$

   From figure III-9, $y_o/D=0.45$

   $y_o=(0.45)(6)=2.7$ ft.
   $TW/y_o=2.0/2.70=0.74$ $TW/y_o<0.75$ O.K.
   Brink Area (A) for $y_o/D=0.45$ is
   $A=(0.343)(36)=12.3$ sq. ft.
   (0.343 is from table III-2) $\because \frac{A}{D^2}=0.343 \therefore A=0.343 \cdot D^2$
   $V_o=Q/A=135/12.3=11.0$ fps

2. $y_e=(A/2)^{1/2}=(12.3/2)^{1/2}=2.48$ ft.

3. $Fr=V_o/[(32.2)(y_e)]^{1/2}=11/[(32.2)(2.48)]^{1/2}=1.23$

4. Try $d_{50}/y_e=0.25$, $d_{50}=(0.25)(2.48)=0.62$ ft.
   From figure XI-2, $h_s/y_e=0.75$, $h_s=(0.75)(2.48)=1.86$ ft.
   check: $h_s/d_{50}=1.86/0.62=3$, $2<h_s/d_{50}<4$ O.K.

5. $L_s=(10)(h_s)=(10)(1.86)=18.6$ ft.
   or $L_s=(3)(W_o)=(3)(6)=18$ ft., Use $L_s=18.6$ ft. for dissipator pool.

   $L_B=(15)(h_s)=(15)(1.86)=27.9$ ft.
   or $L_B=(4)(W_o)=(4)(6)=24$ ft., Use $L_B=27.9$ ft. for basin (pool & apron).

   $d_{50}=0.62$ ft. use $d_{50}=8$ in.

Other basin dimensions are designed in accordance with details shown on figure XI-1.

The design procedure recommended in this chapter is a compromise between the design procedure utilizing the CSU experimentally derived functional relationships and traditional design methods for riprapped basins. It is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 2(d_{50}) thickness of riprap by the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, the damage should be superficial.

Concerning the use of filter material, several factors should be considered. Bank material adjacent to a culvert is not subjected to flow for long continuous periods. Also, the streambed material may be sufficiently well graded and not require a filter. If some siltation of the basin accompanied by plant growth is anticipated, it may be that a filter will not be required. If required, a filter cloth or filter material designed in accordance with instructions in reference XI-3 should be specified.

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

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius
Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.966
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

Drainage Channel Lining Design

VII. Example Problems*

Example Problem No. 1

This first example problem is presented to illustrate the basic use of the design charts and the concepts involved.

The objective is to design a channel lining for a trapezoidal channel with a 4 foot bottom width and 4:1 side slopes. Based on an analysis of the risks of channel failure, it is decided to design the permanent lining for a 10-year recurrence interval runoff and the temporary lining for the mean annual flood flow, with a recurrence interval of 2.33 years.

To determine the runoff rate, the Rational Equation is used for the 4.3 acre drainage area. The soil is judged to have an average erodibility. Due to right-of-way constraints, the channel top width must be restricted to 12 feet. Channel slope is 5 percent. Several permanent and temporary channel lining materials are available.

Detailed calculations are shown in Figure 5. Note that the bare soil would convey very little flow on this 5-percent slope without severe erosion. Bermuda grass or rock riprap are adequate. Since 6-inch Bermuda grass is the lining chosen, temporary linings are evaluated and either a double layer of fiber glass roving and asphalt or excelsior mat are adequate to convey the mean annual flow rate of 5.0 cfs.

Should the grass be permitted to grow to a 12-inch length, the retardance of the channel would be increased. Then, the channel may not convey the 10-year runoff without overtopping its banks. A check of the 12-inch Bermuda grass reveals that d_{max} is greater than 1.0 ft., so that the top width of the flow exceeds 12 feet. Therefore, a 1.0 ft. depth of flow is used to check the channel capacity, which is found to be 15.2 cfs.

The concrete lining has no d_{max}. From Chart 35, it is found that a 1.0 ft. depth of flow in the concrete lining at a 5-percent slope would convey 154 cfs at a velocity of 19 fps. This is the hydraulic advantage and disadvantage of a concrete lining in a nutshell: high capacity coupled with a high, erosive outlet velocity.

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* Normann, J. M., "Design of Stable Channels with Flexible Linings", Federal Highway Administration, U.S. Dept. of Transportation, Washington, D.C., Hydraulic Engineering Circular No. 15, 1975.

Figure 5

DRAINAGE CHANNEL LINING DESIGN

DATE: 11/9/72

PROJECT: FAI - I (87) DESIGNER: JMN

STATION 105 + 40 TO STATION 112 + 80

DRAINAGE AREA = 4.3 ACRES

HYDROLOGIC COMPUTATIONS:

10 year flow

$Q = CiA = (0.7)(5.0)(4.3) = 15$ cfs

Mean Annual flow

$Q = CiA = (0.7)(1.6)(4.3) = 4.8$ cfs

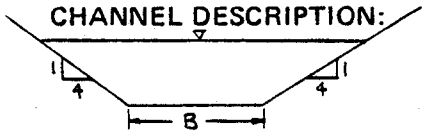
Use 5 cfs

DESIGN FLOW: Q 10 = 15 cfs

DESIGN FLOW FOR TEMPORARY LINING: Q 2.33 = 5 cfs

SOIL ERODIBILITY: Average

CHANNEL DESCRIPTION:



MAX. TOP WIDTH = 12 ft. $T = B + (Z_1 + Z_2)d$
$= 4 + 8d$

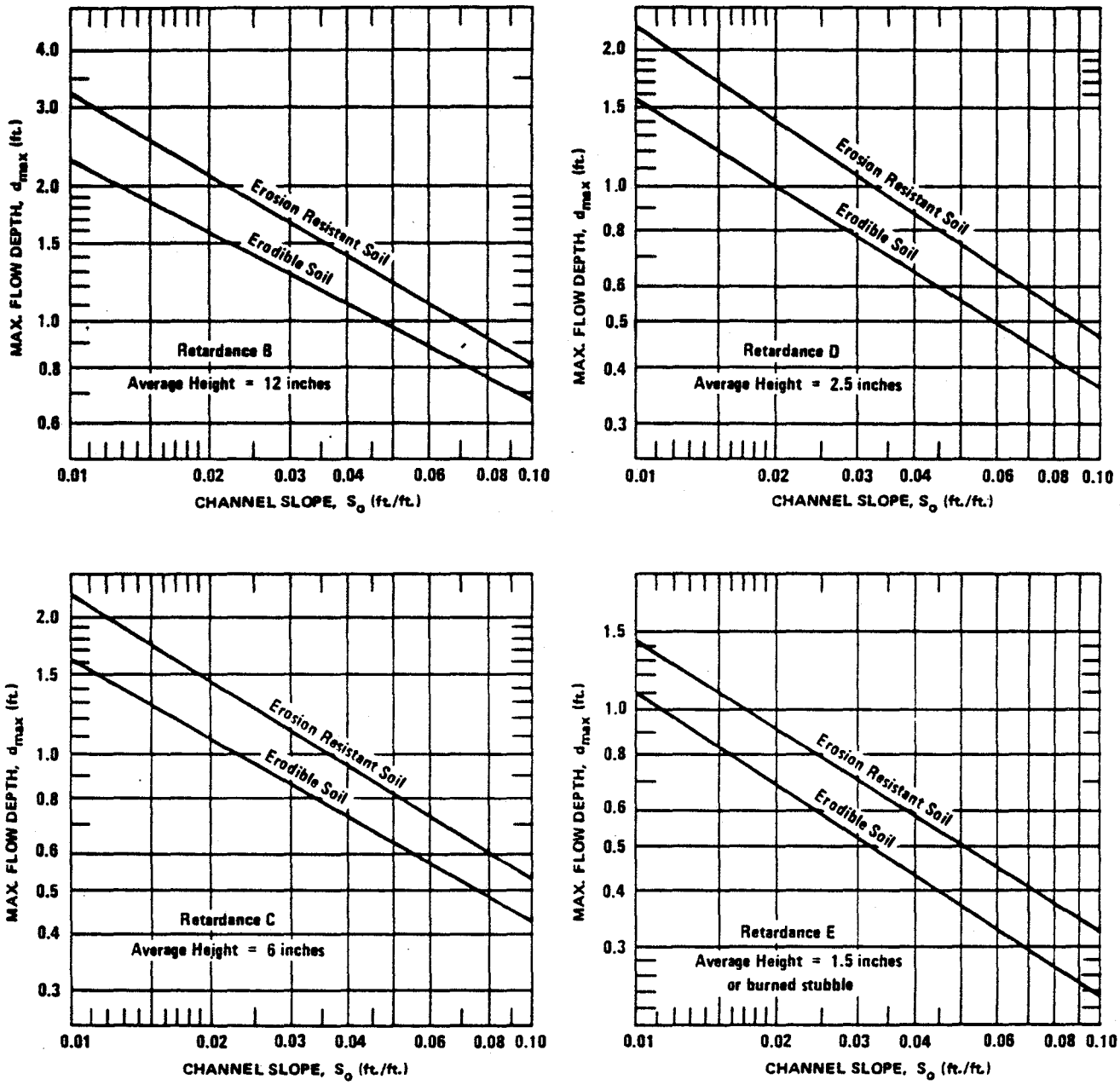
S_o = 0.05

AVAILABLE LININGS: Permanent
1. Bare Soil
2. Bermuda Grass (6")
3. Riprap (3", 6", 12")

Temporary
1. Fiber Glass Roving
2. Excelsior Mat

LINING	d_{max}	B	$\frac{d_{max}}{B}$	$\frac{A}{Bd}$	A	$\frac{R}{d}$	R	V	Q=AV	T	REMARKS
				chart 1		chart 1					
Bare Soil	0.04	4	0.01	—	0.17	—	0.038	1.40	0.24	4.3	No Good
6" Bermuda	0.71	4	0.18	1.7	4.83	0.69	0.49	3.3	15.9	9.7	OK
Grass (retard. c)											
3" Riprap	0.40	4	0.10	1.4	2.24	0.78	0.31	4.9	11.0	7.2	No Good
6" Riprap	0.80	4	0.20	1.8	5.76	0.68	0.54	6.3	36.3	10.4	OK over designed
Therefore,	use	6"	Bermuda	Grass							
Temporary	Linings										
Fiber Glass											
1 Layer	0.16	4	0.04	1.16	0.74	0.87	0.14	2.5	1.85	5.3	No Good
2 Layers	0.40	4	0.10	1.35	2.16	0.77	0.31	6.0	13.0	7.2	OK
Excelsior Mat	0.45	4	0.11	1.4	2.52	0.76	0.34	2.7	6.8	7.6	OK
Check	Capacity	of	12"	Bermuda	Grass						
12" Berm. Grass	1.1	4	0.275	2.1	9.2	0.64	0.70	2.5	23.0	12.8	T>12', Use d=1
	1.0	4	0.25	2.0	8.0	0.66	0.66	1.9	15.2	12.0	
Concrete	1.0*							19	154		High Velocity
*No d_{max}											Rigid Channel

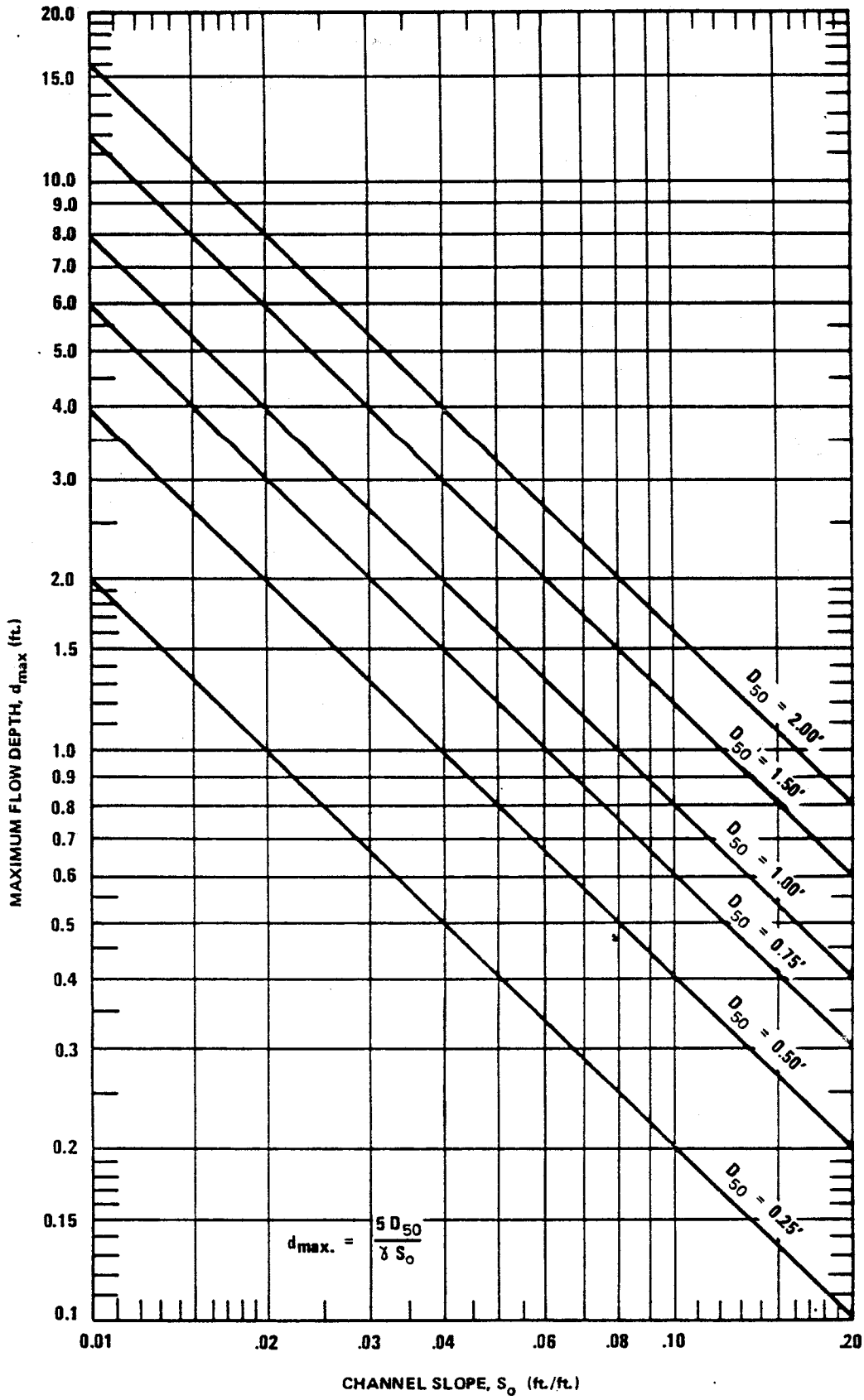
Chart 19



MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH BERMUDA GRASS. GOOD STAND, CUT TO VARIOUS LENGTHS

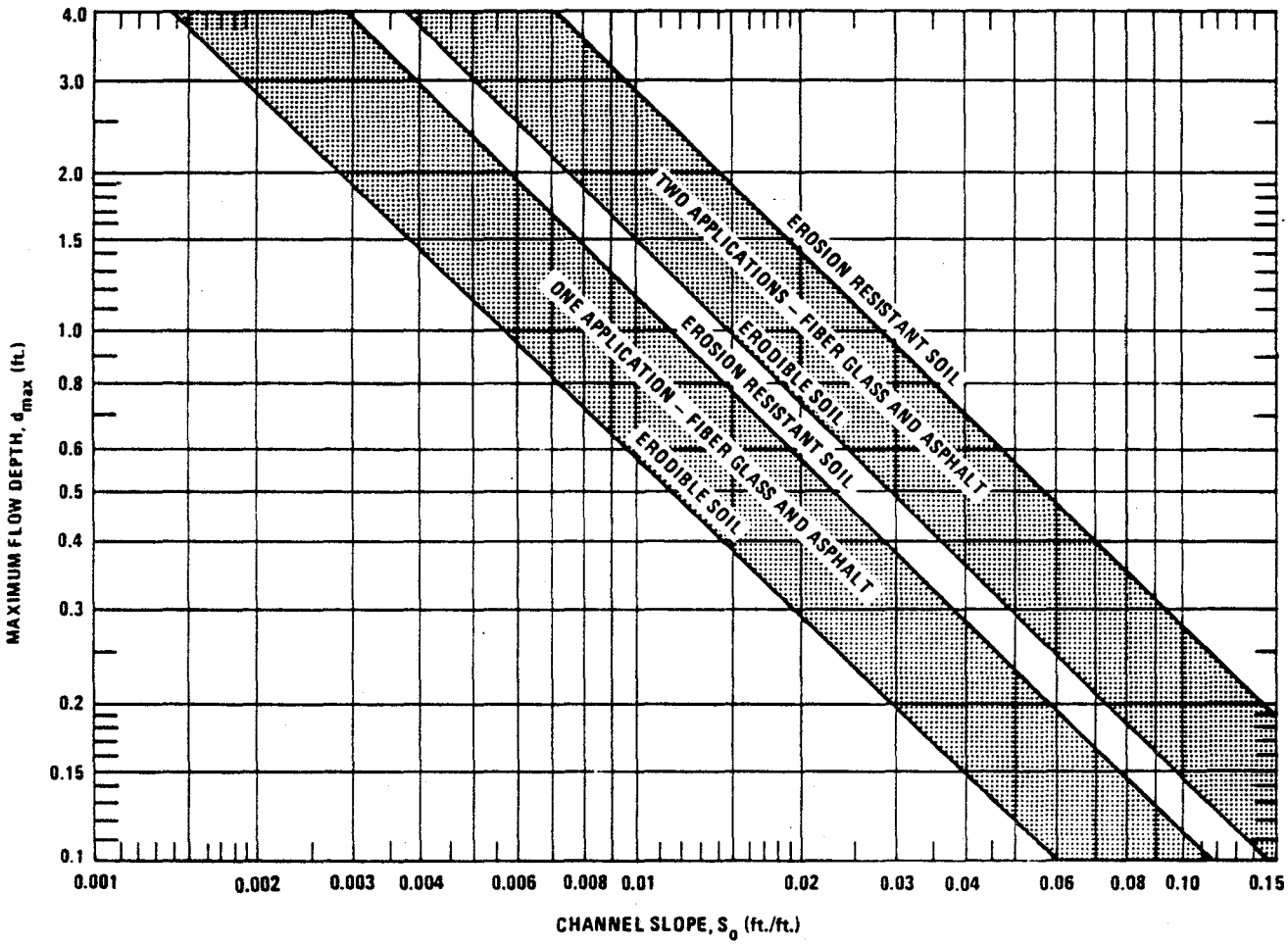
Note: Use on slopes steeper than 10 percent is not recommended

Chart 27

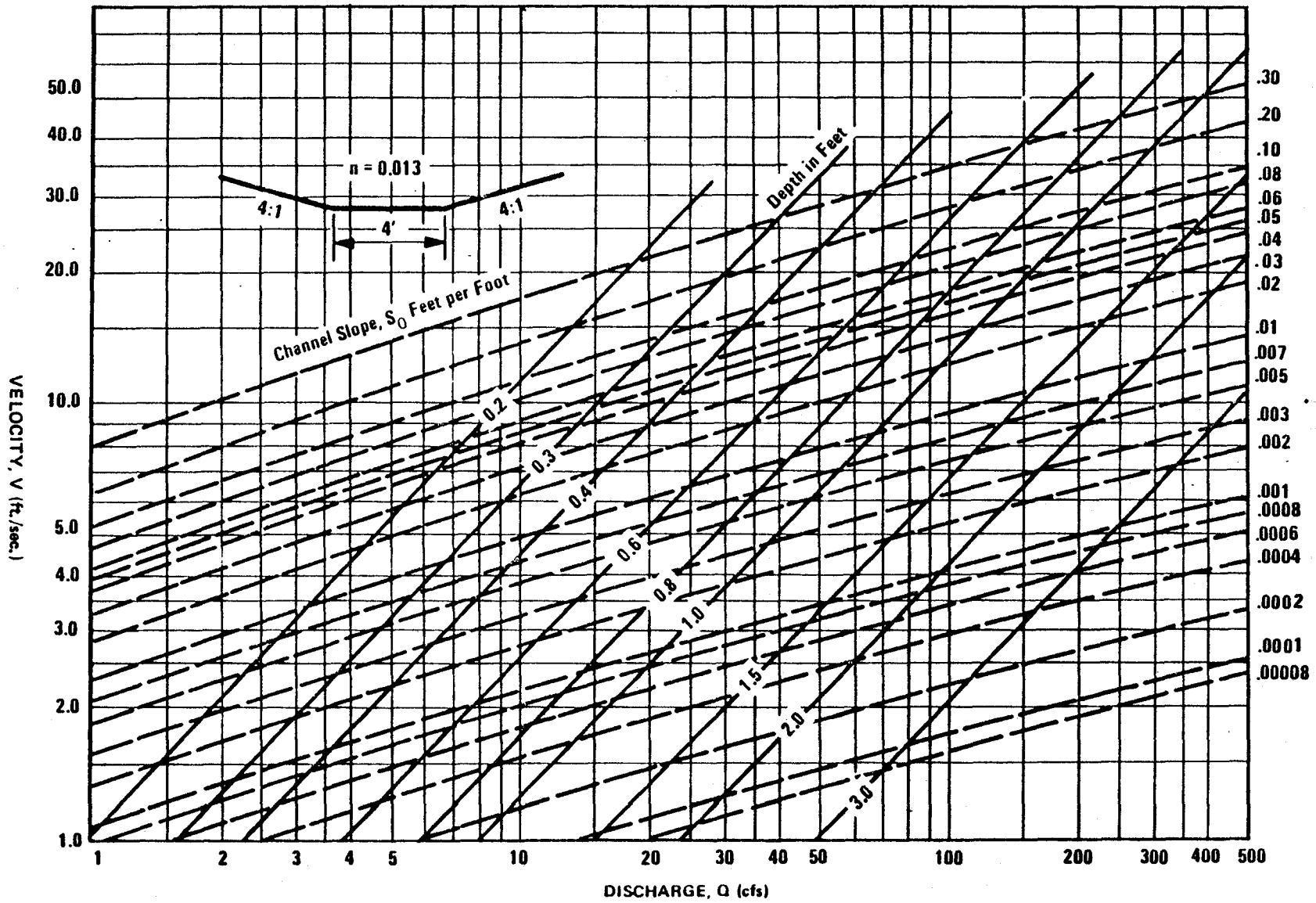


MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max})
FOR CHANNELS LINED WITH ROCK RIPRAP

Chart 4



MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH FIBER GLASS ROVING (SINGLE AND DOUBLE LAYER)



From "Design of Stable Roadside Channels," by the State Highway Commission of Kansas, 1972.

CHANNEL CAPACITY CHART FOR TRAPEZOIDAL CONCRETE CHANNEL

Example Problem No. 2

Problem 2 illustrates the design of a rock riprap lining for channel side slopes steeper than 3:1.

Assume a trapezoidal channel with bottom width of 4 feet, permissible depth of flow of 4 feet, and side slopes of 1.5:1. Slope = 0.01.

1. From design method $(D_{50})_{bottom} = 0.5$ feet. The available stone is classified as crushed rock.

2. Determine the angle of repose. From Chart 30, the angle of repose, $\theta = 42^{o}$.

3. Determine the ratio of maximum side shear to maximum bottom shear. From Chart 31, with $B/d = 1.0$ and $Z = 1.5$, $K_1 = 0.88$.

4. Determine the ratio of critical side shear to critical bottom shear. From Chart 32, with $Z = 1.5$, and $\theta = 42^{o}$, $K_2 = 0.53$.

5. Determine the adjusted rock size for the channel sides.

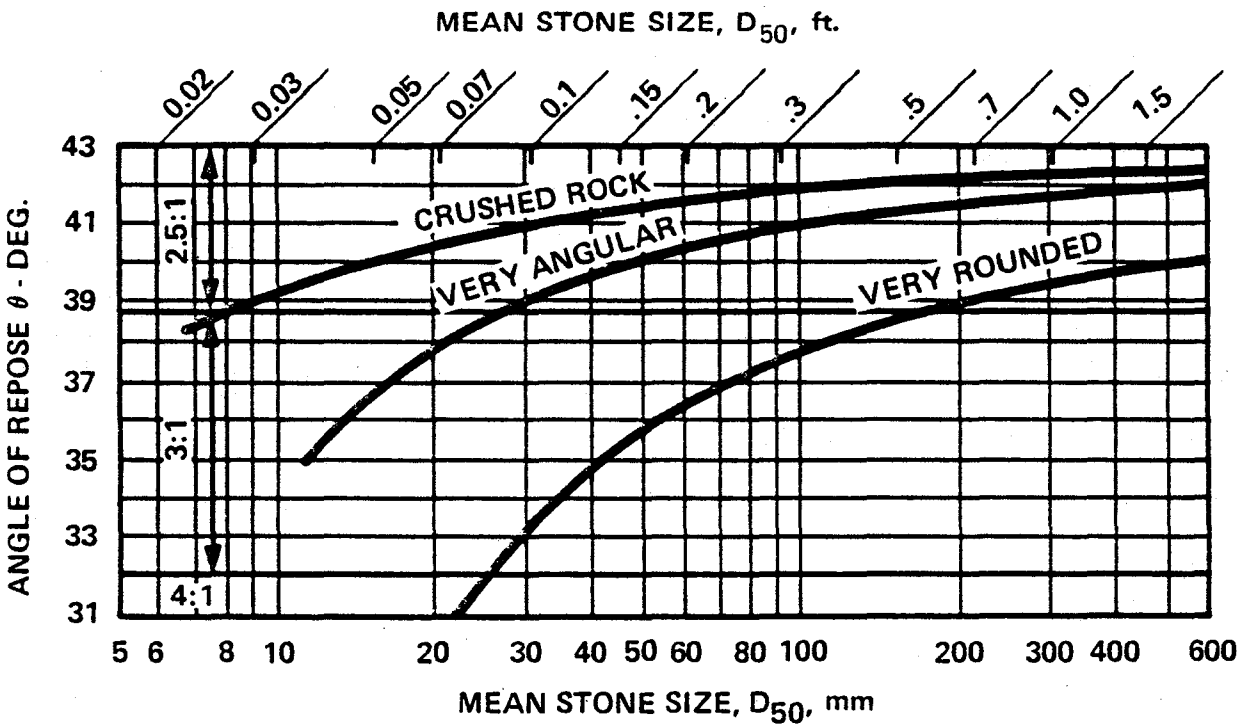
$$(D_{50})_{sides} = \frac{0.88}{0.53} (D_{50})_{bottom}$$

$$= \frac{0.88}{0.53} (0.5) = 0.83 \text{ ft.}$$

<u>Caution</u>: If the angle of the channel side slope exceeds the rock angle of repose, the channel sides are unstable at any flow rate.

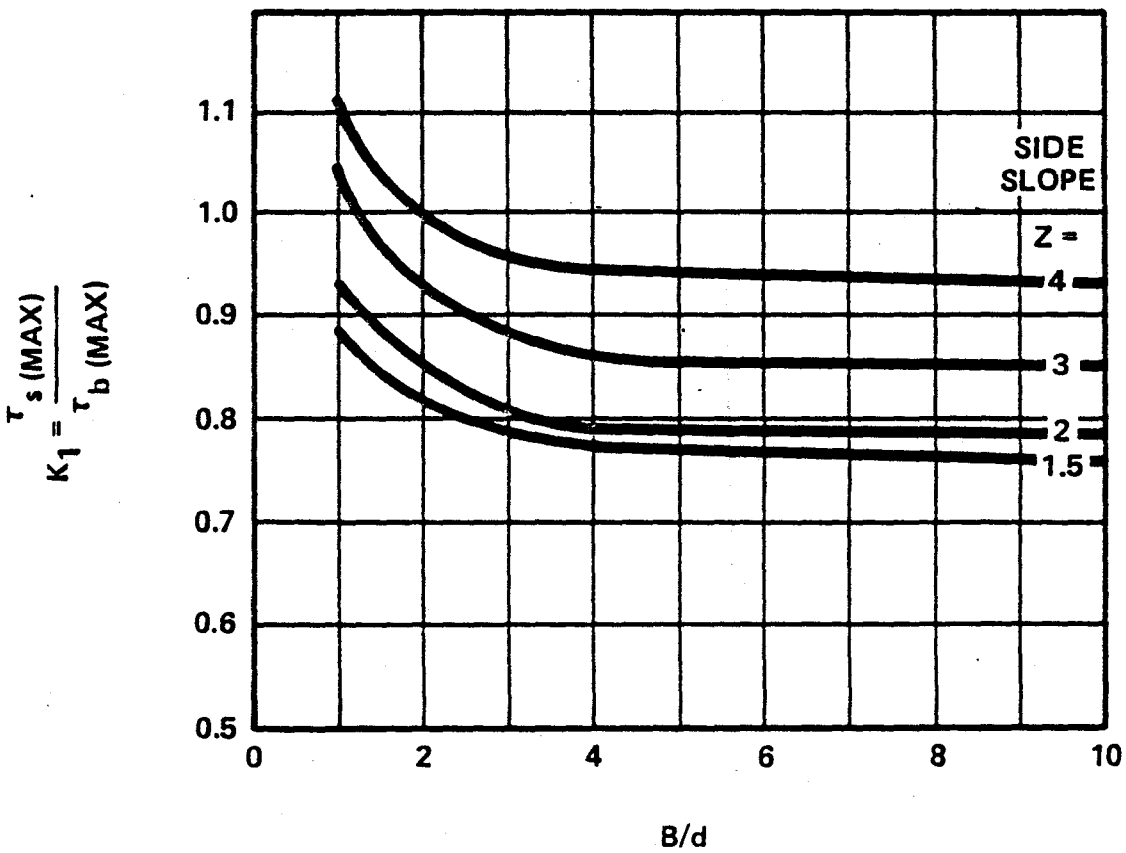
τ_c = Critical Shear Stress; $\frac{\tau_b}{\tau_s}$ $\frac{\tau_{cb}}{\tau_{cs}}$

Chart 30



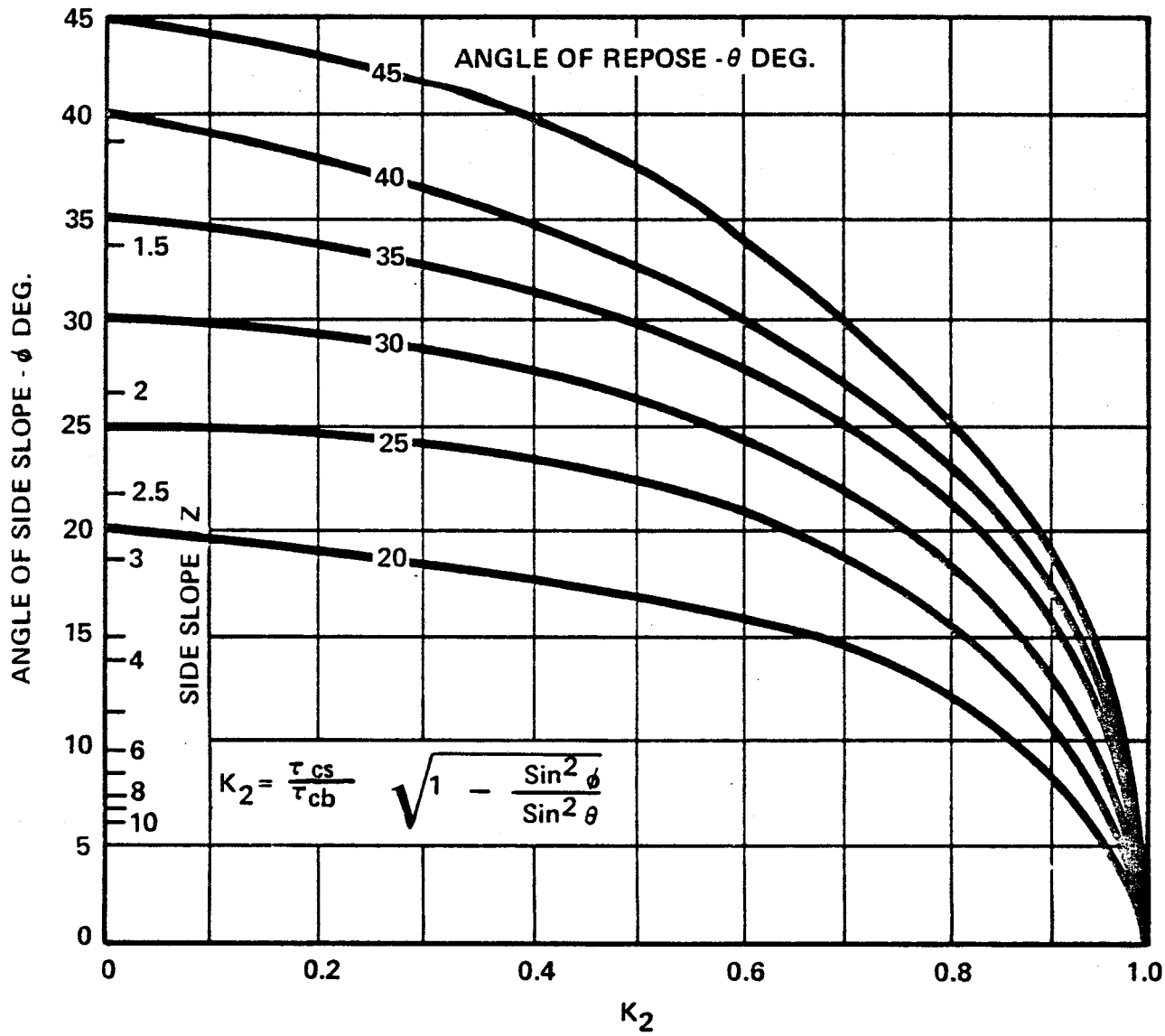
ANGLE OF REPOSE OF RIPRAP IN TERMS OF MEAN SIZE AND SHAPE OF STONE

Chart 31



DISTRIBUTION OF BOUNDARY SHEAR AROUND WETTED PERIMETER OF TRAPEZOIDAL CHANNELS

Chart 32



RATIO OF CRITICAL SHEAR ON SIDES TO CRITICAL SHEAR ON BOTTOM FOR NONCOHESIVE SEDIMENT

$J = \frac{y_2}{y_1}$

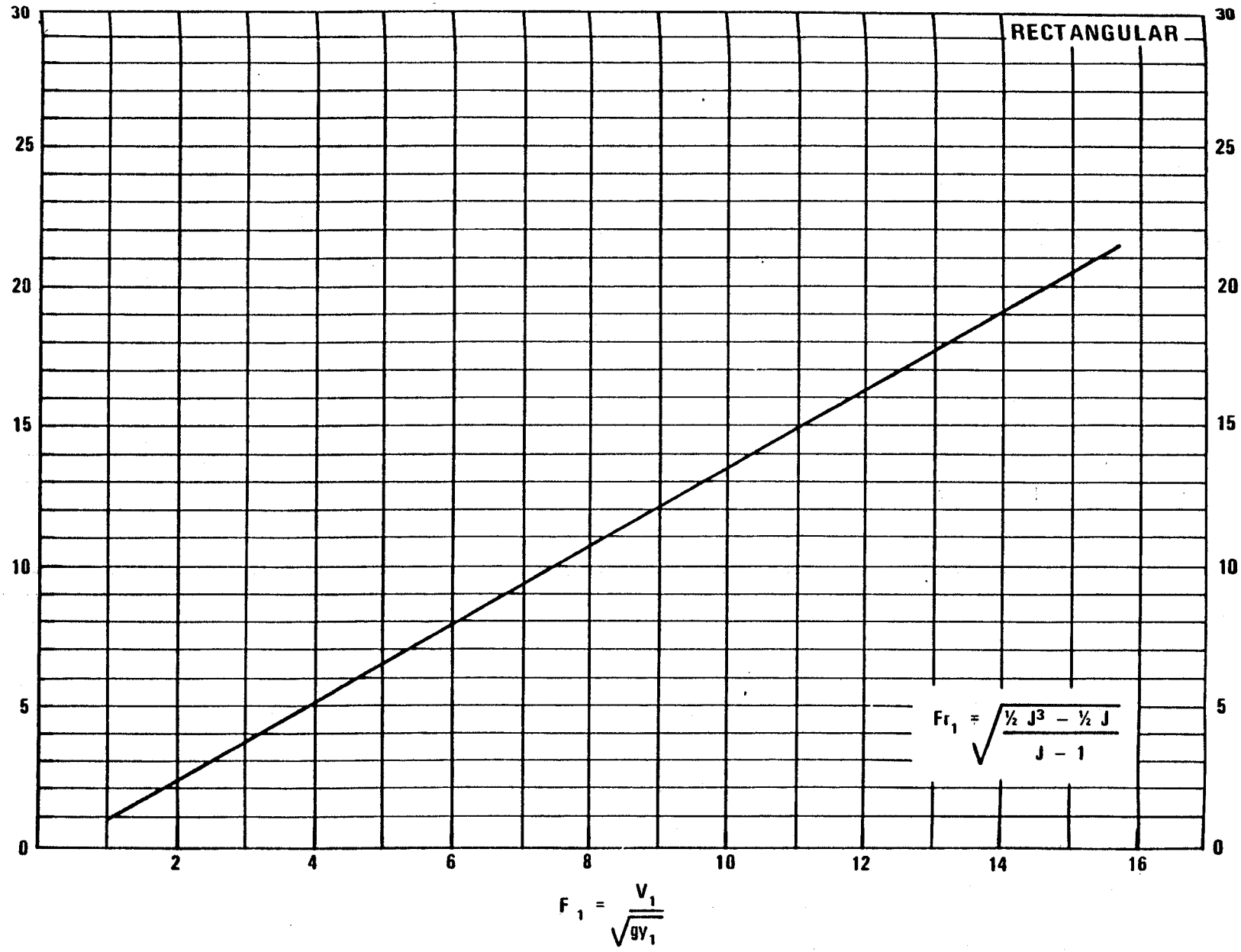


FIGURE VI-4 HYDRAULIC JUMP – HORIZONTAL, RECTANGULAR CHANNEL

* Corry, M. L., et. al., "Hydraulic Design of Energy Dissipators for Culverts and Channels", Federal Highway Administration. U.S. Dept. of Transportation, Washington, D.C., Hydraulic Engineering Circular No. 14, 1975.

Hydraulic Jump:

1. Pre-jump condition, then find $y_1 < y_c > y_2$
2. what is head loss.
3. length of jump
4. location of jump (tailwater effect this)

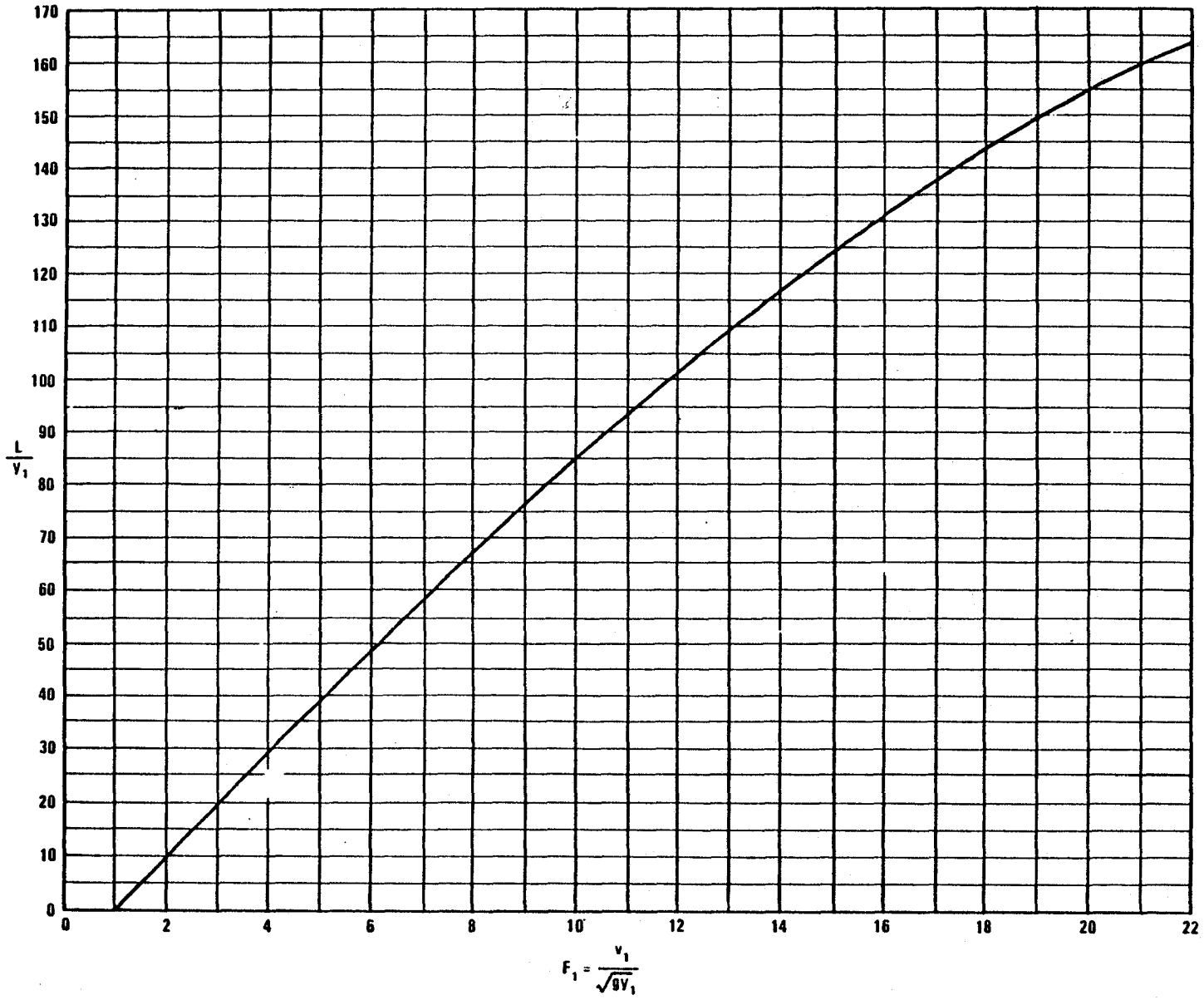


FIGURE VI-11. LENGTH OF JUMP IN TERMS OF y_1, RECTANGULAR CHANNEL

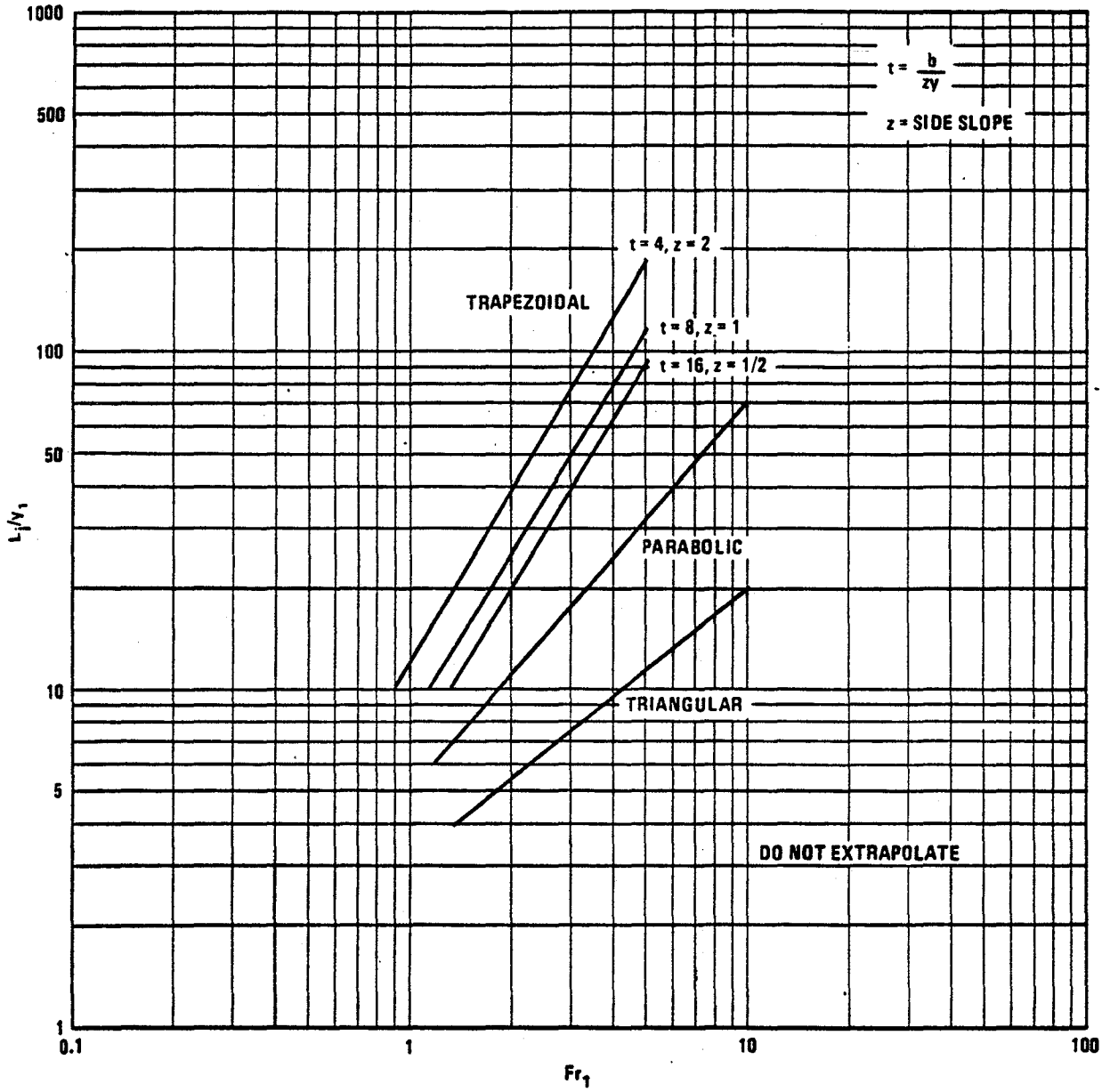


FIGURE VI-12. HYDRAULIC JUMP LENGTH FOR NONRECTANGULAR CHANNELS

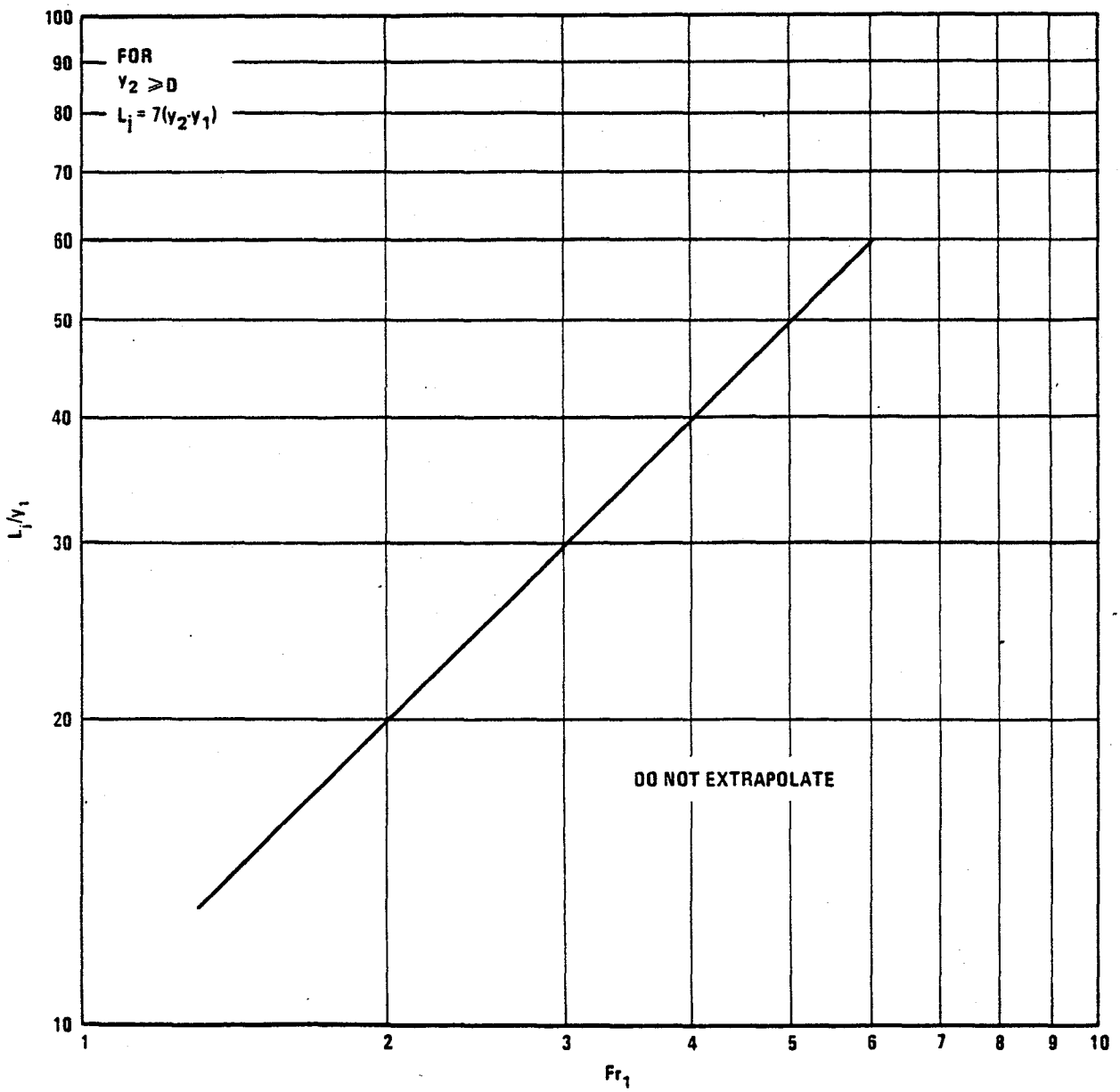


FIGURE VI-13. JUMP LENGTH CIRCULAR CHANNEL WITH $y_2 < D$.

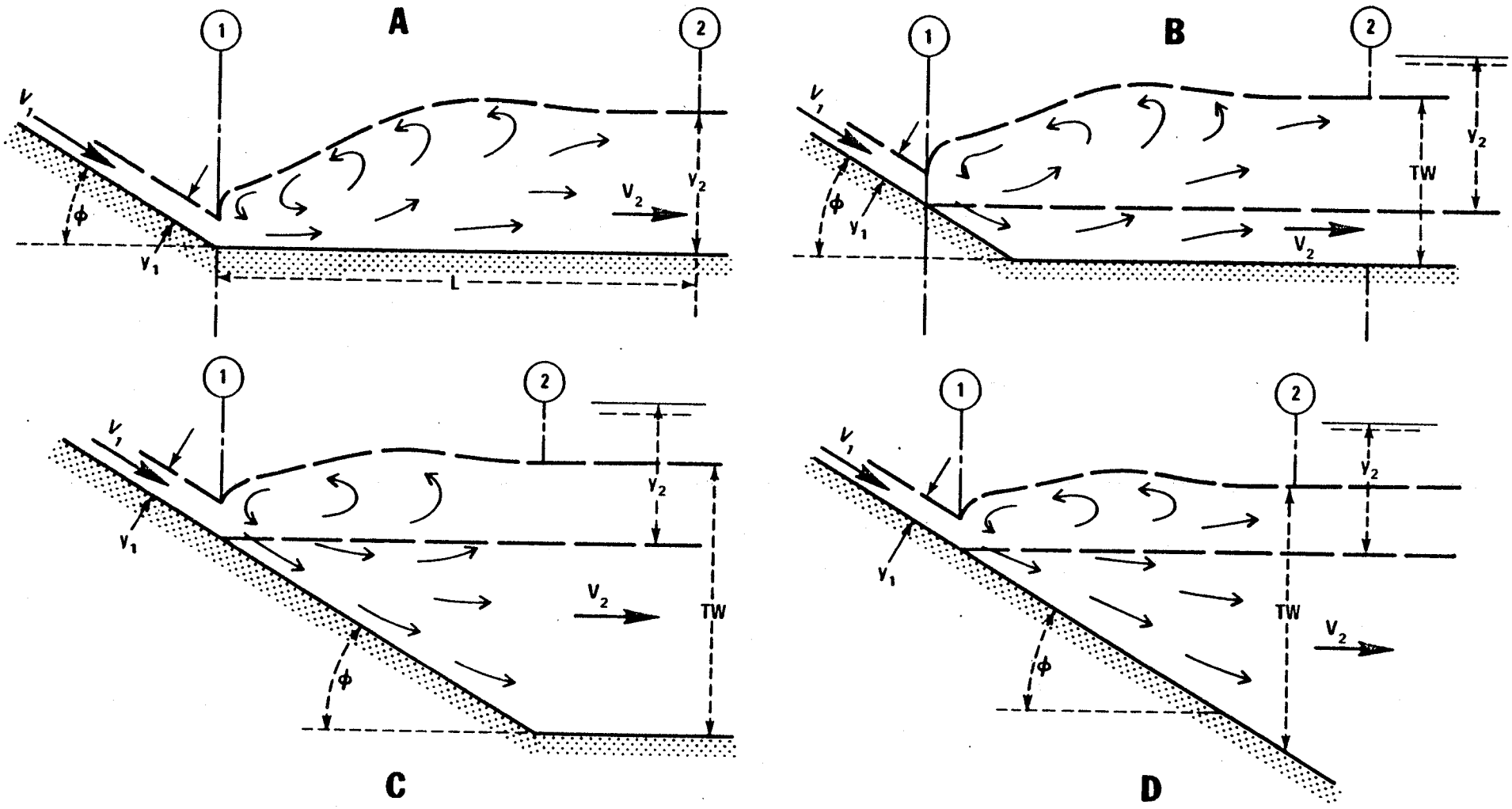


FIGURE VI-14. HYDRAULIC JUMP TYPES SLOPING CHANNELS, FROM REFERENCE VI-2

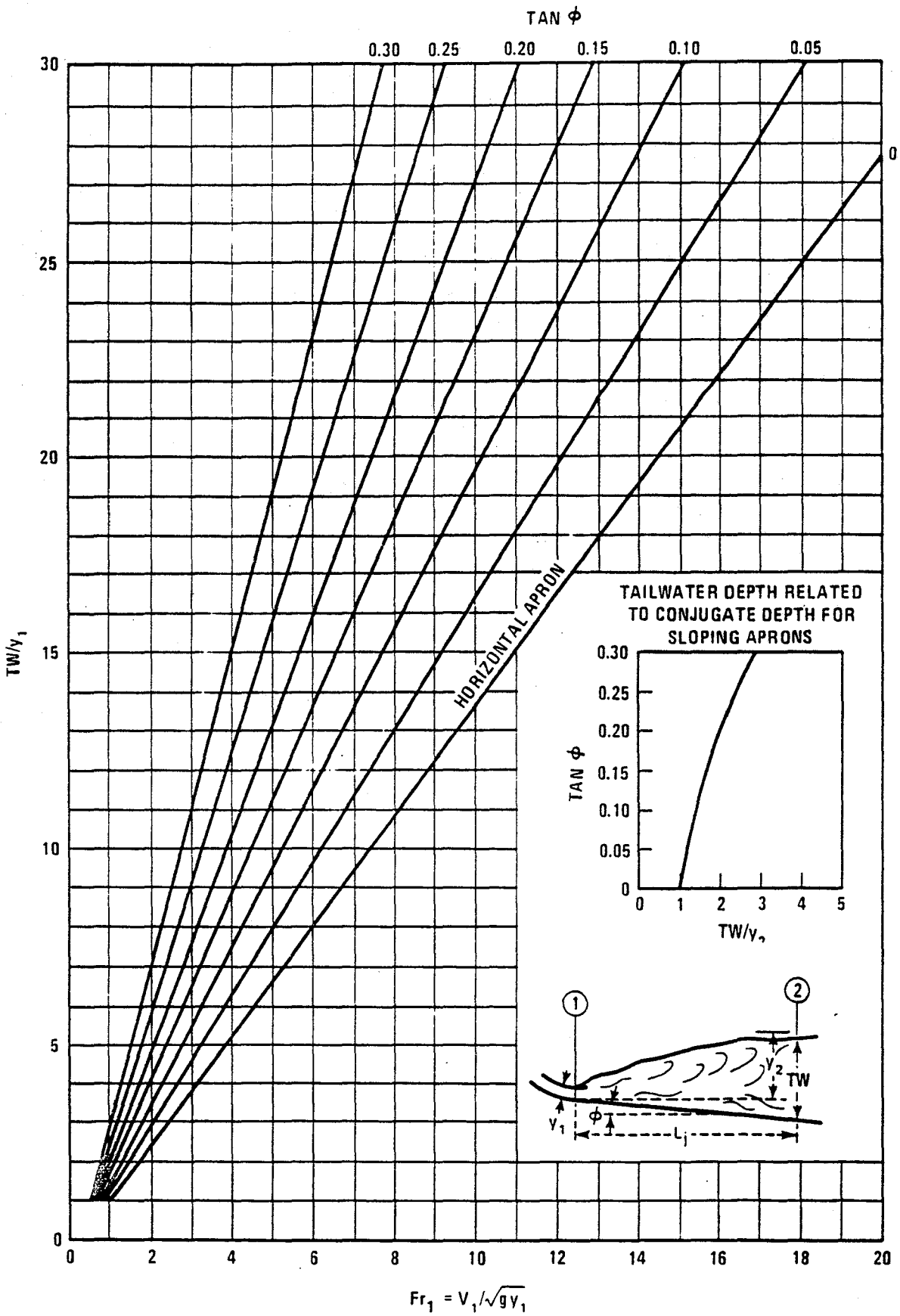


FIGURE VI-15. STILLING BASIN, CASE D, HYDRAULIC JUMP ON SLOPING APRON RATIO OF TAILWATER DEPTH TO y_1, FROM REFERENCE VI-2

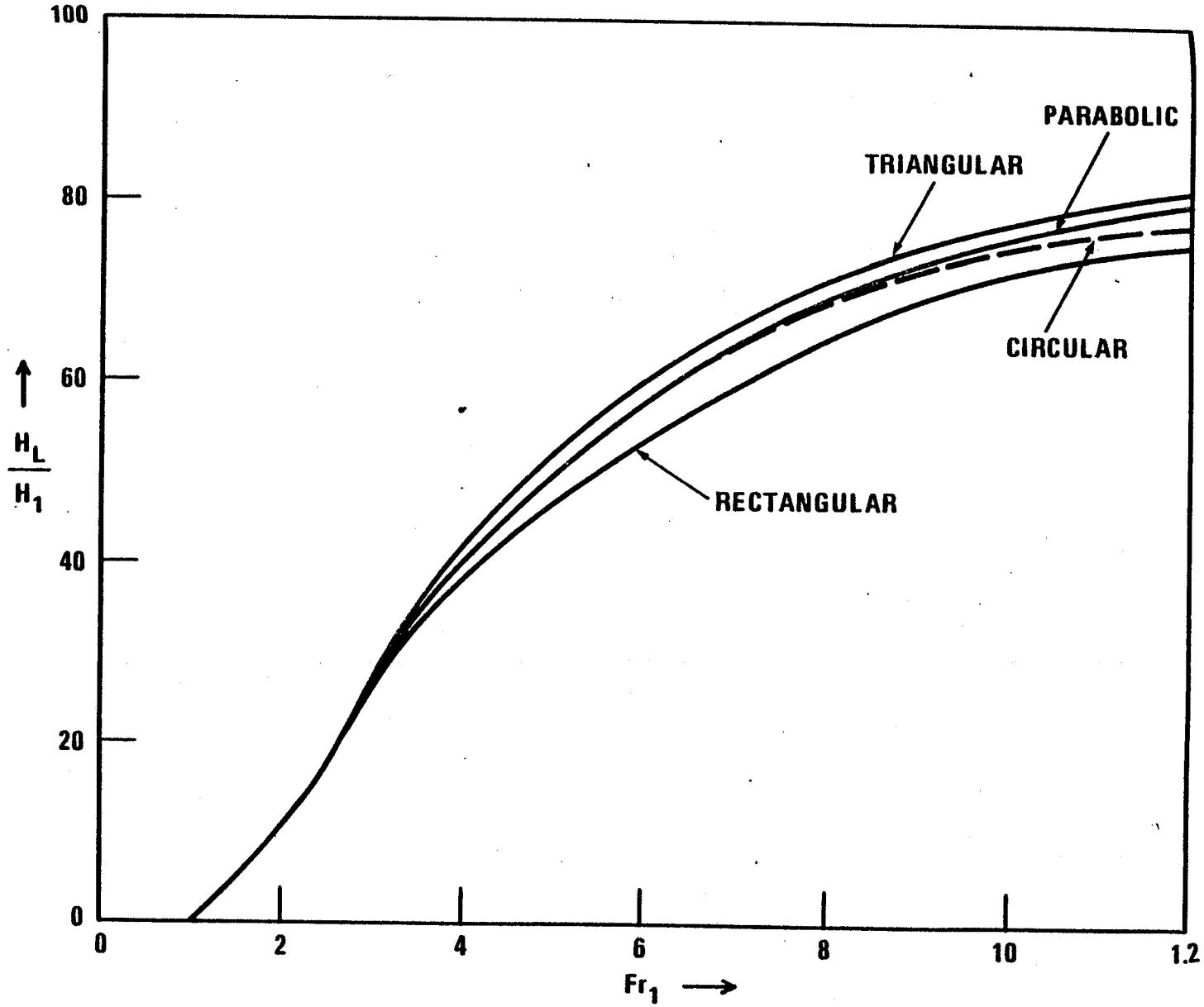


FIGURE VI-20. RELATIVE ENERGY LOSS FOR VARIOUS CHANNEL SHAPES.

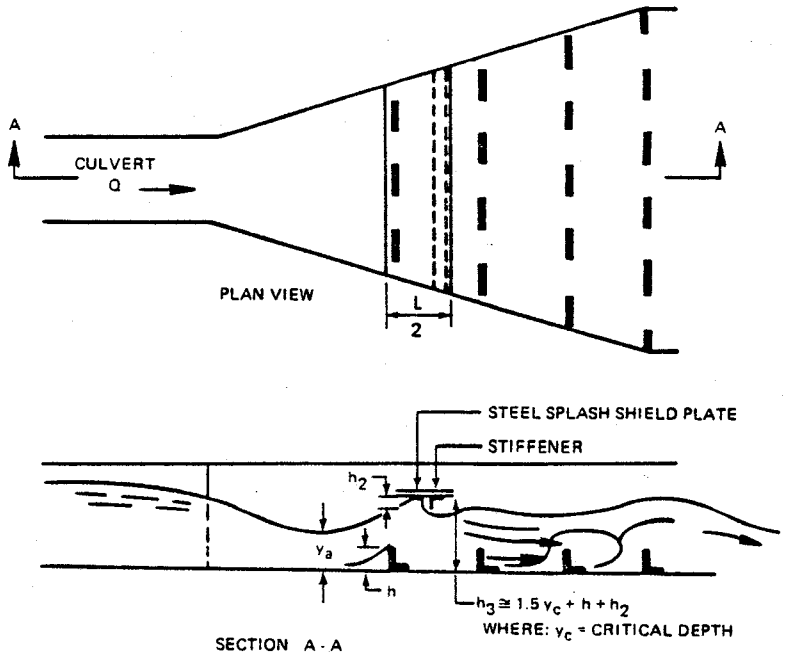


FIGURE VII-A-3. SPLASH SHIELD

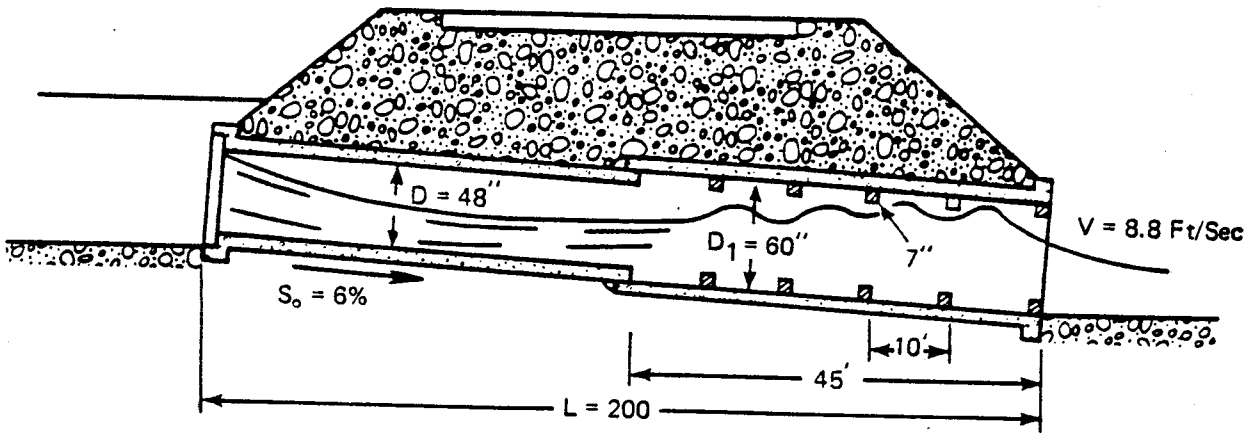


FIGURE VII-B-6. SKETCHED SOLUTION FOR THE TUMBLING FLOW DESIGN EXAMPLE FOR CIRCULAR CULVERTS

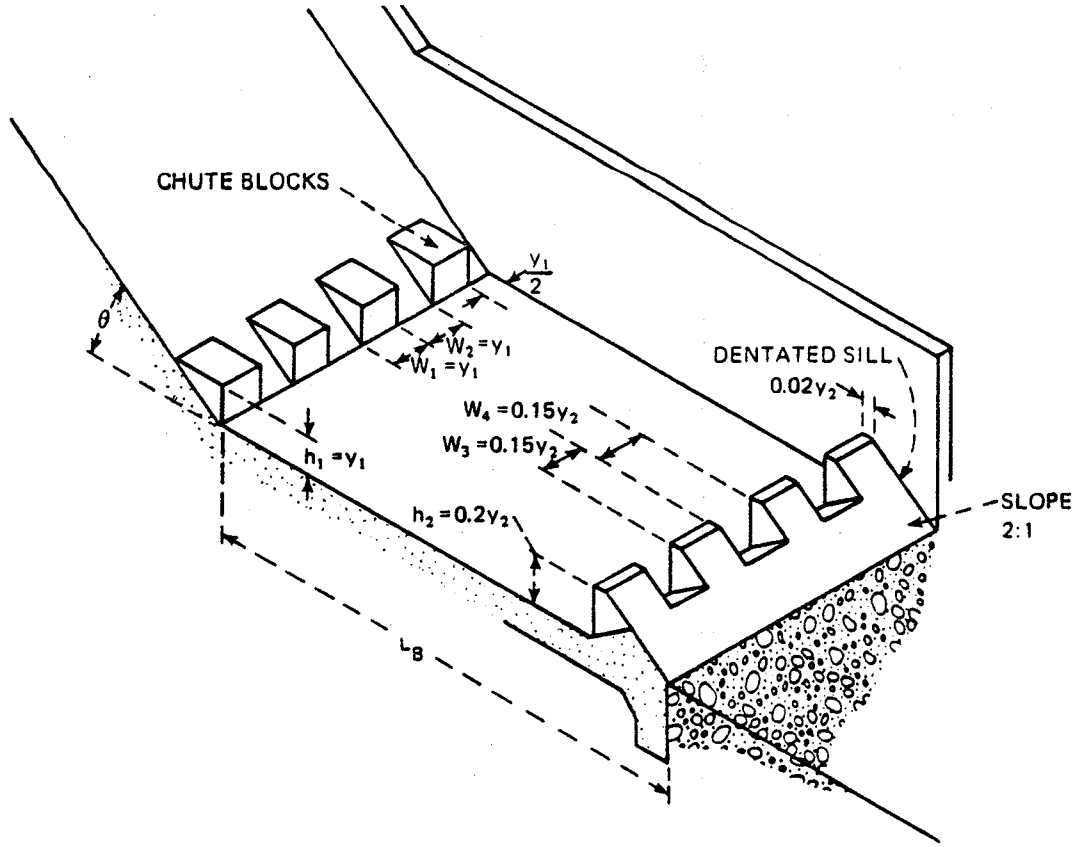


FIGURE VII – D-1 USBR TYPE II BASIN

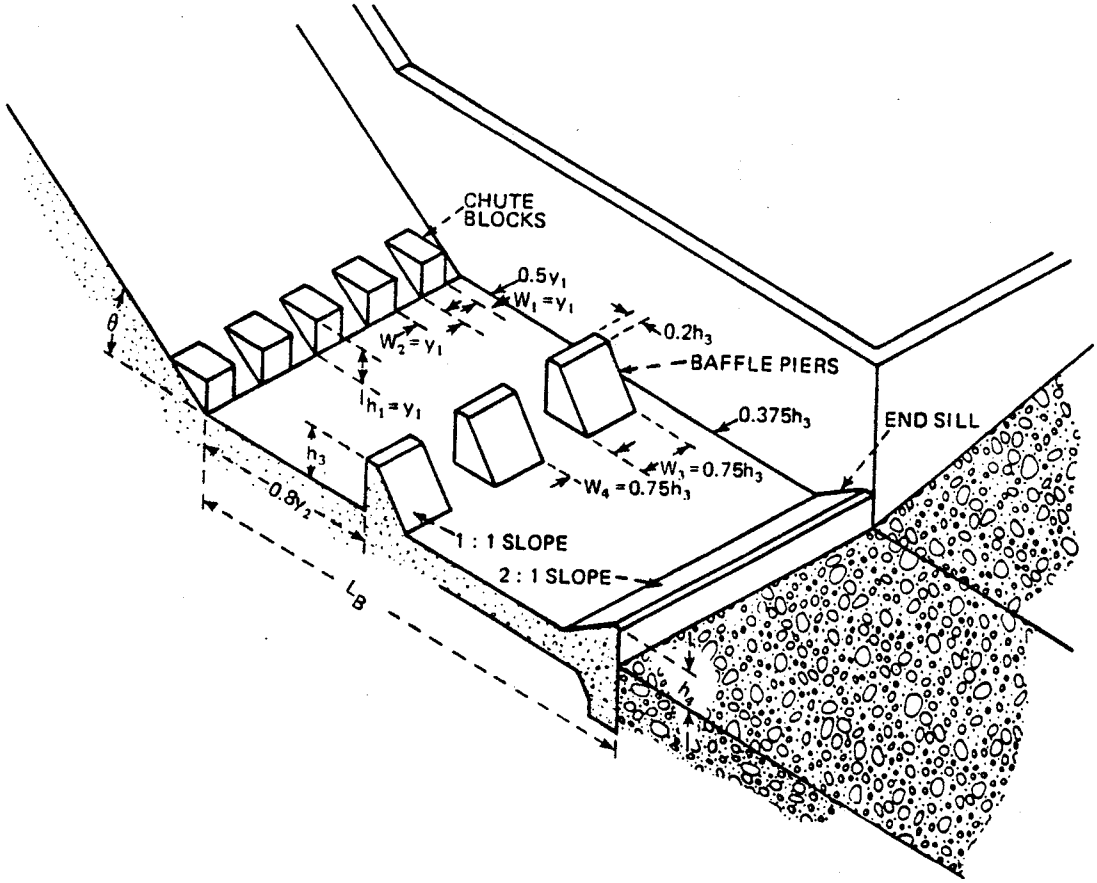


FIGURE VII - E - 1 USBR TYPE III BASIN

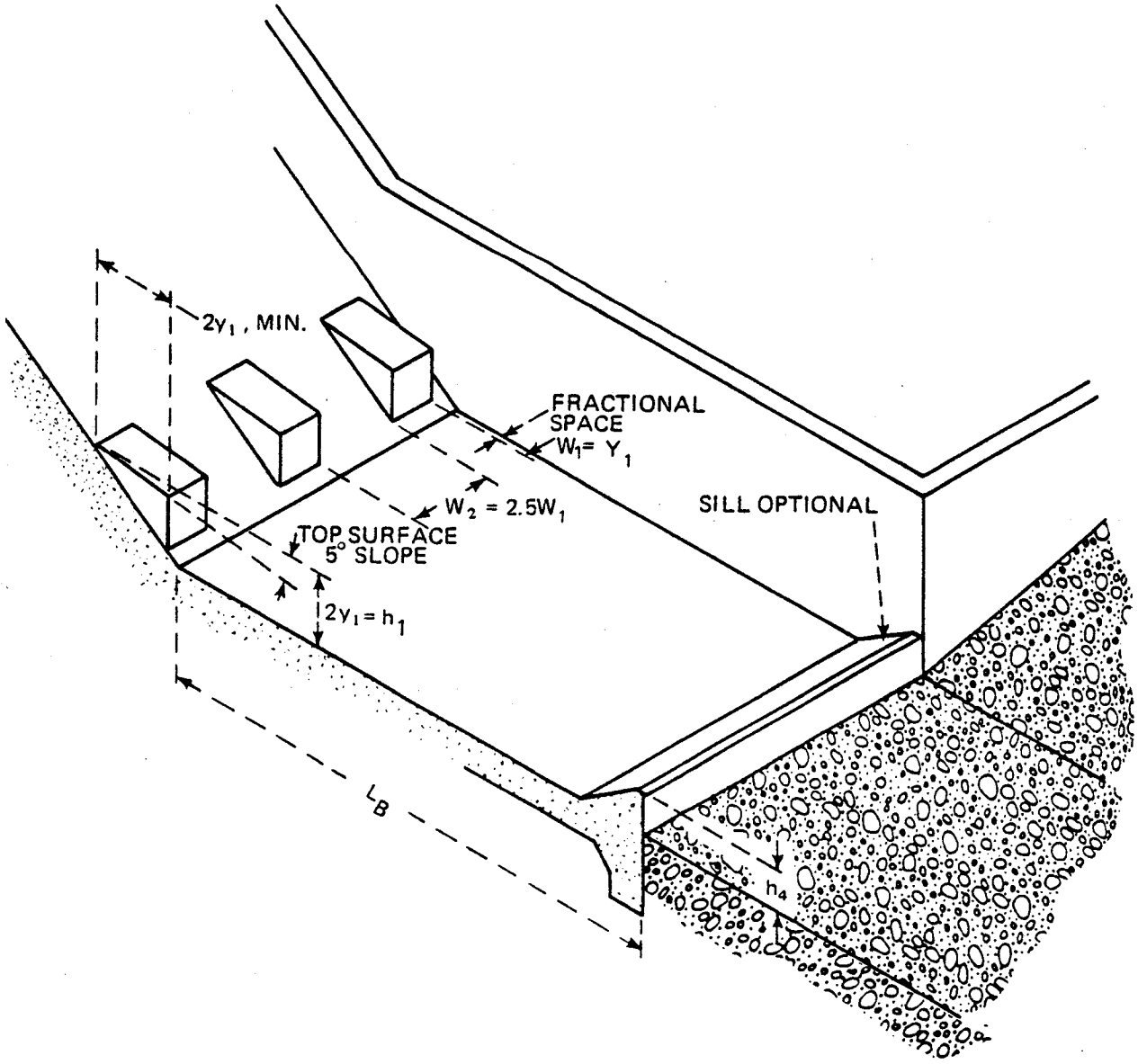
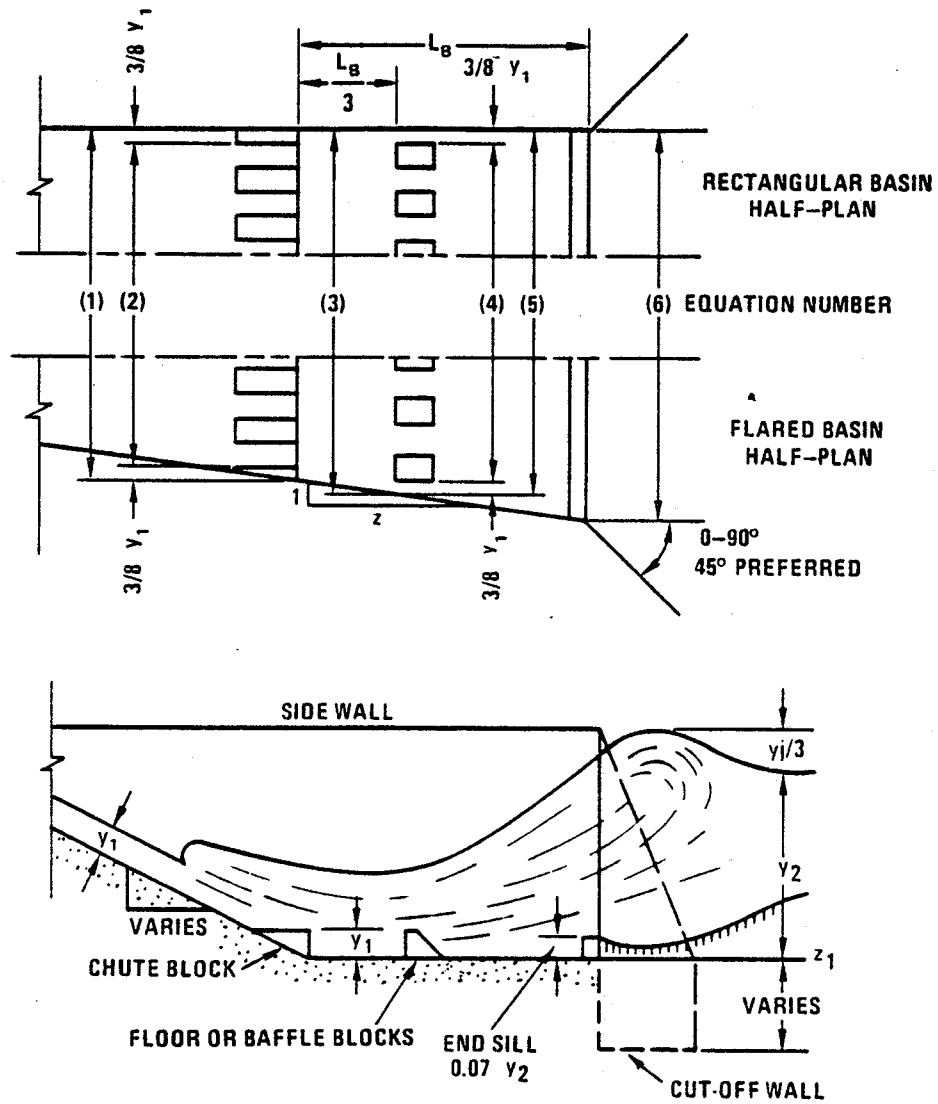


FIGURE VII - F - 1 USBR TYPE IV BASIN



(1) W_B = BASIN WIDTH UPSTREAM

(2) n BLOCKS AT 3/4 y_1 ±

(3) 0.40 W_{B2} ≤ AGGREGATE BLOCK WIDTH ≤ 0.55 W_{B2}

(4) n BLOCKS AT 3/4 y_1 $\frac{W_{B2}}{W_B}$ ±

(5) $W_{B2} = W_B + 2L_B/3z$

(6) $W_{B3} = W_B + 2L_B/z$

FIGURE VII-G-1. SAF STILLING BASIN–FROM REFERENCE VII-G-1

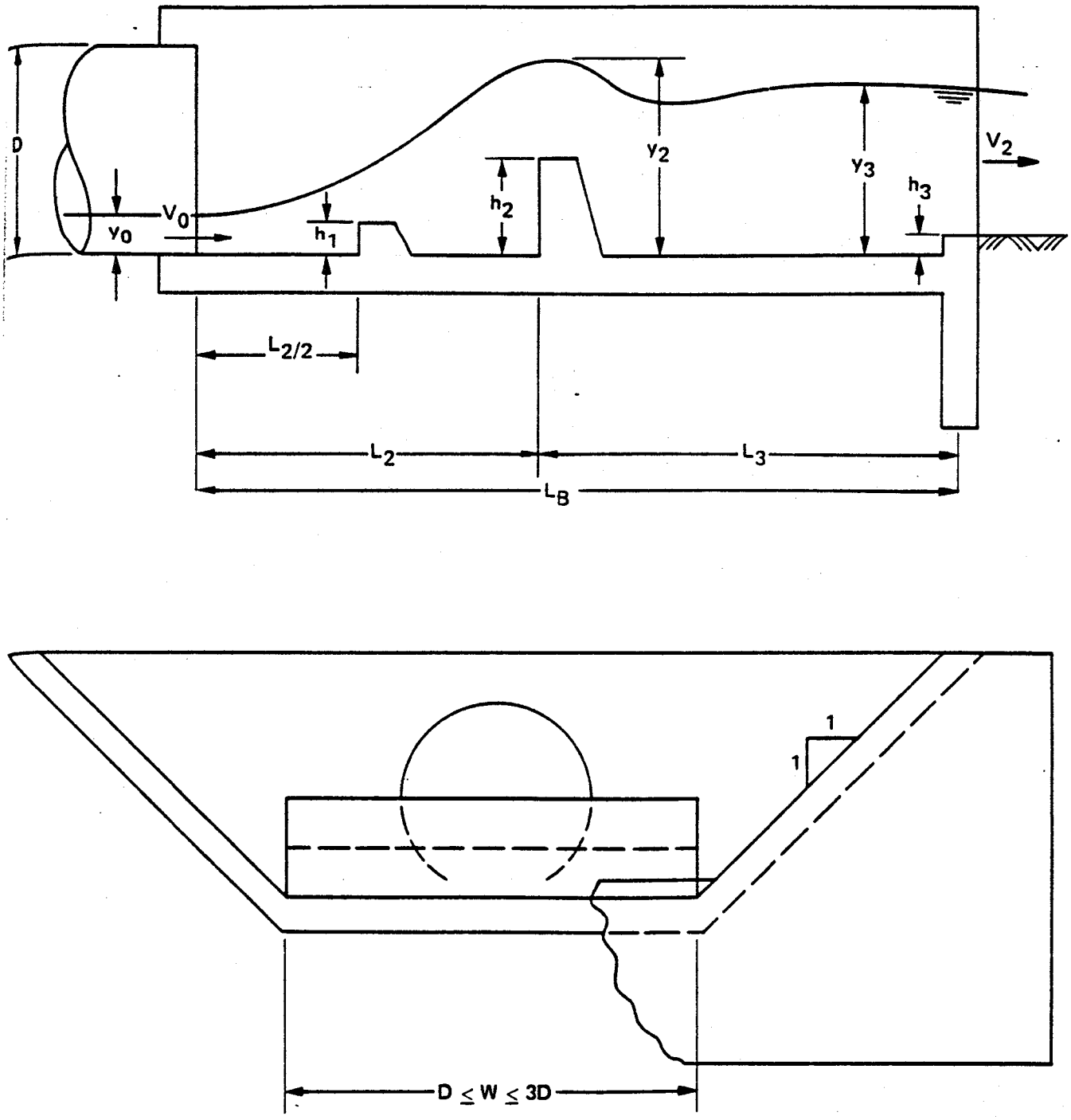


FIGURE VIII–A–1. CONTRA COSTA ENERGY DISSIPATOR

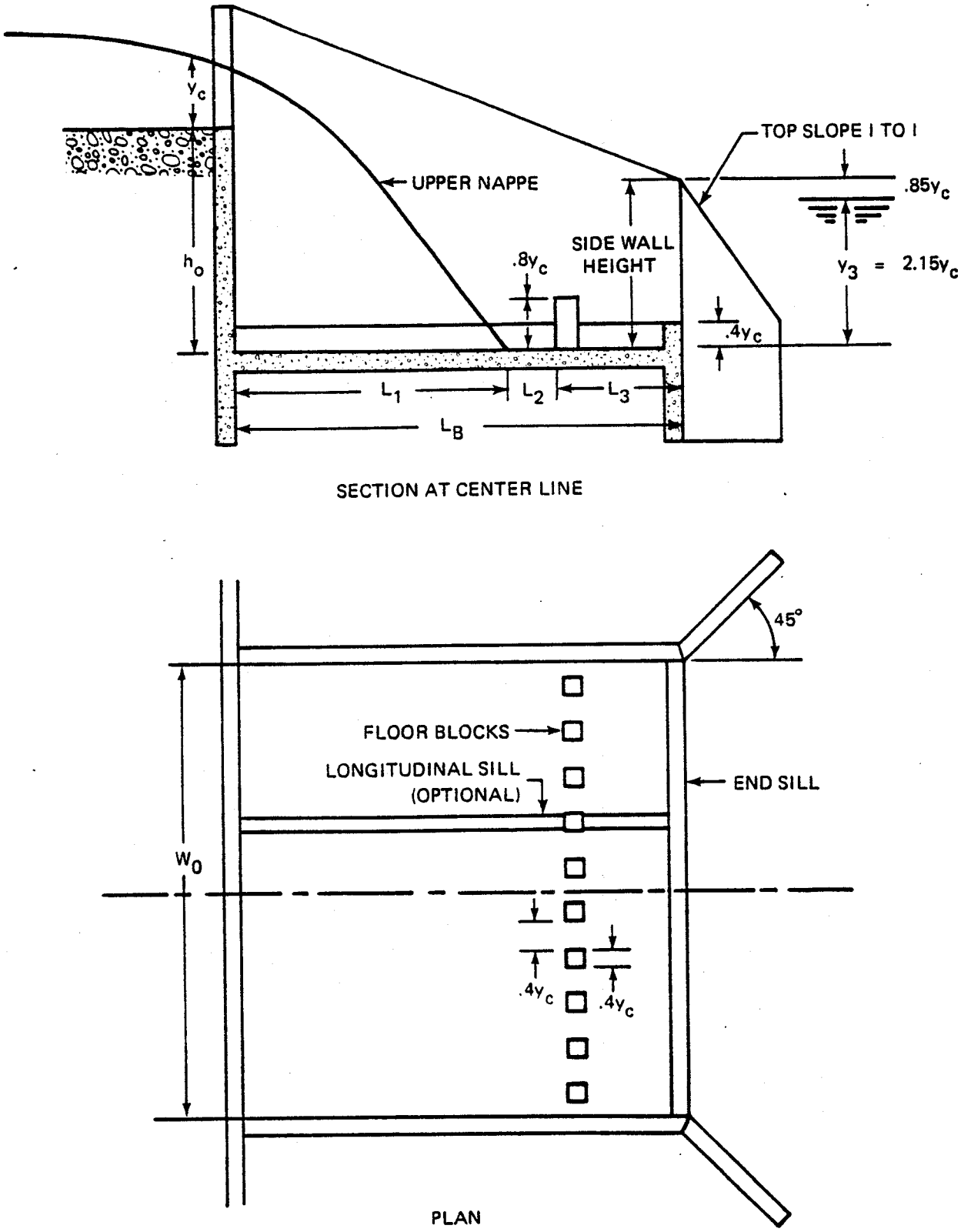


FIGURE IX - A - 1 STRAIGHT DROP SPILLWAY STILLING BASIN FROM REFERENCE IX-I

[9] Miscellaneous Topics

1981 International Symposium on Urban Hydrology, Hydraulics, and Sediment Control
(University of Kentucky, Lexington, Kentucky — July 27-30, 1981)

OPTIMAL DESIGN OF URBAN STORM WATER DRAINAGE SYSTEMS

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Abstract. Several researchers have developed screening models for minimizing the cost of sizing and vertical alignment of storm sewer systems. One of the disadvantages of these models is that they are designed with specific physical/economic assumptions in mind, as well as specific types of cost functions. These assumptions are effectively *hard-wired* into the program, which militates against modifying the program to suit situations not covered in the original assumptions. A generalized dynamic programming computer code has been developed at Colorado State University called CSUDP. A library of subroutines is being prepared as adjuncts to the basic code for solving a wide range of civil engineering problems. One of the packages includes a set of subroutines for storm sewer design. The coding includes data management routines for convenience to the user not familiar with dynamic programming. The code can consider complex branching networks with up to three pipes entering a given manhole. Default options are available for excavation, pipe, and manhole cost functions. If the costs for a particular case have a significantly different structure, the user can supply his own subroutine. There are several other default options which make this program extremely convenient for *normal* design conditions, as well as usable for unique problems. Extensive comparisons have been made between available published results of other dynamic programming screening models and the CSUDP package. Results are extremely close; differing in total cost by 0.8% to In most cases, results are extremely close. The one comparison resulting in a large deviation was due to slightly differing hydraulic and cost assumptions, which caused larger commercial pipe sizes to be chosen by CSUDP, even though actual pipe requirements deviated little.

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12. Robinson, D.K. and Labadie, J.W., "Optimal Design of Urban Stormwater Drainage Systems: User's Manual," Department of Civil Engineering, Colorado State University, Ft. Collins, Colorado, January 1981.

1982 International Symposium on Urban Hydrology, Hydraulics and Sediment Control
(University of Kentucky, Lexington, Kentucky – July 27-29, 1982)

DYNAMIC AND STEADY FLOW MODELING OF SURCHARGED STORM SEWER SYSTEMS

by

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Abstract. Surcharge in a storm sewer system is the condition in which an entire sewer section is submerged and the pipe is flowing full under pressure. Flow in a surcharged storm sewer is essentially slowly varying unsteady pipe flow and methods for analyzing this flow condition are investigated. In this paper the governing equations for unsteady and steady flow in pressurized storm sewer systems are presented. From these governing equations two numerical models are developed using various assumptions and simplifications. These flow models are applied to an example storm sewer system under surcharge conditions. Plots of hydraulic grade and flow throughout the sewer network are presented in order to evaluate the ability of each model to accurately analyze surcharged storm sewer systems.

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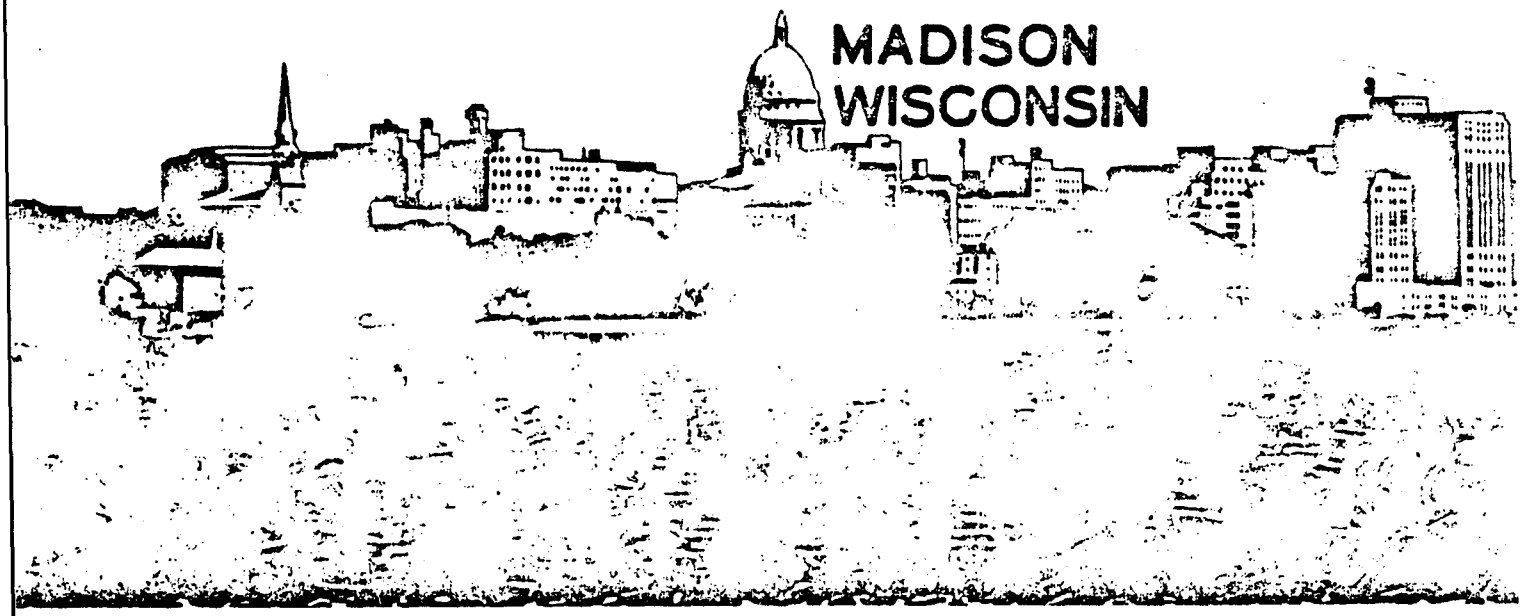
MADISON MODIFIED RRL COMPUTER PROGRAM FOR THE ANALYSIS AND DESIGN OF STORM DRAINAGE SYSTEMS



Source:

DIVISION OF ENGINEERING
DEPARTMENT OF PUBLIC WORKS
CITY OF MADISON, WISCONSIN

COMPUTER INVENTORY CODING FORM * FOR STORM DRAINAGE STRUCTURES

SECTION OR MAP NUMBER

NUMBER OF POINT - UPSTREAM

STRUCTURE TYPE

IF VDH DROP INLET, LENGTH OF OPENING

CONDITION OF STRUCTURE

INVERT OF STRUCTURE

RIM ELEVATION. IF DITCH, LOW BANK ELEVATION

TYPE OF CONDUIT

CONDUIT MATERIAL

CONDITION OF CONDUIT

INVERT OF CONDUIT

CONDUIT SIZE

PIPE-INCHES
DITCH-FEET

DIAMETER, RISE, DITCH BANK WIDTH

SPAN, DITCH BOTTOM WIDTH

SECTION OR MAP NUMBER

NUMBER OF POINT - DOWNSTREAM

* Dept. of Public Works, City of Norfolk, Va.

NUMBER OF POINT - DOWNSTREAM

STRUCTURE TYPE

IF VDH DROP INLET, LENGTH OF OPENING

CONDITION OF STRUCTURE

INVERT OF STRUCTURE

RIM ELEVATION. IF DITCH, LOW BANK ELEVATION

TYPE OF CONDUIT

CONDUIT MATERIAL

CONDITION OF CONDUIT

INVERT OF CONDUIT

CONDUIT SIZE

PIPE-INCHES
DITCH-FEET

DIAMETER, RISE, DITCH BANK WIDTH

SPAN, DITCH BOTTOM WIDTH

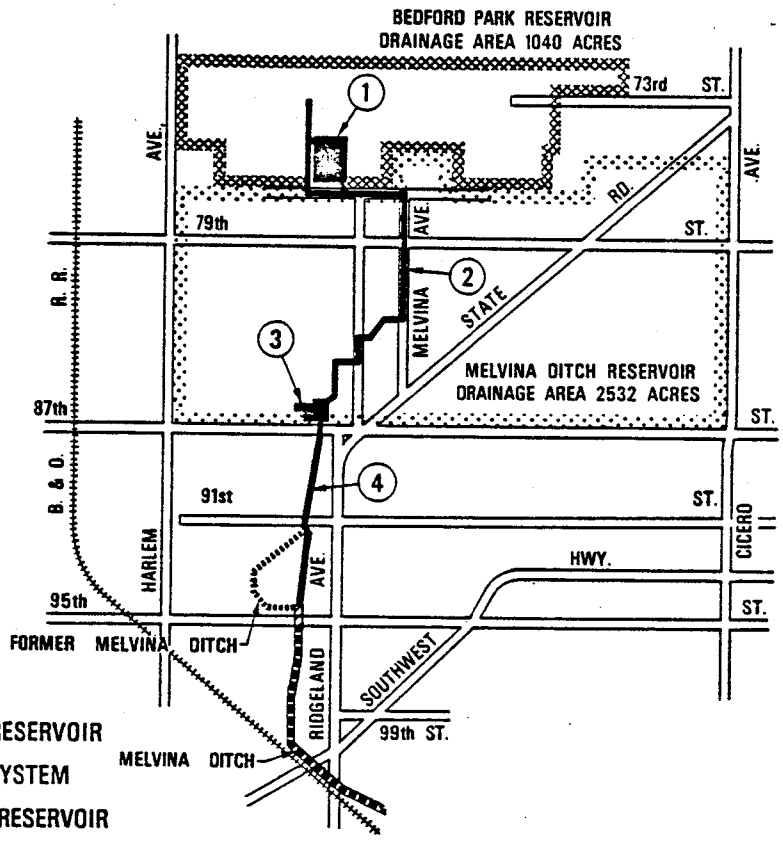
LENGTH BETWEEN POINTS

DATE INSTALLED (TO BE COMPLETED BY CITY)

PRIORITY

MELVINA DITCH RETENTION RESERVOIR
PROJECT NO. 69-218-2F

DRAINAGE AREA	2532 ACRES
DESIGN STORM	100 YR.
PUMPING STA. CAPACITY	214.5 c.f.s.
CONSTRUCTION COMPLETED	7-10-69
CONSTRUCTION COSTS	$ 892,000
LAND AREA	12 ACRES
LAND COST	(furnished by village)



1. BEDFORD PARK RESERVOIR
2. STORM SEWER SYSTEM
3. MELVINA DITCH RESERVOIR
4. MELVINA DITCH IMPROVEMENT (SEWER)

LOCATION MAP

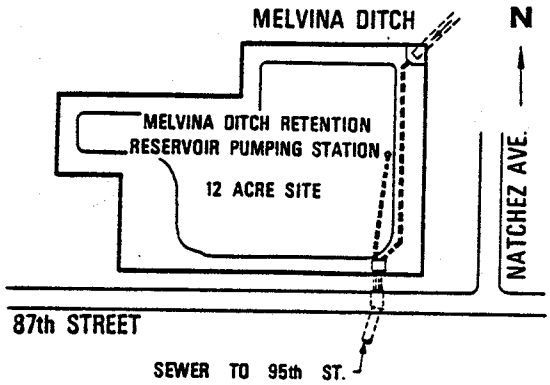
Source: METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

FLOOD CONTROL SECTION

71 F 564 R 2 J.G.N. JULY 1973

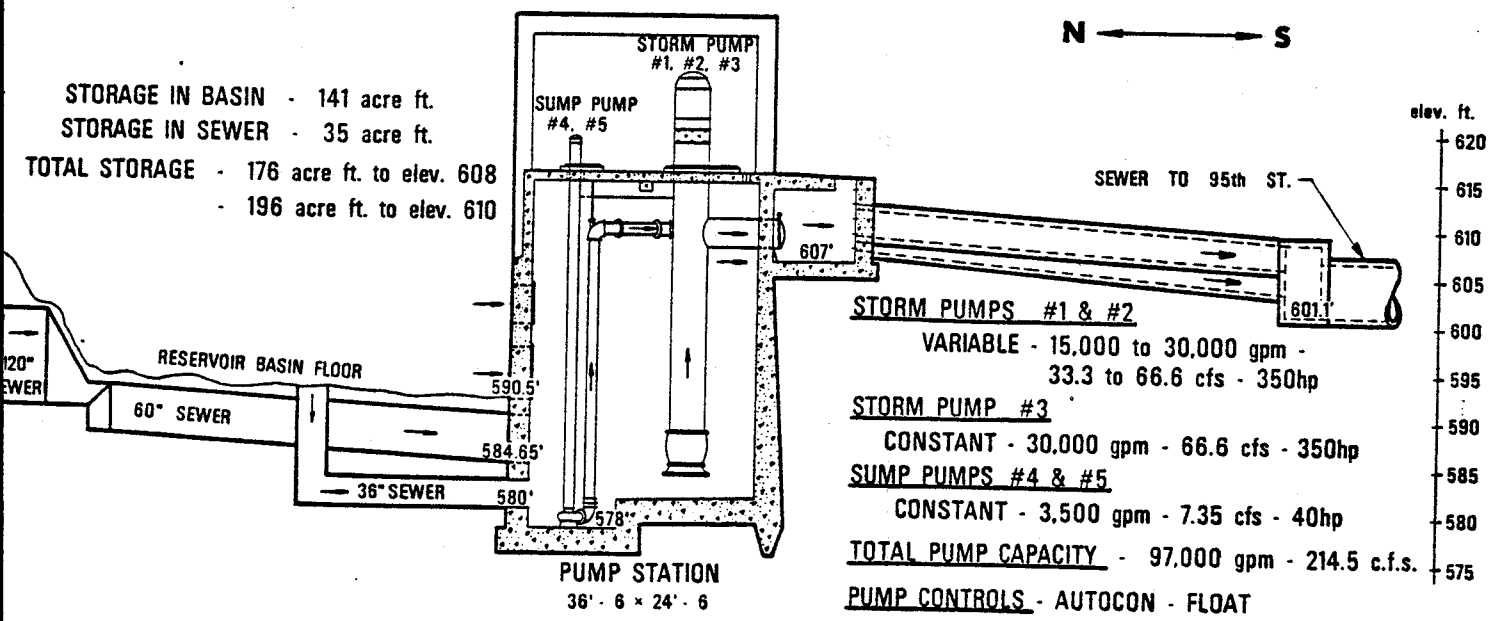
MELVINA DITCH RETENTION RESERVOIR
PROJECT NO. 69-218-2F

SHEET 2 OF 2



RESERVOIR LAYOUT

STORAGE IN BASIN - 141 acre ft.
STORAGE IN SEWER - 35 acre ft.
TOTAL STORAGE - 176 acre ft. to elev. 608
- 196 acre ft. to elev. 610



STORM PUMPS #1 & #2
VARIABLE - 15,000 to 30,000 gpm - 33.3 to 66.6 cfs - 350hp
STORM PUMP #3
CONSTANT - 30,000 gpm - 66.6 cfs - 350hp
SUMP PUMPS #4 & #5
CONSTANT - 3,500 gpm - 7.35 cfs - 40hp
TOTAL PUMP CAPACITY - 97,000 gpm - 214.5 c.f.s.
PUMP CONTROLS - AUTOCON - FLOAT
OVERFLOW @ 607' elev.

PROFILE

Source: **METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO**

FLOOD CONTROL SECTION

71 F 564 R 3 **J.G.N.** **JULY 1973**

International Conference on
WATER RESOURCES DEVELOPMENT
TAIPEI, TAIWAN, REPUBLIC OF CHINA
May 12-14, 1980

SIMULATION OF PUMPING STATION IN DRAINAGE SYSTEM

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Nien-Sheng Hsu
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ABSTRACT

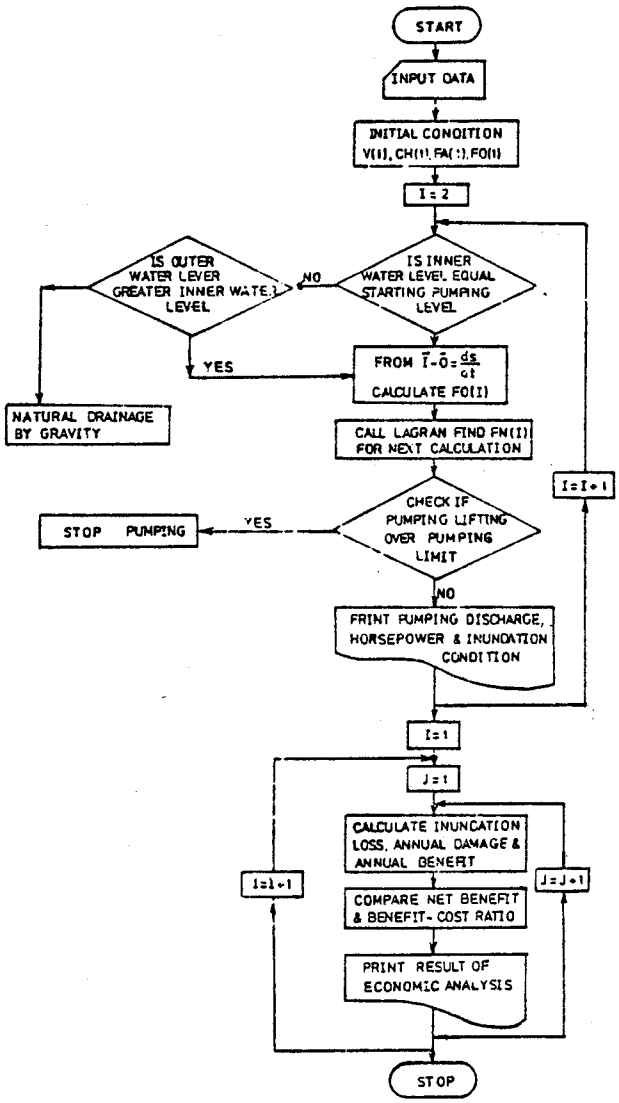
In this study, a completely mathematical model is set up under the conditions that various pumping flow planning and all possible affecting pumping factors in drainage system are combined and analyzed.

The model includes two major parts : inundation model of pumping flow and model of economic analysis. The simulation for inundated condition of pumping flow is based on the continuity equation of unsteady flow which is expressed in numerical forms and set into computer programs. By using the characteristic curve of inundated stage-inundated loss for the calculations of inundated conditions under various storm frequencies in model of economic analysis, the inundated losses and annual benifit can be obtained for the conditions before and after planning of pumping projects. Then, in comparing annual cost with annual benefit, the optimal design head, discharge and all possible pumping machine layouts can be determined.

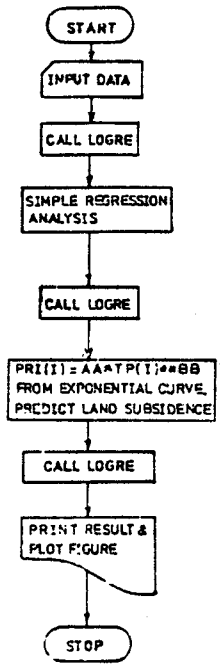
After being applied the model and also with consideration of the future possible trend of land subsidence at Yuan-Shan No. 1 Pumping Station of the pumping planning area of Hsin-Sheng North Road in Taipei City, it reveals that the results of simulation are considerably reasonable and practical. It is expected that the model is well enough for further modifications to apply for all other pumping area plannings in Taiwan.

Table 1 Flow Chart of Computer Program

A. Planning Program for Pumping System



B. Prediction Model for Land Subsidence



Storm Sewer Example *

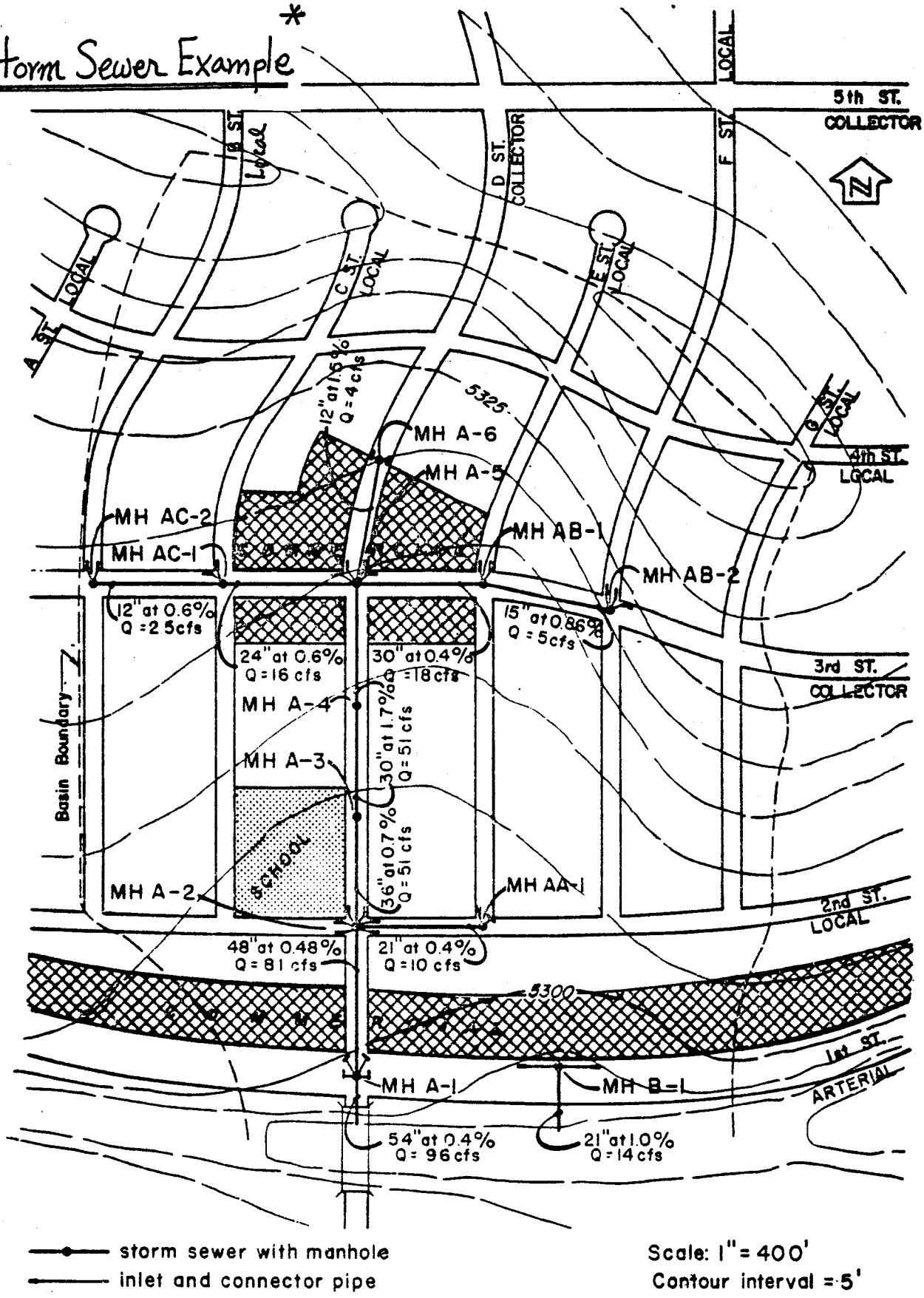


FIGURE II-6. COMPLETED PRELIMINARY STORM SEWER DESIGN

* Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual", Denver Regional Council of Governments, Denver, Colorado, 1969.

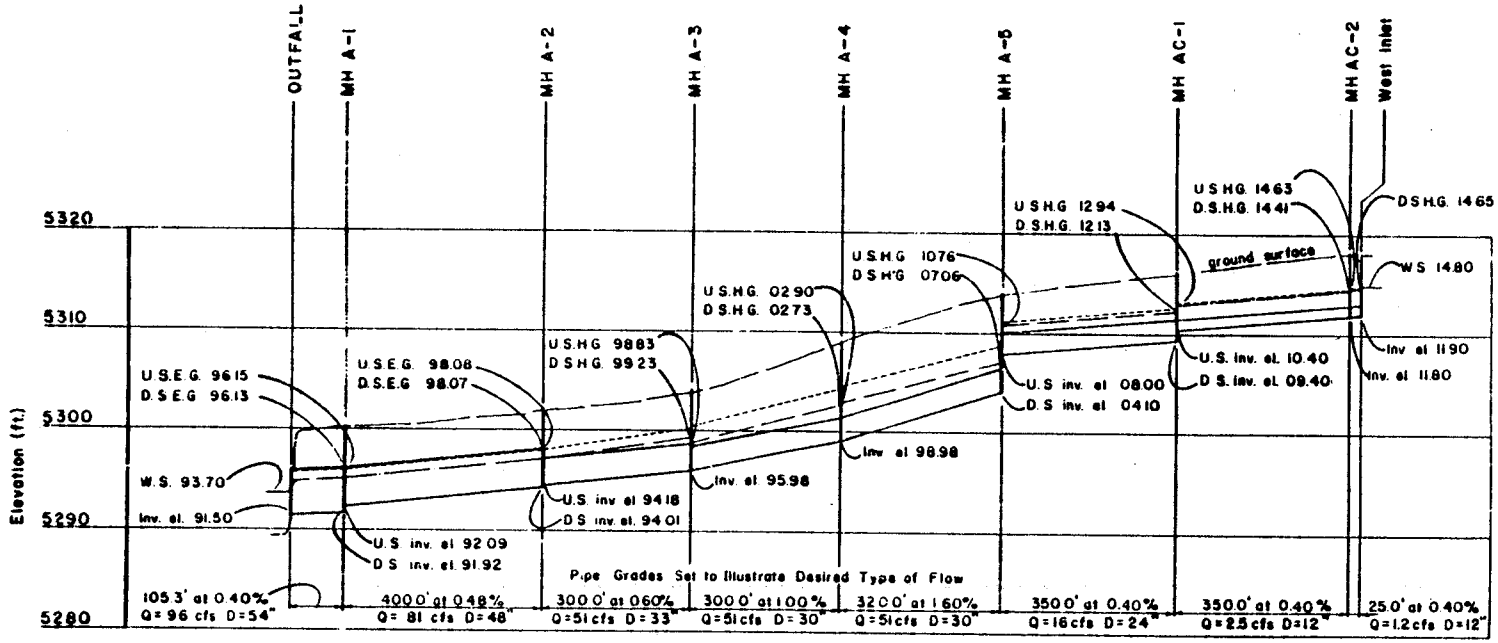


FIGURE 11-8. PROFILE OF EXAMPLE PROBLEM SEWER SHOWING HYDRAULIC PROPERTIES

Workshop Problem

a. Runoff Coefficients - (1) Residential area, 0.3. (2) Business area, 0.6. (3) Average coefficient weighted according to amount of each type of area tributary to a given inlet.

b. 5-yr. frequency rainfall for Davenport as shown in (Figure 2).

c. 20-min. inlet time assumed.

d. Sewer capacity by Manning formula, n = 0.013.

e. Outlet unsubmerged with free outfall.

f. A drop of 0.1 ft. across each manhole where no change in pipe size occurs. When change in pipe size occurs, set elevation of 0.8 of pipe depth points equal and provide corresponding fall in manhole invert.
(Note: in larger systems, a more rigorous analysis of hydraulic losses through manholes, at transitions, and at changes in directions must be made for adequate hydraulic design.)"

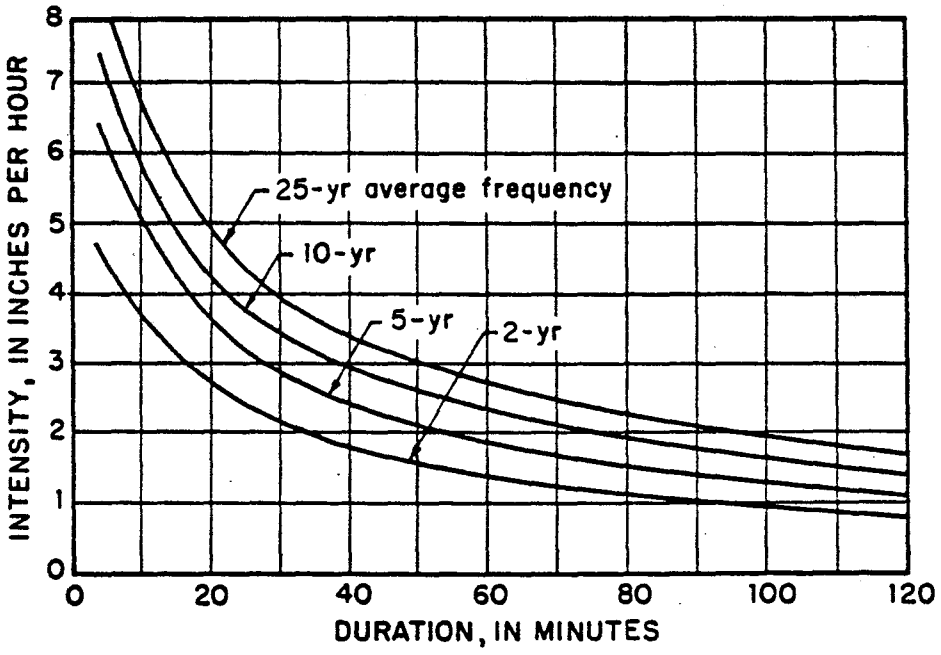


FIGURE 2

INTENSITY-DURATION RAINFALL CURVES FOR DAVENPORT, IOWA
(ASCE Manual No. 37, Fig. 9, p. 44)

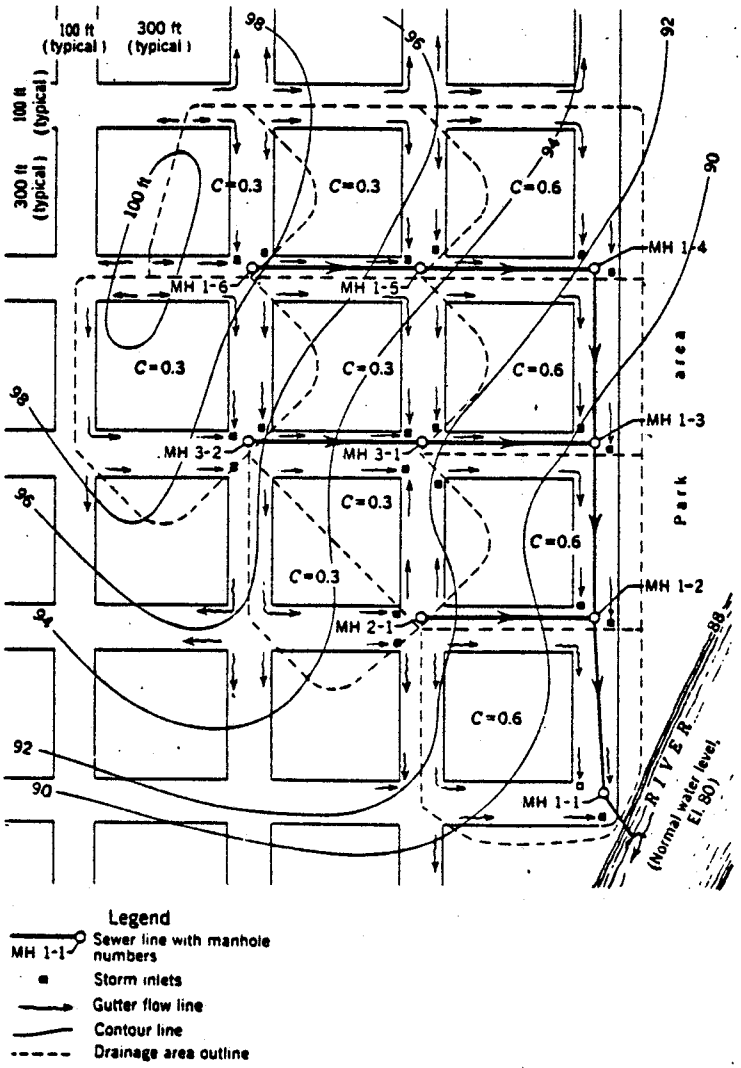


FIGURE 3 - TYPICAL STORM-SEWER DESIGN PLAN

(ASCE Manual No. 37, Fig. 12, p.51)

Line	Manhole No.		Length, ft	Area, Acres		Flow Time, Min		Average runoff coefficient	Rainfall, in./hr.	Runoff, cfs per acre	Total runoff, cfs	Slope of sewer, %	Diameter, in.	Capacity, full	Velocity, full	Design Flow				Manhole losses, ft	Manhole invert drop, ft	Fall in sewer, ft	Sewer Invert		Ground Elevation	
	From	To		Increment	Total	To upper end	In section									Velocity, fps	Velocity head, ft	Depth of flow, in.	Total energy, ft				Upper end	Lower end	Upper end	Lower end
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
1	1-6	1-5	400	2.64				0.3				0.85									....	3.40	93.00	89.60	98.4	94.9
1	1-5	1-4	400	3.61				0.3				0.75									0.40	3.00	89.20	86.20	94.9	91.8
1	1-4	1-3	400	3.88				0.42				0.45									0.40	1.80	85.80	84.00	91.8	89.7
3	3-2	3-1	400	5.55				0.3				1.00									....	4.00	91.00	87.00	96.2	92.3
3	3-1	1-3	400	6.43				0.3				0.60									0.60	2.40	86.40	84.00	92.3	89.7
1	1-3	1-2	400	3.92				0.39				0.30									0.80	1.20	83.20	82.00	89.7	89.5
2	2-1	1-2	400	2.52				0.3				0.90									....	3.60	87.50	83.90	92.7	89.5
1	1-2	1-1	400	3.86				0.41				0.24									0.40	0.96	81.60	80.64	89.5	88.5
1	1-1	Out-fall	125	5.44				0.44				0.30									0.10	0.38	80.54	80.16	88.5	....

TABLE 2 - TYPICAL STORM-SEWER COMPUTATIONS FOR RATIONAL METHOD FOR SYSTEM IN FIGURE 3.

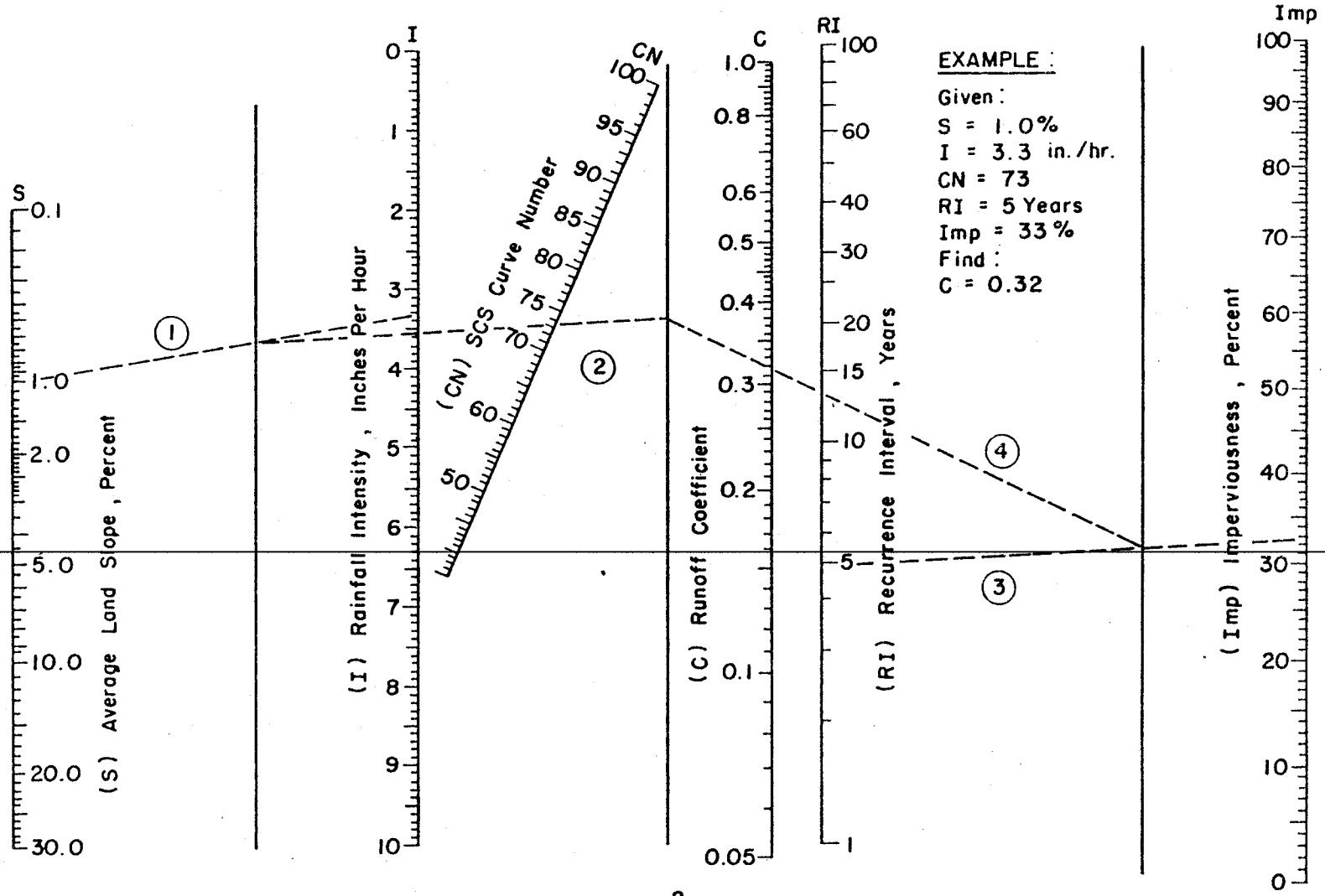
(ASCE Manual No. 37, p. 52)

C coefficients for Rational Method

Typical C coefficients for 5- to 10-year frequency design.

Description of Area	Runoff Coefficients
Business	
Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential	
Single-family areas	0.30-0.50
Multiunits, detached	0.40-0.60
Multiunits, attached	0.60-0.75
Residential (suburban)	0.25-0.40
Apartment dwelling areas	0.50-0.70
Industrial	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.35
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Streets	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and walks	0.75-0.85
Roofs	0.75-0.95
Lawns: Sandy soil	
Flat 2%	0.05-0.10
Average 2-7%	0.10-0.15
Steep 7%	0.15-0.20
Lawns: Heavy soil	
Flat 2%	0.13-0.17
Average 2-7%	0.18-0.22
Steep 7%	0.25-0.35

From ASCE [1972] and Viessman et al. [1977].



$$C = 7.2(10)^{-7} CN^{3} RI^{.05} ((.01CN)^{.6})^{-S^{.2}} (.001CN^{1.48})^{.15-.11} ((Imp+1)/2)^{.7}$$

Nomograph for estimating Rossmiller C-factor in the Rational Formula.

Runoff coefficients for the rational formula by hydrologic soil group and slope range [after Rawls et al., 1981].

ABCD = Hyd. soil group

	A			B			C			D		
Land Use	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Cultivated Land	0.08[a]	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14[b]	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.16	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential Lot Size 1/8 acre	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Lot Size 1/4 acre	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Lot Size 1/3 acre	0.19	0.23	0.26	0 22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Lot Size 1/2 acre	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Lot Size 1 acre	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46

(continued)

	A			B			C			D		
	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open Space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

[a] Runoff coefficients for storm recurrence intervals less than 25 years

[b] Runoff coefficients for storm recurrence intervals of 25 years or more

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