APPENDIX "H"
STORM SEWER DESIGN

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APPENDIX "H"
STORM SEWER DESIGN

1. <u>Review of Criteria</u> Per Section I-B and Section VI-A, storm sewers shall be sized as a minimum for the 5 year storm event. However, if the sewer functions as a major outfall, or passes through an easement on private property, the storm sewer in conjunction with other drainage facilities must be capable of conveying the full 100-year storm runoff within designated drainage tracts, easements, or public rights-of-way, all in accordance with regulations. Consequently, there will be situations where a storm sewer shall be designed for greater than the 5 year event, up to a maximum of the 100-year storm runoff rate.

   Per Section VI-D, minimum flow velocity in pipes shall be 2.5 fps in the 5 year storm event. If pipes flow full or over normal depth, hydraulic gradelines shall be calculated with losses due to friction, bends, expansion, and contraction determined. For the design storm for which a sewer system is designed, the hydraulic gradeline shall not raise within 1.0 foot of any manhole or junction rim, catch basin, inlet grate or other surface opening.

2. <u>Flow Headlosses</u> Unless there is a hydraulic jump condition, water surfaces drop as water flows downhill. It is the difference in water surface elevation that causes the water to flow. The amount of drop or "headloss" required to maintain flow depends upon many conditions. Headlosses in flows are discussed below.

   a. <u>Submerged Exit Headlosses</u> When pipe flow outfalls submerged into a reservoir or transverse to channel flow, the terminal velocity in the direction of flow is essentially zero. Therefore, the headloss is the full velocity head of $H_L = v^2/2g$. For pipe flow which outfalls submerged into a channel in the direction of channel flow, the headloss may be estimated as $H_L = (1/C_v^2 - 1) \times v^2/2g$, where v is the channel flow velocity. The value in the parentheses may be approximated with 0.10, allowing the equation to be rewritten as $H_L = 0.1v^2/2g$.

   b. <u>Pipe Flow Headlosses</u> Below are several forms of the Manning equation for use in calculating storm sewer flow capacity and headloss associated with various flow conditions.

$$Q = \frac{1.486\, a^{1.67} S_f^{0.5}}{n P_w^{.67}}$$

$$V = \frac{1.486\, Rh^{.67} S_f^{0.5}}{n}$$

   For round pipes with <u>full</u> pipe flow, the following are simplified forms:

$$Q = \frac{0.463\, D^{2.67} S_f^{.5}}{n}$$

$$V = \frac{0.590\, D^{.67} S_f^{.5}}{n}$$

$$D = \left[\frac{2.159\, Qn}{Sf^{0.5}}\right]^{0.375}$$

$$D(in) = \left[\frac{1630\, Qn}{Sf^{0.5}}\right]^{0.375}$$

$$Sf = \frac{4.66\, n^2 Q^2}{D^{5.33}}$$

$$Sf = \frac{2.87\, n^2 V^2}{D^{1.33}}$$

$$Hf = Sf \times \text{Pipe Length}$$

where:

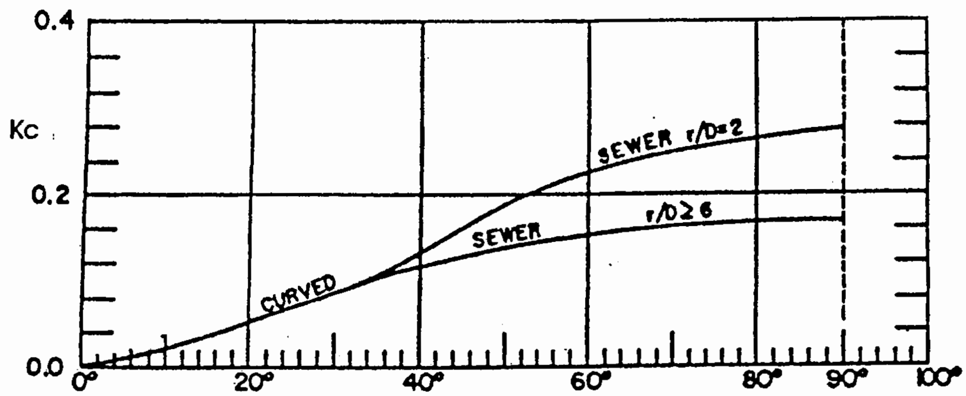
- Q = Flow in cfs;
- a = Area of flow in square feet;
- Sf = Frictional slope in feet/feet;
- n = Manning's "n" value;
- Pw = Wetted perimeter of flow, or length of non-air frictional surface in the flow cross section, feet;
- V = Average velocity of flow in fps;
- D = Pipe diameter in feet;
- Rh = Hydraulic radius, or area of flow/wetted perimeter; and
- D(in) = Pipe diameter in inches; and
- Hf = Frictional headloss, or the headloss in feet required to move flow through the pipe, but does not include headloss required to get flow into the pipe (contraction headloss), out of the pipe (expansion headloss), or bend losses.

c. <u>Curved Sewer Headlosses</u> Some regulatory agencies allow curved storm sewerlines, particularly for large sewers. Bend headloss in a curved sewerline may be added to the pipe friction headloss, and can be estimated using Figure "H-1".

d. <u>Manhole and Junction Headlosses</u> Unless there is a single pipe entering and exiting a manhole at 180 degrees, and both pipes are the same size, and either the pipe passes through the manhole (with an open top) or the manhole is fully shaped to near the top of the pipes, there will be an expansion, contraction, and possibly a redirection of flow within the manhole. These headlosses are usually calculated as a function of the velocity head $V^2/2g$, but recommended procedures vary. Some promote use of the velocity (V) from the smaller pipe, while others suggest use of the difference in velocities between the smaller and larger pipe. Each procedure has its own set of "K"

FIGURE "H-1"
HEADLESS COEFFICIENT Kc FOR CURVED SEWERS

MODIFIED FROM FIGURE 4.16 IN MARICOPA COUNTY



DEFLECTION ANGLE, DEGREES
(From point of curvature to point of tangency)

factors to be multiplied by the velocity head $V^2/2g$ to obtain headlosses. The inconvenience with using either of these methods with hand calculations is that one must know the velocity of flow in all inflowing and outflowing pipes before being able to calculate headloss through a manhole, which in turn will affect upstream flow velocity. Thus, the procedure is iterative.

The major drawback of typical procedures and K values provided in handbooks and manuals is that they are based upon an application not found in storm sewer systems. The experimental basis is for a single pipe in and a single pipe out; pipe size enlargements are either sudden (instantaneous) or gradual (through a cone), neither of which approximates flow through a manhole; they do not account for bends through the manhole nor ratios of incoming flows from various directions, nor the possibility of plunge flow from a higher inlet in addition to a main lower inlet, nor do they account for manhole benching types. In short, they are not applicable for storm sewer systems.

The procedure presented herein is taken from a FHWA publication that accompanies FHWA design software (FHWA Hydrain), which procedures are based upon various studies of flow through manholes and junctions. Procedures are provided for calculation of both headlosses at a pipe junction and at manholes.

(1) <u>Pipe Junction</u> A pipe junction is the connection of a single lateral pipe to a larger trunk pipe without the use of a manhole structure. The minor loss equation for a pipe junction is a form of the momentum equation:

$$Hm = \frac{Q_o \times V_o - Q_i \times V_i - Q_L \times V_L \times \cos\theta}{0.5 \times g \times (A_o - A_i)} + h_i - h_o$$

where:

Hm	=	Junction head loss, in ft;
Q_o, Q_i, Q_L	=	Outlet, inlet, and lateral flows, respectively in ft^3/s;
V_o, V_i, V_L	=	Outlet, inlet, and lateral velocities, respectively in ft/s;
h_o, h_i =		Outlet and inlet velocity heads, in ft;
A_o, A_i	=	Outlet and inlet cross-sectional areas, in ft^2;
θ	=	Angle of lateral with respect to centerline of outlet pipe, in degrees; and
g	=	Gravitational acceleration, (32.2 ft/s^2).

(2) Manholes The basis for determining the minor head loss experienced at manholes (or junction manholes) is the energy equation, which can be reduced to $Hm = KV_o^2/2g$.

Several experimental studies have supplemented the theoretical value understanding of headloss in manholes. From these experiments, it was determined that the value K can be approximated as follows:

$$K = K_o \times C_D \times C_d \times C_Q \times C_p \times C_B$$

where:

K = Adjusted headloss coefficient;
K_o = Initial headloss coefficient based on relative manhole size;
C_D = Correction factor for pipe diameter;
C_d = Correction factor for flow depth;
C_Q = Correction factor for relative flow;
C_B = Correction factor for benching; and
C_p = Correction factor for plunging flow.

(i) K_o The initial head loss coefficient K_o is estimated as a function of the relative manhole size and angle between the inflow and outflow pipes:

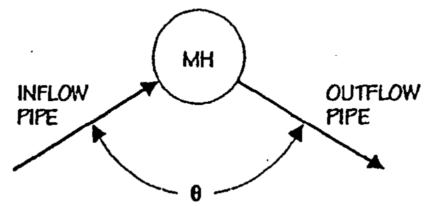
$$K_o = 0.1 \times \left[\frac{D_{MH}}{D_o}\right] \times [1 - \sin\theta] + 1.4 \times \left[\frac{D_{MH}}{D_o}\right]^{0.15} \times \sin\theta$$

where:

K_o	=	Initial headloss coefficient based on relative manhole size;
θ	=	The angle in degrees between the inflow and outflow pipes (see Figure "H-2");
D_{MH}	=	Manhole diameter, ft; and
D_o	=	Outlet pipe diameter, ft.

FIGURE "H-2"
ANGLE OF SEWERLINE BEND

REPRODUCED FROM FIGURE 7 IN FHWA HYDRAIN



It has been shown that there are only slight differences in headloss coefficient between round and square manholes. Therefore, manhole shape can be ignored when estimating headlosses for design purposes.

(ii) <u>C_D</u>. A change in headloss due to differences in pipe diameter was found to only be significant in pressure flow situations when the depth in the manhole to outlet pipe diameter ratio (d/K_o) is greater than 3.2. Therefore, it is only applied in such cases. The correction factor for pipe diameter (C_D) was determined to be

$$C_D = \left[\frac{D_o}{D_i}\right]^3$$

where:

C_D = Correction factor for variation in pipe diameter;
D_i = Incoming pipe diameter, ft; and
D_o = Outgoing pipe diameter, ft.

(iii) <u>C_d</u>. This correction factor was found to be significant only in cases of free surface flow or low pressures, when the d/D_o ratio is less than 3.2, and is only applied in such cases. Water depth in the manhole is approximated as the level of the hydraulic guideline at the upstream end of the outflow pipe.

The correction factor for flow depth, C_d, is calculated by the following:

$$C_d = 0.5 \times \left[\frac{d}{D_o}\right]^{3/5}$$

where:

C_d = Correction factor for flow depth;
d = Water depth in manhole above outlet pipe invert, ft; and
D_o = Outlet pipe diameter, ft.

(iv) $\underline{C_Q}$ C_Q is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming and outflow pipes. The correction factor for relative flow (C_Q) is computed by:

$$C_Q = 1 + (1 - 2 \times \sin\theta) \times \left[1 - \frac{Q_i}{Q_o}\right]^{0.75}$$

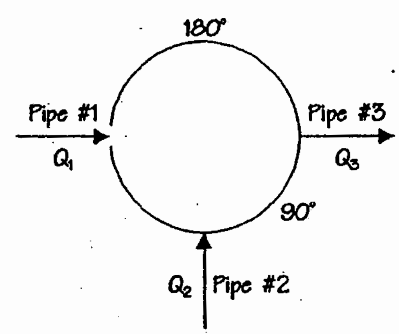
where:

C_Q = Correction factor for relative flow;
θ = The angle between the inflow and ouflow pipes, degrees;
Q_i = Flow in the inflow pipe of interest, cfs; and
Q_o = Flow in the outflow pipe, cfs.

To illustrate the effect of C_Q, consider the manhole shown in Figure "3" and assume that $Q_1 = 3$ ft^3/s, $Q_2 = 1$ ft^3/s, and $Q_3 = 4$ ft^3/s. Solving for the relative flow correction factor in going from the outlet pipe (number 3) to one of the inflow pipes (number 2):

FIGURE "H-3"
RELATIVE FLOW Qc SCHEMATIC

REPRODUCED FROM FIGURE 8 IN FHWA HYDRAIN



$$C_{Q_{3-2}} = [1 - 2 \times \sin(90°)] \times \left[1 - \frac{1}{4}\right]^{0.75} + 1 = 0.19$$

For a second example, consider the following flow regime: $Q_1 = 1$ ft^3/s, $Q_2 = 3$ ft^3/s, and $Q_3 = 4$ ft^3/s. Calculating C_Q for this case:

$$C_{Q_{3-2}} = [1 - 2 \times \sin(90°)] \times \left[1 - \frac{3}{4}\right]^{0.75} + 1 = 0.65$$

In both of these cases, the flow coming in through pipe number 2 has to make a 90-degree bend before it can go out pipe number 3. In case 1, the larger flow traveling straight through the manhole, from pipe number 1 to pipe number 3, assists the flow from pipe number 2 in making this bend. In case 2, a majority of the flow is coming in through pipe number 2. There is less assistance from the straight through flow in directing the flow from pipe number 2 into pipe number 3. As a result, the correction factor for relative flow in case 1 (0.19) was much smaller than the correction factor for case 2 (0.65)

(v) $\underline{C_p}$. This correction factor corresponds to the effect of inflow from an elevated pipe or surface inlet, resulting in flow plunging into the manhole. The factor is applied to the inflow pipe for which the headloss is being calculated. Using the notations in Figure "H-3", for example, C_p is calculated for pipe number 2 when pipe number 1 discharges plunging flow. Plunging flow that results from inlet inflow into the manhole is considered in the same manner.

The correction factor for plunging flow (C_p) is calculated by the following:

$$C_p = 1 + 0.2 \times \left[\frac{h}{D_o}\right] \times \left[\frac{h - d}{D_o}\right] \qquad \text{(h must exceed d)}$$

where:

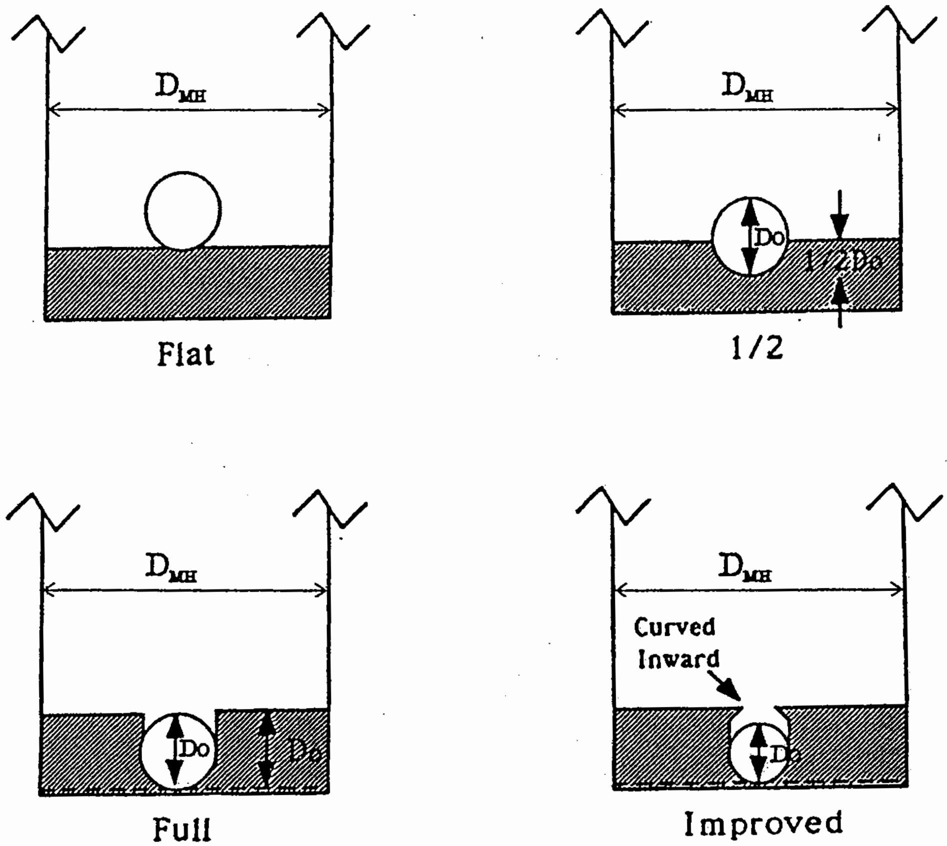
C_p = Correction for plunging flow;
h = Vertical distance of plunging flow from the center of the outlet pipe or surface inlet elevation to the bottom of the manhole, ft;
D_o = Outlet pipe diameter, ft; and
d = Water depth in the manhole, ft.

(vi) $\underline{C_B}$. The final correction factor multiplied by the initial headloss coefficient K_o to get the adjusted headloss coefficient K is the correction for benching in the manhole (C_B). Benching tends to direct flows through the manhole, resulting in reductions in head loss. Figure "H-4" shows various types of benching and benching correction factors.

To estimate the headloss through a manhole from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

e. <u>Entrance Headloss</u> Entrance headlosses are those associated with flow into an inlet. Some advocate not only calculating the hydraulic grade line (HGL) through the inlet connection pipe to an inlet, but calculating the entrance

MODIFIED FROM FIGURE 9 AND TABLE 2 IN FHWA HYDRAIN



D_{MH} = Diameter of Manhole, ft.
d = Depth of flow in manhole, taken at entrance of outflow pipe, ft.
Do = Diameter of outflow pipe, ft.

Bench Type	Outlet Conditions	
	Submerged*	Unsubmerged**
Flat floor	1.00	1.00
Benched one-half of pipe diameter	0.95	0.15
Benched one pipe diameter	0.75	0.07
Improved	0.40	0.02

* Pressure flow, d/Do > 3.2
** Free surface flow, d/Do <1.0
For flow depths between the submerged and unsubmerged conditions, interpolate between the values shown.

MANHOLE BENCH SHAPING FACTOR C_B | **FIGURE "H-4"**

headloss as well. However, a simplified procedure is to ignore entrance headloss calculations, but require instead a 1.0 foot freeboard between the HGL in the connection pipe and the inlet grate or curb opening elevation. Such is the procedure advocated herein.

HGL calculations are not required if the outfall is unsubmerged and all pipeline frictional slopes (Sf) are less than corresponding pipe slopes for the design storm. Nonetheless, flow velocities must conform with the minimum 2.5 fps rate for 2-year storm conditions.

3. Hydraulic Guidelines (HGL)

a. Outfalls Hydraulic design of storm drains begins at the lowest point in the storm drain system. The beginning hydraulic grade line is the higher of the normal flow depth in the pipe at the point of discharge, the water surface in open channel flow, or the hydraulic grade line in pressurized conduits of the receiving drainage facility. The hydraulic grade line or water surface elevation of the outfall must be determined coincident with the time of peak flow from the storm drain.

If the outlet is submerged or if the receiving water surface is higher than the normal depth in the storm drain, the beginning hydraulic grade line is the hydraulic grade line in the receiving stream. With a submerged outlet, the design proceeds up the pipeline after inclusion of exit losses (see subsection 2-a, p. H-1). For unsubmerged (free discharge) outlets, design can begin assuming normal depth at the end of the storm sewer pipe.

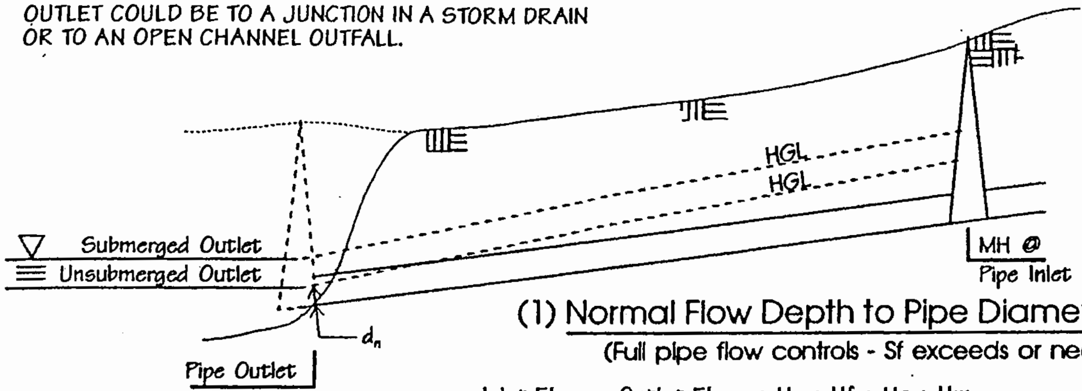
Figure "H-5" illustrates exit conditions.

b. Outlets The lowest outlet in a drainage system is the outfall discussed in (a) above. All upstream pipes outlet to a manhole or junction, and the calculated water surface therein, which is based upon the hydraulic grade line (HGL) in the downstream pipe plus losses in the manhole, becomes the outlet HGL for the upstream pipe.

c. Pipeline Reaches and Manholes The headloss through a pipeline and manhole depends upon the outlet condition, whether submerged or unsubmerged, and upon whether the pipe frictional slope Sf is greater or less than the pipe slope S. Figure "H-5" shows design procedures.

4. Manhole Design Guidelines The following are not design requirements, but are intended for use when junction losses are an important design consideration.

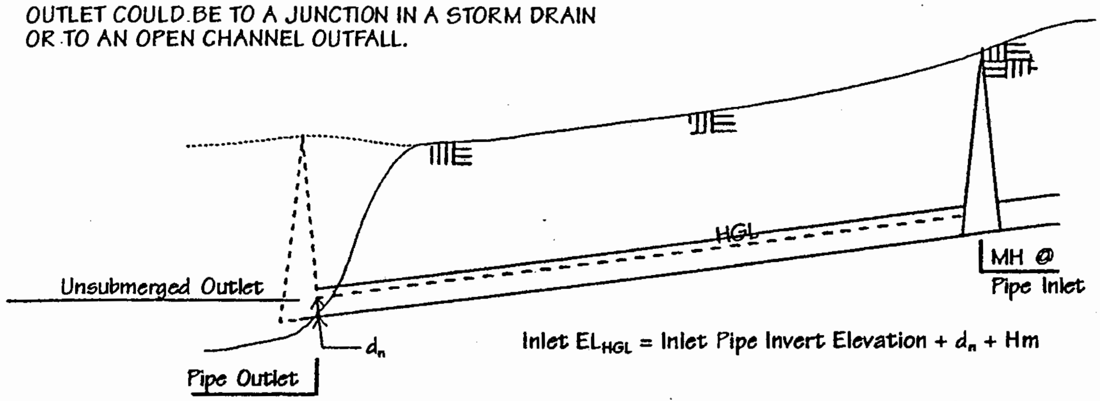
a. Alignment of Pipe in Manholes The following discussion applies to the location of pipes within a manhole to achieve maximum efficiency.



(1) Normal Flow Depth to Pipe Diameter $d_n/D > 0.80$

(Full pipe flow controls - Sf exceeds or nearly exceeds S)

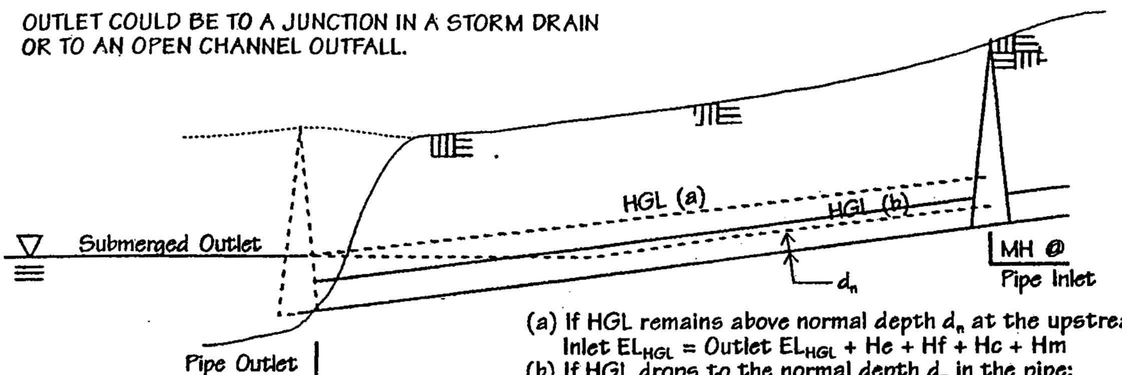
Inlet EL_{HGL} = Outlet EL_{HGL} + He + Hf + Hc + Hm



Inlet EL_{HGL} = Inlet Pipe Invert Elevation + d_n + Hm

(2) $d_n/D \leq 0.8$ and Outlet is Unsubmerged

(Partial pipe flow controls)



(a) If HGL remains above normal depth d_n at the upstream junction:
Inlet EL_{HGL} = Outlet EL_{HGL} + He + Hf + Hc + Hm
(b) If HGL drops to the normal depth d_n in the pipe:
Inlet EL_{HGL} = Inlet Pipe Invert Elevation + d_n + Hm

(3) $d_n/D \leq 0.8$ and Outlet is Submerged

(The higher of condition (a) or (b) controls)

Outlet EL_{HGL} = Higher of Tailwater (TW) or Normal pipe flow depth (d_n) elevation in outfall,
He = Submerged exit expansion loss (applicable to submerged outfalls only - see subsection 2-a)
Hf = Pipe frictional loss (see subsection 2-b)
Hc = Curved Sewer Bend Loss (Does not apply to bends in manholes — see subsection 2-c)
Hm = Manhole or Junction Losses (see subsection 2-d)

NOTE: If all pipeline flows have normal depths to pipe diameter $d_n/D \leq 0.80$, calculations are not required if the outlet is unsubmerged; or, if the outlet is submerged, calculations are required only to the point where the HGL coincides with the normal flow depth d_n. However, the minimum flow velocity of 2.5 fps in the 2-year storm condition must be met.

HYDRAULIC GRADE LINE CONDITIONS	FIGURE "H-5"

For a straight through-flow, pipes should be positioned vertically so that they are between the limits of inverts aligned or crowns aligned. A horizontal offset is allowable provided the projected area of the smaller pipe falls within that of the larger. Where feasible, the manhole bottom should support the bottom of the jet issuing from the upstream pipe.

When two inflowing laterals intersect in a manhole, the alignment should be quite different. If lateral pipes are aligned opposite one another so the jets may impinge upon each other, the magnitude of the losses are extremely high. Consequently, this arrangement should be avoided wherever possible. If the installation of directly opposed inflow laterals is necessary, the installation of a deflector, as shown in Figure "H-6" will result in significantly reduced losses.

Lateral inflow pipes larger than 24 inches in diameter shall not be located directly opposite; rather, use a manhole on each lateral with a 45° bend, and a third junction manhole on the outfall line with inflow pipes at a 45° deflection angle from the outflow pipe.

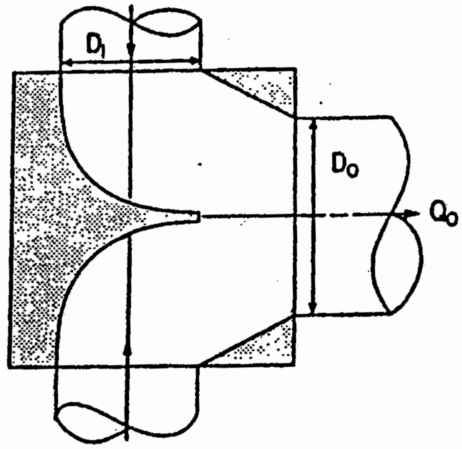
The University of Missouri presented additional information in 1958 regarding manhole inflow/outflow configurations, which information is presented in both Maricopa County's and UD & FCD's drainage manuals.

b. <u>Shaping Inside of Manholes</u> Jets issuing from upstream and lateral pipes must be considered when attempting to shape the inside of manholes.

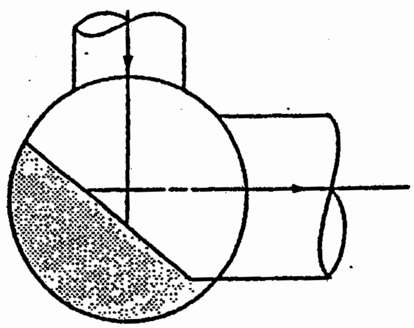
Figure "H-6" details several types of deflector devices that have been found efficient in reducing losses at junctions and bends. In all cases, the bottoms are flat, or only slightly rounded, to handle low flows. This is because University of Missouri tests for <u>full flow</u> revealed that very little, if anything, was gained by shaping the bottom of a manhole to conform to the pipe invert. Shaping of the invert was even found to be detrimental when lateral flows were involved, as the shaping tended to deflect the jet upwards, causing unnecessary headloss. On the other hand, more recent studies used by the FHWA indicate that manhole shaping does provide limited hydraulic improvement for submerged flow conditions, and considerable improvement for unsubmerged flow conditions (see Figure "H-4"). It would appear that, for single inflow manhole conditions, invert shaping can be very beneficial, whereas only limited bottom shaping is advisable when laterals are involved.

5. <u>Flow Routing Using the Rational Method</u> The Rational Method does not produce hydrographs which can be routed and combined at different points of interest, such as at manholes along a storm sewer line. However, if only peaks are of interest, then reasonable results may generally be obtained. Peak runoff using the Rational Method involves rainfall intensity, which is time dependent. Keeping track of elapsed time for runoff from various watersheds to reach a given point allows for adjustments to the combined peak which reflects

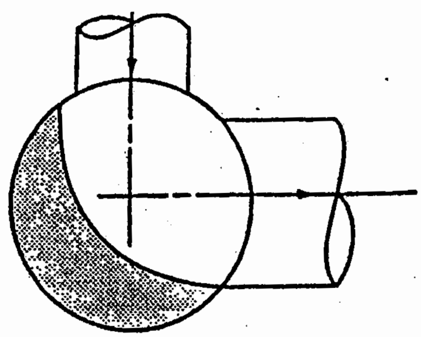
REPRODUCED FROM FIGURE 4.1 IN MARICOPA COUNTY



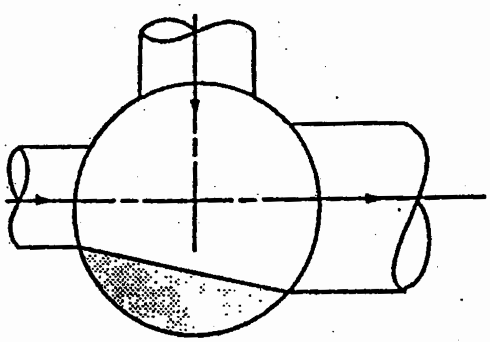
Directly opposed lateral with deflector (head losses are still excessive with this method, but are significantly less than when no deflector exists.) Do not use this method for laterals over 24 inches in diameter. Instead, use a manhole on each lateral with a 45° bend, and a third junction manhole on the outfall line with inflow pipes at a 45° angle to the outflow pipe.



Bend with straight deflector



Bend with curved deflector



Inline upstream main and 90° lateral with deflector

EFFICIENT MANHOLE SHAPING | **FIGURE "H-6"**

routing and time dependent conditions. There are certain limitations, but, in smaller systems, inaccuracies of "routing" are usually tolerable and generally conservative.

Unfortunately, there seems to be a prevalent misunderstanding as to how the intensity adjustment should be applied. It is thought by some that, as one proceeds downstream in a drainage system, at each point where additional flow is added, all upstream contributing watersheds are also analyzed at the new (longer) time which accounts for travel time. Actually, only the additional contributing flow should be estimated using the longer time; the flows from upstream are essentially the same as they were when they entered the system. The continuity equation indicates that, if a peak flow enters a system (street, channel, or pipe) at one point, the same peak will be realized at a point downstream when the travel time has passed for that peak to arrive there. Of course, this is true only if there is no storage enroute. However, in urban drainage systems, storage capacity of channels and pipes is usually minimal, is mostly filled prior to the arrival of peak flows, and therefore attenuation of peak flows limited.

The only watershed that should be analyzed at a larger time is the one adding flow at that given point, and then only under certain conditions. Catch basin inlets and connector pipes should be designed for the peak flow resulting from the watershed time of concentration (Tc), not the accumulated time. Additionally, if the contributing watershed analyzed at its own Tc is greater than the combined flows at a longer time, then the main channel or pipe should be designed to convey the higher flow from the single watershed.

The above arguments are not only consistent with hydrologic and hydraulic principles, but can also be supported by analyzing equal systems by hydrograph routing techniques. Table "H-1" provides data for a schematic drainage basin typical in urban systems. Using the information, Table "H-2" shows correct and incorrect procedures and estimated flow rates for pipe reaches using the Rational Method. Also shown are flow rates obtained by using SCS methods in HEC-1 which produce identical individual basin runoffs (rounded to nearest cfs), where actual hydrograph routing can be accomplished. It can be seen that hydrograph routing supports the above discussion.

6. <u>Dynamic Storm Sewer Design</u> Few systems are designed any more by purely static means: that is, estimating peak runoffs for a given reach of a drainage system simply by adding peak flows estimated from individual watersheds. However, in order to "simplify" calculations, procedures and assumptions are sometimes made less than dynamic. The result is an inconsistency in system capacity: some reaches are overdesigned for the design storm; and, worse, others are underdesigned. This would apply to both inlets and their laterals and also main conveyance facilities.

Regardless of the hydrologic method used, hydraulic routing should be performed so that, at any given point, facilities are being designed for realistic peak flows. If the drainage system is a single conveyance element, the job is simple. However, systems are generally more complex, consisting of catch basin inlets, pipes, and overflow channels or streets. Under these

TABLE "H-1"
SCHEMATIC DRAINAGE PLAN

Watershed A	Watershed B	Watershed C	Intensities
A = 5.0 AC	A = 5.0 AC	A = 5.0 AC	I [10 min] = 4.4 in/hr
C = 0.80	C = 0.80	C = 0.80	I [14 min] = 3.8 in/hr
Tc = 10 min	Tc = 10 min	Tc = 10 min	I [18 min] = 3.3 in/hr

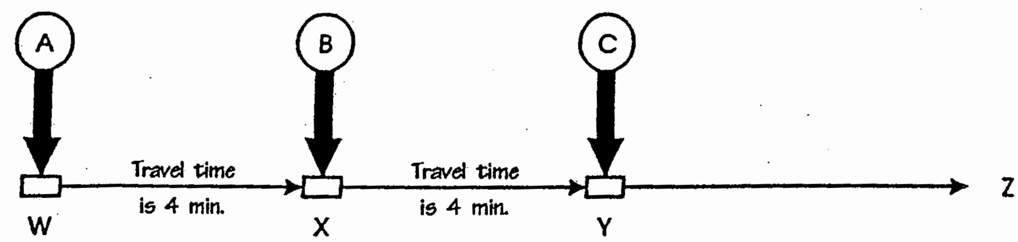


TABLE "H-2"
FLOW CALCULATION PROCEDURES AND RATES
(Rates given are cfs)

PIPE REACH IDENTIFIER	INCORRECT PROCEDURE	CORRECT PROCEDURE	HYDROGRAPH ROUTING RESULTS
AW	- -	$Q_T = Q_A$ (10) = 17.6	18
WX	- -	$Q_T = Q_A$ (10) = 17.6	18
BX	$Q_T = Q_B$ (14) = 15.2	$Q_T = Q_B$ (10) = 17.6	18
XY	$Q_T = Q_{A+B}$ (14) = 30.4	$Q_T = Q_A$ (10) + Q_B (14) = 32.8	33
CY	$Q_T = Q_C$ (18) = 13.2	$Q_T = Q_C$ (10) = 17.6	18
YZ	$Q_T = Q_{A+B+C}$ (18) = 39.6	$Q_T = Q_A$ (10) + Q_B (14) + Q_C (14) = 46.0	47

NOTE : Q_T = TOTAL FLOW IN A REACH OF PIPE
Q_A (10) = RUNOFF FROM WATERSHED A DETERMINED AT A TIME OF 10 MINUTES.

conditions, a detailed but not difficult or lengthy analysis and accounting process must be utilized to determine inlet interception capacity for respective flow rates, inlet overflow, and pipe/channel flow routing.

Inlets should be designed to intercept a desired percentage of the design storm peak, be that from a single adjacent watershed under a short time of concentration, or from an adjacent watershed at a later time with the addition of overflow from upstream inlets. If inlets lead to connector pipes or laterals, the laterals should be designed for the same peak flows.

Double checking to see which condition results in the greatest peak and keeping track of flows can be cumbersome. However, if a good chart is used for hand analyses, or, if computerized methods are used, the process can be done without undue effort.

a. <u>Hand Semi-Dynamic Storm Sewer Design</u> It would be hard to justify performing hydrograph routing, combining, and diverting analyses by hand. However, using the Rational Method, semi-dynamic results which are acceptable for most projects can be obtained. A number of design charts have been prepared through the years by various agencies and individuals. However, the principles of semi-dynamic routing and procedures contained in this manual have been incorporated into a set of design charts prepared by the author, which is presented in Table "H-3a" and "H-3b". At first sight, the tables will have a formidable appearance, but, once the worksheets are used, the process becomes relatively quick and reasonably simple. Furthermore, the worksheets guide the user through most of the checks necessary to ensure that appropriate estimated runoff peaks are used to size drainage facilities. However, in many systems, abnormalities exist, and the user must be constantly aware of what is going on, and should be liberal in use of footnotes, particularly for the benefit of a reviewer.

b. <u>Computerized Dynamic Storm Sewer Design</u> Although the above procedures are really not overly tedious, computer methods are less tedious, faster, possibly allow for less chance of error, and are capable of hydrograph routing which should increase the likelihood of achieving more realistic results.

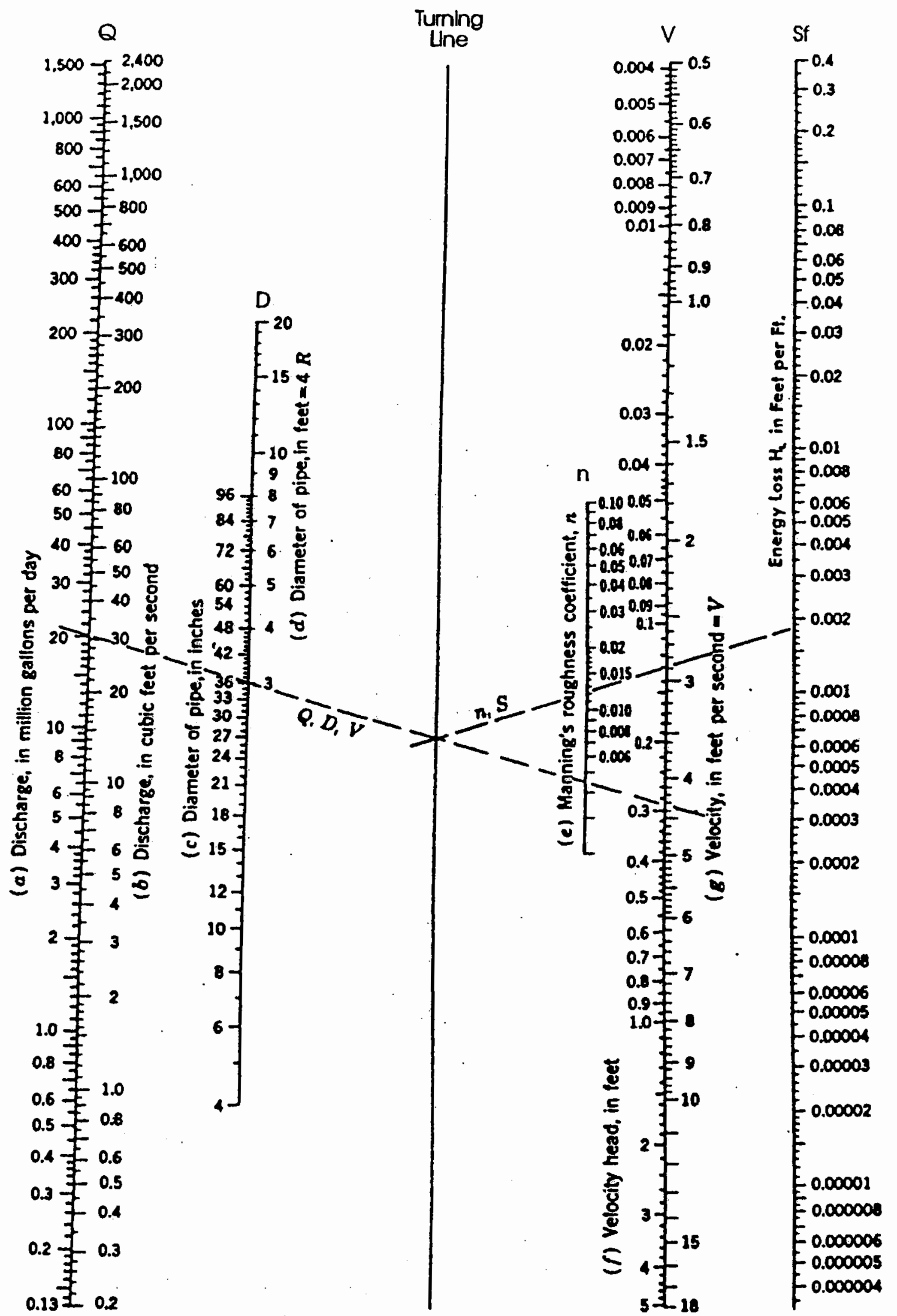
Since there are many different programs on the market, they cannot be discussed specifically. Consequently, procedures and options will only be discussed in general terms.

The computer methods will generate hydrographs for the various watersheds contributing to the system. These hydrographs may be routed as required for the specific project conditions. Inlet interception capacities can generally be entered in table form to indicate interception rates over a range of surface flow rates, which will include the limits of no flow and the peak flow expected for the design storm. The inlet interception then creates its own hydrograph diverted from the surface flow, and can be routed and combined with other diverted (intercepted) flows to establish peak flows and volumes being conveyed by the primary drainage facility (pipe or channel). Unintercepted flow remains on the surface as a hydrograph and is combined and routed with other surface hydrographs with diversions due to inlet interception or side overflows out of the basin as required for the specific project conditions. The end results are totally time consistent hydrographs which are based on flow rates experienced by the surface inlets, laterals, channels, and pipes.

7. <u>Design Aids</u> Several figures are provided as design aids. Figure "H-7" is a nomograph for pipes flowing just full or surcharged. Figure "H-8" allows conversion from full to partial pipe flow characteristics. When dealing with partial pipe flows, Figures "H-9" and "H-10" may be easier to use than Figures "H-7" and "H-8". Figures "H-11" and "H-12" allow determination of critical flow depth and hydraulic radius for circular conduits. For flow in open channels other than circular, such as rectangular, triangular, trapezoidal, or otherwise, see Appendix "I".

Tables "H-3a" and"H-3b" provide comprehensive worksheets for surface runoff and storm sewer design, incorporating procedures presented in this manual.

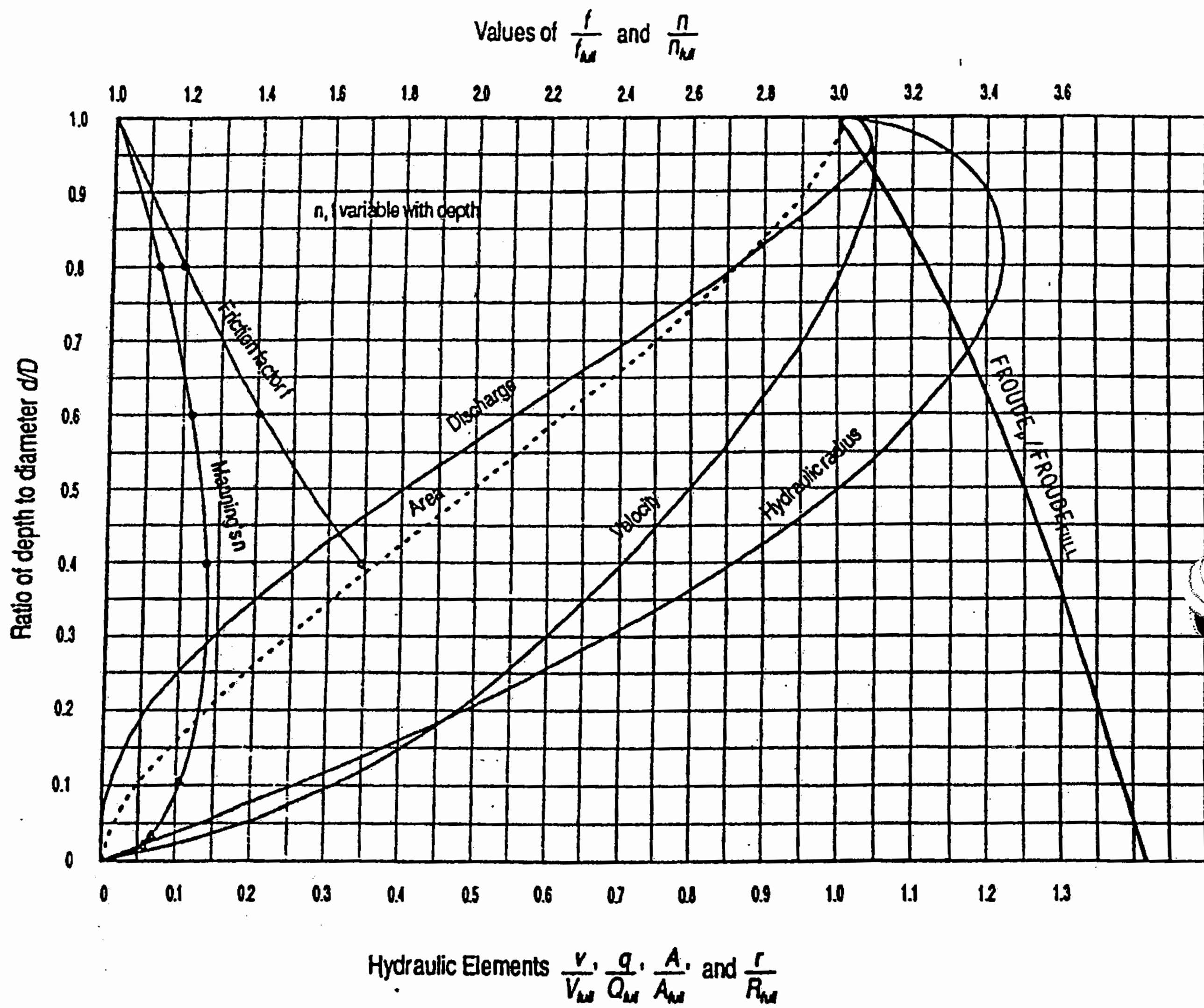
REPRODUCED FROM FIGURE 3-29a IN (AISI 1980)



Alignment chart for energy loss in pipes flowing full using Manning's formula.
Enter Q and D, obtain V. Start at Turning Line, enter "n", read Sf.
For partial flow, adjust values using Figure "H-8", or use Figures "H-9" and "H-10" instead of Figures "H-7 and "H-8".

UNIFORM FLOW FOR PIPES FLOWING FULL	FIGURE "H-7"

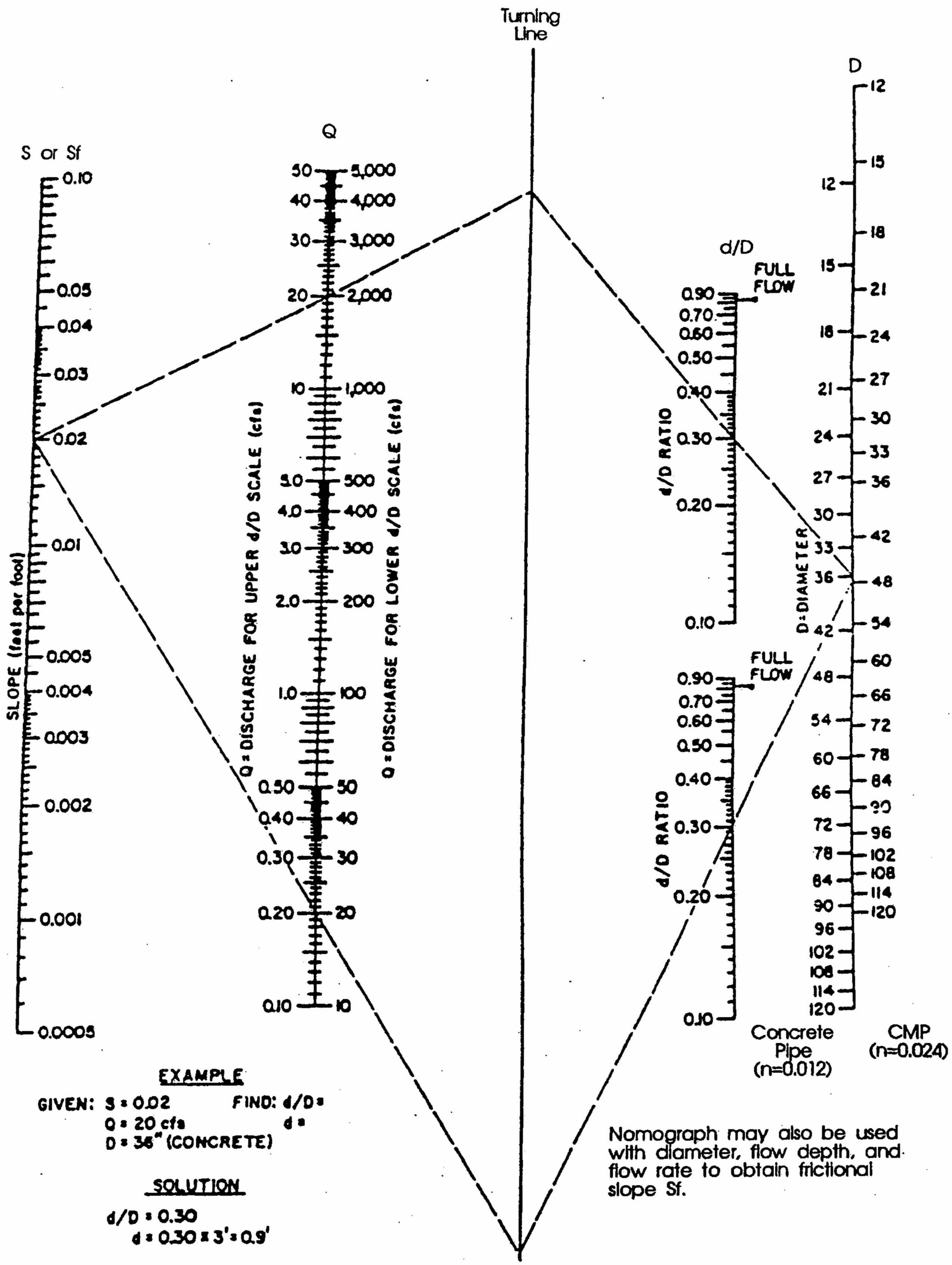
MODIFIED FROM FIGURE 4.6 IN MARICOPA COUNTY ADDING INFORMATION OBTAINED FROM FHWA HEC-14.



v = Actual velocity of flow, fps	A = Area occupied by flow ft^2
V_{full} = Velocity flowing full, fps	A_{full} = Area of pipe, ft^2
q = Actual quantity of flow, cfs	r = Actual hydraulic radius, ft
Q_{full} = Capacity flowing full, cfs	R_{full} = Hydraulic radius of full pipe, ft

HYDRAULIC ELEMENTS OF CIRCULAR CONDUITS | **FIGURE "H-8"**

REPRODUCED FROM FIGURE 8-1 IN (COLORADO SPRINGS)



EXAMPLE

GIVEN: S = 0.02
Q = 20 cfs
D = 36" (CONCRETE)

FIND: d/D =
d =

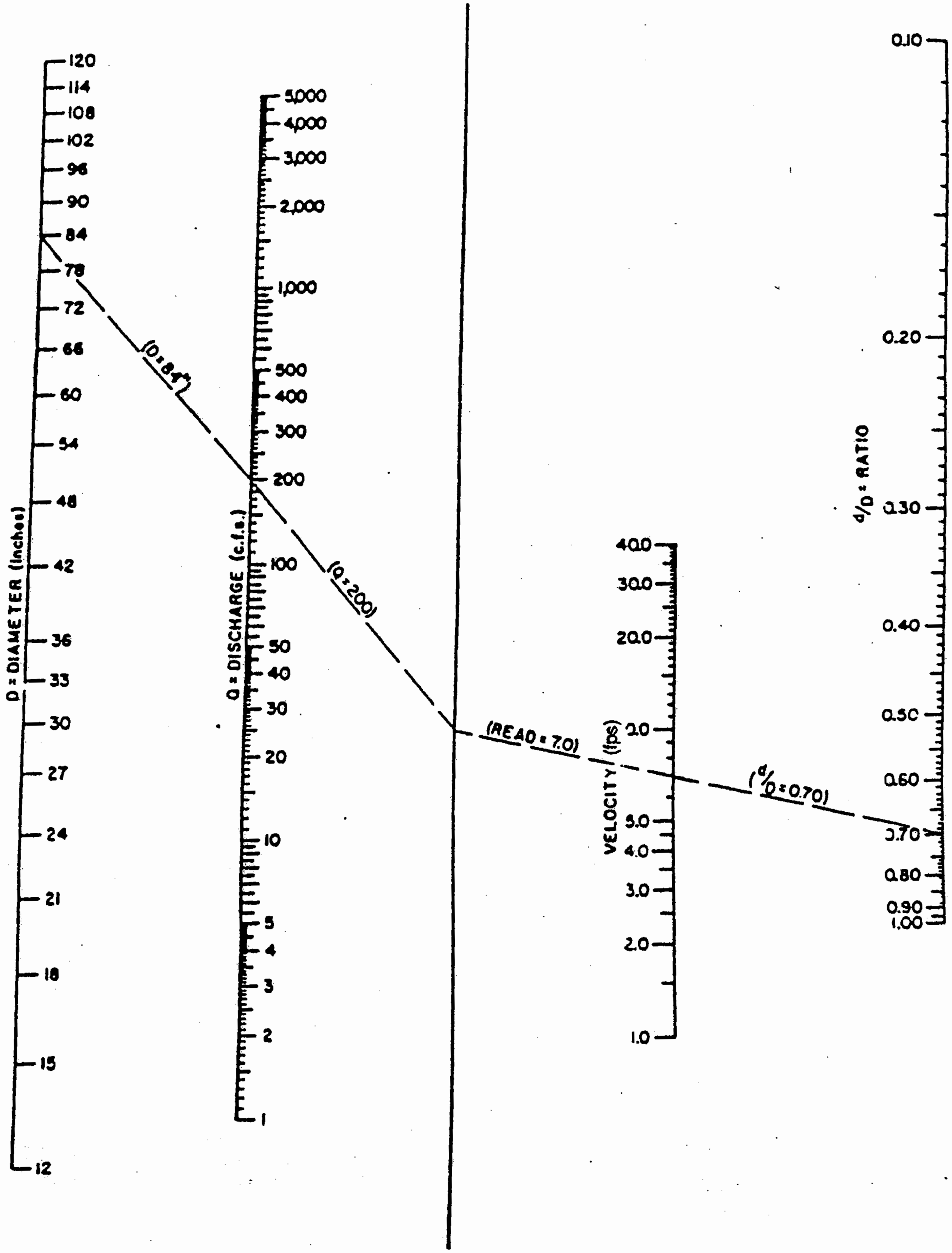
SOLUTION

d/D = 0.30
d = 0.30 x 3' = 0.9'

Nomograph may also be used with diameter, flow depth, and flow rate to obtain frictional slope Sf.

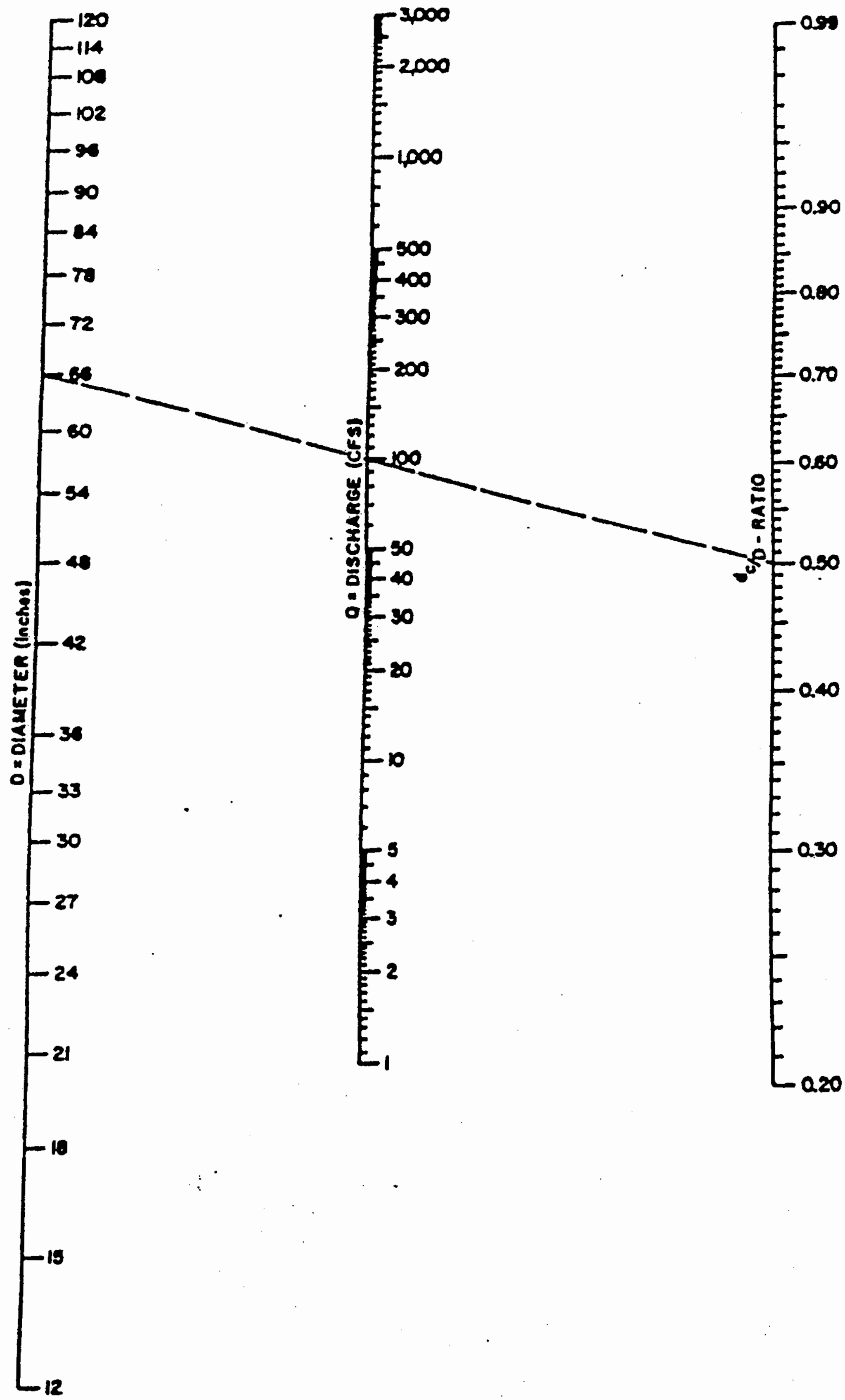
UNIFORM FLOW FOR PIPES FLOWING PARTIALLY FULL	FIGURE "H-9"

REPRODUCED FROM FIGURE 8-3 IN (COLORADO SPRINGS)



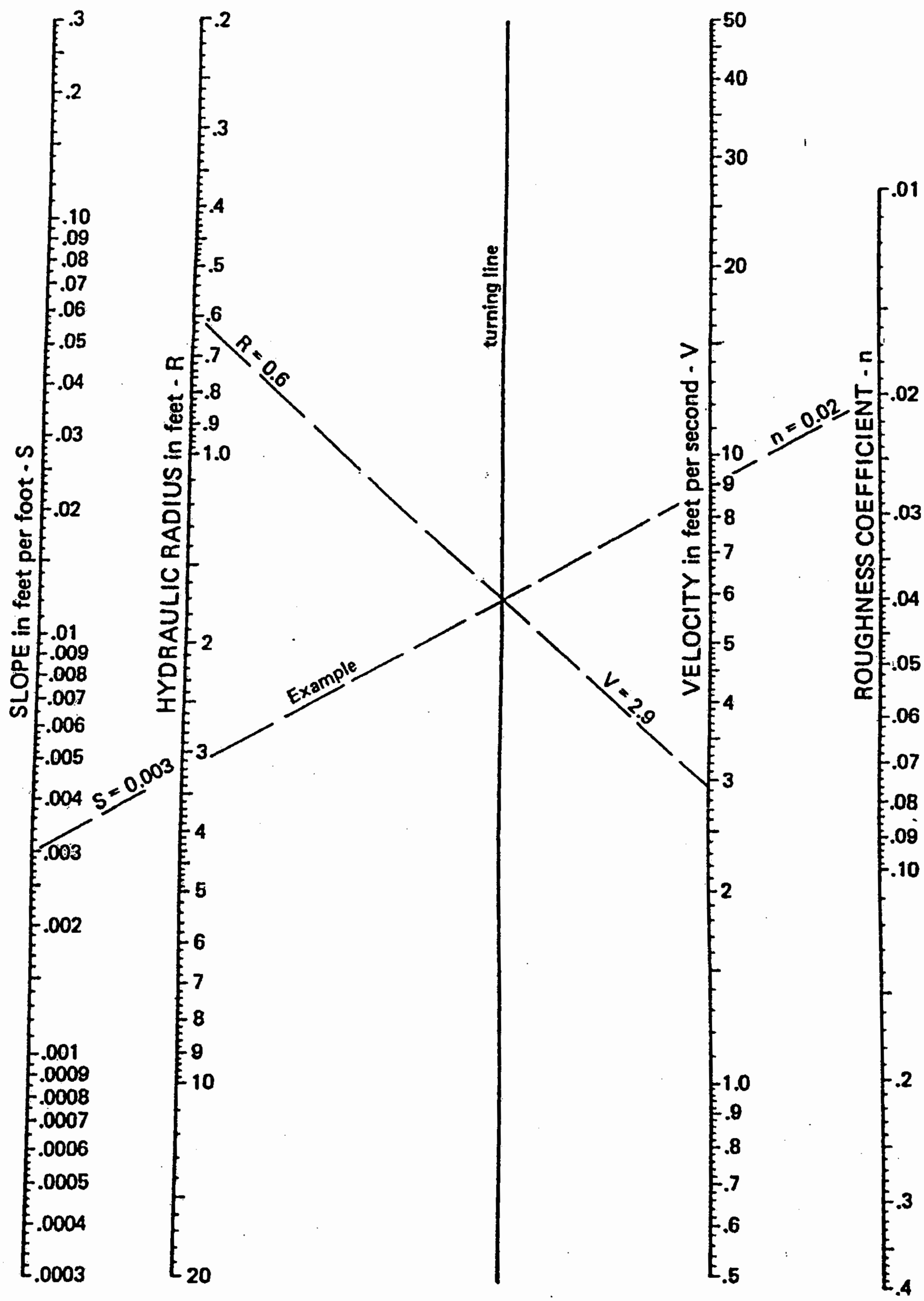
VELOCITY IN PIPE FLOWING PARTIALLY FULL	FIGURE "H-10"

REPRODUCED FROM FIGURE 8-2 IN (COLORADO SPRINGS)



CRITICAL DEPTH OF FLOW IN CIRCULAR CONDUITS	FIGURE "H-11"

REPRODUCED FROM (LINDEBURG)



Enter S and n, then V to R

HYDRAULIC RADIUS OF FLOW IN CIRCULAR CONDUITS	FIGURE "H-12"

NOTE: This worksheet pertains only to surface hydro| this worksheet would only be used for the 2-year storm condition.
However, under some conditions, there may

(1) Provide name of basin or subbasin to be analyzed.
(2,3) Identify limits of the watershed basin or subbasin, as applicable. The the top of a watershed to the uppermost set of inlets, from below a set o manhole to another set, or even half street flow or some other designat 24.
(4) - (8) Using Table "B-2" Reference "B-2" in step 4, and enter the value 8. column of Table "B-2" into step 8, skipping steps 5-7.
(4) - (8) Without Using Table "B-2" Describe each land use type on as ma required and enter the corresponding "c" value from Table "B-1" on pa acres of the land use; and the incremental addition of "c" times the area incremental "CA" values in step 7 for the basin or subbasin described i FOR enter the total in step 8.
(9) - (25) Using Table "E-3", "E-4", or "E-5" Reference in step 9 the applic tream "E-4", or "E-5" that is used to calculate the basin time of concentration so, the time for upstream runoff to flow through the reach or basin (Tr). Skip st enter Tc and Tr in steps 24 and 25, respectively. Table
(9) - (25) Without Using Table "E-3", "E-4", or "E-5" Select a method of es flow time. Options are the TR-55, HEC-12, and FAA methods (See Ap Provide a surface description and overland flow resistance factor "N" from Table "E-1" for the TR-55 & HEC-12 methods. If using the FAA m a surface description and runoff coefficients "C" from Table "A-1" (or s er it is

a single, double, or triple length inlet.
(37) Enter the intercepted runoff (Qi) as the lesser of step 32 and the value obtained from Figure "G-7" (on-grade) or Table "G-1" (sag), as applicable. Note that Figure "G-7" assumes "standard conditions" - if not applicable, the inlet capacity will have t be calculated (see Appendix "G").
(38) The summation of intercepted runoff ΣQi along the storm sewer system is general equal to (Qi + upstream ΣQi). However, if Qt = Qc, begin a new summation, i.e., ΣQi = Note that ΣQi is not necessarily the peak pipe flow, but is used with Qt to determine Qs or the flow remaining on the surface that must be intercepted.
(39) Use this step only if this is the lower end of a lateral or branch line where it will enter the main sewerline. Note that where half-street flows are analyzed separately, one si could be considered an inlet and connector, and the other side a "lateral line", becaus it falls in a lateral subbasin analysis. The additional equivalent CA ("c" value times acreage) that will enter the main sewer system is LAT CA = Qi/Ia (step 37/step 29), or, if Qi is at the uppermost inlet in a system, then LAT CA = Qi/Ic (step 37/step 27).
(40) By use of an identifier or footnote, one may record information as to which MH the "Qi" goes to, or which inlet that flow by Qs-Qi goes to, or any other information helpful t the designer and reviewer.

SURFACE DRAINAGE AREA								ULATIONS			INLET CONDITIONS				INLET INFLOW			
LOCATION			"C" VALUE					i Ta										
BASIN I.D. (1)	FROM PT. (2)	TO PT. (3)	LAND USE (4)	"C" VALUE (5)	AREA (AC) (6)	INCR CA (7)	TOTAL ADD'L CA (8)	Qa n/hr) (30)	Qt (cfs) (31)	Qs (cfs) (32)	Street Slope (%) (33)	Sag or on-grade ? (34)	Grate or Comb ? (35)	Sgl, Dbl or Tpl. ? (36)	Qi (cfs) (37)	ΣQi (cfs) (38)	LAT. CA (39)	FO NC ? (4

Project: ______________________

Page ____ of ____ Ref. Table "H-3b" sh

TABLE "H-3a"

not required if the outlet is unsubmerged; or if the outlet is submergec

NOTE: This worksheet pertains to storm sewers o
calculations are required only to the poi

(41)-(43) Identify the sewerline number or watershed basin which ... ll), and reach of sewerline to be analyzed. Start at the upper end of the se ... 2-a&c generally more convenient to analyze the main storm sewerline picking up side mains in the process, and then go back and analy the main sewerline. Note that the upper endpoint of a sewerline watershed that contributes to it. For example, a drainage subbas MH 8 contributes to the storm sewer at MH 8 (or its laterals), and sewerline is either the mainline between MH 8 and MH 7 or an in transferring information from table "H-3a" to this table, this must

(44) If the pipe reach or segment is a mainline or lateral branch lin connector pipe to a catch basin inlet, do this step and skip steps design flow intercepted by inlets, and the inlet connector pipe m ... se values flow. The value Qi is taken from step 37 on Table "H-3a".

(45) Identify the lateral branch line, which could also be a single in from a branch subbasin.

(46) The time independent "CA" that enters the main sewerline is Table "H-3a".

(47) The peak design flow that the main sewerline will convey is b pipeline. The Ta is provided in step 26 of Table "H-3a". Be sure ... ions by line; that is the "To Point" in Step 3 of Table "H-3a" should corres Inlet" in step 42 on this table. ... d lateral

(48) Using the Table "A-1" in Appendix "A," obtain the intensity la

sewerlines from the top to outfall of the system, begin with step 73 at the outfall an up on the chart (upstream in the system).

(73) Beginning at the outfall, enter the higher of the tailwater or step 59 d_n/D partial elevation at the end or brink of the pipe. Upstream pipe outlet HGL elevations are same as the downstream MH pipe inlet HGL elevations in step 74.

(74) The elevation of the HGL (EL_{HGL}) in the MH at the pipe inlet is determined as fo (See Figure "H-5"):
If $d_n/D > 0.80$, Inlet EL_{HGL} = Outlet EL_{HGL} + Hf + He + Hc + Hm (Steps 73 + 61 + 63 +
If $d_n/D \leq 0.80$ and outlet is unsubmerged,
Inlet EL_{HGL} = Inlet EL_{HGL} + Hm (Steps 72 + 70)
If $d_n/D \leq 0.80$, and outlet is submerged, use higher of:
Outlet EL + Hf + He + Hc + Hm (Steps 73 + 61 + 63 + 70); or
Inlet EL_{HGL} + Hm (Steps 72 + 70)

(75) Enter the elevation of the manhole rim, inlet grate, or storm sewer surface ope the upper end of the pipe reach. This value must be at least 1.0 foot below the ste value. If not, either the storm sewer must be redesigned to lower the HGL, or the opening must be raised. Note also that if step 75 minus step 74 is close to 1.0 foo manholes or junctions, that the system will likely be inadequate if catch basin inl connected at that point.

(76) Provide comment or list footnotes that may assist the designer and reviewer i following the analysis.

LOCATION			DESIGN FL[O]				[LO]SSES			PARTIAL		ELEVATIONS		
			Conn.	LATERAL LINE										
LINE NO. (41)	From MH or Inlet No. (42)	To MH or Inlet No. (43)	Qi (cfs) (44)	LAT. I.D. (45)	LAT. CA (46)	Ta (min) (47)	C_B (68)	K (69)	Hm (ft) (70)	$D(1-\frac{d_n}{D})$ (ft) (71)	EL_{HGL} @ PIPE INLET (ft) (72)	EL_{HGL} @ PIPE OUTLET (ft) (73)	EL_{HGL} in MH or Junction @ PIPE INLET (ft) (74)	MH RIM or INLET GRATE (ft) (75)

Project: ______

TABLE "H-3b"

Ref. Table "H

NOTE: This worksheet pertains only to surface
However, under some conditions, therlly, this worksheet would only be used for the 2-year storm condition.

(1) Provide name of basin or subbasin to be analyzed.
(2,3) Identify limits of the watershed basin or subbasin, as appl the top of a watershed to the uppermost set of inlets, from be or the manhole to another set, or even half street flow or some other
(4)-(8) Using Table "B-2" Reference "B-2" in step 4, and enter step 24. column of Table "B-2" into step 8, skipping steps 5-7. step 8.
(4)-(8) Without Using Table "B-2" Describe each land use type required and enter the corresponding "c" value from Table "B acres of the land use; and the incremental addition of "c" time incremental "CA" values in step 7 for the basin or subbasin de enter the total in step 8. PED FOR
(9)-(25) Using Table "E-3", "E-4", or "E-5" Reference in step 9 "E-4", or "E-5" that is used to calculate the basin time of conce upstream time for upstream runoff to flow through the reach or basin (Tr 5". If so, the enter Tc and Tr in steps 24 and 25, respectively.
(9)-(25) Without Using Table "E-3", "E-4", or "E-5" Select a me enter Table flow time. Options are the TR-55, HEC-12, and FAA methods Provide a surface description and overland flow resistance fa from Table "E-1" for the TR-55 & HEC-12 methods. If using the a surface description and runoff coefficients "C" from Table "A ition; whether it is

a single, double, or triple length inlet.
(37) Enter the intercepted runoff (Qi) as the lesser of step 32 and the value obtai from Figure "G-7" (on-grade) or Table "G-1" (sag), as applicable. Note that Figu "G-7" assumes "standard conditions" - if not applicable, the inlet capacity will b be calculated (see Appendix "G").
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(39) Use this step only if this is the lower end of a lateral or branch line where it will e the main sewerline. Note that where half-street flows are analyzed separately, o could be considered an inlet and connector, and the other side a "lateral line", be it falls in a lateral subbasin analysis. The additional equivalent CA ("c" value time acreage) that will enter the main sewer system is LAT CA = Qi/Ia (step 37/step 29 if Qi is at the uppermost inlet in a system, then LAT CA = Qi/Ic (step 37/step 27).
(40) By use of an identifier or footnote, one may record information as to which MH "Qi" goes to, or which inlet that flow by Qs-Qi goes to, or any other information he the designer and reviewer.

SURFACE DRAINAGE AREA						
LOCATION			"C" VALUE			
BASIN I.D. (1)	FROM PT. (2)	TO PT. (3)	LAND USE (4)	"C" VALUE (5)	AREA (AC) (6)	INCR CA (7)

ALCULATIONS				INLET CONDITIONS				INLET INFLOW		
n at Ta										
r))	Qa (in/hr) (30)	Qt (cfs) (31)	Qs (cfs) (32)	Street Slope (%) (33)	Sag or on-grade ? (34)	Grate or Comb ? (35)	Sgl, Dbl or Tpl. ? (36)	Qi (cfs) (37)	ΣQi (cfs) (38)	LAT. CA (39)

Project: ____________

Page ___ of ___ Ref. Table "H-3b"

TABLE "H-3a"

e not required if the outlet is unsubmerged; or if the outlet is submerge

NOTE: This worksheet pertains to storm sewer
calculations are required only to the p

(41)-(43) Identify the sewerline number or watershed basin w[...]all), and reach of sewerline to be analyzed. Start at the upper end of th[...]s 2-a&c generally more convenient to analyze the main storm sewer[...] picking up side mains in the process, and then go back and a[...]e the main sewerline. Note that the upper endpoint of a sewer[...] watershed that contributes to it. For example, a drainage sub[...]b)[35], MH 8 contributes to the storm sewer at MH 8 (or its laterals), [...] sewerline is either the mainline between MH 8 and MH 7 or a [...] transferring information from table "H-3a" to this table, this n[...]

(44) If the pipe reach or segment is a mainline or lateral branc[...] connector pipe to a catch basin inlet, do this step and skip st[...] design flow intercepted by inlets, and the inlet connector pip[...] Use values flow. The value Qi is taken from step 37 on Table "H-3a".

(45) Identify the lateral branch line, which could also be a sing[...] from a branch subbasin.

(46) The time independent "CA" that enters the main sewerlin[...] Table "H-3a".

(47) The peak design flow that the main sewerline will convey[...]itions by pipeline. The Ta is provided in step 26 of Table "H-3a". Be s[...] line; that is the "To Point" in Step 3 of Table "H-3a" should co[...]nd lateral Inlet" in step 42 on this table.

(48) Using the Table "A-1" in Appendix "A," obtain the intensi[...]

sewerlines from the top to outfall of the system, begin with step 73 at the outfall a[...] up on the chart (upstream in the system).

(73) Beginning at the outfall, enter the higher of the tailwater or step 59 d_n/D partia[...] elevation at the end or brink of the pipe. Upstream pipe outlet HGL elevations ar[...] same as the downstream MH pipe inlet HGL elevations in step 74.

(74) The elevation of the HGL (EL_{HGL}) in the MH at the pipe inlet is determined as f[...] (See Figure "H-5"):
If $d_n/D > 0.80$, Inlet EL_{HGL} = Outlet EL_{HGL} + Hf + He + Hc + Hm (Steps 73 + 61 + 63[...]
If $d_n/D \leq 0.80$ and outlet is unsubmerged,
Inlet EL_{HGL} = Inlet EL_{HGL} + Hm (Steps 72 + 70)
If $d_n/D \leq 0.80$, and outlet is submerged, use higher of:
Outlet EL + Hf + He + Hc + Hm (Steps 73 + 61 + 63 + 70); or
Inlet EL_{HGL} + Hm (Steps 72 + 70)

(75) Enter the elevation of the manhole rim, inlet grate, or storm sewer surface o[...] the upper end of the pipe reach. This value must be at least 1.0 foot below the s[...] value. If not, either the storm sewer must be redesigned to lower the HGL, or th[...] opening must be raised. Note also that if step 75 minus step 74 is close to 1.0 fo[...] manholes or junctions, that the system will likely be inadequate if catch basin i[...] connected at that point.

(76) Provide comment or list footnotes that may assist the designer and reviewe[...] following the analysis.

LOCATION			DESIGN				[...]ES			PARTIAL		ELEVATIONS		
			Conn.	LATERAL LIN[...]			[...]OSSES							
LINE NO. (41)	From MH or Inlet No. (42)	To MH or Inlet No. (43)	Qi (cfs) (44)	LAT. I.D. (45)	LAT. CA (46)	Ta (mi[...] (47)	C_B (68)	K (69)	Hm (ft) (70)	$D(1-\frac{d_n}{D})$ (ft) (71)	EL_{HGL} @ PIPE INLET (ft) (72)	EL_{HGL} @ PIPE OUTLET (ft) (73)	EL_{HGL} in MH or Junction @ PIPE INLET (ft) (74)	MH RIM or INLET GRATE (ft) (75)

Project: ______________________ [...]RS

TABLE "H-3b"

Page ___ of ___ Pof Table "H[...]

APPENDIX "I"
CHANNEL FLOW

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List of Figures

List of Tables

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APPENDIX "I"
CHANNEL FLOW

A. BASIC CONCEPTS OF OPEN CHANNEL FLOW

Open channel flow is complex and not easily described in a short appendix. Entire books have been devoted to the subject of open channel flow. What is provided herein is a brief review of definitions, flow types, parameters, and procedures. For additional information, refer to other sources such as FHWA's HEC-15, HDS-3, and HDS-4; and also to books on open channel flow.

1. Definition of Open Channel Flow Flow that is conveyed in such a manner that the top surface is bounded by the atmosphere is open channel flow. This occurs in natural channels, and also in artificial covered and uncovered conveyance facilities, such as canals, ditches, drainage channels, culverts, and pipes under partially full flow conditions.

2. Hydraulic Flow Types Open channel flow is classified as follows: (1) uniform or nonuniform flow; (2) steady or unsteady flow; and (3) subcritical or supercritical flow. In uniform flow, the depth and discharge remain constant along the channel. In steady flow, no change in discharge occurs over time. Most natural flows are unsteady and are described by runoff hydrographs. It can be assumed in most cases that the flow will vary gradually and can be described as steady, uniform flow for short periods of time. Subcritical flow is distinguished from supercritical flow by a dimensionless number called the Froude number (Fr), which is defined as the ratio of inertial forces to gravitational forces in the system. Subcritical flow ($Fr < 1.0$) is characterized as tranquil and has deeper, slower velocity flow. Supercritical flow ($Fr > 1.0$) is characterized as rapid and has shallower, higher velocity flow.

   For design purposes, uniform flow conditions are usually assumed with the energy slope approximately equal to average bed slope. This allows the flow conditions to be defined by a uniform flow equation such as Manning's equation. Supercritical flow creates surface waves that may approach the depth of flow. For very steep channel gradients, the flow may splash and surge in a violent manner and special considerations for freeboard are required.

3. Parameters and Terms Used in Open Channel Flow Analyses Several terms and parameters are used and must be understood when analyzing open channel flow. These are described below.

   a. Area (A) The area always means the cross-sectional area of the flow, and is measured perpendicular to the direction of flow.

b. Wetted Perimeter (Pw) The wetted perimeter is the portion of the perimeter of a flow conveyance facility that is in contact with the flowing liquid.

c. Hydraulic Radius (Rh) The hydraulic radius is the cross-sectional area of flow divided by the wetted perimeter, or Rh =A/Pw.

d. Depth (d) If not specified otherwise, depth of flow refers to the maximum depth of water in the cross section.

e. Surface Spread (T) The surface spread is the width at the top of the flow, measured perpendicular to the flow direction.

f. Hydraulic Depth (Dh) The hydraulic depth is the ratio of area in flow to the width of the channel at the fluid surface, or Dh = A/T.

g. Slope (S) Slope may refer to the channel bed, the hydraulic grade line, or energy grade line.

h. Hydraulic Grade Line (HGL) In an open channel, the hydraulic grade line is the profile of the free water surface.

i. Hydraulic Gradient (Hg) The slope of the hydraulic grade line is the hydraulic gradient.

j. Energy Grade Line (EGL) The grade line of the water surface profile plus the velocity head, or the specific energy line.

k. Critical Flow This refers to flow at critical depth or velocity, where the specific energy is a minimum for a given discharge. Critical flow is very unstable.

l. Critical Depth (dc) This refers to the depth of flow under critical flow conditions.

m. Critical Velocity This refers to the velocity of flow under critical flow conditions.

n. Critical Slope This refers to the slope which, for a given cross-section and flow rate, results in critical flow.

o. Froude Number (Fr) This is a dimensionless number, equal to the ratio of the velocity of flow to the velocity of very small gravity waves, the latter being equal to the square root of the product of the acceleration of gravity and the flow depth, or

$$Fr = \frac{V}{\left(\frac{gA}{T}\right)^{0.5}} = \frac{V}{(gd)^{0.5}}$$

When: $Fr < 1.0$, flow is subcritical;
$Fr = 1.0$, flow is critical; and
$Fr > 1.0$, flow is supercritical.

p. Normal Depth When the flow depth is constant along a channel reach; that is, when neither the flow depth nor velocity is changing, the depth is said to be normal.

q. Uniform Flow Uniform flow occurs when flow has a constant water area, depth, discharge, and average velocity through a reach of channel.

B. DESIGN PROCEDURES

1. Superelevation The centrifugal force caused by flow around a curve results in a rise in the water surface on the outside embankment and a potential depression of the surface along the inside embankment. This phenomenon is called superelevation. In addition, curved channels tend to create secondary flows (helicoidal motion) that may persist for many channel widths downstream. The shifting of the maximum velocity from the channel center line may possibly cause a disturbing influence downstream. The latter two phenomena could lead to serious local scour and deposition or poor performance of a downstream structure. There may also be a tendency toward separation near the inner wall, especially for very sharp bends. Because of the complicated nature of curvilinear flow, the amount of channel alignment curvature should be kept to a minimum as discussed in Subsection 3 that follows.

In tranquil flow channels, superelevation is usually small for the channel size. In rapid flow channels, the main problem is standing waves generated in simple curves. These waves not only affect the curved flow region but exist over long distances downstream. The total rise in water surface for rapid flow has been found experimentally to be about twice that for tranquil flow.

Generally, the most economical design for rapid flow in a curved channel results when wave effects are reduced as much as practical and embankment heights are kept to a minimum. Channel design for rapid flow usually involves low rates of channel curvature, the use of spiral transitions with circular curves, and consideration of invert banking.

The equation for the transverse water-surface slope around a curve can be obtained by balancing outward centrifugal and gravitational forces. The superelevation equation commonly used is

$$\Delta d = C\frac{V^2T}{gRc}$$

where:

Δd = rise in water surface between a theoretical level water surface at the center line and outside water-surface elevation (superelevation), ft;
C = coefficient (see Table "I-1");
V = mean channel velocity, fps;
T = surface spread, or the channel width at the water surface elevation, ft;
g = acceleration of gravity (assume 32.2 ft/s^2); and
Rc = radius of channel center-line curvature, ft.

TABLE "I-1"
SUPERELEVATION COEFFICIENTS

Flow Type	Froude Number	Channel Cross Section	Type of Curve	Value of C
Tranquil	Fr ≤ 0.86	Rectangular	Simple circular	0.6
Tranquil	Fr ≤ 0.86	Trapezoidal	Simple circular	0.5
Rapid	Fr ≤ 0.86	Rectangular	Simple circular	1.0
Rapid	Fr ≤ 0.86	Trapezoidal	Simple circular	1.0
Rapid	Fr ≤ 0.86	Rectangular	Spiral transitions	0.5
Rapid	Fr ≤ 0.86	Trapezoidal	Spiral transitions	1.0
Rapid	Fr ≤ 0.86	Rectangular	Spiral with bottom banked	0.5

This is a reproduction of information found in (ACOE 1970).

2. <u>Freeboard</u> Freeboard above 100-year runoff levels to finish floors is specified in Section I-A-3-b on page I-2. In addition, channel embankments shall have freeboard if they are higher than the surrounding terrain. The raised embankment freeboard requirement is presented in Table "I-2".

TABLE "I-2" RAISED EMBANKMENT FREEBOARD REQUIREMENTS			
Flow Regime	Froude Number	Freeboard Equation(1)	Minimum Freeboard(2)
Tranquil	$Fr \leq 0.86$	$0.15\left(d + \frac{v^2}{2g}\right) + \Delta d$	1.0' (2)
Near Critical or Supercritical	$Fr \leq 0.86$	$0.15\left(d + \frac{v^2}{2g}\right) + \Delta d$	2.0' (2)

where: d = depth of flow, ft;
Δd = increase in height due to superelevation, ft;
v = average velocity of flow, fps; and
g = acceleration of gravity (assume value of 32.2 ft/s^2).

(1) Superelevation calculations are only required for bends where the channel centerline radius (Rc) is ≤ 10T, where "T" is the top surface width of flow.
(2) For slopes greater than 10%, wave height usually reaches twice the mean flow depth. Freeboard shall be provided accordingly.

3. <u>Channel Curvature</u> The maximum centerline curvature of a channel shall be limited as shown in Table "I-3".

TABLE "I-3" MAXIMUM CENTERLINE CURVATURE Rc		
Flow Regime	Froude Number	Radius Equation
Tranquil	Fr $\leq$ 0.86	Rc $\geq$ 3T
Near critical to supercritical	Fr > 0.86	$Rc \geq \frac{4V^2T}{gd} = \frac{0.12V^2T}{d}$

where: Rc = channel centerline radius, ft;
T = surface width of water, ft;
V = average flow velocity, fps;
g = acceleration of gravity (assume 32.2 ft/s^2); and
d = average depth of flow, ft.

Note: If Rc $\geq$ 10%, the channel may be considered straight.

4. __Transitions__ Where an abrupt channel cross-section change occurs, special design considerations must be given to prevent an increase in the flow depth from back water effects, excessive velocities, hydraulic jumps, or other disturbances. Flow may remain subcritical or supercritical, change from subcritical to supercritical, or experience hydraulic jump in changing from supercritical to subcritical. If either of the channel flows are near critical depth, special care should be taken to prevent a change to the opposite type of flow that could result in channel overbanking or excessive erosion.

   For proper transition design, one must calculate critical and normal depth for each section at the peak design flow rate (see Figures "I-5" and "I-6"). This will indicate if flows are critical or subcritical. Use normal depth for the design depth (see Figure "I-1").

   For channel constrictions, the water surface level downstream must be set below the upstream water surface level by at least as much as the drop due to increased velocity head and fractional losses, or Δd as shown on Figure "I-1".

   Once the transition design is completed for peak design flow rate, the analysis should be repeated at a lower flow to ensure suitability of design at lower flows. Flow transitions must be complete before exiting a site onto property owned by others.

   There may be other conditions superimposed on the transition reach that may affect the final design. Transition deigns shown in Figure "I-1" are for the simplest cases with all other features held constant. Conditions may vary so that additional requirements may be called for in the transition design.

5. Drop Structures Rock drop structures and other channel flow energy dissipation and grade control structures shall be designed in accordance with engineering practices. Excellent resources are the UD & FCD and Maricopa County drainage manuals. Figure "I-2" provides guidance on the application of various types of facilities that may aid in selecting a type of drop structure prior to researching design procedures.

6. Permissible Velocities To mitigate erosion, flow velocities shall not exceed that allowed for liners per procedures presented in Appendix "J", nor the velocities shown below in Table "I-4".

7. Channel Liners There are many types of liners that may be used for channels. Appendix "J" presents design procedures for all types of flexible liners for flows less than 50 cfs, and for larger flows with use of riprap.

TABLE "I-4"
ALLOWABLE CHANNEL FLOW VELOCITIES

Channel Cover*	Maximum Velocity	
	Erosion Resistant Soil	Easily Eroded Soil
a) Bare soil	4	2.5
b) Buffalo Grass, Bluegrass, Smooth Brome, Blue Grama Native Grass Mix	7	5
c) Lespedeza, Lovegrass, Kudzu, Alfalfa, Crabgrass	4.5	3

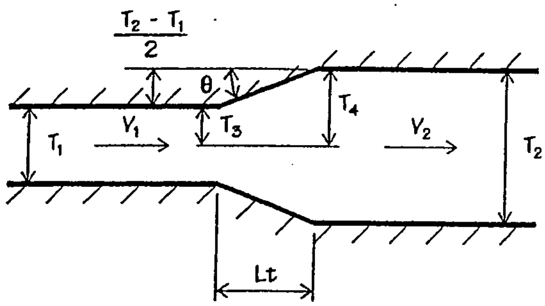
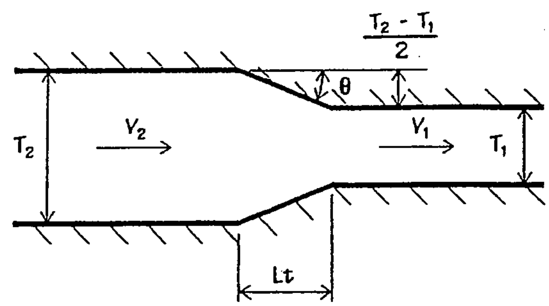
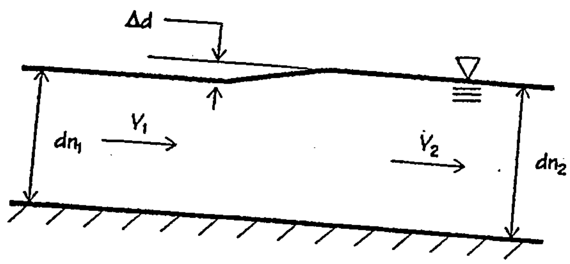
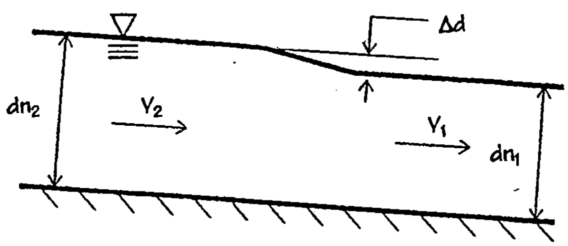
*Assuming a good stand of grass

Source: UD & FCD

C. DESIGN AIDS An assortment of nomographs, graphs, and chart are provided which may assist in the hydraulic design of open channels. These comprise Figure "I-3" through Figure "I-9". Worksheets for channel design are provided in Appendix "J" — Flexible Lining Erosion Protection". Table "I-5" provides a matrix of design charts that can be used in channel design. They do not account for transitions, however. These must be addressed separately.

TABLE "I-5"
CHANNEL DESIGN WORKSHEET MATRIX

Slope	Q ≤ 50 cfs	Q > 50 cfs
0-2%	Method: HEC-15 Reference: Appendix "J", Section II-A Worksheet: Table "J-2" Liner Types: Bare soil; straw net, jute, and other temporary liners; and riprap.	Method: HEC-11 Reference: Appendix "J", Section III-A Worksheet: Table "J-7" Liner Type: Riprap
2%-10%	Method: HEC-15 Reference: Appendix "J", Section II-A Worksheet: Table "J-2" Liner Types: Bare soil; straw net, jute, and other temporary liners; and riprap.	Method: HEC-11, CSU/Abt, ACOE Reference: Appendix "J", Sections III-A, III-B-1; & III-B-2 Worksheet: Table "J-7" (HEC-11); Table "J-9" (CSU/Abt & ACOE) Liner Type: Riprap
10%-20%	Method: HEC-15 Reference: Appendix "J", Section II-B Worksheet: Table "J-5" Liner Type: Riprap	Method: CSU/Abt ACOE (1) Reference: Appendix "J", Sections III-B-1 & III-B-2 Worksheet: Table "J-4" Liner Type: Riprap

CHANNEL WIDENING	CHANNEL CONSTRICTION
	
PLAN VIEW	PLAN VIEW
	
PROFILE	PROFILE
$\Delta d = 0.8 \frac{(V_1 - V_2)^2}{2g}$ - FRICTION LOSS	$\Delta d = 1.1 \frac{(V_1 - V_2)^2}{2g}$ + FRICTION LOSS
$\theta = \tan^{-1}[1/(3.375Fr)]$ (Pima County) $\tan\theta = 1/3.375Fr$	$\theta = \tan^{-1}[1/(3.375Fr)]$ (Pima County) $\tan\theta = 1/3.375Fr$
$Lt = 0.5[T_2 - T_1]/\tan\theta = 1.69Fr[T_2 - T_1]$ (From improved to improved channel) $Lt = 6.65Fr[T_4 - 0.7T_3]$ (From improved to natural channel — Pima County)	$Lt = 0.5(T_2 - T_1)/\tan\theta = 1.69Fr[T_2 - T_1]$ (From natural or improved to improved channel)

CAUTION: If $Fr \le 0.51$, use $\theta \le 30°$ or set $Fr = 0.51$ in the Lt equation unless it can be demonstrated that $\Delta d \le 0.1$ foot, $\Delta V \le 1$ fps, and V changes by less than 10 %.

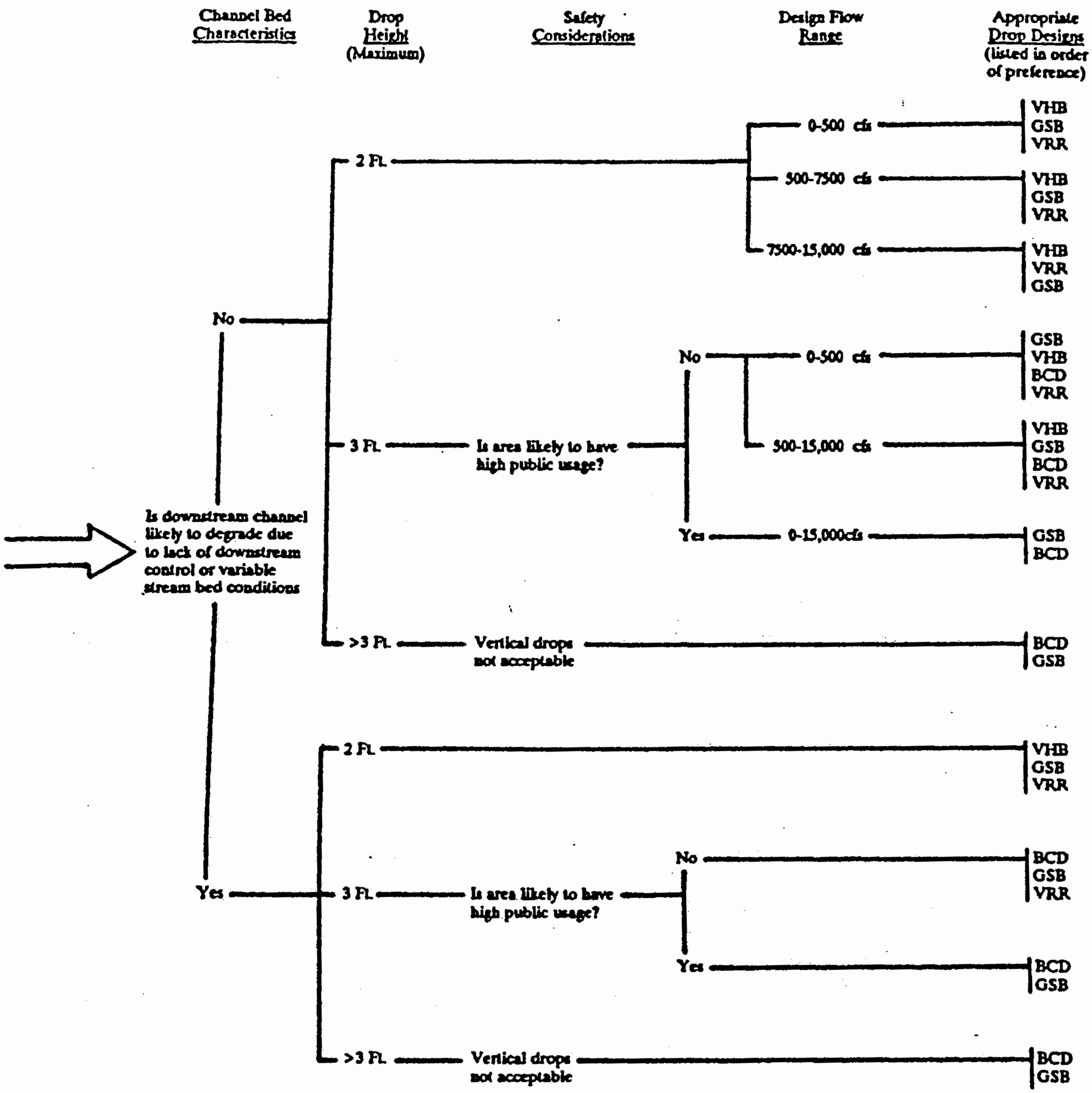
TERMS

V_1 = Velocity in narrow channel, ft
V_2 = Velocity in wide channel, ft
T_1 = Surface width of narrow channel, ft
T_2 = Surface width of wide channel, ft
T_3 = One-half of the upstream channel water surface, ft
T_4 = One-half of the downstream channel water surface, ft

Δd = Change in flow depth, ft
Lt = Length of channel transition, ft
θ = Transition angle, degrees
g = Acceleration of gravity (assume at 32.2 fps)
Fr = Froude number (Use upstream Froude in these equations)

TRANSITIONS BETWEEN TWO OPEN CHANNELS — **FIGURE "I-1"**

REPRODUCED FROM UD & FCD, FIGURE 2-1



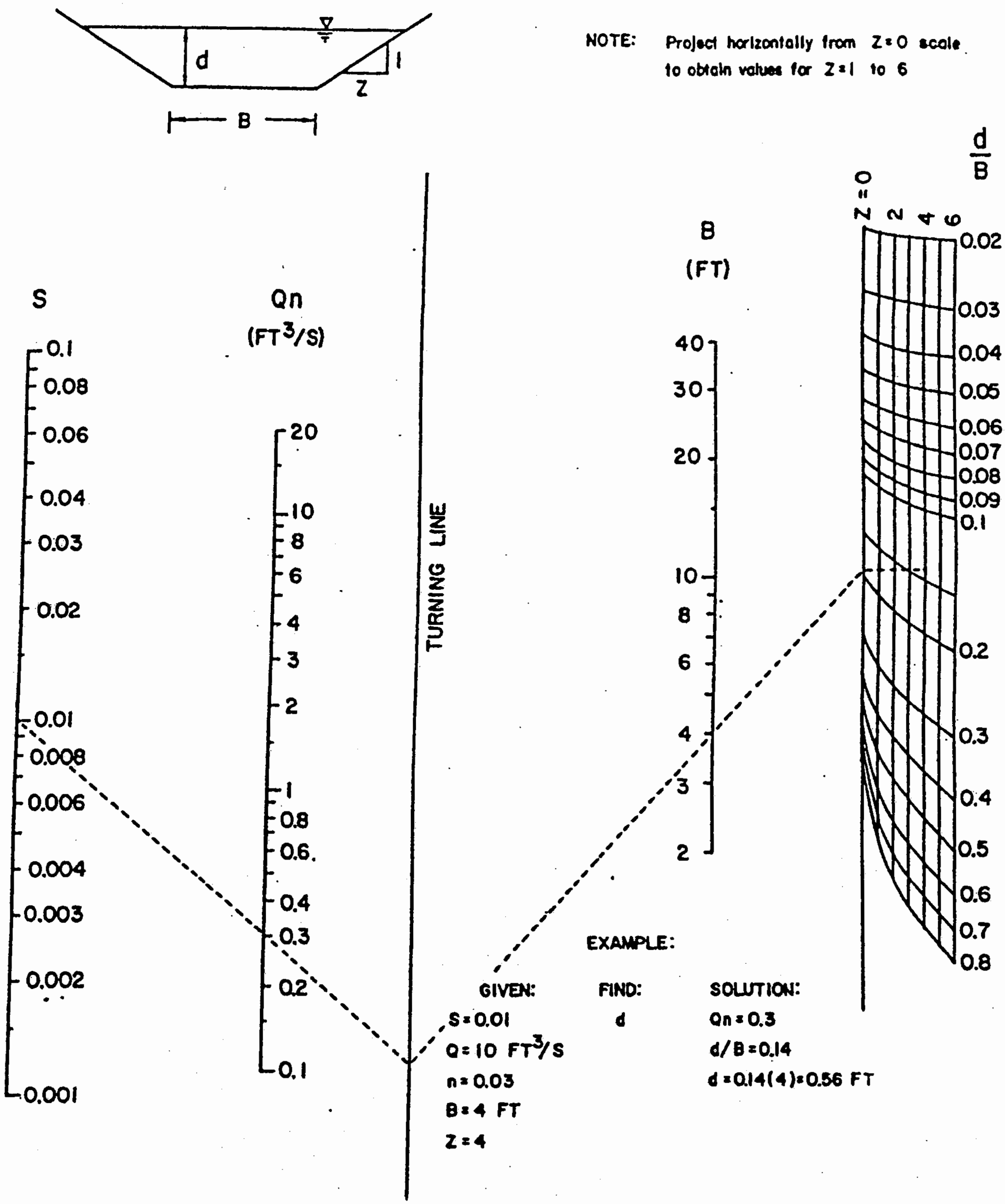
WHERE:

VRR = VERTICAL RIPRAP BASIN
VHB = VERTICAL HARD BASIN DROPS
GSB = GROUTED SLOPING BOULDER DROPS
BCD = BAFFLE CHUTE DROPS

DROP STRUCTURE SELECTION CHART

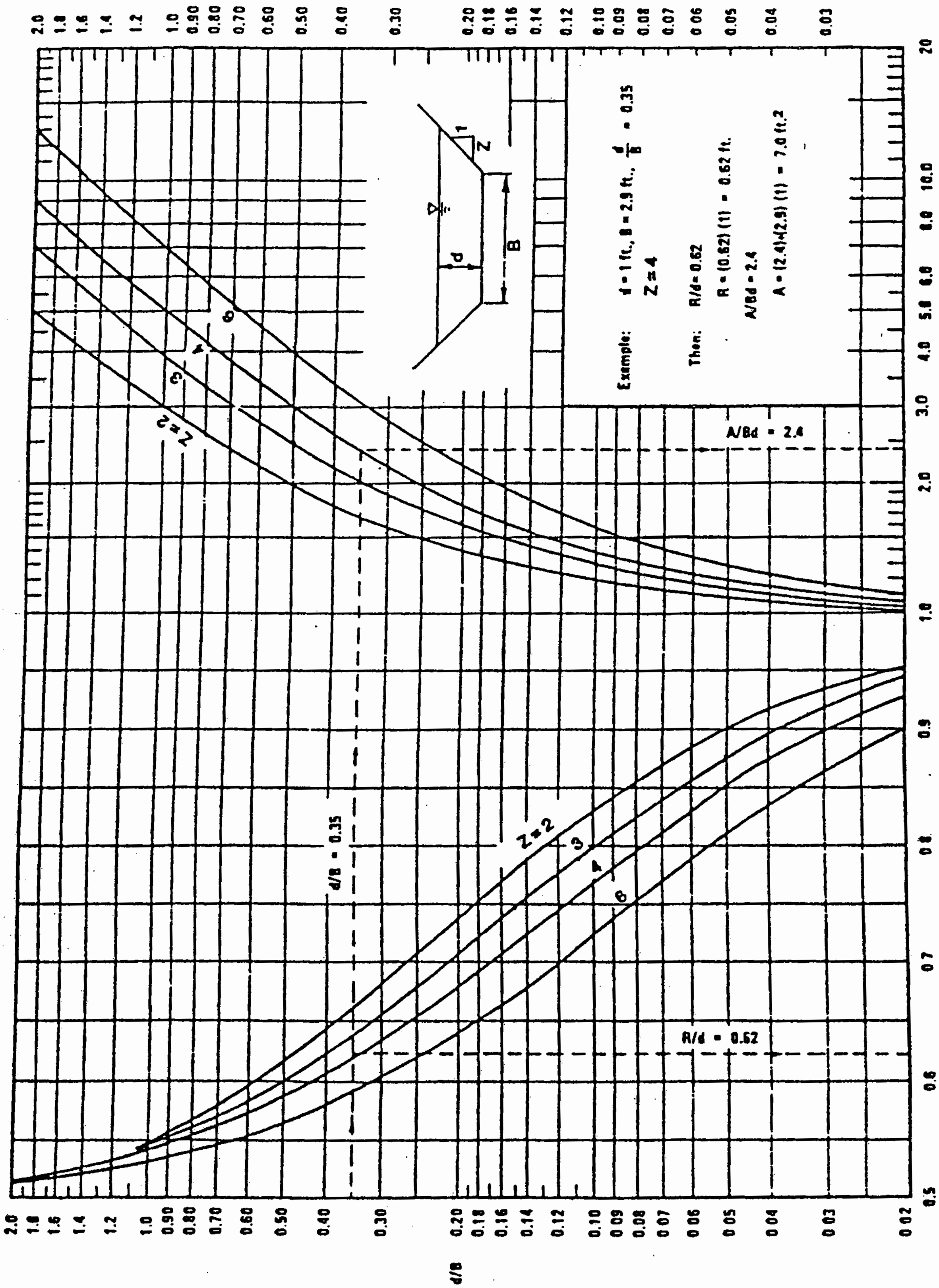
FIGURE "I-2"

REPRODUCED FROM FHWA HEC-15, CHART 3



TRAPEZOIDAL CHANNEL UNIFORM FLOW NOMOGRAPH	FIGURE I-3

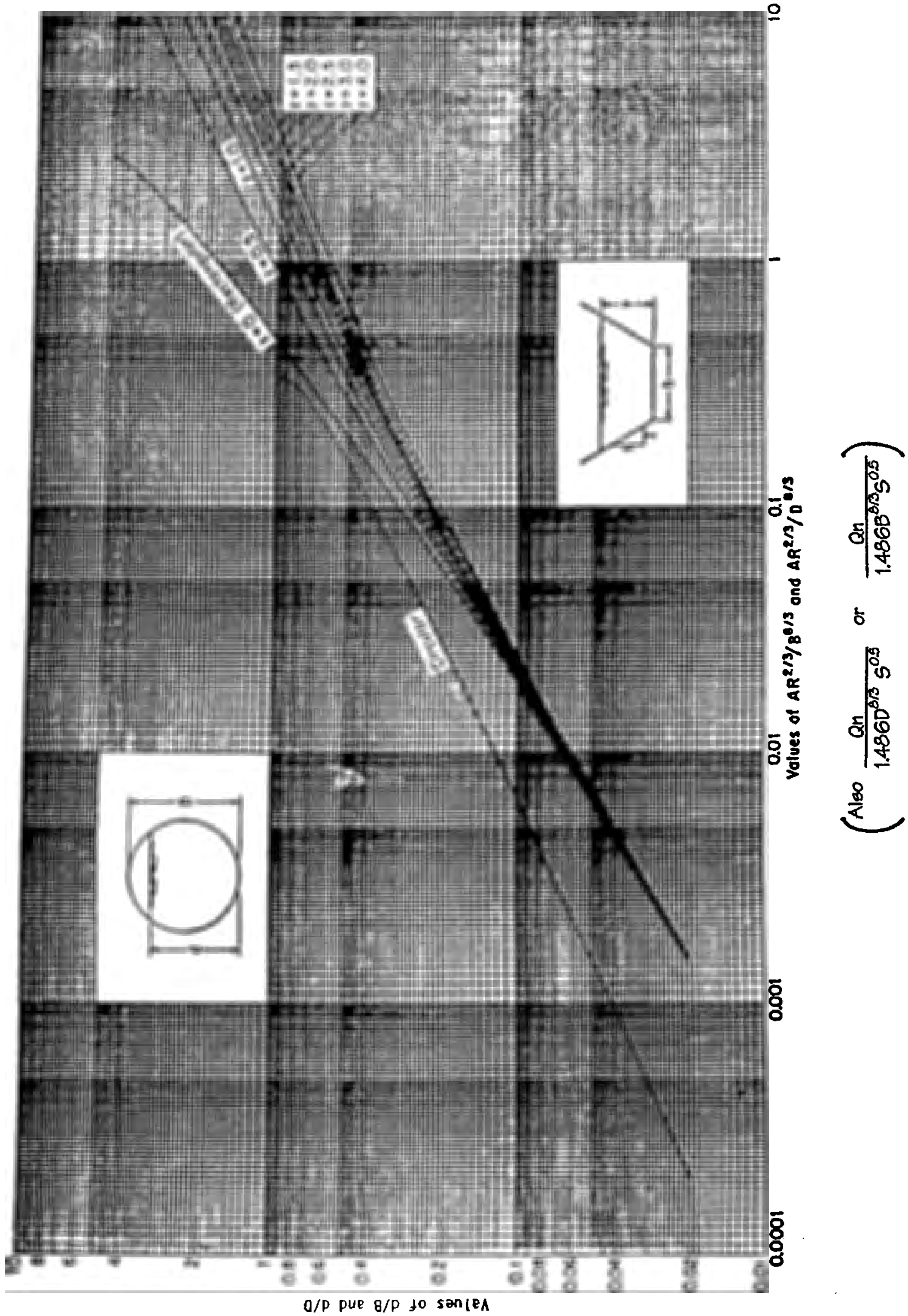
REPRODUCED FROM FHWA HEC-15, CHART 4



NOTE: R = HYDRAULIC RADIUS WHICH IS DENOTED AS "Rh" ELSEWHERE IN THIS MANUAL

TRAPEZOIDAL CHANNEL UNIFORM FLOW CHART	FIGURE "I-4"

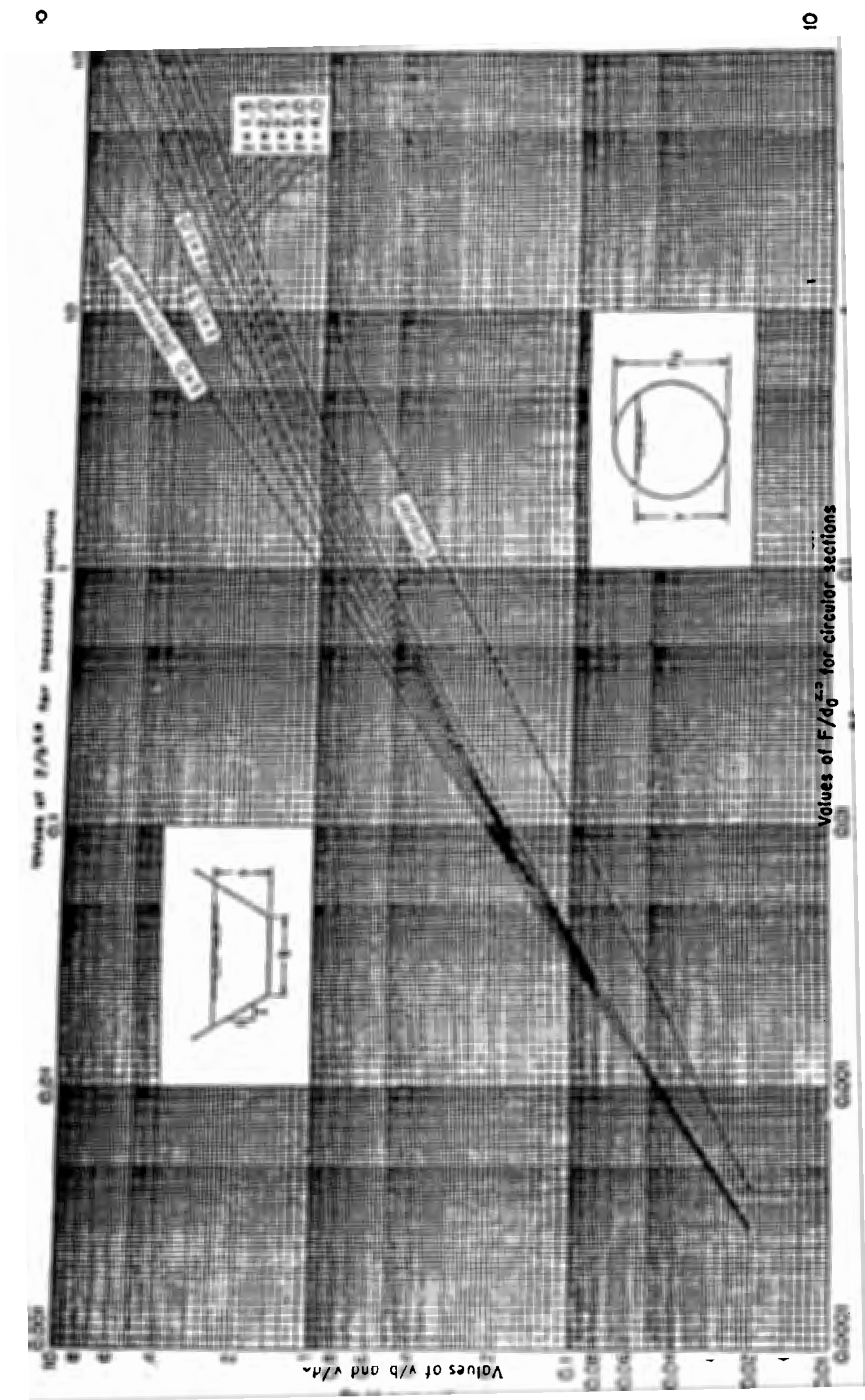
REPRODUCED FROM UD & FCD, FIGURE 2-1



NORMAL DEPTH FOR UNIFORM FLOW GRAPH

FIGURE "I-5"

REPRODUCED FROM UD & FCD, FIGURE 2-2



CRITICAL DEPTH FLOW GRAPH

FIGURE "I-6"

REPRODUCED FROM FHWA HEC-14, TABLE III-2

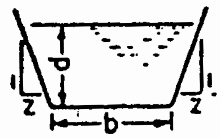
d = depth of flow (ft)
D = diameter of pipe (ft)
A = area of flow (ft^2)
R_h = hydraulic radius (ft)
Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface (ft/ft)

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

CIRCULAR CONDUIT UNIFORM FLOW TABLE

FIGURE "I-7"

REPRODUCED FROM FHWA HEC-14, TABLE III-1



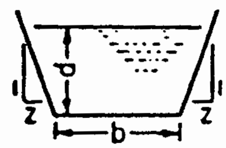
d/b[1]	Values of $\frac{Qn}{b^{8/3}S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.02	.00213	.00215	.00216	.00217	.00218	.00219	.00220	.00220	.00221	.00223
.03	.00414	.00419	.00423	.00426	.00429	.00431	.00433	.00434	.00437	.00443
.04	.00661	.00670	.00679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

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

TRAPEZOIDAL CHANNEL UNIFORM FLOW TABLE	FIGURE "I-8a"

REPRODUCED FROM FHWA HEC-14, TABLE III-1

Values of $\frac{Qn}{b^{8/3}S^{1/2}}$

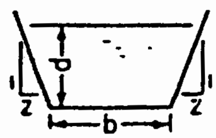


d/b	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.45	.256	.303	.346	.385	.422	.455	.487	.519	.548	.665
.46	.263	.313	.359	.401	.439	.475	.509	.541	.574	.696
.47	.271	.323	.371	.417	.457	.494	.530	.565	.600	.729
.48	.279	.333	.384	.432	.475	.514	.552	.589	.626	.763
.49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

TRAPEZOIDAL CHANNEL UNIFORM FLOW TABLE

FIGURE "I-8b"

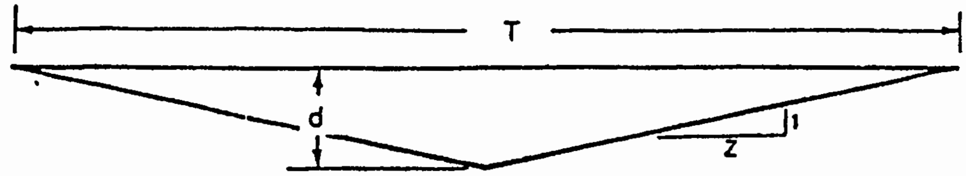
REPRODUCED FROM FHWA HEC-14, TABLE III-1

d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$ 									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
1.50	1.16	1.94	2.78	3.63	4.46	5.26	6.04	6.81	7.55	10.4
1.55	1.20	2.05	2.96	3.88	4.78	5.65	6.50	7.33	8.14	11.3
1.60	1.25	2.15	3.14	4.14	5.12	6.06	6.99	7.89	8.79	12.2
1.65	1.30	2.27	3.33	4.41	5.47	6.49	7.50	8.47	9.42	13.2
1.70	1.34	2.38	3.52	4.69	5.83	6.94	8.02	9.08	10.1	14.2
1.75	1.39	2.50	3.73	4.98	6.21	7.41	8.57	9.72	10.9	15.2
1.80	1.43	2.62	3.93	5.28	6.60	7.89	9.13	10.4	11.6	16.3
1.85	1.48	2.74	4.15	5.59	7.01	8.40	9.75	11.1	12.4	17.4
1.90	1.52	2.86	4.36	5.91	7.43	8.91	10.4	12.4	13.2	18.7
1.95	1.57	2.99	4.59	6.24	7.87	9.46	11.0	12.5	14.0	19.9
2.00	1.61	3.12	4.83	6.58	8.32	10.0	11.7	13.3	14.9	21.1
2.10	1.71	3.39	5.31	7.30	9.27	11.2	13.1	15.0	16.8	23.9
2.20	1.79	3.67	5.82	8.06	10.3	12.5	14.6	16.7	18.7	26.8
2.30	1.89	3.96	6.36	8.86	11.3	13.8	16.2	18.6	20.9	30.0
2.40	1.98	4.26	6.93	9.72	12.5	15.3	17.9	20.6	23.1	33.4
2.50	2.07	4.58	7.52	10.6	13.7	16.8	19.8	22.7	25.6	37.0
2.60	2.16	4.90	8.14	11.6	15.0	18.4	21.7	25.0	28.2	40.8
2.70	2.26	5.24	8.80	12.6	16.3	20.1	23.8	27.4	31.0	44.8
2.80	2.35	5.59	9.49	13.6	17.8	21.9	25.9	29.9	33.8	49.1
2.90	2.44	5.95	10.2	14.7	19.3	23.8	28.2	32.6	36.9	53.7
3.00	2.53	6.33	11.0	15.9	20.9	25.8	30.6	35.4	40.1	58.4
3.20	2.72	7.12	12.5	18.3	24.2	30.1	35.8	41.5	47.1	68.9
3.40	2.90	7.97	14.2	21.0	27.9	34.8	41.6	48.2	54.6	80.2
3.60	3.09	8.86	16.1	24.0	32.0	39.9	47.8	55.5	63.0	92.8
3.80	3.28	9.81	18.1	27.1	36.3	45.5	54.6	63.5	72.4	107
4.00	3.46	10.8	20.2	30.5	41.1	51.6	61.9	72.1	82.2	122
4.50	3.92	13.5	26.2	40.1	54.5	68.8	82.9	96.9	111	164
5.00	4.39	16.7	33.1	51.5	70.3	89.2	108	126	145	216

TRAPEZOIDAL CHANNEL UNIFORM FLOW TABLE

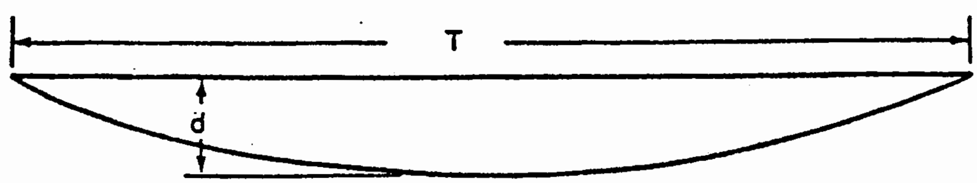
FIGURE "I-8c"

REPRODUCED FROM FHWA HEC-15, FIGURE 29



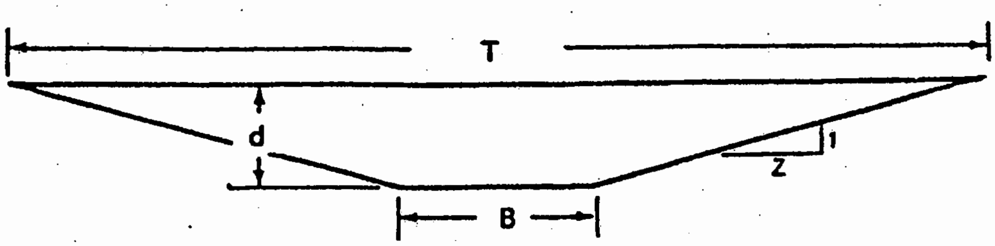
$$A = Zd^2$$
$$Wp = 2d\sqrt{Z^2 + 1}$$
$$T = 2dZ$$

V-SHAPE



$$A = \frac{2}{3}Td$$
$$Wp = \frac{1}{2}\sqrt{16d^2 + T^2} + \left(\frac{T^2}{8d}\right)\ln_e\left(\frac{4d + \sqrt{16d^2 + T^2}}{T}\right),$$
$$T = 1.5\frac{A}{d}$$

PARABOLIC

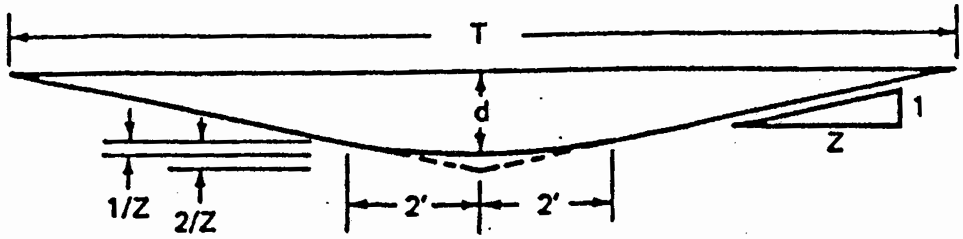


$$A = Bd + Zd^2$$
$$Wp = B + 2d\sqrt{Z^2 + 1}$$
$$T = B + 2dZ$$

TRAPEZOIDAL

EQUATIONS FOR VARIOUS CHANNEL GEOMETRIES | FIGURE I-9a

REPRODUCED FROM FHWA HEC-15, FIGURE 29



2 CASES

NO. 1

IF $d \leq 1/Z$, THEN:

$$A = \frac{8}{3} d \sqrt{dZ}$$

$$Wp = 2Z \ln_e \left(\sqrt{\frac{d}{Z}} + \sqrt{1 + \frac{d}{Z}} \right) 2 \sqrt{d^2 + dZ}$$

$$T = 4 \sqrt{dZ}$$

NO. 2

IF $d > 1/Z$, THEN:

$$A = \frac{8}{3} Z + 4 \left(d - \frac{1}{Z} \right) + Z \left(1 - \frac{1}{Z} \right)^2$$

$$Wp = 2 Z \ln_e \left(\frac{1}{Z} + \sqrt{\frac{Z^2 + 1}{Z}} \right) + 2 \sqrt{\frac{1 + Z^2}{Z}} + 2 \left(d - \frac{1}{Z} \right) \sqrt{1 + Z^2}$$

$$T = 2Z \left(d + \frac{1}{Z} \right)$$

V-SHAPE WITH ROUNDED BOTTOM

EQUATIONS FOR VARIOUS CHANNEL GEOMETRIES | FIGURE I-9b

APPENDIX "J"
FLEXIBLE LINING EROSION PROTECTION

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APPENDIX "J"
FLEXIBLE LINING EROSION PROTECTION

I. GENERAL DISCUSSION

Runoff produces erosive forces which may, if not mitigated by some means of erosion protection, cause undesirable damage to drainage facilities and nearby properties. Protection in channels, whether for the entire section or only the embankment or revetment, and at outlets of culverts or other drainage structure, is often desirable to maintain the integrity of the drainage system. The type of protection best suited for a particular purpose will depend upon a variety of factors, including hydraulic conditions, economic factors, soil conditions, material availability, aesthetic considerations, and site restraints.

The principles of hydraulic erosion can be quite complex, and are treated at length in many publications. This appendix will not present all aspects of erosion protection, but is intended to provide a basis that may assist the designer with more common applications.

A. LINING MATERIALS

Traditional materials used in rigid erosion protection systems include cast-in-place concrete, concrete modular block or stone masonry, asphaltic concrete, soil cement, grouted riprap, and use of metal products. Common materials used in flexible erosion protection systems include ordinary and wire-enclosed riprap, gravel, and vegetation. For temporary conditions during bed stabilization, products such as straw with or without netting, curled wood or synthetic mats, jute, paper, and synthetic nets have been used. In more recent years, geofabrics and geogrids, alone and with vegetation (soil bioengineering) have been used with success in many applications.

Rock riprap is the most commonly used material for erosion protection of drainage systems, and is therefore the main focus of this appendix. The term "riprap" may include rubble, broken concrete slabs, and preformed concrete shapes, but, as discussed in this appendix, the term riprap will only have reference to rock.

B. CHANNEL LINER DESIGN GUIDELINES

1. **Flow Range** Channel flows of 50 cfs or less are usually sufficiently uniform that average hydraulic conditions may be assumed. Also, these channels are small enough that, where erosion protection is necessary, usually the entire channel section is lined. Channel flows greater than 50 cfs are usually not uniform, flow concentrations may develop, and erosive forces can be quite dynamic. Consequently, liner design procedures are separated based upon flow rate.

2. **Channel Slope** Flow velocities tend to increase with channel slope, which results in more drag force which liner materials must resist. Also, the greater the slope, the

more that the gravitational force is applied to materials in a downstream direction. Consequently, procedures developed for liner design are different for mild and steep channel slopes.

3. <u>Flow Transitions</u> Forces of erosion are usually high at locations of flow transition. Expansion and contraction cause turbulence that is highly erosive. Liners are often required at these locations. Protection upstream and downstream, as applicable, must be of adequate length to allow flow to fully transition.

   a. <u>Channel Bends</u> At channel bends, transition length, but not length of protection, is discussed in Appendix "I". Where protection lengths are covered in Sections II and III of this Appendix as part of a Methods procedure, follow them. Otherwise a general rule of thumb provided in Subsection "4" below may be used.

   b. <u>Outlet Flow Transition</u> For culvert and other concentrated discharge from outlets, protection lengths are covered in Section IV of this appendix.

   c. <u>Steep to Mild Gradient Change</u> Another application is extent of riprap between the toe of a steep gradient and the beginning of a mild slope. The transition distance should be between 3 and 5 times the mean rock diameter (D_{50}) required on the steep gradient. The transition from a steep gradient channel to a mild gradient channel may require an energy dissipation structure such as a plunge pool.

   d. <u>Mild to Steep Gradient Change</u> The transition from a mild gradient to a steep gradient should be protected against local scour upstream of the transition for a distance of approximately five times the downstream uniform depth of flow.

4. <u>Channel Alignment</u> Channels for which no protection is required in straight sections may have need of protection at bends, and channel reaches where protection is required may have need of larger riprap at bends. For design purposes, a channel may be considered straight if the centerline of channel radius (Rc) is at least 10 times greater than the surface width of flow (T).

   a. <u>Flow ≤ 50 cfs, Slope ≤ 10%</u> Protection or increased protection, as applicable, shall be provided in accordance with HEC-15 procedures outlined in Section II-A.

   b. <u>Flow > 50 cfs, Slope ≤ 10%</u> If the design procedures used lack specific guidelines for protection length, it is advisable that the protection be at least 1.0 times the surface water width (T) in an upstream direction from the bend, all the way through the bend, and a distance of at least 1.5T beyond the bend in the downstream direction.

c. Slope ≥ 10% Bends should be avoided on steep gradient channels. A design requiring a bend in a steep channel should be reevaluated to eliminate the bend or designed using a culvert.

5. Superelevation Where flow superelevation occurs, the amount shall be calculated per Section B-1 of Appendix "I". A protective liner shall be high enough to protect embankments for the superelevated condition plus required freeboard.

6. Freeboard Channel freeboard shall conform to requirements given in Section B-2, Appendix "I". Freeboard embankments shall receive the same liner protection that the adjacent channel bottom and side slopes receive. Note that freeboard height shall, as a minimum, equal twice the mean depth of flow for slopes exceeding 10%.

7. Liner Thickness Most of the design methods presented herein pertain to riprap. All riprap procedures result in determination of a minimum required rock size, usually the mean rock size D_{50}, although one procedure results in a minimum D_{30} rock size. The riprap blanket, measured perpendicular to the grade, shall meet or exceed the thickness of 2.25 D_{30}, 2.0 D_{50}, and 1.5 D_{100}. In addition to the riprap, a liner includes 0.25 foot of granular bedding below the riprap and over a filter fabric. Riprap and fabric specifications are provided in Section V of this Appendix.

8. Liner Flanks At the approach and discharge edges of a channel liner, and at the termination of an outlet liner, a liner flank is required. Typically, upstream liner flanks have a thickness of 5.0 feet or 3 times the liner thickness, whichever is less. Downstream flanks are typically 3.0 feet or 2 times the liner thickness, whichever is less.

C. CHANNEL LINER DESIGN METHOD SELECTION There are several published methods of designing a channel liner. Each of the methods has applicable conditions for which they are best suited. Table "J-1" provides a matrix that shows method application, allowing proper method selection for a given problem. Also provided in the table are references to where in this appendix design procedures are presented.

TABLE "J-1"
CHANNEL DESIGN WORKSHEET MATRIX

Slope	Q ≤ 50 cfs	Q > 50 cfs
0-2%	Method: HEC-15 Reference: Appendix "J", Section II-A Worksheet: Table "J-2" Liner Types: Bare soil; straw net, jute, and other temporary liners; and riprap.	Method: HEC-11 Reference: Appendix "J", Section III-A Worksheet: Table "J-7" Liner Type: Riprap
2%-10%	Method: HEC-15 Reference: Appendix "J", Section II-A Worksheet: Table "J-2" Liner Types: Bare soil; straw net, jute, and other temporary liners; and riprap.	Method: HEC-11, CSU/Abt, ACOE Reference: Appendix "J", Sections III-A, III-B-1; & III-B-2 Worksheet: Table "J-7" (HEC-11); Table "J-9" (CSU/Abt & ACOE) Liner Type: Riprap
10%-20%	Method: HEC-15 Reference: Appendix "J", Section II-B Worksheet: Table "J-5" Liner Type: Riprap	Method: CSU/Abt ACOE (1) Reference: Appendix "J", Sections III-B-1 & III-B-2 Worksheet: Table "J-9" Liner Type: Riprap

Note (1) The FHWA does not limit usage of HEC-11 based upon slope, and therefore it could be used in the 10% to 20% range. However, the riprap size determined by HEC-11 procedures in this range becomes extremely large, and does not appear realistic. Through the 2% to 20% range, the CSU/Abt equation will likely yield the smallest required rock, the ACOE equation slightly larger, and HEC-11 the largest of the three methods. Judgement as to which procedure produces the best results for a given application is left up to the user. It is noted, however, that at slopes of 10% and above, loose riprap is usually not a very economical design, and other means of erosion protection ought to be explored.

II. CHANNEL DESIGN, FLOW ≤ 50 CFS (HEC-15)

This section is divided into two parts, depending upon channel slope. Subsection "A" pertains to slopes of 10% or less; subsection "B" pertains to greater slopes.

A. FLEXIBLE LINER CHANNEL DESIGN FOR SLOPES ≤ 10%

1. **General Guidelines** This subsection outlines design procedures for flexible linings on channel slopes less than or equal to 10% and flows not exceeding 50 cfs. Primary emphasis is on riprap; however, other flexible liners will be presented as well for comparative purposes. When riprap is used on steeper gradients, the design procedure must take into consideration the additional forces acting on the riprap. Designs involving riprap on grades approaching 10% should be checked and compared to results obtained from design procedures presented in Subsection B which follows. The more conservative results, i.e., the largest riprap size, shall be used for design.

   Design procedures for flexible linings involves only two computations and several comparisons of lining performance. Computations include a determination of uniform normal flow depth in the channel (see Appendix "I" — Channel Flow), and determination of shear stress at maximum flow depth. The comparison required in the design procedure is that of permissible to computed shear stress for a lining. If the computed shear stress exceeds the permissible shear, the lining is considered unacceptable, and a lining with a higher permissible shear stress is selected and calculations repeated. Channels lined with gravel or riprap on side slopes steeper than 3H:1V must be designed using the steep side slope design procedure presented in Subsection B which follows.

2. **Design Procedures** The design procedure for flexible linings is presented in the worksheet on Table "J-2".

Project: ______________________ Channel I.D. __________ (1) Design Q___=________cfs Design by: ____________________ Date: ____________

(2) Trial/Reach ___S = ___ft/ft	Trial/Reach ___S = _____ft/ft	Trial/Reach ___S = _____ft/ft	Trial/Reach ___S = _____ft/ft	Trial/Reach ___S = _____ft/ft
Lining Category:__________	Lining Category:__________	Lining Category:__________	Lining Category:__________	Lining Category:__________
Lining Type: __________	Lining Type: __________	Lining Type: __________	Lining Type: __________	Lining Type: __________

STEPS (1) - (26)

(1) & (2) Enter Q, trial/reach no., channel slope, lining description from Table "J-3", and sketch the channel section with base width B and side slope "Z" horizontal to 1 vertical.

(3) - (5) From Tbl. "J-3", obtain τ_p and estimated flow depth (di) and corresponding Manning "n" value.

(6) - (8) For non-vegetative liners, skip these steps.

(6) Use values in steps 1, 2, & 5 with Fig. "I-3" to get first iteration of di.

(7) Enter Fig. "I-4" with di/B and obtain R/d (Rh/di). Multiply by di to obtain R (or Rh).

(8) Use Chart "F-1" in Appendix F to determine "n".

(9) Calculate flow depth (d) using Fig. "I-3".

(a) For non-vegetative linings, d must fall within the depth range assumed when selecting an "n" value from Table "J-3" in Step 5. If not, repeat steps 4, 5, & 9.

(b) For vegetative linings, if "d" differs by more than 0.1 foot from "di", repeat steps 4-9.

(10) Calculate shear stress $\tau_d = 62.4$ (d) (S) [see steps 2 & 9] If $\tau_d \leq \tau_p$ (step 3), the lining is acceptable for straight reaches. If not, repeat steps 4-10.

(11, 12) Calculate water surface width T and area of flow (See Fig. "I-9").

(13) V = Q/A (See steps 1 & 12)

(14) $Fr = 0.176V/(A/T)^{0.5}$

(15) - (19) = If there are no bends for which Rc < 10T, skip to step 19, enter zero, and continue. Otherwise, steps 15-19 must be done. However, if τ_d is close to τ_p, it may be well to skip steps 15-18 for the current trial liner type, and complete the straight channel liner design. Then, select another liner type with a higher τ_p, and start a new trial for use in the bend. If τ_d is significantly larger than τ_p, one may as well continue with step 15.

(15) For $Fr \leq 0.86$, minimum Rc = 3T
For Fr > 0.86, minimum $Rc = 0.12V^2T/d$
Enter design Rc which must not be less than minimum Rc.

(16) Determine bend shear stress Kb from Fig. "J-3"

(17) Calculate $\tau_b = (Kb)(\tau_d)$ - See steps 10 & 16.

(18) Calculate R (or Rh), if not done in step 7. Use Fig "J-4" to obtain Lp.

(19) Superelevation $\Delta d = CV^2T/(32.2Rc)$, where C values are presented in Table "I-1".

(20) & (21) For $Fr \leq 0.86$, freeboard $= 0.15(d+v^2/2g) + \Delta d$ at bends.
For Fr > 0.86, freeboard $= 0.25(d+v^2/2g) + \Delta d$ at bends.
Enter freeboard without Δd (step 20) and with Δd (step 21) for straight and bend reaches, respectively.

(22) Determine the rock angle of repose from Fig. "J-5".

(23) (24) Determine K1 and K2 from Fig. "J-6" and "J-7", respectively.

(25) Calculate D_{50} (side slopes) = K1/K2 times D_{50} (bottom)

(26) If the slope was near 10%, the liner should also be analyzed by HEC-11 procedures. Use the most conservative results.

Trial/		Estimate		Vegetative			Channel Hydraulics						Channel Bends					Freeboard		Side slope rock size				HEC
																				(Do only if $2 \leq Z < 3$				11
Rch. No. (2)	τ_p (psf) (3)	di (ft) (4)	'n' (5)	di (ft) (6)	Rh (ft) (7)	n (8)	d (ft) (9)	τ_d (psf) (10)	T (ft) (11)	A (ft^3) (12)	V (fps) (13)	Fr (14)	Rc (ft) (15)	Kb (16)	τ_b (psf) (17)	Lp (ft) (18)	Δd (ft) (19)	Straight (ft) (20)	Bend (ft) (21)	θ (22)	K1 (23)	K2 (24)	D_{50} (25)	D_{50} (26)

FLEXIBLE LINER CHANNEL DESIGN WORKSHEET (HEC-15)

$Q \leq 50$ cfs, $0\% \leq$ SLOPE $\leq 10\%$

TABLE "J-2"

TABLE "J-3"
PERMISSIBLE SHEAR STRESSES AND "n" VALUES FOR LINING MATERIALS
(Taken from Tables 2 and 3 in FHWA's HEC-15)

Lining Material		τp(1) lb/ft^2	Manning "n" at Depth Ranges (2)		
Category	Type		0.0-0.5 ft	0.5-2.0 ft	> 2.0 ft
Temporary (3)	Woven paper net	0.15	0.016	0.015	0.015
	Jute net	0.45	0.028	0.022	0.019
	Fiberglass roving:				
	single	0.60	0.029	0.021	0.019
	double	0.85	0.028	0.021	0.019
	Straw with net	1.45	0.065	0.033	0.025
	Curled wood mat	1.55	0.066	0.035	0.028
	Synthetic mat	2.00	0.036	0.025	0.021
Vegetative	Class A	3.70	—	—	(0.14)
	Class B	2.10	—	(0.18)	(0.075)
	Class C (4)	1.00	—	(0.075)	(0.050)
	Class D	0.60	(0.35)	(0.055)	(0.042)
	Class E	0.35	(0.19)	(0.046)	(0.038)
Gravel Riprap	1-inch D_{50}	0.33	0.044	0.033	0.030
	2-inch D_{50}	0.67	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	2.00	0.104	0.069	0.035
	12-inch D_{50}	4.00	—	0.078	0.040
Bare Soil	Non-cohesive	See Fig. "J-1"	0.023	0.020	0.020
	Cohesive	See Fig. "J-2"	0.023	0.020	0.020

1) τp is the permissible unit shear stress
2) See Appendix "F" for Manning's "n" value for other conditions. Values shown for vegetative liners in parentheses are preliminary, to be used only for an initial estimate in Table "J-2".
3) Some temporary linings become permanent when buried.
4) See Table "J-4" for vegetative descriptions.

NOTE: THIS IS A REPRODUCTION OF TABLE 1 IN HEC-15

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 30") (76 cm)
	Yellow bluestem Ischaemum	Excellent stand, tall (average 36") (91 cm)
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12") (30 cm)
	Native grass mixture (little bluestem, blue- stem, blue gamma, and other long and short midwest grasses)..........	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24") (61 cm)
	Lespedeza sericea	Good stand, not woody, tall (average 19") (48 cm)
	Alfalfa	Good stand, uncut (average 11") (28 cm)
	Weeping lovegrass	Good stand, unmowed (average 13") (33 cm)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13") (28 cm)
C	Crabgrass	Fair stand, uncut (10 to 48") (25 to 120 cm)
	Bermuda grass	Good stand, mowed (average 6") (15 cm)
	Common lespedeza	Good stand, uncut (average 11") (28 cm)
	Grass-legume mixture-- summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)....	Good stand, uncut (6 to 8 inches) (15 to 20 cm)
	Centipedegrass.............	Very dense cover (average 6 inches) (15 cm)
	Kentucky bluegrass.........	Good stand, headed (6 to 12 inches (15 to 30 cm)
D	Bermuda grass..............	Good stand, cut to 2.5-inch height (6 cm)
	Common lespedeza	Excellent stand, uncut (average 4.5") (11 cm)
	Buffalo grass	Good stand, uncut (3 to 6 inches (8 to 15 cm)
	Grass-legume mixture-- fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)...............	Good stand, uncut (4 to 5 inches) (10 to 13 cm)
	Lespedeza sericea	After cutting to 2-inch height (5 cm) Very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5 inch height (4 cm)
	Bermuda grass	Burned stubble

NOTE: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

CLASSIFICATION OF VEGETAL LINERS

TABLE "J-4"

THIS IS A REPRODUCTION OF CHART 1 IN HEC-15.

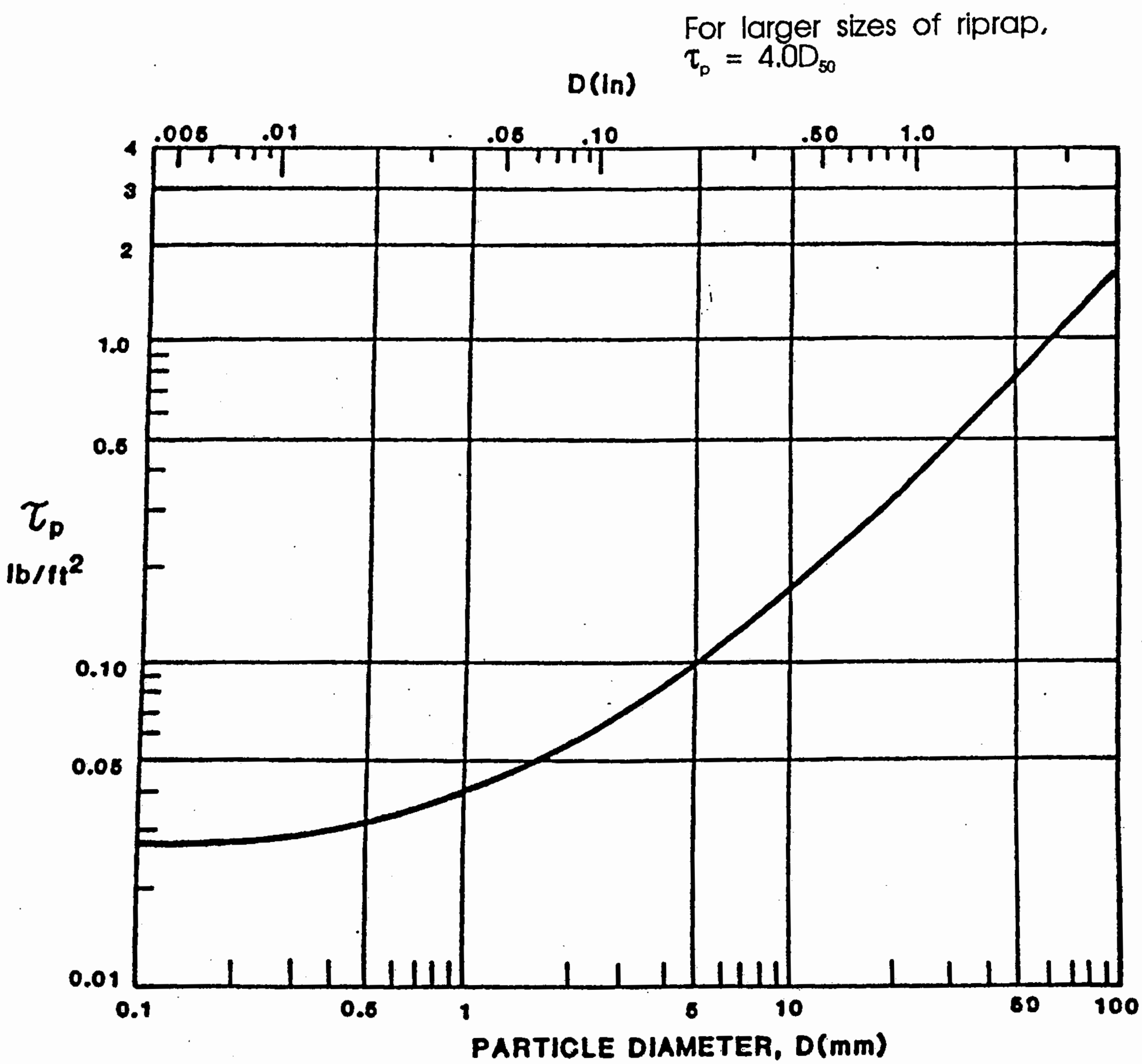
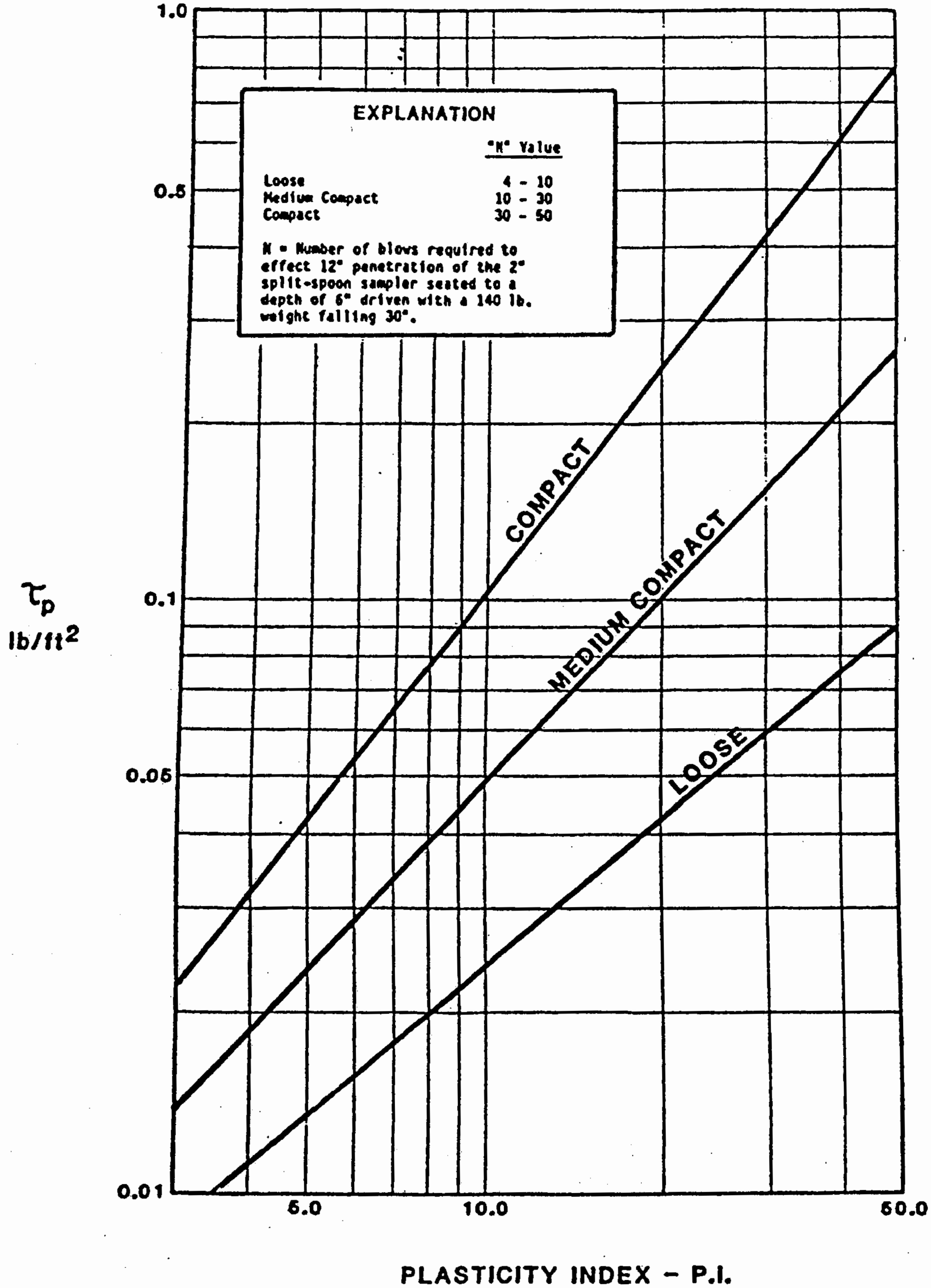


Chart 1. Permissible shear stress for non-cohesive soils.(after 15)

PERMISSIBLE SHEAR STRESS FOR NON-COHESIVE SOILS

FIGURE "J-1"

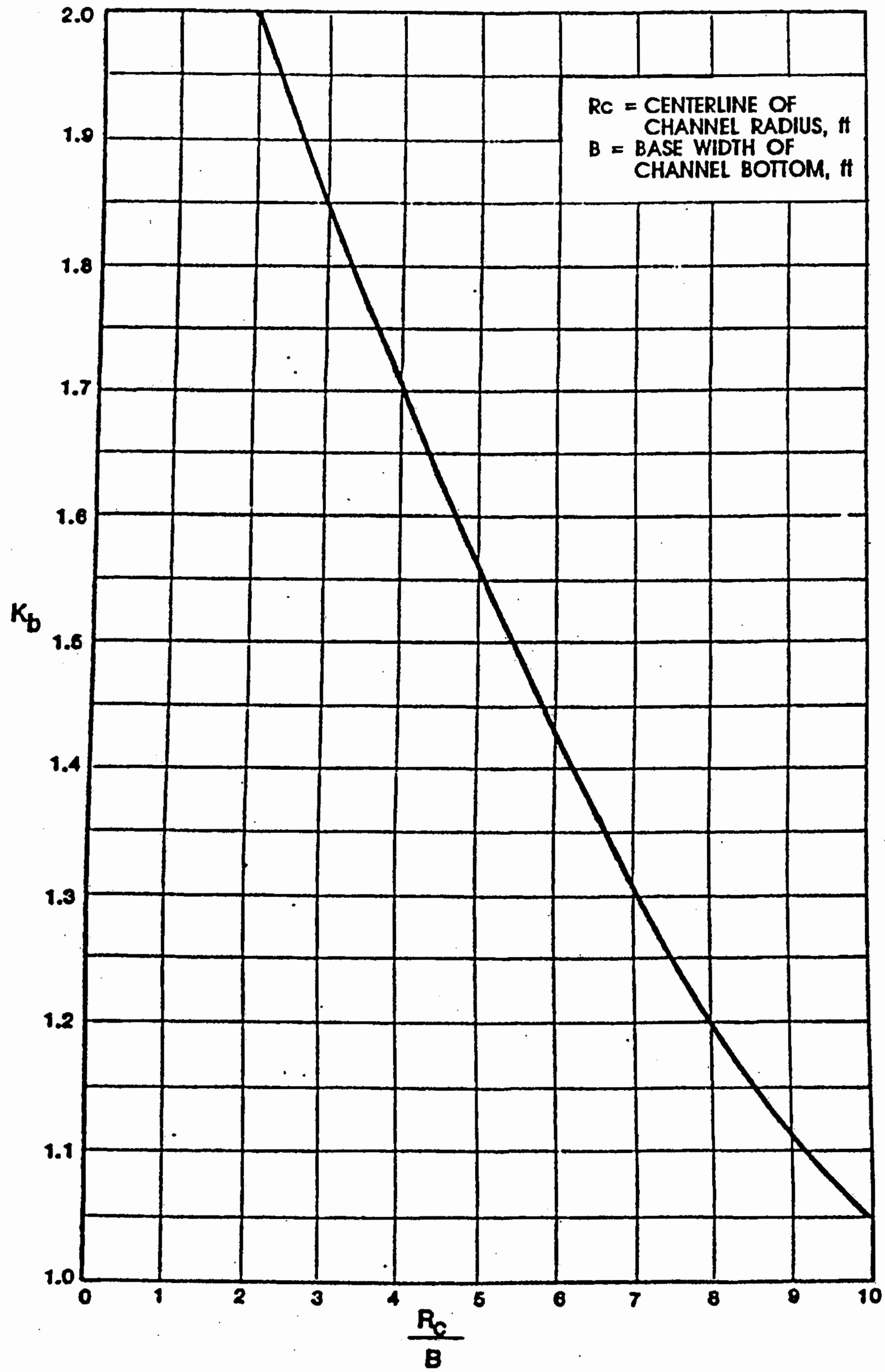
THIS IS A REPRODUCTION OF CHART 2 IN HEC-15.



PERMISSIBLE SHEAR STRESS FOR COHESIVE SOILS

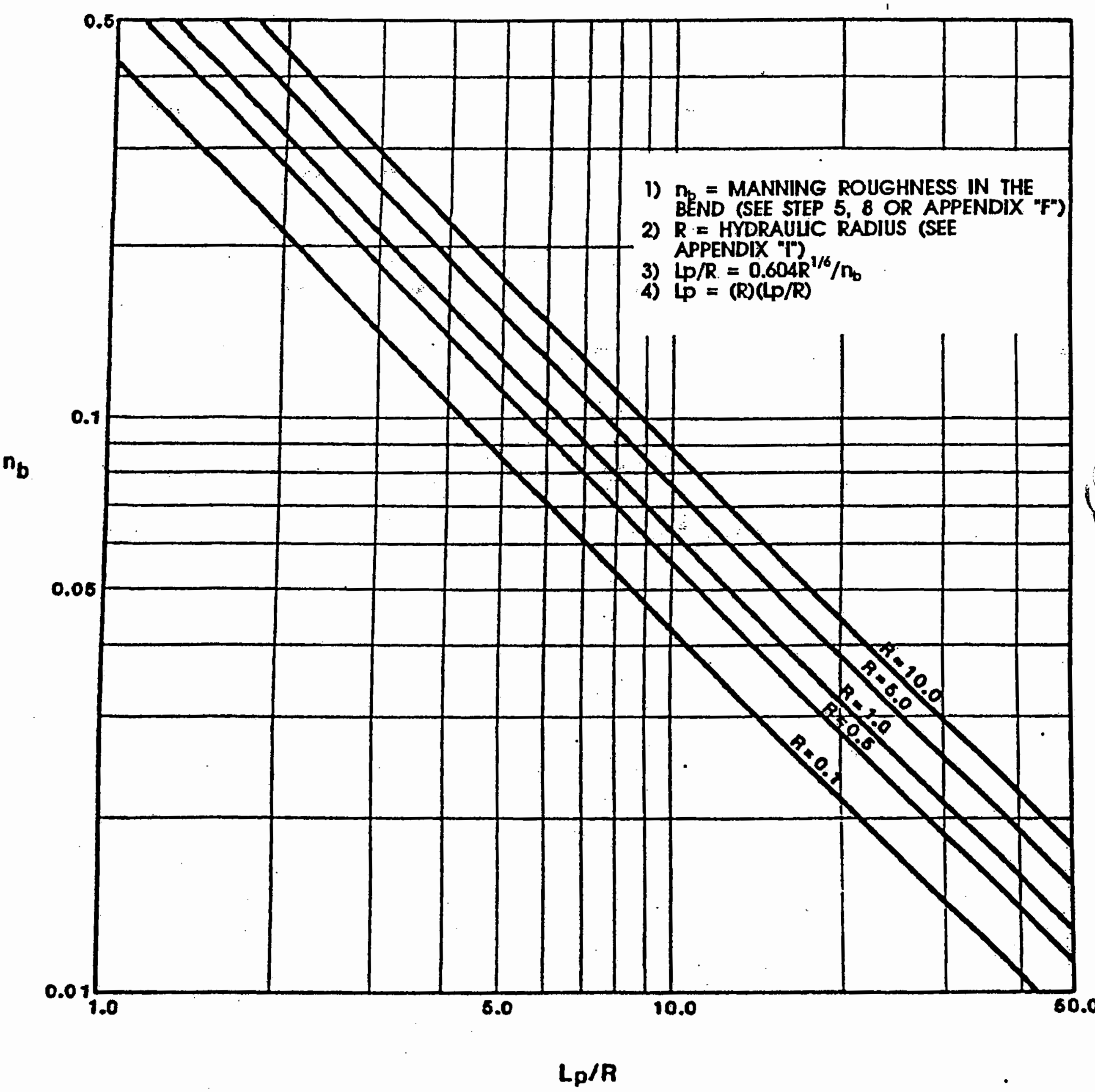
FIGURE "J-2"

THIS IS A REPRODUCTION OF CHART 10 IN HEC-15.



Kb FACTOR FOR MAXIMUM SHEAR STRESS ON CHANNEL BENDS	FIGURE "J-3"

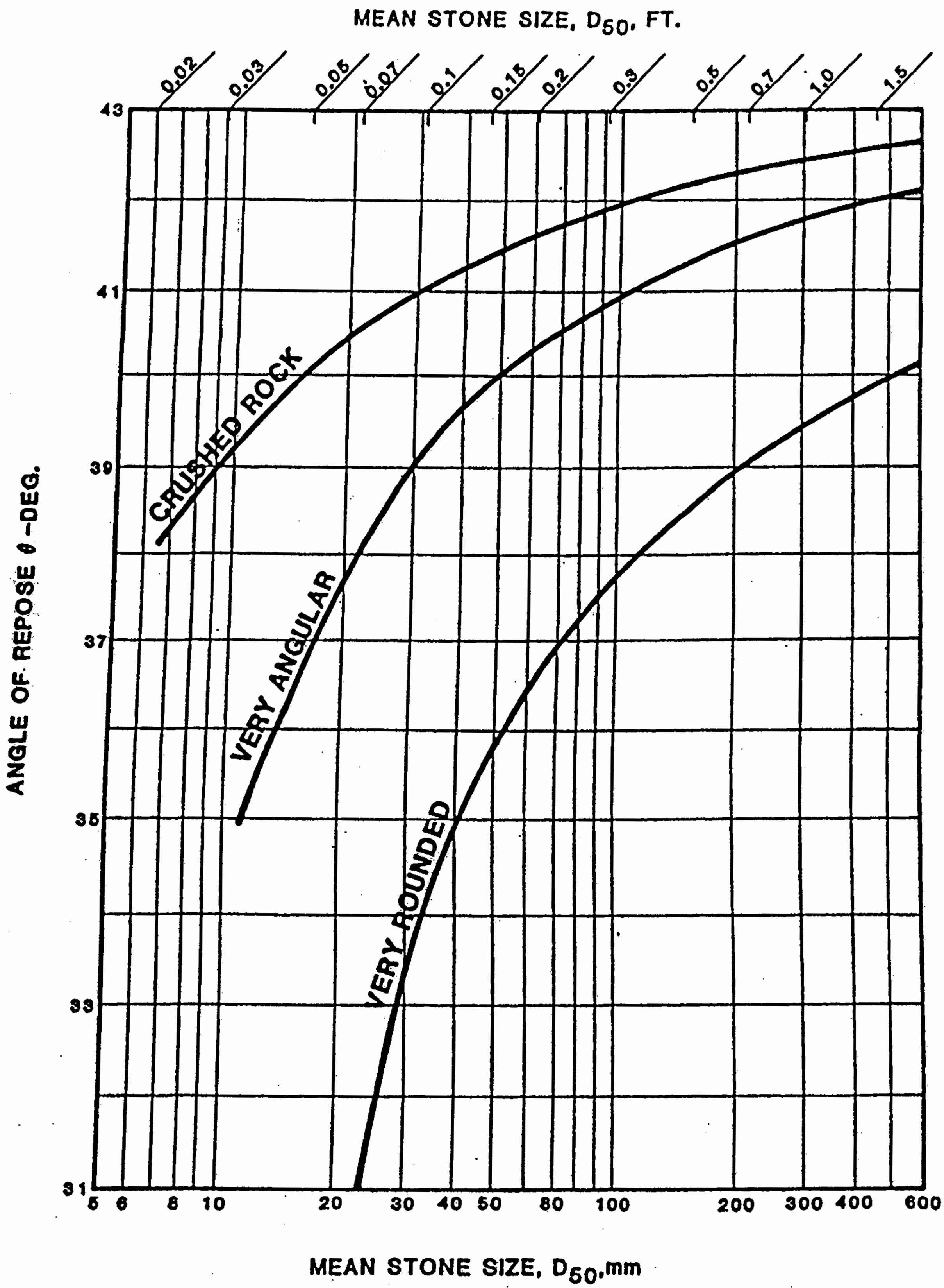
THIS IS A REPRODUCTION OF CHART 11 IN HEC-15.



NOTE: "R" herein is hydraulic radius, denoted as "Rh" in some places.

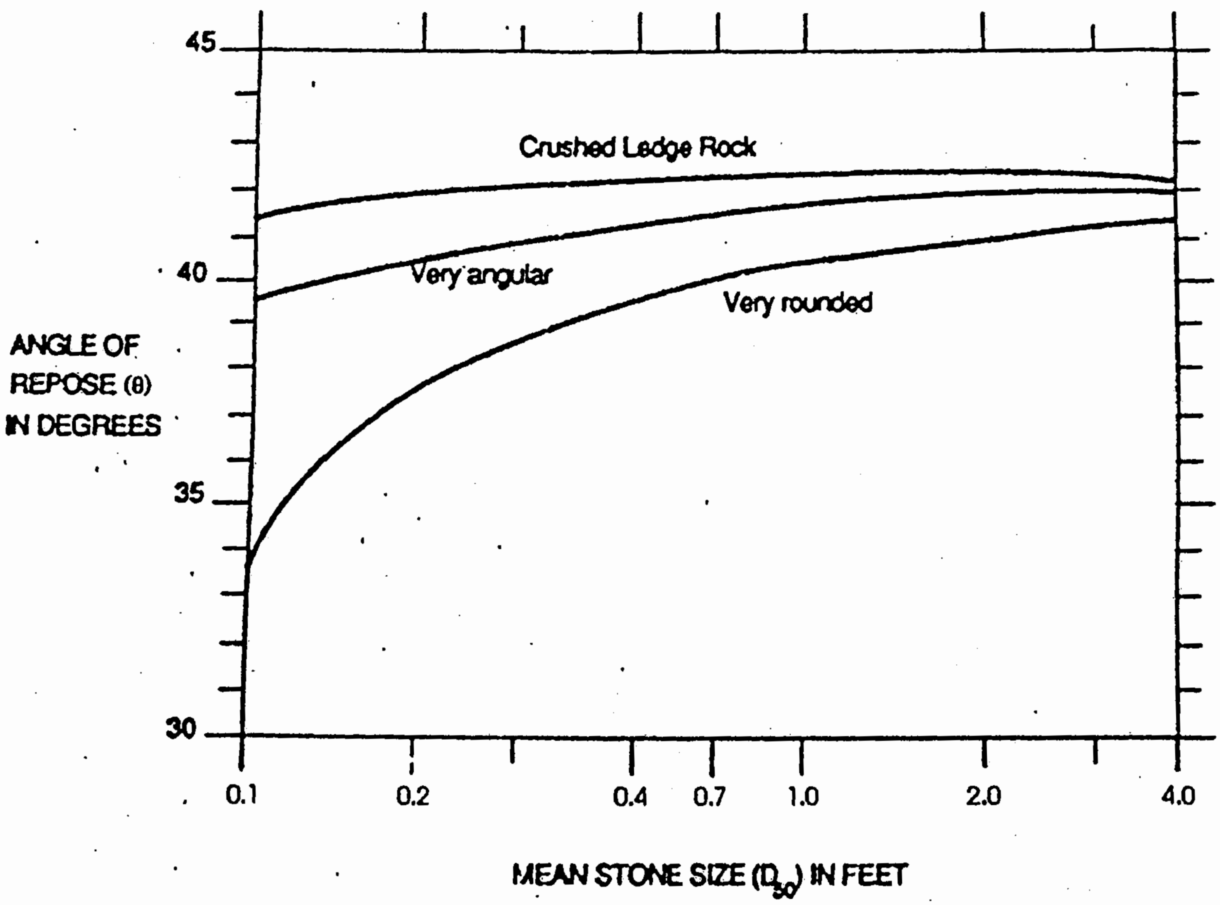
PROTECTION LENGTH (Lp) IN FEET DOWNSTREAM OF CHANNEL BENDS	FIGURE "J-4"

THIS IS A REPRODUCTION OF CHART 12 IN HEC-15.



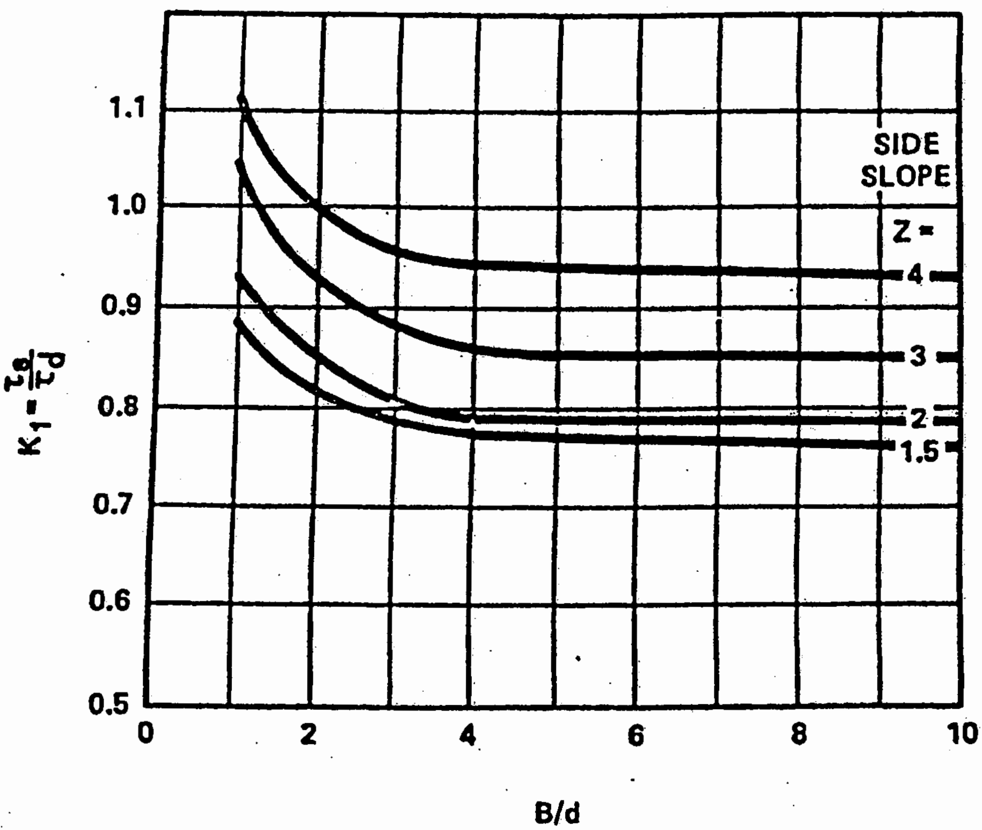
ANGLE OF REPOSE OF RIPRAP	FIGURE "J-5a"

THIS IS A REPRODUCTION OF CHART 4 IN HEC-11.



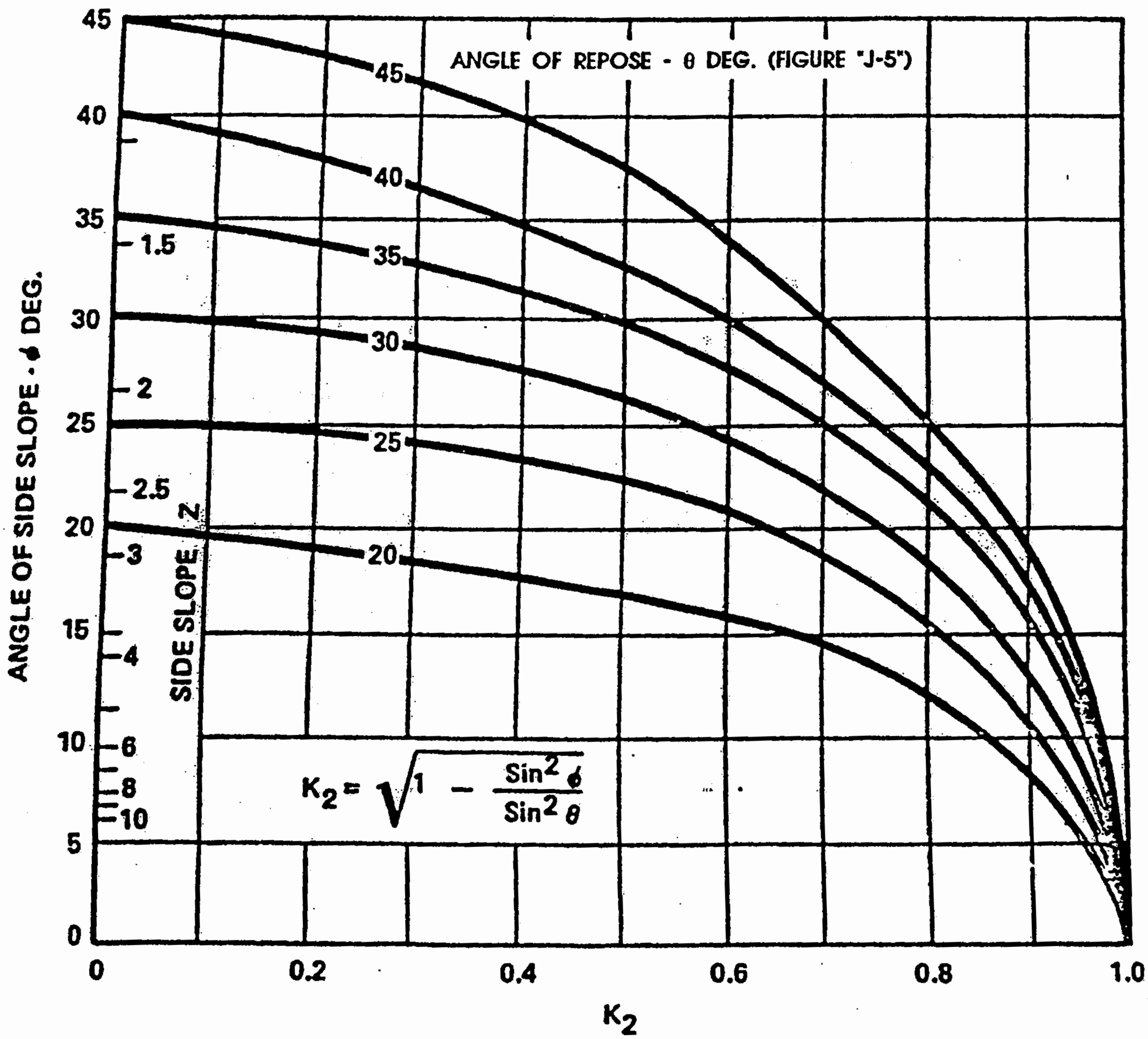
ANGLE OF REPOSE OF RIPRAP	FIGURE "J-5b"

THIS IS A REPRODUCTION OF CHART 13 IN HEC-15.



CHANNEL SIDE SHEAR STRESS TO BOTTOM SHEAR STRESS RATIO, K1	FIGURE "J-6"

THIS IS A REPRODUCTION OF CHART 14 IN HEC-15.



TRACTIVE FORCE RATIO K2	FIGURE "J-7"

B. RIPRAP-LINED CHANNEL DESIGN FOR SLOPES > 10%

1. General Guidelines This subsection outlines the design of riprap flexible channel linings for steep gradients. Because of the additional forces acting on riprap, results obtained using the previous design procedure should be compared to results obtained using these steep gradient procedures when channel gradients approach 10 percent.

   The size of riprap (even wire-enclosed riprap liners) increases significantly as discharge and channel gradient increase. If the size of riprap or cost of riprap becomes excessive per these design requirements, it may be well to look at rigid channel linings as a potentially more cost-effective alternative.

2. Design Procedures The design procedure is presented in the worksheet provided on Table "J-5".

Project: ______________________ Designed by: ______________ Date: __________

Channel Identification: ______________________ (1) Design Flow Q ____=______cfs

(2) TRIAL 1	TRIAL 2	TRIAL 3
REACH ______________	REACH ______________	REACH ______________

NOTE: WHEN USING THIS METHOD, THE CHANNEL CENTERLINE CURVATURE Rc MUST EXCEED 10T, WHERE T = SURFACE WATER WIDTH. THEREFORE, SUPERELEVATION NEED NOT BE ANALYZED.

STEPS: (These correspond with numbers on the chart below.)

1. Enter pre-determined design flow.

2-6. Select trial or reach channel geometric configuration and slope.

7 & 8. Enter Figures "J-8a" through "J-8d", as appropriate, with the discharge Q of (1), read horizontal to the slope in (4), and vertically to determine the preliminary flow depth (di) and mean rock size (D_{50i}).

a. For channel widths not given, interpolate between values on two figures to determine the correct values. For channel bottom widths greater than 6 feet wide, see detailed design procedures in HEC-15, Appendix C.

b. If the side slope (Z) in step 6 is 3, then step 7 and 8 values do not need adjustment, di is the flow depth, D_{50i} is the required mean rock size, and steps 9 through 13 may be skipped.

Steps 9-13 are only applicable if step 6 $Z \neq 3$.

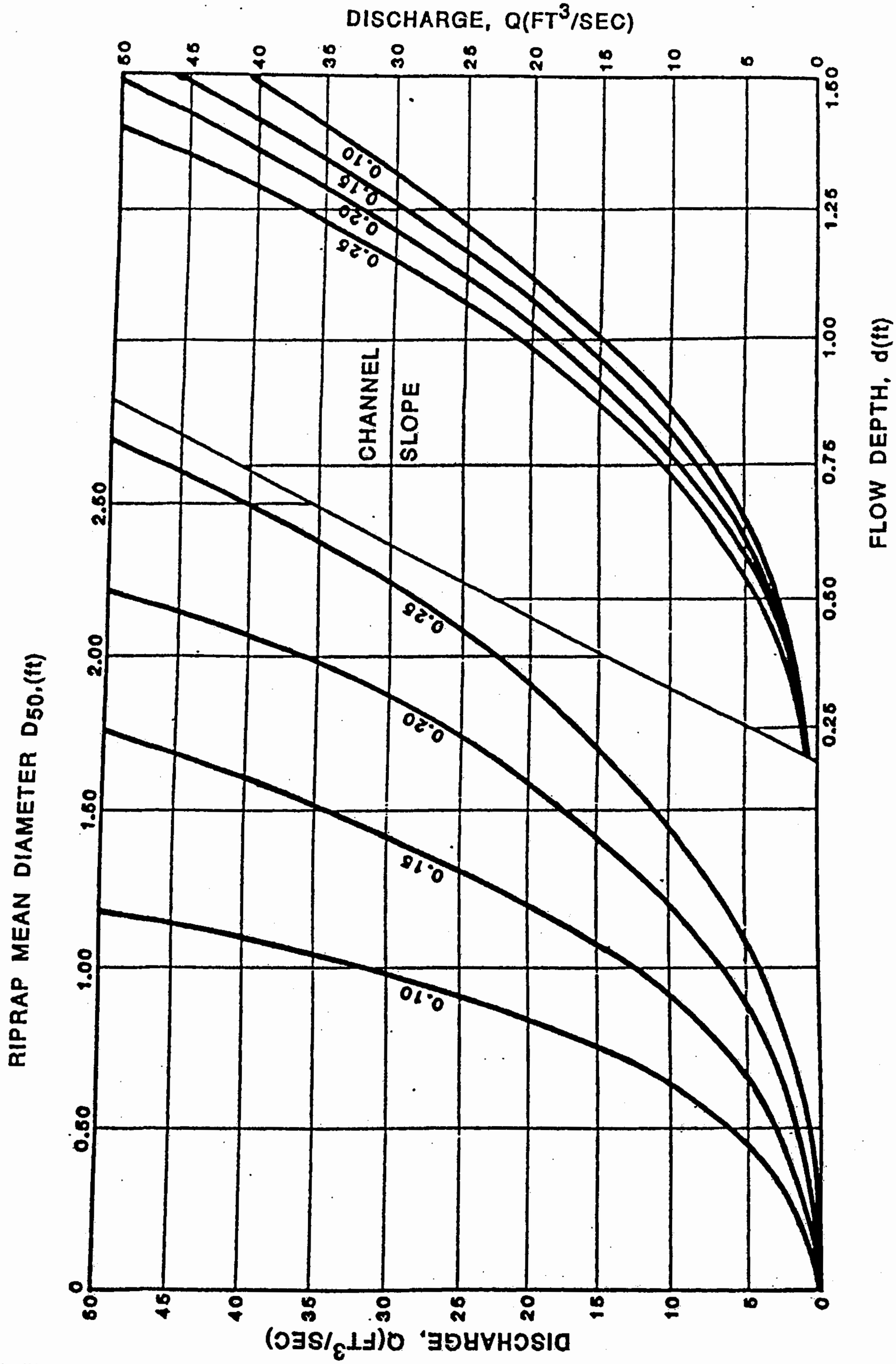
9. Divide step 7 "di" by step 5 "B".
10. From Table "J-6", read or interpolate the side slope K factor.
11. The corrected flow depth (d) = K(di), or (7) times (10).
12. Divide step (7) "di" by step 11 "d".
13. The corrected mean rock size (D_{50}) = D_{50i}(di/d), or (8) times (12).
14. The required channel depth is 2 times the flow depth (i.e., 2di or 2d, as appropriate).
15. Using the final design flow depth from step (7) or (11), as appropriate, calculate surface water spread T (See Fig. "I-9"). The design radius of channel centerline Rc must be greater than 10T.

						USE IF $Z \neq 3$						
TRIAL NO. (3)	SLOPE (ft/ft) (4)	Bottom Width B (ft) (5)	Side Slope Z (ZH:1V) (6)	di (ft) (7)	D_{50i} (ft) (8)	di/B (9)	K (10)	d (ft) (11)	di/d (12)	D_{50} (ft) (13)	Channel Depth (ft) (14)	T (ft) (15)

STEEP SLOPE RIPRAP – LINED CHANNEL DESIGN WORKSHEET (HEC-15)
$Q \leq 50$ cfs, SLOPE > 10%

TABLE "J-5"

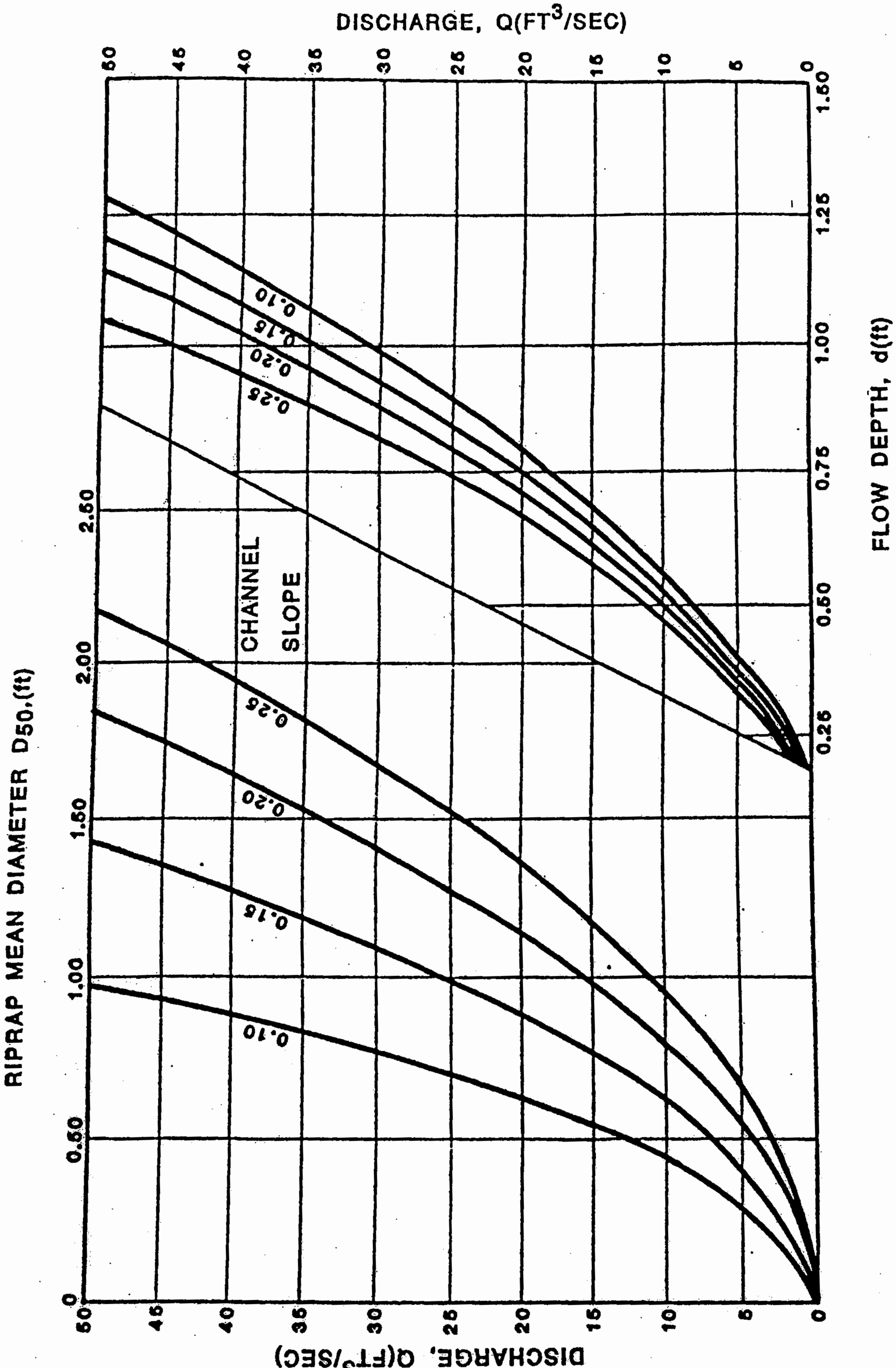
REPRODUCED FROM FHWA HEC-15, CHART 15



STEEP SLOPE RIPRAP DESIGN (HEC-15):
TRIANGULAR CHANNEL (B=0), Z=3

FIGURE J-8a

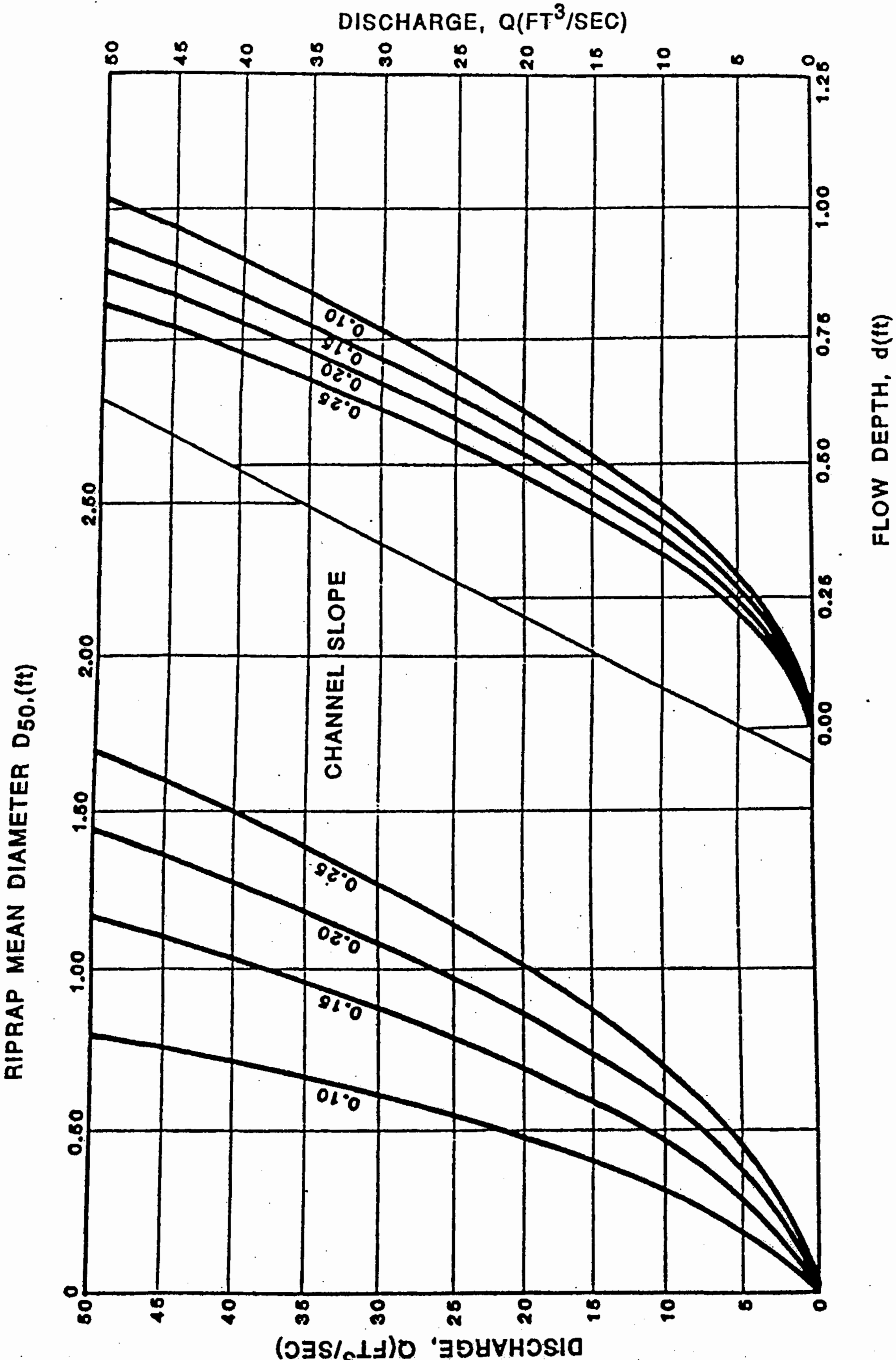
REPRODUCED FROM FHWA HEC-15, CHART 16



STEEP SLOPE RIPRAP DESIGN (HEC-15):
TRAPEZOIDAL CHANNEL (B=2), Z=3

FIGURE J-8b

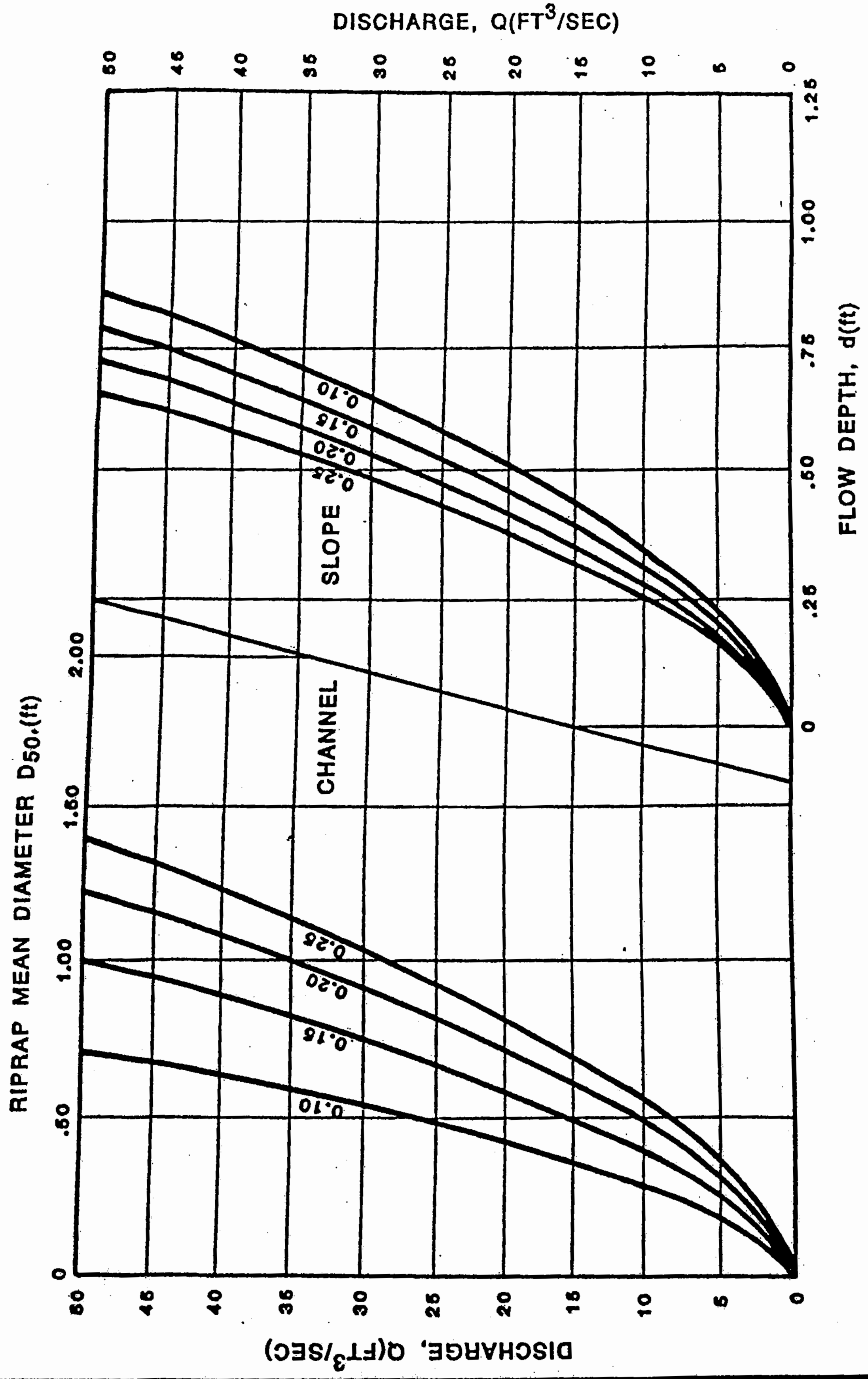
REPRODUCED FROM FHWA HEC-15, CHART 17



STEEP SLOPE RIPRAP DESIGN (HEC-15):
TRAPEZOIDAL CHANNEL (B=4), Z=3

FIGURE J-8c

REPRODUCED FROM FHWA HEC-15, CHART 18



STEEP SLOPE RIPRAP DESIGN (HEC-15):
TRAPEZOIDAL CHANNEL (B=6), Z=3

FIGURE J-8d

TABLE "J-6"
SIDE SLOPE K FACTOR
(This is a reproduction of Table 4 in HEC-15)

di/B	Channel Side Slopes				
	2H:1V	3H:1V	4H:1V	5H:1V	6H:1V
0.10	1.083	1.000	0.928	0.866	0.812
0.20	1.142	1.000	0.888	0.800	0.727
0.30	1.187	1.000	0.853	0.760	0.678
0.40	1.222	1.000	0.846	0.733	0.647
0.50	1.250	1.000	0.833	0.714	0.625
0.60	1.272	1.000	0.823	0.700	0.608
0.70	1.291	1.000	0.815	0.688	0.596
0.80	1.307	1.000	0.809	0.680	0.586
0.90	1.321	1.000	0.804	0.672	0.578
1.00	1.333	1.000	0.800	0.666	0.571
1.10	1.343	1.000	0.796	0.661	0.565
1.20	1.352	1.000	0.793	0.657	0.561
1.30	1.361	1.000	0.790	0.653	0.556
1.40	1.368	1.000	0.787	0.650	0.553
1.50	1.378	1.000	0.785	0.647	0.550
1.60	1.381	1.000	0.783	0.644	0.547
1.70	1.386	1.000	0.782	0.642	0.544
1.80	1.391	1.000	0.780	0.640	0.542
1.90	1.395	1.000	0.779	0.638	0.540
2.00	1.400	1.000	0.777	0.636	0.538

III. CHANGEL DESIGN, FLOW > 50 CFS

A. RIPRAP-LINED CHANNEL DESIGN FOR SLOPES ≤ 10% (FHWA HEC-11)

The entire design procedure is presented in the worksheet provided on Table "J-7".

TABLE J-8
RIPRAP STABILITY FACTOR CSF
(Reproduced from Table 1 in FHWA HEC-11)

Condition	Stability Factor Range
Uniform flow: Straight or mildly curving reach [curve radius (Rc)/channel width (T) > 30]; Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow; Moderate bend curvature (30 > curve radius (Rc)/channel width (T > 10); Impact from waves or floating debris moderate.	1.3 - 1.6
Approaching rapidly varying flow; sharp bend curvature (10 > curve radius (Rc)/channel width (T); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (1 - 2 ft (.30 - .61m)); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 - 2.0*

*The stability factor for a sharp bend curvature alone would be 1.75

Project: ______________________ Designed by: ______________________ Date: ____________

Channel Identification: ______________________ (1) Design Flow Q _______ = _______ cfs Q___ (Main Channel) = ___________ cfs

(2) TRIAL 1	TRIAL 2	TRIAL 3	TRIAL 4	TRIAL 5
REACH _____ S = _____ ft/ft	REACH _____ S = _____ ft/ft	REACH _____ S = _____ ft/ft	REACH _____ S = _____ ft/ft	REACH _____ S = _____ ft/ft

STEPS (1) - (22)

(1) Enter total and main channel Q, if known.

(2) Select channel trial/reach geometry, including sketch of channel section with base width B and side slope "Z" horizontal to 1 vertically and overbank areas, if any.

(3) Estimate a required rock size and whether the liner will be used only on the embankments or the full channel section. Use Table "F-2" to select base "n", then adjust per Table "F-3".

(4) Determine the design water surface and average flow depth "d" in the main channel.

a. If the design section approximates a trapezoid, and flow can be assumed to be uniform, use Fig. "I-3".

b. If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface. Computer methods such as WSPRO and HEC-2 are recommended.

c. Any backwater analysis conducted must be based on conveyance weighting of flows in the main channel, right bank, and left bank.

Check the calculated depth "d" against that assumed in step 3 in determining the "n" value. If calculated "d" is more than 0.25 foot different than that assumed in step 3, return to step 3.

(5) Determine water surface width T (see Fig. "I-9").

(6) Determine water flow area (see Fig. "I-9").

(7) Determine average flow velocity ($V = Q/A$).

(8) $Fr = 0.176V/(A/T)^{0.5}$; If $Fr \leq 0.86$, riprap is not required and steps 9-16, and 19 may be skipped.

(9) Calculate the riprap angle of repose from Fig. "J-5".

(10) Determine the traction force ratio K2 from Figure "J-7".

(11) Preliminarily determine the riprap size required to resist particle erosion using Fig. "J-9".

(12) Select the appropriate stability factor from Table "J-8".

(13) Enter the rock riprap specific gravity (2.65 is typical).

(14) Determine combined adjustment factor from Figure "J-10".

(15) If designing riprap for piers or abutments, apply pier/abutment correction (Cpa) of 3.38. Otherwise, enter 1.0.

(16) Calculate corrected rock riprap size as:

$D_{50} = C(Cpa)D_{50l}$

If the entire channel perimeter is being stabilized, and the D_{50} assumed in selection of Manning's "n" is more than 0.25' different from the value obtained in step 16, return to step 3 and repeat steps 3-16.

(17) Enter design channel centerline radius of curvature Rc. If $Rc > 10T$, skip steps 18-20 and 22.

(18) Calculate minimum Rc as:

For $Fr \leq 0.86$, minimum $Rc = 3T$; and

For $Fr > 0.86$, minimum $Rc = 0.12V^2T/d$

The design Rc of step 17 must be equal to or larger than this value.

(19) Determine longitudinal extent of riprap protection (Lp) req'd. The liner should be used at least 1T and 1.5T upstream and downstream, respectively, from bend.

(20) Superelevation $\Delta d = CV^2T/(32.2Rc)$, where C values are presented in Table "I-1".

(21) & (22) For $Fr \leq 0.86$, freeboard $= 0.15(d+v^2/2g) + \Delta d$ at bends.

For $Fr > 0.86$, freeboard $= 0.25(d+v^2/2g) + \Delta d$ at bends.

Enter freeboard without Δd (step 21) and with Δd (step 22) for straight and bend reaches, respectively.

CHANNEL HYDRAULICS							CHANNEL ROCK SIZE								CHANNEL BENDS				FREEBOARD	
Trial/ Reach No. (2)	"n" Value (3)	Avg. depth d (ft) (4)	Width T (ft) (5)	Area A (ft^2) (6)	Avg. V (fps) (7)	Fr (8)	θ (deg) (9)	K2 (10)	Prelim. D_{50l} (ft) (11)	Csf (12)	SG (13)	C (14)	Cpa (15)	Adj. D_{50} (ft) (16)	Rc dsgn. (ft) (17)	Rc min. (ft) (18)	Lp (ft) (19)	Δd (ft) (20)	Straight (ft) (21)	Bend (ft) (22)

RIPRAP — LINED CHANNEL DESIGN WORKSHEET (HEC-11)

Q > 50 cfs, 0% ≤ SLOPE ≤ 10%

TABLE "J-7"

REPRODUCED FROM CHART 1 IN HEC-11

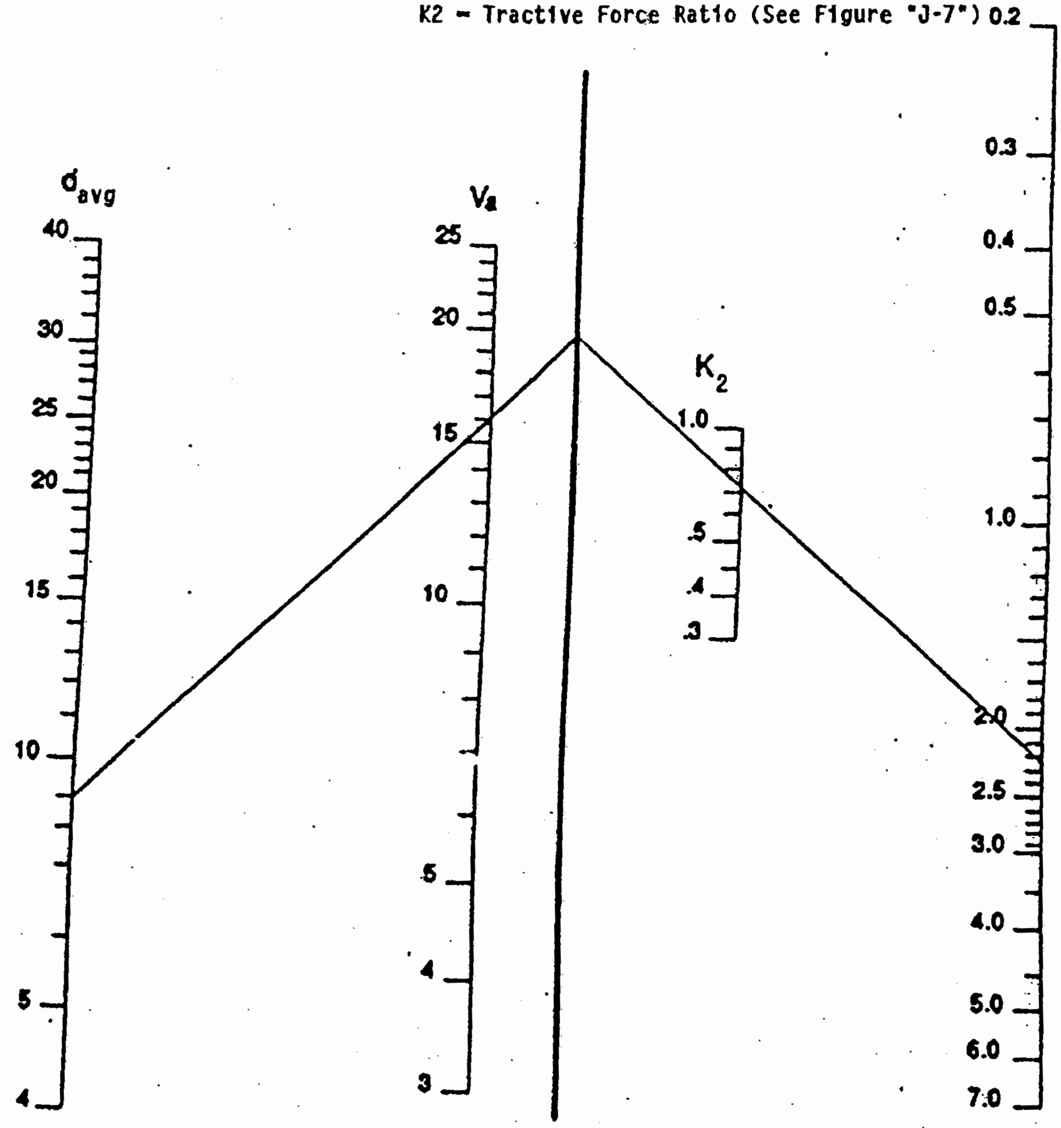
$$D_{50} = 0.001\, V_a^3 / (d_{avg}^{1/2} K_2^{3/2})$$

D_{50} = Median Riprap Size (ft.)

V_a = Average velocity in main channel (ft/sec)

d_{avg} = Average depth in main channel (ft)

K2 = Tractive Force Ratio (See Figure "J-7")



Example

Given:

V_a = 16 ft/sec

d_{avg} = 9 ft

K_2 = 0.72

Find:

D_{50}

Solution:

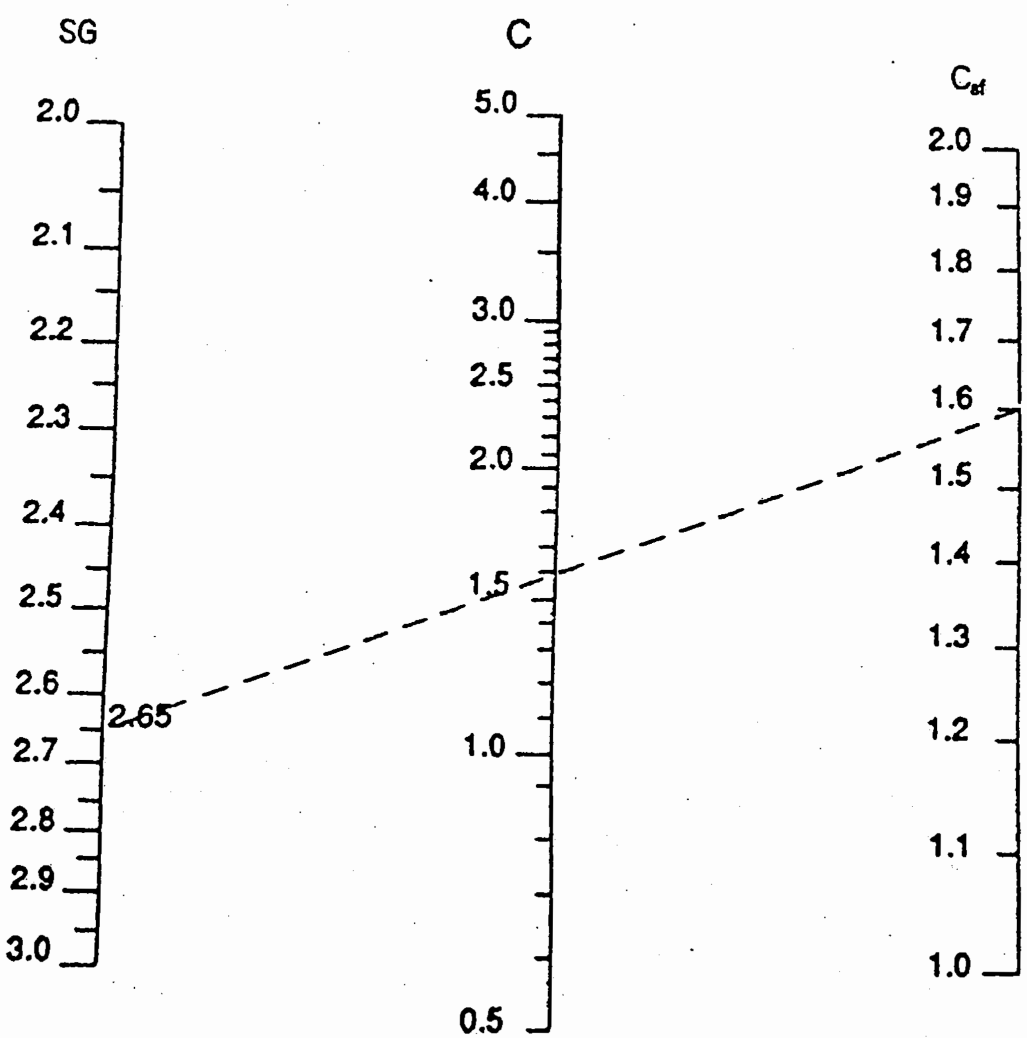
D_{50} = 2.25

PRELIMINARY D_{50} RIPRAP SIZE (HEC-11) | FIGURE "J-9"

REPRODUCED FROM CHART 2 IN HEC-11

$$C = 1.61 C_{sf}^{1.5} / (SG-1)^{1.5}$$

C = COMBINED ADJUSTMENT FACTOR
C_{sf} = STABILITY FACTOR (TABLE "J-8")
SG = SPECIFIC GRAVITY OF ROCK



Exampie:

Given:	Find:	Solution:
SG=2.75	C	C=1.59
C_{sf} = 1.60		

COMBINED ADJUSTMENT FACTOR

FIGURE "J-10"

B. RIPRAP-LINED CHANNEL DESIGN FOR SLOPES BETWEEN 2% AND 20% (CSU & ACOE)

Although the HEC-11 manual does not preclude its use for slopes above 10%, the method results in very large riprap sizes that appear to be unrealistic. More recent methods have been developed based upon analyses of riprap sizing for steep slopes performed by Abt et al. at Colorado State University. Near prototype flume analyses were conducted with riprap-protected liners for slopes up to 20%. Results and conclusions were published, providing an equation and recommended rock sizing procedures (Abt 1991). Based upon this study and published results, the Army Corps of Engineers proposed revisions to their published procedures (ACOE 1994). Notwithstanding the influence of the CSU study, the ACOE equation is quite dissimilar in form. Both methods are presented herein.

1. CSU/Abt Procedure The equation recommended by the CSU study is
D_{50} (in) = $5.23\ S^{0.43}\ q^{0.56}$ (for rock with a specific gravity of 2.65).

However, several adjustment factors were also recommended:

1) A stability factor of 1.2 to be applied to the riprap size per the observed deviation envelope;

2) A rock shape factor to be applied to the riprap size;

3) A stone failure/stone movement factor of 1.35 to be applied to the unit discharge; and

4) A flow concentration factor of 1 to 3 to be applied to the unit discharge.

The CSU/Abt study recommends use of a rock shape factor of 1.0 for angular rock and 1.4 for rounded rock. However, the study was based upon experimental observations on a scaled-down near-prototype flume using 2-inch and 4-inch rock. Observing Figure "J-5b", one may see that the angle of repose is significantly different for angular and rounded rock in the 2-inch to 4-inch range. On the other hand, rock sizes that will be required for practical channel flows on high slopes will be considerably larger where the difference in angle of repose values is significantly reduced. Using HEC-11 procedures, riprap sizes do not vary between angular and rounded riprap for channel bottoms. This is because while the permissible unit shear stress is less with rounded rock, FHWA experience is that the reduction in drag force on the smooth rock is compensating, even for bed slopes up to 20%. Using the FHWA's HYCHL computer program, the author performed analyses for the most severe stability condition (2H:1V side slopes). It appears that from 2% to 20% channel bed slopes, the increase in rock size between angular and rounded riprap for the 2H:1V side slopes is only 4% per HEC-11 procedures. Consequently, it does not appear reasonable to apply a shape factor of 1.4 for rounded riprap. Instead, the author recommends use of the same size riprap in channel beds regardless of rock shape. On side slopes, which generally are much steeper than channel beds, the difference can be adjusted by the same procedures presented in HEC-15; that is, the rock shape determines angle of repose, which in turn impacts the factor to be applied to D_{50} (channel) to calculate the D_{50} (side slopes).

A flow concentration factor of 1 to 3 is recommended. This is to account in part for variations in velocity between the peak and average, but more particularly for subchannelization that occurs. However, subchannelization will be minimal without stone movement, which the stone failure/stone movement factor should help prevent. Furthermore, CSU's tests were on small riprap, unlike the larger riprap that would be used in real projects. Based upon their previous experience, the ACOE decided to adopt a flow channelization factor of 1.25 for use in their equation. It is noted that, if a flow concentration factor of 2.0 were used, which is the mean value, the resultant riprap size requirement is similar to that obtained using the ACOE equation. It would seem justifiable, then, to use a channelization factor between 1.25 and 2.0.

The CSU/Abt equation below is modified to give the riprap size in feet, apply the stability factor of 1.2, and insert other recommended factors.

Channel Bottom $\quad D_{50}(ft) = 0.523\ S^{0.43}\ (K_{fm}\ K_{fc}\ q)^{0.56}$

Channel Embankment $\quad D_{50}(embankment) = (K1/K2)\ D_{50}\ (channel\ bottom)$

where:

S	=	channel bed slope, ft/ft;
K_{fc}	=	channel flow concentration factor, ranging between 1.25 and 2.0;
K_{fm}	=	stone failure/stone movement factor equal to 1.35;
q	=	unit discharge, or Q/B;
K1	=	shear stress ratio (see Figure "J-6"); and
K2	=	tractive force ratio (see Figure "J-7"), which is dependent upon the angle of repose, which in turn is dependent upon the rock shape (see Figure "J-5b").

The CSU/Abt method is presented along with other channel design procedures in the design worksheet on Table "J-9".

2. <u>ACOE Procedure</u> The procedure recommended by the Army Corps of Engineers is to use the following equation (ACOE 1994):

$$D_{30} = \frac{1.95\ S^{0.555}\ (1.25\ q)^{0.67}}{g^{0.333}}$$

where D_{30} = Rock size where 30% of the rock is finer by weight;
s = Channel bed slope, ft;
q = Unit discharge, or Q/B; and
g = Acceleration of gravity (assume 32.2 fps)

Conditions of equation use are:

1) Rock is angular (per the previous discussion, this may not be critical);

2) Minimum riprap thickness = 1.5 D_{100} (provided per specifications);

3) Unit weight of rock is at least 167 pcf (SG $\geq$ 2.59) [the SG $\leq$ 2.5 required by specifications may be adequate];

4) Use a uniform riprap gradation having $1.7 \leq D_{85}/D_{15} \leq 2$ (specified gradation is adequate); and

5) Restrict application to straight channels with a side slope of 2.5H:1V or flatter.

Only condition (5) deserves additional comment. Riprap embankments are allowed to 2H:1V; however, procedures presented in the design worksheet on Table "J-9" allow for riprap enlargement on steeper side slopes, and should be adequate. Also limitations on curvature prevent use of small radii for flatter grades, and preclude use of bends at slopes over 10% in grade. It is felt that the ACOE method is applicable to procedures outlined herein, and as presented in Table "J-9".

Project: ______________________ Channel I.D. ____________ (1) Design Q__ = ______ cfs Design by: ______________________ Date: ____________

(2) TRIAL/REACH ____	TRIAL/REACH ____	TRIAL/REACH ____	TRIAL/REACH ____	TRIAL/REACH ____
SLOPE = _____ft/ft	SLOPE = _____ft/ft	SLOPE = _____ft/ft	SLOPE = _____ft/ft	SLOPE = _____ft/ft

STEPS (1) - (22)

(1), (2) Enter Q, trial/reach no., channel slope, and sketch channel section with base width B and slope "Z" horizontal to 1 vertical.

(3) Estimate a required rock size and whether the liner will be used only on the embankments or the full channel section. Use Table "F-2" to select base "n", then adjust per Table "F-3".

(4) Determine channel flow depth using Fig. "I-3". Check the calculated depth "d" against that assumed in step 3 in determining the "n" value. If calculated "d" is more than 0.25 foot different than that assumed in step 3, return to step 3.

(5) Determine water surface width T (see Fig. "I-9").

(6) Determine water flow area (see Fig. "I-9")

(7) Determine average flow velocity (V=Q/A).

(8) Fr = 0.176V/(A/T)0.5 If Fr ≤ 0.86, riprap is not required and steps 9-16 and 19 may be skipped.

(9) Calculate the unit discharge. This is typically taken as q = Q/B. For a non-rectangular channel, use q = Qd/A.

(10) K_{fm} = 1.35 for CSU/ABT method, 1.00 for ACOE method

(11) K_{fc} = 1.25-2.0 for CSU/ABT method (designer choice, higher value is more conservative - see text for explanation), or 1.25 for the ACOE method.

(12) Calculate the channel bed D_{50} as follows:

For CSU/ABT method,

$$D_{50}(ft) = 0.523S^{0.43}(K_{fm}K_{fc}q)^{0.56}$$

For ACOE method,

$$D_{30}(ft) = 0.614S^{0.555}(K_{fc}q)^{0.67}$$

Enter D_{50} or D_{30} in the tabulation heading and the rock size in the column.

(13) Determine the rock angle of repose from Fig. "J-5".

(14), (15) Determine K1 and K2 from Fig. "J-6" and "J-7", respectively.

(16) Calculate the rock size required on the side slope.

D_{xx} (side slope) = K1/K2 times D_{xx} (bottom)

(17) Enter design channel centerline radius of curvature Rc. If Rc > 10T, skip steps 18-20 and 22.

(18) Calculate minimum Rc as:

For Fr ≤ 0.86, minimum Rc = 3T; and

For Fr > 0.86, minimum Rc = $0.12V^2T/d$

The design Rc of step 17 must be equal to or larger than this value.

(19) Calculate R (or Rh). Use Fig "J-4" to obtain Lp.

(20) Superelevation $\Delta d = CV^2T/(32.2Rc)$, where C values are presented in Table "I-1".

(21) & (22) For Fr ≤ 0.86, freeboard = $0.15(d+v^2/2g)$ + Δd at bends.

For Fr > 0.86, freeboard = $0.25(d+v^2/2g)$ + Δd at bends

Enter freeboard without Δd (step 21) and with Δd (step 22) for straight and bend reaches respectively.

CHANNEL HYDRAULICS							CHANNEL ROCK SIZE				SIDE SLOPE ROCK SIZE				CHANNEL BENDS				FREEBOARD	
Trial/ Reach No. (2)	"n" Value (3)	Avg. depth d (ft) (4)	Width T (ft) (5)	Area A (ft^2) (6)	Avg. V (fps) (7)	Fr (8)	q (ft^2/s) (9)	K_{fm} (10)	K_{fc} (11)	D__ (ft) (12)	θ (13)	K1 (14)	K2 (15)	D__ (ft) (16)	Rc dsgn. (ft) (17)	Rc min. (ft) (18)	Lp (ft) (19)	Δd (ft) (20)	Straight (ft) (21)	Bend (ft) (22)

RIPRAP - LINED CHANNEL DESIGN WORKSHEET (CSU/ABT)

Q > 50 cfs, 2% ≤ SLOPE ≤ 20%

TABLE "J-9"

VI. OUTLET PROTECTION

A. GENERAL GUIDELINES

Where concentrated flows are discharged, velocities are usually high, and there is a significant flow transition, both of which are conducive to high erosion potential. A general understanding of problems and mitigating facilities may be helpful as a background prior to presenting facility design procedures.

1. Outlet Velocities Conduit outlet velocity is one of the primary indicators of erosion potential, and has been used as a guide to what erosion control facilities may be required (Pima County, Maricopa County). These are categorized depending upon the receiving channel.

   a. Natural Channel Outlets Natural channel outlet protection is often based on the ratio of the culvert outlet velocity to the average natural stream velocity as follows:.

      1) Culverts with outlet velocities less than or equal to 1.3 times the average natural stream velocity for the design discharge usually require a cutoff wall or flared end section as a minimum for protection;

      2) Where the outlet velocity is greater than 1.3 times the natural stream velocity, but less than 2.5 times, a riprap apron should be provided; and

      3) When outlet velocities exceed 2.5 times the natural stream velocity, an energy dissipator should be provided.

   b. Artificial Channel and Side Channel Outlets Artificial channel and side channel outlet protection is often based on the ratio of the culvert outlet velocity to the allowable velocity for the channel lining material. Outlet discharge must be transitioned to limit the velocity to that allowed by criteria and the channel design and liner, if any. Typical guidelines are:

      1) Conduits with outlet velocity less than or equal to the allowable require no outlet protection;

      2) Conduits with outlet velocity greater than one and less than 2.5 times the allowable velocity must be provided with a riprap, concrete, or other suitable apron to transition the velocity to the allowable channel velocity; and

      3) When outlet velocities exceed 2.5 times the natural stream velocity, an energy dissipator should be provided.

2. Velocity of Flow Reduction by Conduit Size Change This subsection summarizes a discussion presented in Section III of FHWA's HEC-14 on the same subject.

Culvert outlet velocities are often quite high, and therefore it is reasonable to investigate measures to modify or reduce velocity within the culvert before considering an energy dissipator. Several possibilities exist, but the degree of velocity reduction is, in most cases, limited and must always be weighed against the increased costs which are generally involved.

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

a. Culverts on Mild Slopes Where high tailwater controls the culvert outlet velocity, outlet velocity is determined using the full barrel area. With this flow condition, it is possible to reduce the velocity by increasing the culvert size. The degree of reduction is proportional to the reciprocal of the culvert area. However, for high tailwater conditions above the top of culvert outlet, erosion may not be a serious problem. It may be more important to determine if tailwater will always control or only sometimes.

When the tailwater depth is low, culverts on mild or horizontal slopes will flow with critical depth near the outlet. When culverts discharge with critical depth near the outlet, changing the barrel slope will have no effect on the outlet velocity as long as the slope is less than critical slope. Changing the resistance factor will change the depth at the outlet an insignificant degree and will, therefore, not modify the outlet velocity.

b. Culverts on Steep Slopes Increasing the barrel size for a given discharge and slope has little effect on velocity if the flow reaches normal depth, as it will within most culverts on steep slopes.

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

Outlet velocities can also be modified by substituting a rough barrel for a smooth barrel, or by using flow tumblers. These restrictive rings go inside the culvert and change the nature of the culvert flow. Tumbling flow is discussed in detail in HEC-14, Sections VII-B and VII-C. When using this method of velocity reduction, it should be remembered that changing the flow from supercritical to subcritical may result in a marked change in the headwater.

Substituting a "broken-slope" flow line for a steep, continuous slope is not recommended for controlling outlet velocity. Such a design is based on the assumption that the reduced slope of the lower barrel will control depth and velocity, as indicated by the Manning formula. Where the total fall from inlet to outlet remains the same, a broken-slope flow line reduces the outlet velocity only slightly. The initial steeper slope will bring about a lesser depth and greater velocity at the break in grade, followed by a small increase in depth in the lesser slope section. In supercritical flow, the total loss of energy by resistance will be somewhat greater with the steeper and then flatter slope because a lesser depth is produced over a greater portion of the barrel length. This increased loss due to resistance will be small, however, as will the reduction in outlet velocity. Formation of a hydraulic jump in the lower barrel is rare, as the downstream depth required to force a jump will seldom be encountered. If this type of design is attempted, water surface profile calculations must be made to insure that the hydraulic jump relationship is fulfilled.

For culverts on slopes greater than critical, rougher material will cause greater depth of flow and less velocity in equal size pipes. Velocity varies inversely with resistance; therefore, using a corrugated metal pipe instead of a concrete pipe will reduce velocity approximately 40 percent, and substitution of a structural corrugated metal plate pipe for concrete will result in about 50 percent reduction in velocity. Barrel resistance is obviously an important factor in reducing velocity at the outlets of culverts on steep slopes. Sections VII-B and VII-C of HEC-14 contain detailed discussions and specific design information for increasing barrel resistance.

3. <u>Downstream Scour Holes</u> Without outlet protection, a scour hole will typically form downstream of the outlet. Two reasons for understanding the geometry of the predicted hole are:

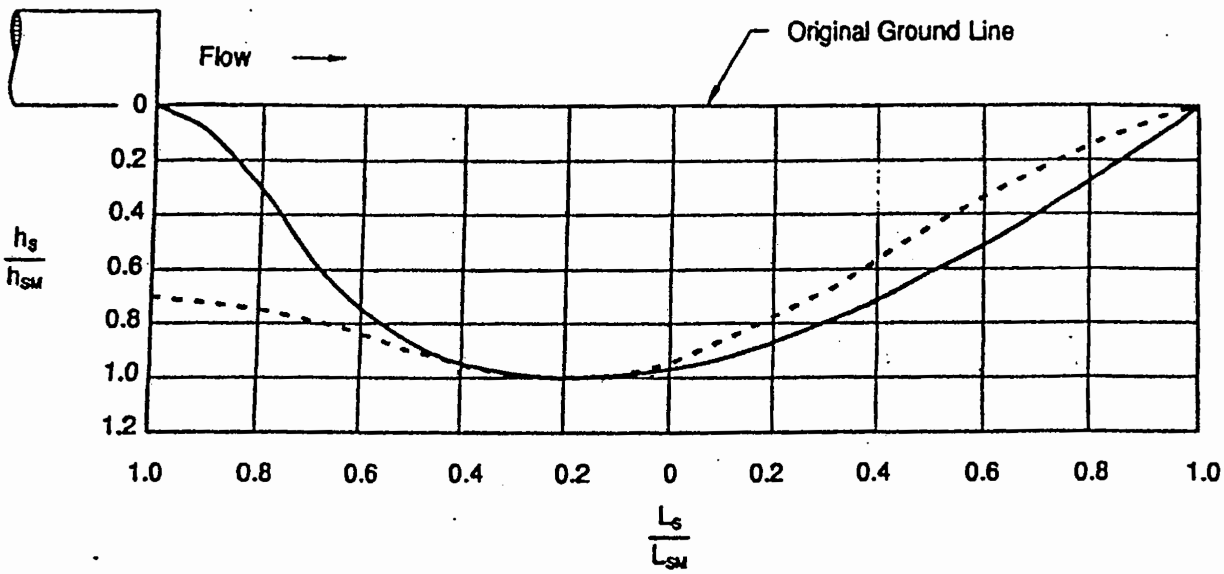
    1) It may be that the scour hole is acceptable, and therefore no protection is needed; and

    2) Knowing the region of predicted scour, one may predict the area of proposed mitigation facilities.

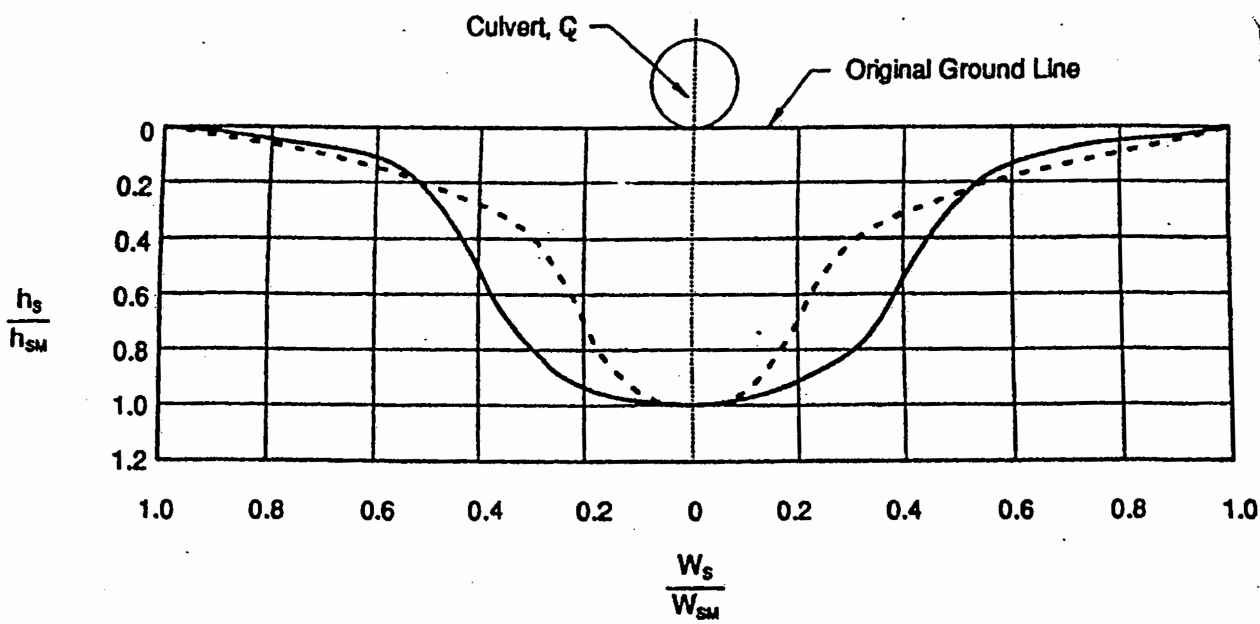
A dimensionless scour hole is shown in Figure "J-11". Procedures have been prepared to estimate the dimensions of the scour hole. A simple method was developed by the ACOE which uses culvert diameter, culvert hydraulic radius, and the hydrological time of concentration, and three coefficients which depend upon tailwater conditions (Pima County). The drawback of this straightforward procedure is that it does not account for soil types. FHWA's HEC-14 presents a more detailed but not difficult procedure that takes into consideration soil characteristics. The procedure will not be presented herein, however, because erosion protection is usually desired and/or necessary anyway. One may as well simply design a protection facility directly per acceptable procedures, such as those presented in Subsection B which follows.

THIS IS A REPRODUCTION OF FIGURE 4.78 IN (MARICOPA COUNTY)

Dimensionless Centerline Profile



Dimensionless Cross Section at 0.4 L_{SM}



Legend:

h_s = Depth of scour
h_{SM} = Maximum depth of scour
W_S = Width of scour
W_{SM} = Maximum width of scour

L_S = Length of scour
L_{SM} = Maximum length of scour
——— Maximum Tailwater
- - - - - Minimum Tailwater

DIMENSIONLESS SCOUR HOLE GEOMETRY	FIGURE "J-11"

B. OUTLET PROTECTION DESIGN PROCEDURES

Riprap aprons and energy dissipation basins placed downstream of culverts provide protect against scour immediately around the culvert as well as providing for the uniform spreading of flow and decreasing flow velocity, thus mitigating downstream damages. Several design methods and the range of their allowed use are shown in Table "J-10".

TABLE "J-10"
OUTLET PROTECTION DESIGN METHODS

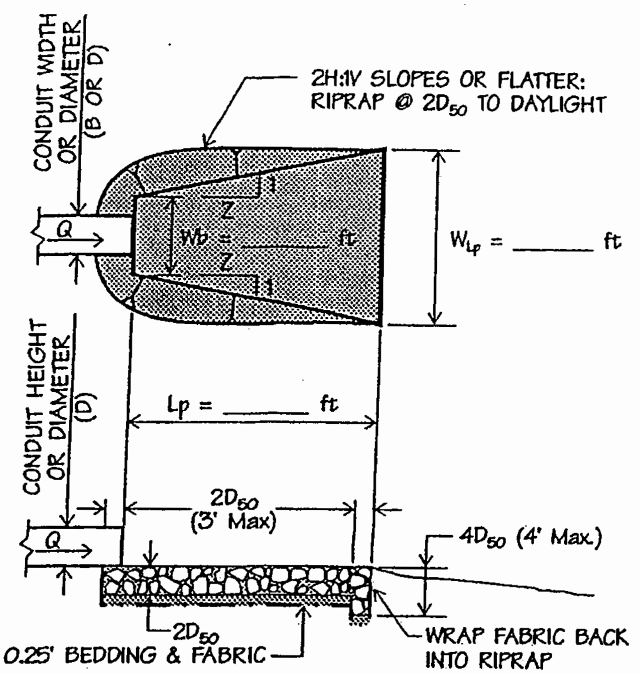
Conduit Outflow Froude No.	Flow Velocity		
	$\frac{Vo}{Vp} \leq 1.3$	$1.3 < \frac{Vo}{Vp} \leq 2.5$	$2.5 < \frac{Vo}{Vp}$
FR < 0.86	– Use a standard flared end section	– Pima Co. Riprap Apron – Pima Co. Riprap Plunge Basin – UD & FCD Riprap Apron – HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures	– HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures
0.86 ≤ Fr ≤ 1.7	– Pima Co. Riprap Apron – Pima Co. Riprap Plunge Basin – UD & FCD Riprap Apron – HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures	– Pima Co. Riprap Apron – Pima Co. Riprap Plunge Basin – UD & FCD Riprap Apron – HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures	– HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures
1.7 ≤ Fr ≤ 2.5	– HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures	– HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures	– HEC-14 Riprap Plunge Basin – Other standard energy dissipation structures
2.5 < Fr	– Other standard energy dissipation structures	– Other standard energy dissipation structures	– Other standard energy dissipation structures

Fr = Froude number, Vo = culvert brink flow velocity (fps), Vp = permissible velocity for the channel (fps).

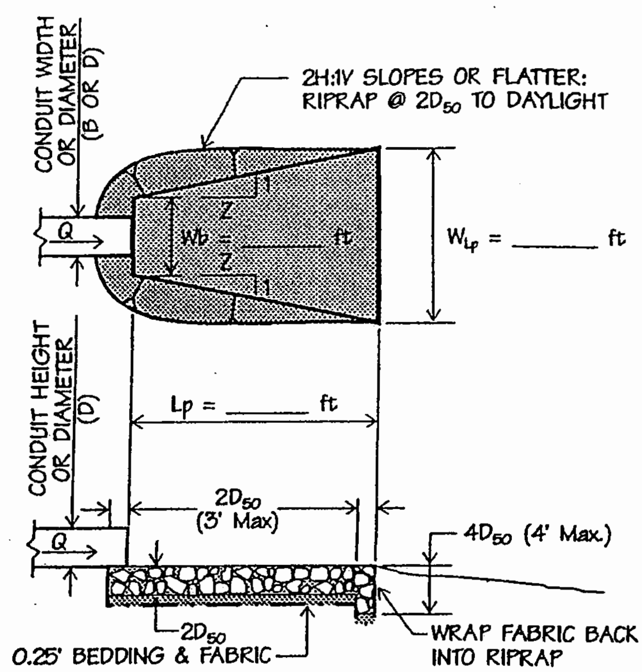
1. <u>Pima County Riprap Apron Method</u> This method is taken from a Pima County publication (Pima County), although Maricopa County presents the same material (Maricopa County). Neither manual identifies the originator or source of the material, which is unknown to the author, hence the method name of "Pima County". The procedure is presented in a worksheet format in Table "J-11"

2. <u>Pima County Riprap Plunge Basin Method</u> A modification to the riprap apron is a riprap plunge basin. The advantage of plunge basins is that the required riprap size is reduced; however, the reduced thickness may be offset by the enlarged area. Nonetheless, the Pima County procedure is presented in worksheet format in Table "J-12".

CULVERT I.D. No. ________(1) D_{50} = ________ft



CULVERT I.D. No. ________(1) D_{50} = ________ft



NOTE: SEE TABLE "J-10" REGARDING APPLICABILITY OF THIS PROCEDURE (SEE TABLE "L-5" FOR PARAMETER VALUES)

STEPS

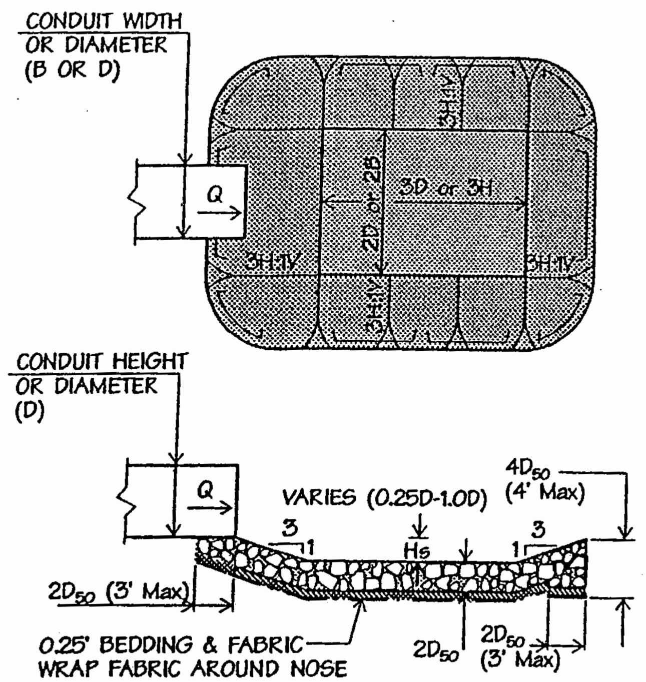
(1) - (2) Enter culvert identification number and corresponding Table "L-5" worksheet number.
(3) - (9) Enter data from Table "L-5" worksheet from the following columns: (22); (9); (5) or (6); (7); (13); (25), (26), or (27); and (33).
(10) Calculate D/2.
(11) Calculate Lp:
For TW ≤ D/2, Lp = D(8 + 17 log Fr)
For TW > D/2, Lp = D(8 + 55 log Fr)
(12) Adjust Lp, if necessary as follows:
3D ≤ Lp ≤ 10D. Enter value in column and on figure.
(13) Enter Z value as follows:
For TW ≤ D/2, Z=2
For TW > D/2, Z=5
(14) Wb = Width of protection at the culvert brink. If the protected embankments are at least D/2 high, then use double the outlet width (2D or 2B); otherwise, use triple (3D or 3B). Enter value in column and on figure.
(15) W_{Lp} = Width of protection at the end of protection = 2Lp/Z + Wb. Enter value in column and on figure..
(16) $D^{50} = 0.02U^{1.33}D^2/TW$. Enter size in column and on figure.

Culv. I.D. No. (1)	Ref. "L-5" Sheet No. (2)	DATA FROM TABLE "L-5" Trial No. (3)	Q (cfs) (4)	Dia. or Depth D (ft) (5)	Width B (ft) (6)	TW (ft) (7)	Froude Param. U (8)	Fr (9)	D/2 (ft) (10)	Lp (ft) (11)	Adj. Lp (ft) (12)	Z (13)	Wb (ft) (14)	W_{Lp} (ft) (15)	D_{50} (ft) (16)

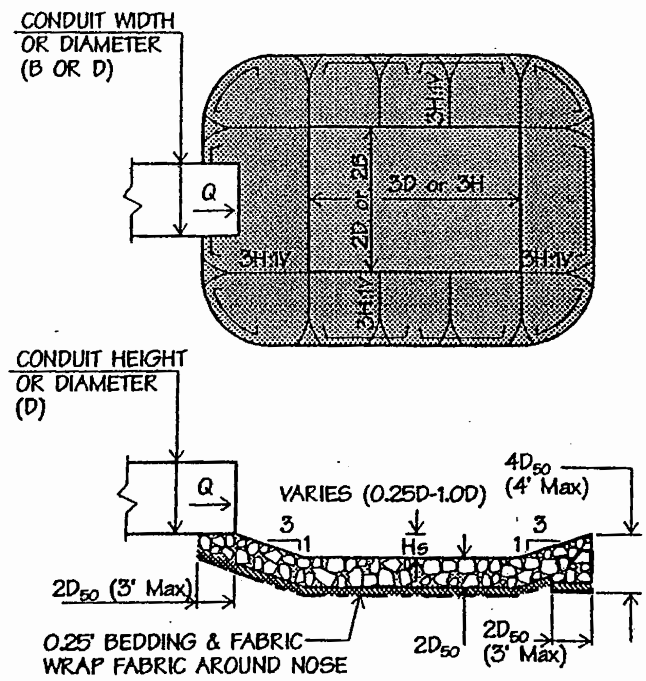
OUTLET RIPRAP PROTECTION DESIGN WORKSHEET
PIMA COUNTY APRON

TABLE "J-11"

CULVERT I.D. No. ________ D_{50} = ______ft



CULVERT I.D. No. ________ D_{50} = ______ft



NOTE: SEE TABLE "J-10" REGARDING APPLICABILITY OF THIS PROCEDURE (SEE TABLE "L-5" FOR PARAMETER VALUES)

STEPS

(1) - (2) Enter culvert identification number and corresponding Table "L-5" worksheet number.
(3) - (9) Enter data from Table "L-5" worksheet from the following columns: (22); (9); (5) or (6); (7); (13); (25), (26), or (27); and (33).
(10) Enter drop height Hs.
(11) Calculate D/2
(12) Calculate D_{50}:
For $Hs \leq D/2$, $D_{50} = 0.0125u^{1.33}D^2/TW$
For $Hs > D/2$, $D_{50} = 0.0082u^{1.33}D^2/TW$

Culv. I.D. No. (1)	Ref. "L-5" Sheet No. (2)	DATA FROM TABLE "L-5" Trial No. (3)	Q (cfs) (4)	Dia. or Depth D (ft) (5)	Height H (ft) (6)	Width B (ft) (7)	TW (ft) (8)	Froude Param. U (9)	Hs (ft) (10)	D/2 (ft) (11)	D_{50} (ft) (12)	Remarks

OUTLET RIPRAP PROTECTION DESIGN WORKSHEET
PRIMA COUNTY PLUNGE BASIN

TABLE "J-12"

3. UD & FCD Riprap Apron Method The Urban Drainage & Flood Control District (Denver Metropolitan) provides another method of designing riprap outlet protection. The procedure is presented in a worksheet format in Table "J-13".

The UD & FCD method may also be used to design outlet erosion protection for multi-barrel culvert installations by hypothetically replacing the multiple barrels with a single hydraulically equivalent rectangular conduit. The dimensions of the equivalent conduit may be established as presented below:

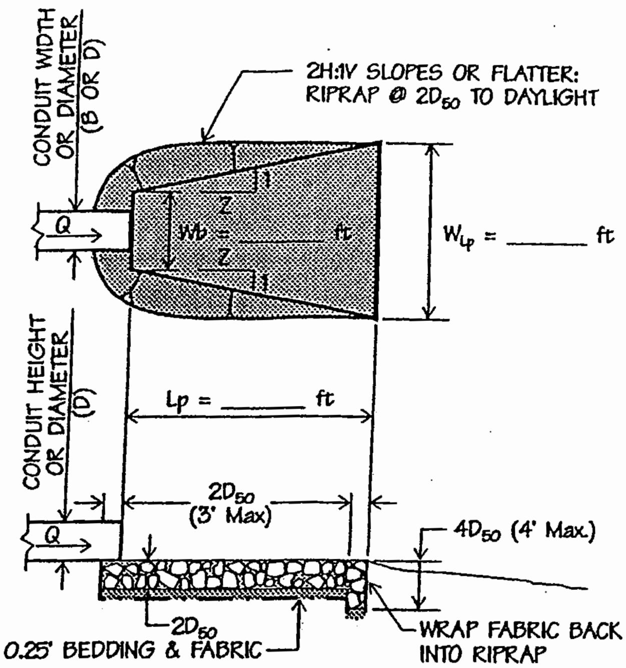
1) Distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed; otherwise, the flow through each barrel must be computed.

2) Compute the Froude parameter $Q_i/D_i^{2.5}$ (circular conduit) or $Q_i/W_iH_i^{1.5}$ (rectangular t), where the subscript i indicates the discharge and dimensions associated with an individual conduit. If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit.

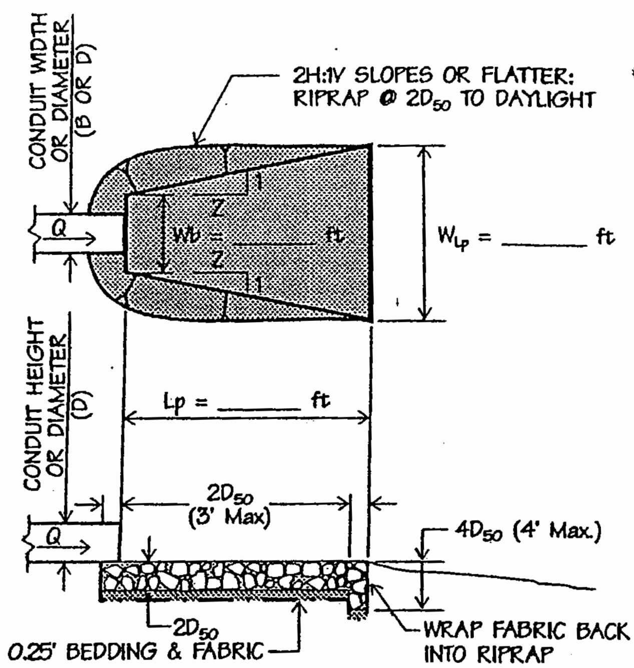
3) Make the height of the equivalent conduit, H_e, equal to the height, or diameter, of the selected individual conduit.

4) The width of the equivalent conduit, W_e, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $Q/W_eH_e^{1.5}$.

CULVERT I.D. No. ________(1) D_{50} = _______ft



CULVERT I.D. No. ________(1) D_{50} = _______ft



NOTE: SEE TABLE "J-10" REGARDING APPLICABILITY OF THIS PROCEDURE (SEE TABLE "L-5" FOR PARAMETER VALUES)

STEPS

(1) - (2) Enter culvert identification number and corresponding Table "L-5" worksheet number.
(3) - (10) Enter data from Table "L-5" worksheet from the following columns: (22); (9); (5) or (6); (7); (13); (25), (26), or (27); and (34).
(11) Determine D_{50} from Figure "J-12". Enter in column and on figure.
(12) $W_{Lp} = Q / (TW \cdot Vp)$ Enter value in column and on figure.
(13) Determine coefficient of expansion Ce from Figure "J-13".
(14) Calculate minimum length of protection $Lp = Ce(W_{Lp} - B)$. Enter value in column and on figure.
(15) Adjust Lp, if necessary, as follows: $3D \leq Lp \leq 10D$. Enter value in column and on figure.
(16) Wb = Width of protection at the culvert brink. If the protected embankments are at least D/2 high, then use double the outlet width (2D or 2B); otherwise, use triple (3D or 3B). Enter value in coulumn and on figure.

Culv. I.D. No. (1)	Ref. "L-5" Sheet No. (2)	DATA FROM TABLE "L-5" Trial No. (3)	Q (cfs) (4)	Dia. or Depth D (ft) (5)	Width B (ft) (6)	TW (ft) (7)	Froude Param. U (8)	Fr (9)	Vp (fps) (10)	D_{50} (ft) (11)	W_{Lp} (ft) (12)	Ce (13)	Lp (ft) (14)	Adj. Lp (ft) (15)	Wb (ft) (16)

OUTLET RIPRAP PROTECTION DESIGN WORKSHEET
UD&FCD APRON

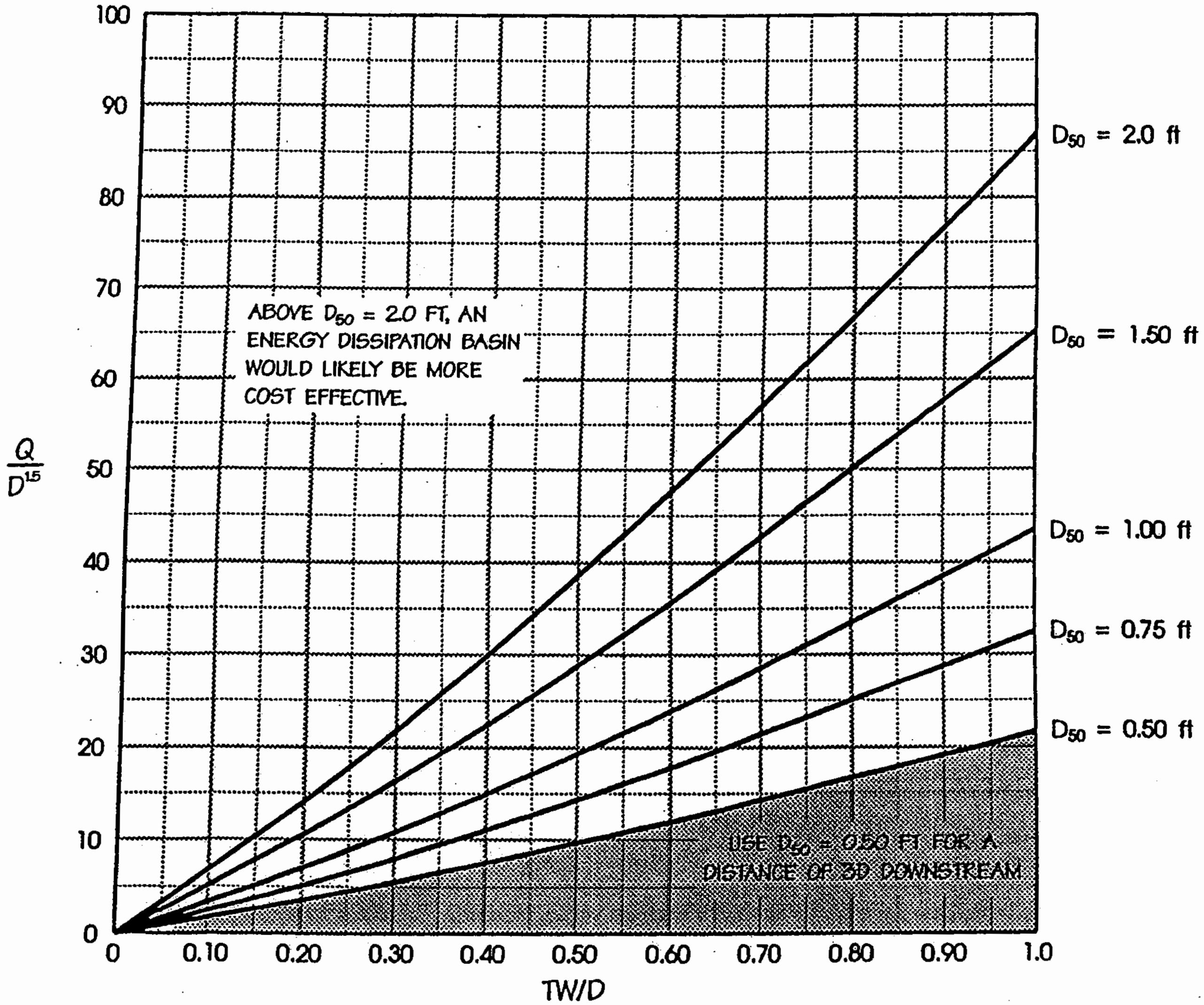
TABLE "J-13"

SOLUTION BY EQUATION

TW (ft)	FLOW CONDITION AT THE END OF THE CONDUIT: SUBCRITICAL ($dc < dn$; $Fr < 1.0$)	FLOW CONDITION AT THE END OF THE CONDUIT: SUPERCRITICAL ($dn < dc$; $Fr > 1.0$)
KNOWN	$D_{50} = \dfrac{0.023Q}{TW^{1.2}D^{0.3}}$	$D_{50} = \dfrac{0.028Q}{TW^{1.2}(D+dn)^{0.3}}$
UNKNOWN (Assume at 0.4D)	$D_{50} = \dfrac{0.069Q}{D^{1.5}}$	$D_{50} = \dfrac{0.195Q}{(D+dn)^{1.5}}$

IF $D_{50} < 0.50$ FT PER THE ABOVE, USE $D_{50} = 0.50$ FT FOR A DISTANCE OF 3D DOWNSTREAM

SOLUTION BY GRAPHICS



D_{50} = Mean rock diameter, ft
Q = Conduit flow, cfs
TW = Tailwater downstream, ft
D = Diameter of conduit, ft
dn, dc = Normal and Critical depth, respectively, ft
Fr = Froude Numer

NOTE: WHEN USING GRAPH, IF FLOW IS SUPERCRITICAL, SUBSTITUTE 0.5 (D+dn) FOR D. [0.5(D+dn) SHALL NOT EXCEED D]

RIPRAP D_{50} SIZE AT OUTLETS - UD&FCD
(CIRCULAR OR SQUARE CONDUITS)

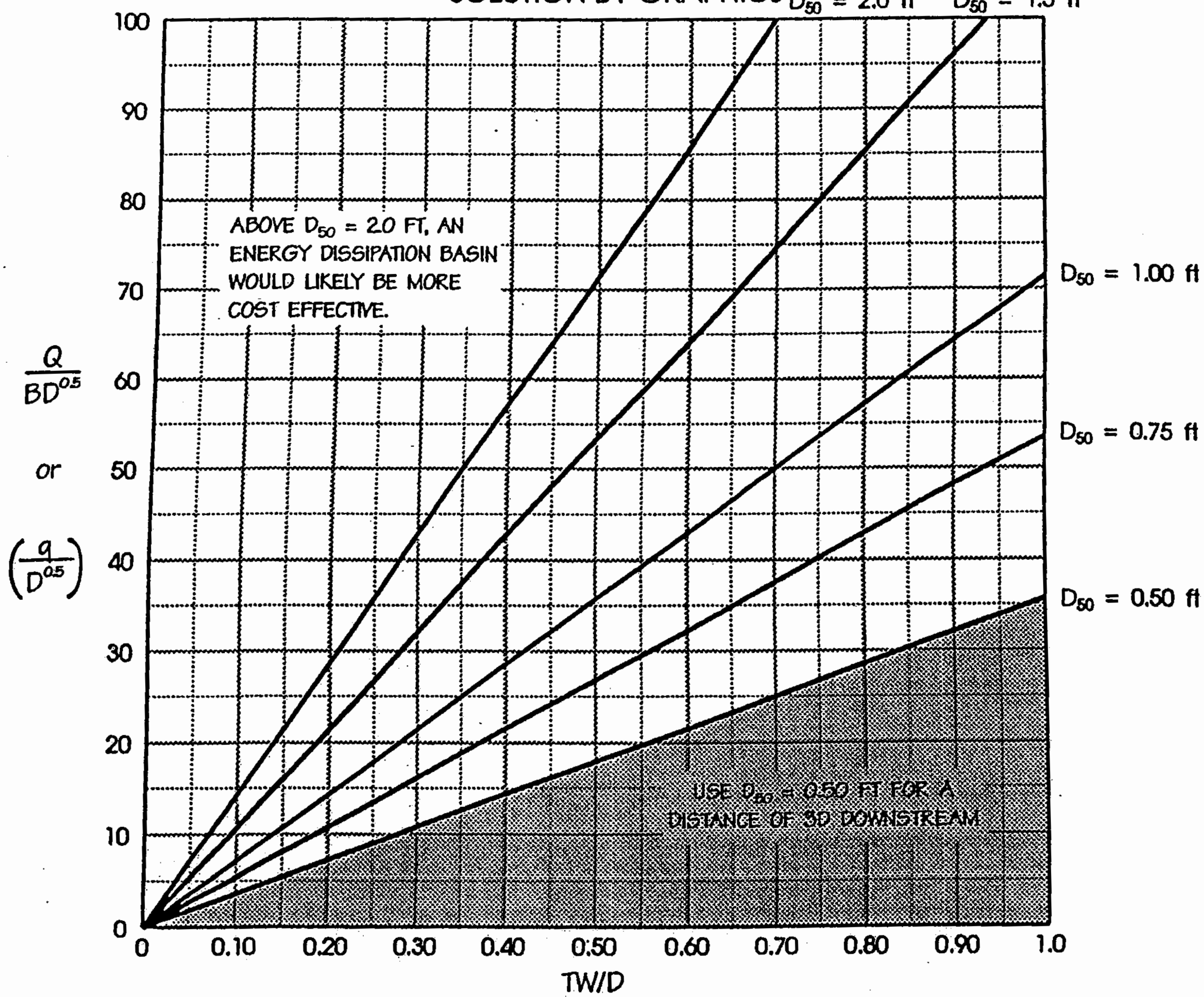
FIGURE "J-12a"

SOLUTION BY EQUATION

TW (ft)	FLOW CONDITION AT THE END OF THE CONDUIT: SUBCRITICAL ($dc < dn$; $Fr < 1.0$)	SUPERCRITICAL ($dn < dc$; $Fr > 1.0$)
KNOWN	$D_{50} = \dfrac{0.014D^{0.5}Q}{B(TW)} = \dfrac{0.014D^{0.5}q}{TW}$	$D_{50} = \dfrac{0.0099(D+dn)^{0.5}Q}{B(TW)} = 0.0099(D+dn)^{0.5}q/TW$
UNKNOWN (Assume at 0.4D)	$D_{50} = \dfrac{0.035Q}{B(D^{0.5})} = \dfrac{0.035q}{D^{0.5}}$	$D_{50} = \dfrac{0.049Q}{B(D+dn)^{0.5}} = \dfrac{0.049q}{(D+dn)^{0.5}}$

IF $D_{50} < 0.50$ FT PER THE ABOVE, USE $D_{50} = 0.50$ FT FOR A DISTANCE OF 3D DOWNSTREAM

SOLUTION BY GRAPHICS



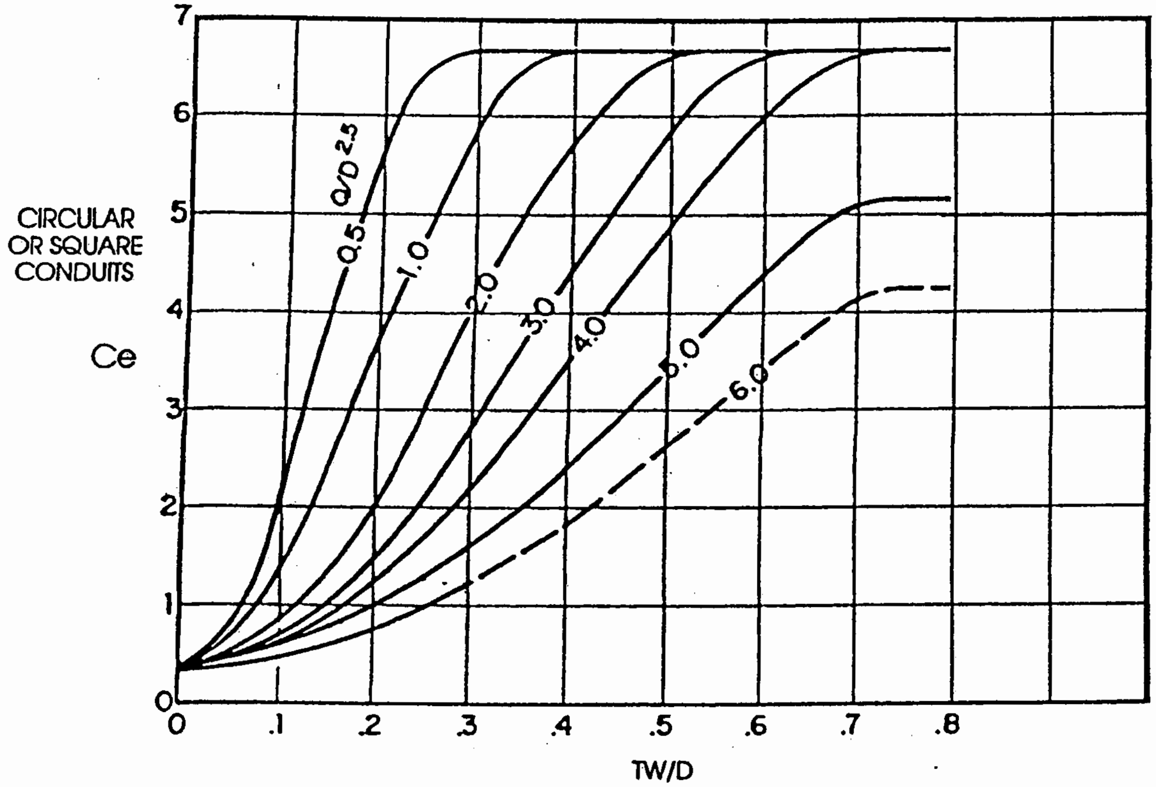
D_{50} = Mean rock diameter, ft
Q = Conduit flow, cfs
TW = Tailwater downstream, ft
D = Diameter of conduit, ft
dn, dc = Normal and Critical depth, respectively, ft
Fr = Froude Numer

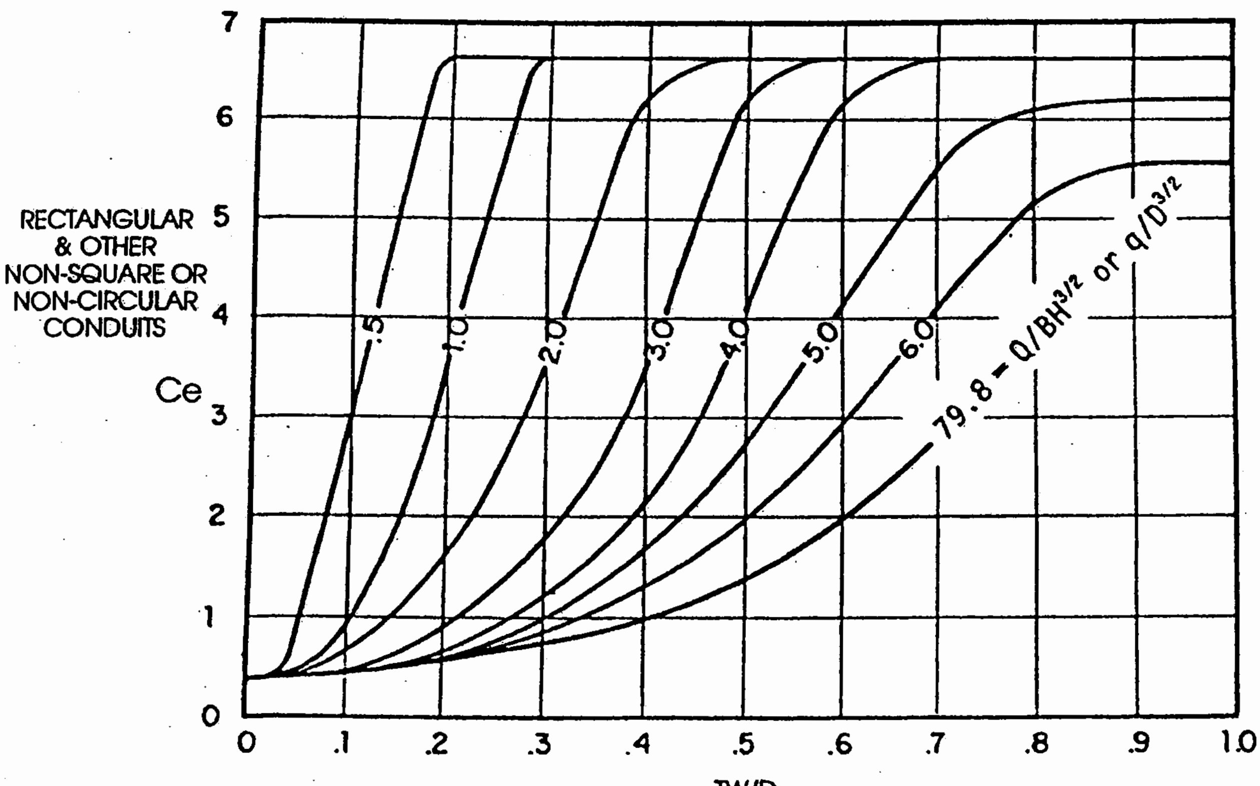
NOTE: WHEN USING GRAPH, IF FLOW IS SUPERCRITICAL, SUBSTITUTE 0.5 (D+dn) FOR D. [0.5(D+dn) SHALL NOT EXCEED D]

RIPRAP D_{50} SIZE AT OUTLETS - UD&FCD
(RECTANGULAR CONDUITS)

FIGURE "J-12b"

REPRODUCED FROM FIGURES 5-9 & 5-10 IN (UD&FCD).





COEFFICIENTS OF EXPANSION - UD&FCD

FIGURE "J-13"

4. HEC-14 Riprap Plunge Basin Method The design procedure presented herein for riprap energy dissipators is based on data obtained during a study 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 effective riprapped basins at culvert outfalls.

   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.

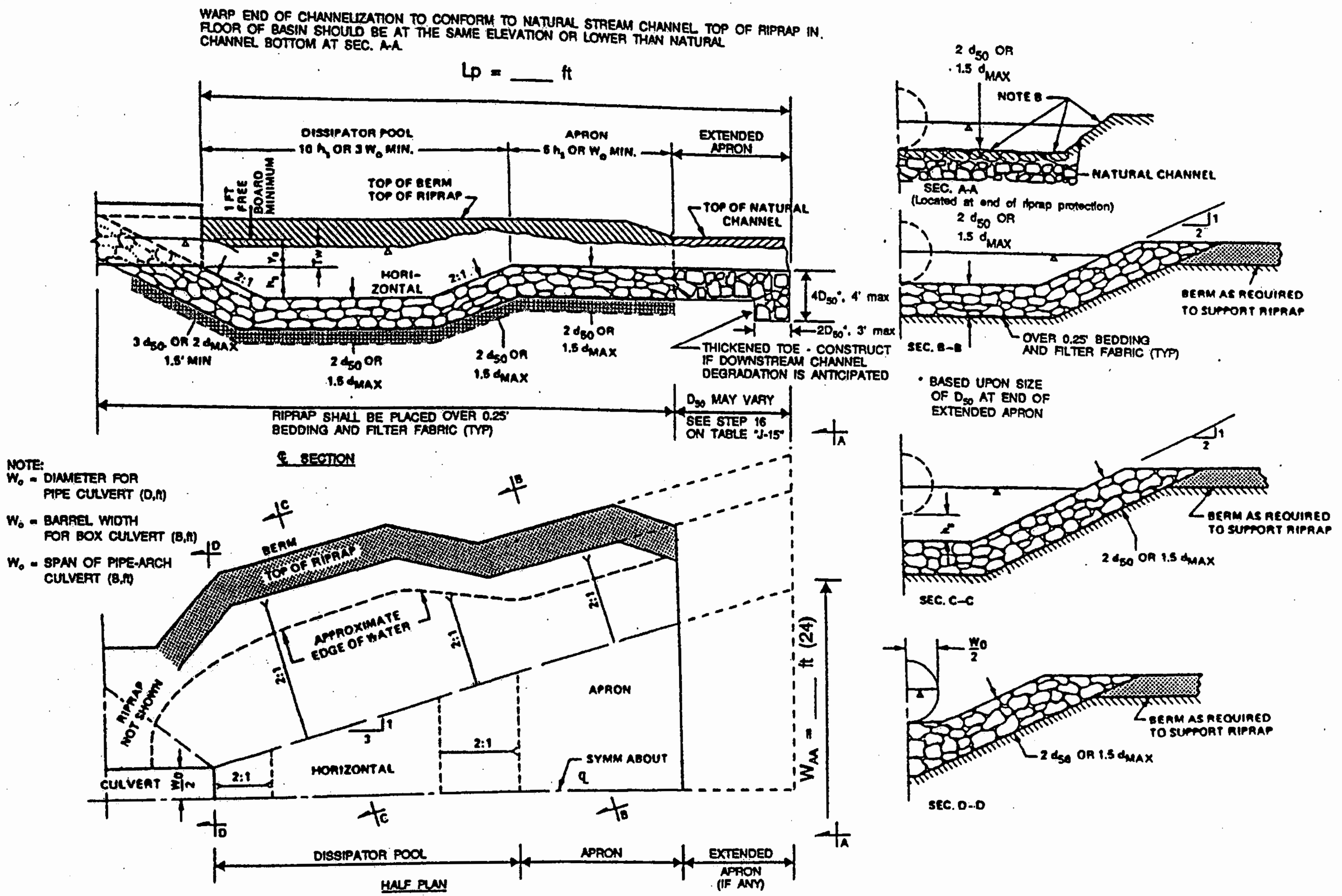
   When the ratio of tailwater depth to brink depth (TW/Yo) was less than 0.75 and the ratio of scour depth to size of riprap (h_s/d_{50}) was greater than 2.0, the scour hole functioned very efficiently as an energy dissipator. For high tailwater basins (TW/Yo greater than 0.75), the high velocity core 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 are shown on Figure "J-14", and a design worksheet which presents the design procedures is provided in Table "J-14".

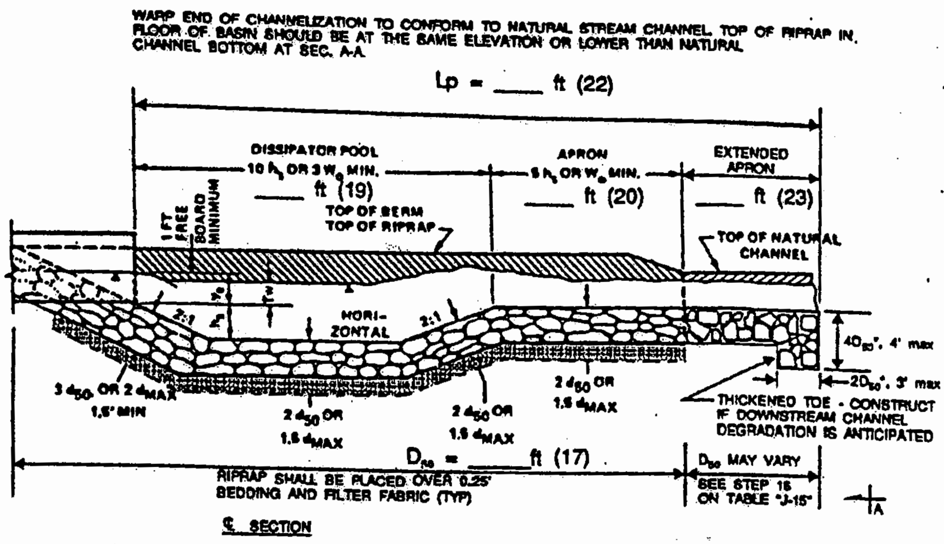
5. Other Standard Energy Dissipation Structures Concrete energy dissipation or stilling basin structures are required to prevent scour damages caused by high exit velocities and flow expansion turbulence at conduit outlets. Outlets structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection at conduit outlets is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical. Concrete outlet structures can be designed easily and are suitable for a wide variety of site conditions. In some cases, they are more economical than large rock basins, particularly where long term costs are considered.

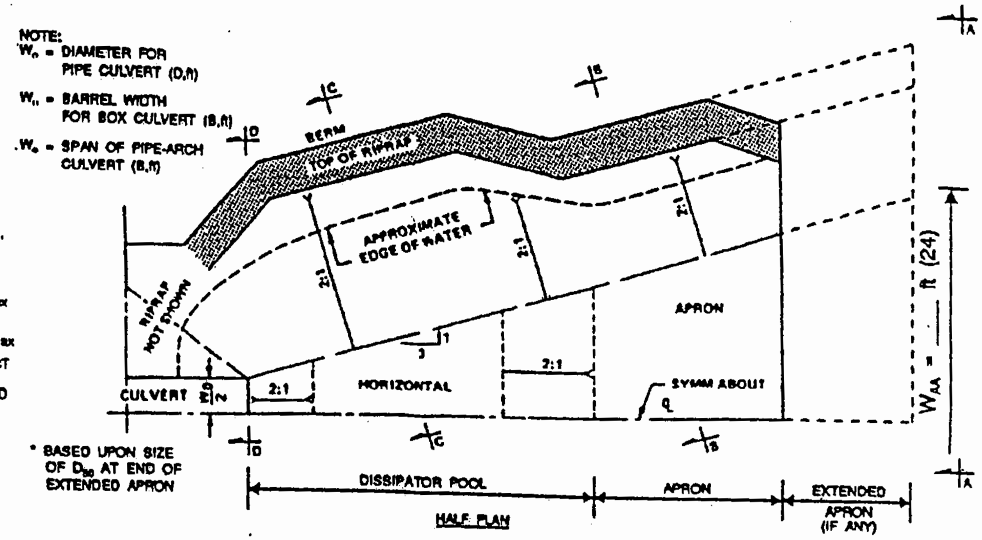
   Details of specific grouted rock and concrete energy dissipation structures are not provided in this manual. For these other methods of erosion control, reference is made to (FHWA HEC-14), (UD & FCD), (Maricopa County), and USBR publications listed in Section II of this manual.

MODIFIED FROM FIGURE XI-1 IN HEC-14



HEC-14 PLUNGE BASIN DESIGN DETAILS	FIGURE "J-14"





NOTE: DO NOT USE THIS PROCEDURE IF Fr > 2.5 (SEE TABLE "L-5" FOR Fr VALUE)

STEPS

(1) - (2) Enter culvert identification number and corresponding Table "L-5" worksheet number.
(3) - (10) Enter data from Table "L-5" worksheet from the following columns: (22); (9); (5) or (7); (13); (24) or (29) times (5) or (6); (32); (33) & (34).
(11) Enter Figure "J-15" with Fr; read vertically. Select D_{50}/Ye value in mid-range of shaded area.
(12) Read horizontally to hs/Ye from trial D_{50}/Ye & Fr intersection.
(13) Divide (12) by (11) to obtain hs/D_{50}. If value is between 2 and 4, skip to step (17).
(14) - (18) Repeat steps (11) - (13) if (13) is not between 2 and 4.
(17) Multiply acceptable D_{50}/Ye value by step (8) to get D_{50}. Round to conventional size, and enter in column and on figure.
(18) Multiply acceptable hs/D_{50} value by step (17) to get hs. Enter value rounded to nearest 0.1 foot in column and on figure.
(19) Pool length is greater of 10hs and 3Wo. Enter value in column and on figure.
(20) Apron length is greater of 5hs and Wo. Enter value in column and on figure.
(21) If TW/Yo > 0.75, additional downstream riprap may be req'd to protect against jet stream. Go now to Table "J-15" for calculations of Lp and rock size to use for the extended basin.
(22) If Tw/Yo > 0.75, enter Lp value from step 17 on Table "J-15". If TW/Yo ≤ 0.75, the minimum length of protection Lp is the greater of (19) + (20) and

$$1.5\left(\frac{Q}{V_p TW} - W_o\right)$$

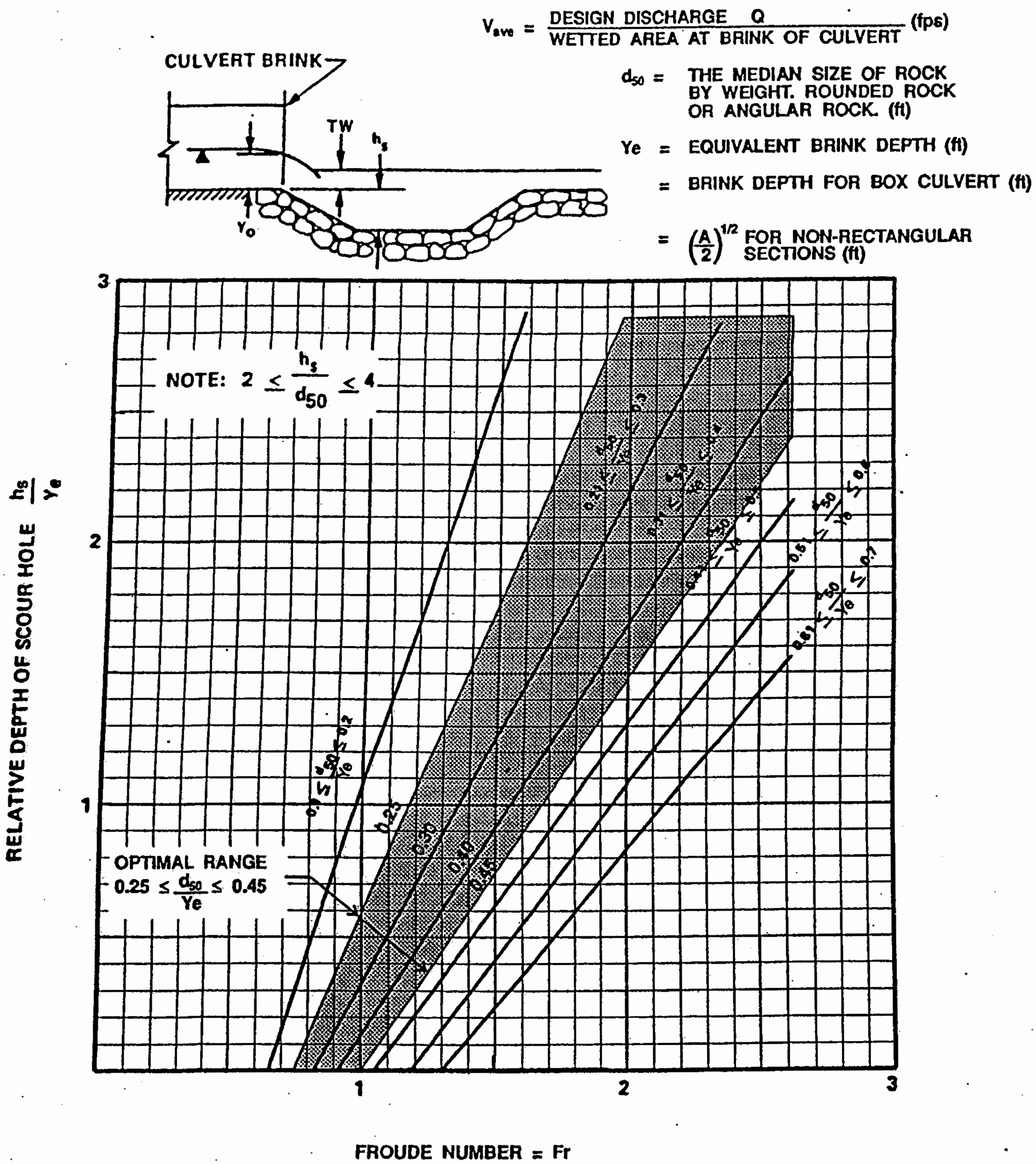
Enter in column and on figure.
(23) Extended apron length, if any, is Lp - [(19) + (20)]. Enter value in column and on figure. If this value is a positive number, Table "J-15" should be used to design smaller riprap req'd for the extended apron.
(24) The width of the end of the apron or extended apron is $W_{AA} = 2Lp/3 + Wo$. Enter in column and on figure.

		DATA FROM TABLE "L-5"								FIRST TRIAL			SECOND TRIAL			DESIGN VALUE							
Culv. I.D. No. (1)	Ref. "L-5" Sheet No. (2)	Trial No. (3)	Q (cfs) (4)	Culv. Width Wo (D or B) (5)	TW (ft) (6)	Yo (ft) (7)	Ye (ft) (8)	Fr (9)	Vp (cfs) (10)	D_{50}/Ye Value (11)	hs/Ye (12)	hs/D_{50} (13)	D_{50}/Ye Value (14)	hs/Ye (15)	hs/D_{50} (16)	D_{50} (ft) (17)	hs (ft) (18)	Pool Length (ft) (19)	Apron Length (ft) (20)	TW/Yo (21)	Lp (ft) (22)	Ext. Apron length (ft) (23)	W_{AA} (ft) (24)

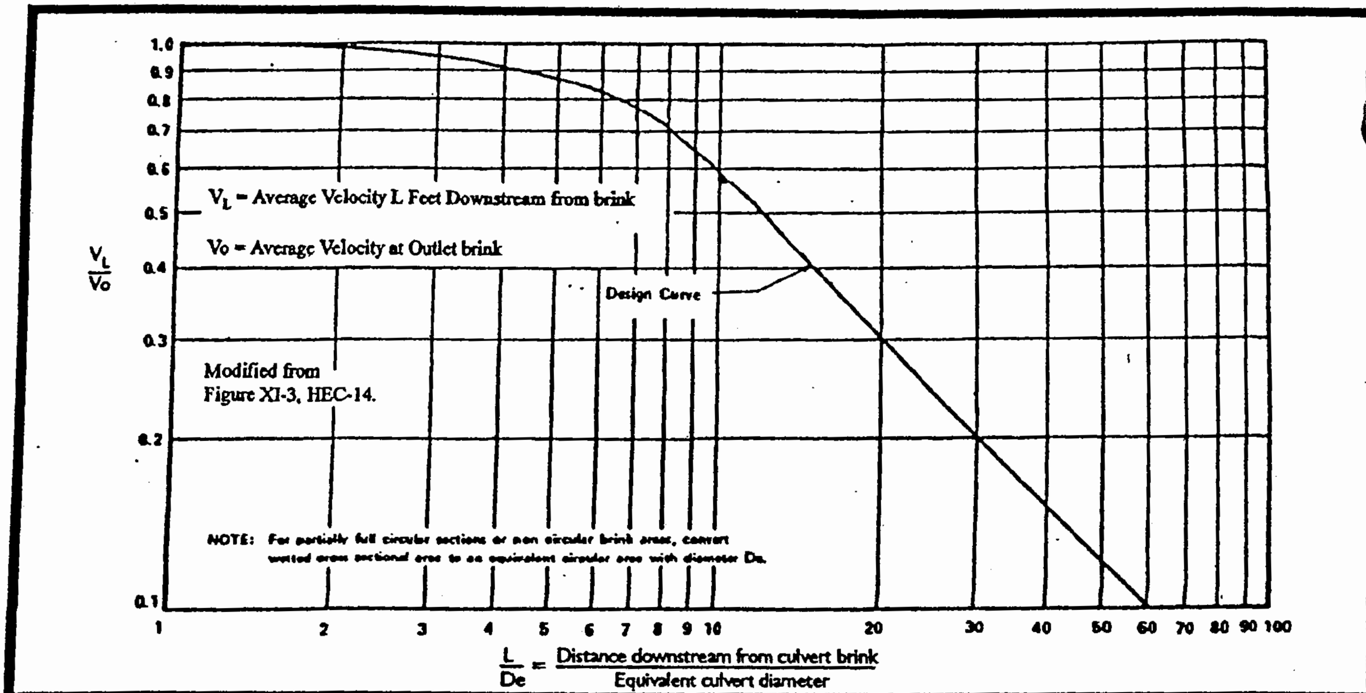
OUTLET RIPRAP PROTECTION DESIGN WORKSHEET
HEC-14 PLUNGE BASIN

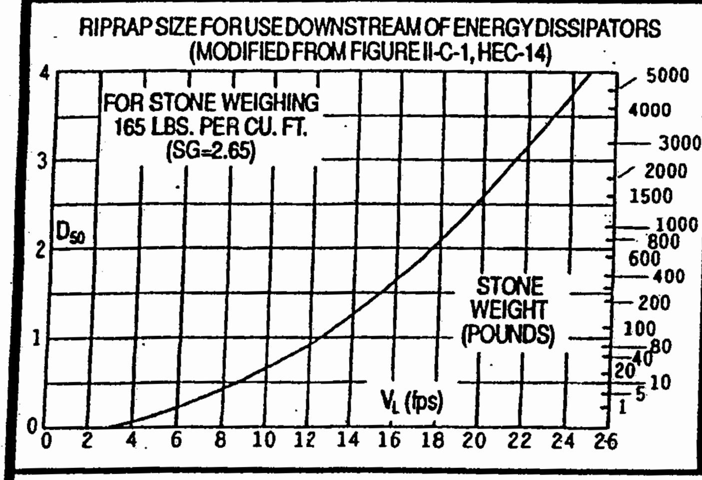
TABLE "J-14"

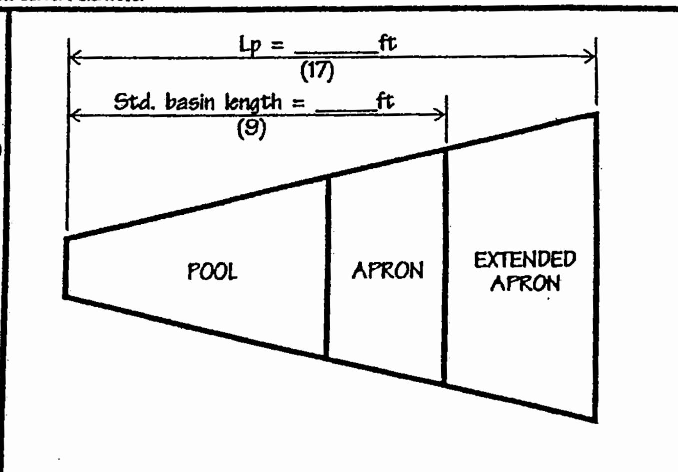
MODIFIED FROM FIGURE XI-2 IN HEC-14



HEC-14 PLUNGE BASIN DESIGN GRAPH	FIGURE "J-15"







STEPS (1-13 here, 14-17 below)

(1) - (2) Enter culvert I.D. and corresponding Table Sheet No.
(3) - (6) Enter data from Table "J-11d" from the following columns: (8); (10); (19); and (20).
(7) & (8) Enter values from Table "L-5", columns (30) & (31).
(9) Add steps (5) and (6). Enter in column and on figure.
(10) De = Ye for circular conduits, and $[4Ao/\pi]^{0.5}$ otherwise.

(11) - (15) Provide an incremental tabulation to determine req'd Lp and where riprap size reductions could occur.
(11) Select ratios of L/De from the graph at top, starting with the equivalent of the STD basin length in step (9). With each value selected, do steps 12-15, then return to step (11) until a length is reached where $V_L \le Vp$.
(12) Enter graph at top with L/De and read V_L/Vo. Enter value.
(13) Multiply step (12) by step (8) to get V_L. Calculations may end on the row where $V_L \le Vp$.

Culv. I.D. No. (1)	Ref. "J-14" Sheet No. (2)	DATA FROM TABLE "J-14" Ye (ft) (3)	Vp (fps) (4)	Pool Length (ft) (5)	Apron Length (ft) (6)	TABLE "L-5" Ao (ft²) (7)	Vo (fps) (8)	STD. Basin Length (ft) (9)	De (ft) (10)	L - V - D_{50} Tabulation $\frac{L}{De}$ (11)	$\frac{V_L}{Vo}$ (12)	V_L (fps) (13)	MIN. D_{50} (ft) (14)	L (ft) (15 & 17)	Design D_{50} (ft) (16)

(14) Enter step (13) V_L value into graph at mid-page and read D_{50}.
(15) Multiply step (11) by step (10) to get distance L where D_{50} in step 4 is acceptable.
(16) Do not do step 16 until steps 11-15 have been completed for all V_L > Vo. Then, using data in columns (14) and (15), select design D_{50} to be used between logical increments of L. D_{50} shall not be less than 0.5, feet however.
(17) The distance L where $V_L \le Vp$ is Lp. Enter this value on the figure above, and on Table "J-14" in column (22) and the figure. Complete Table "J-14" by performing steps (23) & (24).

EXTENDED RIPRAP APRON DESIGN WORKSHEET

TABLE "J-15"

V. SPECIFICATIONS

A. RIPRAP

Work involving riprap shall conform to this subsection.

1. Materials

a. Specifications Stone used for riprap shall be hard and durable; resistant to weathering and to water action; free from overburden, spoil, shale and organic material; and shall meet the gradation requirements specified. Neither breadth nor thickness of a single stone should be less than one-third its length. Shale and stone with shale seams are not acceptable. The minimum weight of the stone shall be 155 pounds per cubic foot as computed by multiplying the specific gravity (bulk-saturated-surface-dry basis, AASHO Test T 85) times 62.3 pounds per cubic foot. This corresponds with a specific gravity of 2.5. A size-weight-specific gravity relationship is provided in Figure "J-16".

Each load of riprap shall be reasonably well graded from the smallest to the maximum size specified. Stones smaller than $D_{50}/3$ will not be permitted in an amount exceeding 5 percent by weight of each load. For an economical thickness, the largest stones (D_{100}) should not exceed 1.5 D_{50}; even so, only 10% by weight of the riprap may exceed 1.5 D_{50}, and then only up to a maximum size of 2.0 D_{50}. The slope of gradation σ shall be within the range of 1.50 and 2.25 as determined by the following equation

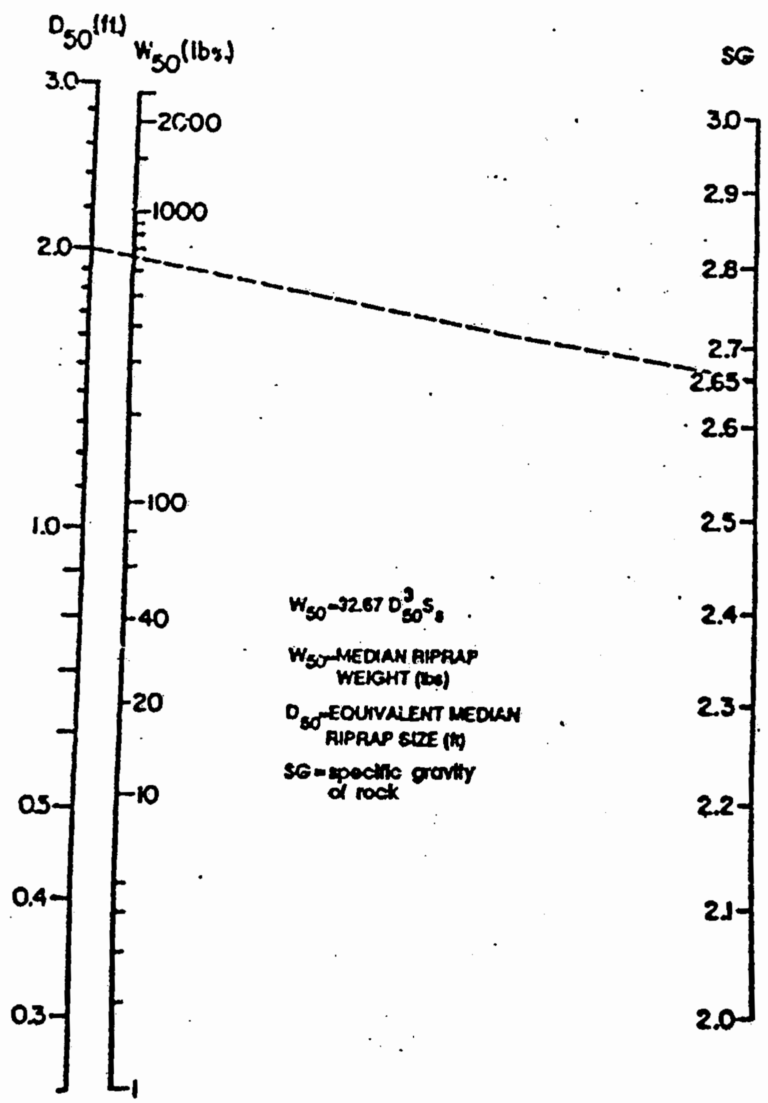
$$\sigma = 0.5\left[\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right]; \text{ or}$$

$$3 \leq \left[\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right] \leq 4.5$$

(D_X represents the rock size for which "x" percent of riprap particles by weight are finer)

b. Engineer Approval The sources from which the stone will be obtained shall be selected in advance of the time when the stone will be required in the Work. The acceptability of the stone by the Engineer must precede riprap use, and will be determined by Engineer approval of service records, suitable tests, or by visual inspection. The approval of some rock fragments from a particular quarry site shall not be construed as constituting the approval of all rock fragments taken from that quarry.

REPRODUCED FROM CHART 5 IN HEC-11



EXAMPLE

GIVEN:
D_{50}=2.0 ft.
SG =2.65

FIND:
W_{50}

SOLUTION:
W_{50}=680 lbs

RIPRAP SIZE - WEIGHT-SPECIFIC GRAVITY RELATIONSHIP

FIGURE "J-16"

In the absence of service records, resistance to disintegration from the type of exposure to which the stone will be subjected will be determined by any or all of the following tests:

1) When the riprap must withstand abrasive action from material transported by the stream, the abrasion test in the Los Angeles machine shall also be used. When the abrasion test in the Los Angeles machine (AASHO Test T 96) is used, the stone shall have a percentage loss of not more than 40% after 500 revolutions.

2) In locations subject to freezing or where the stone is exposed to salt water, the sulfate soundness test (AASHO Test T 104 for ledge rock using sodium sulfate) shall be used. Stones shall have a loss not exceeding 10 percent with the sulfate test after 5 cycles.

3) When the freezing and thawing test (AASHO Test 10 for ledge rock procedure A) is used as a guide to resistance to weathering, the stone should have a loss not exceeding 10 percent after 12 cycles of freezing and thawing.

2. Construction Requirements

a. General Slopes to be protected by riprap shall be free of brush, trees, stumps, and other objectionable materials and be dressed to smooth surface. All soft or spongy material shall be removed to the depth shown on the plans or as directed by the Engineer and replaced with approved native material. Filled areas will be compacted and a toe trench as shown on the plans shall be dug and maintained until the riprap is placed.

Filter fabric covered with 0.25 feet of clean granular material shall be placed on the prepared slope or area as specified before the stone is placed.

The Contractor shall maintain the riprap until all work on the contract has been completed and accepted. Maintenance shall consist of the repair of areas where damaged by any cause.

b. Rock Riprap Stone for riprap shall be placed on the prepared slope or area in a manner which will produce a reasonably well-graded mass of stone with the minimum practicable percentage of voids. The entire mass of stone shall be placed so as to be in conformance with the lines, grades, and thicknesses shown on the plans. Riprap shall be placed to its full course thickness at one operation and in such a manner as to avoid displacing the underlying material. Placing of riprap in layers, or by dumping into chutes, or by similar methods likely to cause segregation, will not be permitted.

The larger stones shall be well distributed and the entire mass of stone shall conform to the specifications on the construction drawings. All material going into riprap protection shall be so placed and distributed that there will be no large accumulations of either the larger or smaller sizes of stone.

It is the intent of these specifications to produce a fairly compact riprap protection in which all sizes of material are placed in their proper proportions. Hand placing or rearranging of individual stones by mechanical equipment may be required to the extent necessary to secure the results specified.

Unless otherwise authorized by the Engineer, the riprap protection shall be placed in conjunction with the construction of the embankment with only sufficient lag in construction of the riprap protection as may be necessary to allow for proper construction of the portion of the embankment protected and to prevent mixture of embankment and riprap.

B. FILTER FABRIC

Underlying riprap liners shall be an engineering filter fabric and 0.25 foot of granular bedding material to protect the fabric during riprap placement. Filter fabric shall conform to the specifications of this subsection.

1. Materials

a. Specifications Minimum properties for geotextiles shall be as presented in Table "J-16".

b. Engineer Approval The Contractor shall submit fabric specifications to the Engineer for approval prior to use on the project.

2. Construction Requirements

a. General All subgrade compaction and grading shall be completed and approved by the Engineer prior to placement of fabric. Also, surfaces where fabric shall be placed shall be placed shall be free of brush, trees, stumps, sharp rocks exceeding 1½" in size, and other objectionable materials, and shall be dressed to a smooth surface.

b. Placement Filter fabric shall be placed similar to erosion control nettings as shown on Figure "J-17", except that check slots are not required. Figure "J-18" shows fabric wrapping at the toe of revetments.

Table "J-16"
Filter Fabric Specifications

Property	Class A[1]	Class B[2]	Test Method
Grab Strength, lbs	200	90	ASTM D-4632
Elongation, % min.	15	15	ASTM D-4632
Seam Strength, lbs[3]	180	80	ASTM D-4632
Puncture Strength, lbs	80	40	ASTM D-3787
Burst Strength, psi	320	140	ASTM D-3786
Trapezoid Tear, lbs	50	30	ASTM D-4533
Apparent Opening Size (AOS) US Std Sieve	AOS less than 0.297 mm (greater than No. 50 sieve)		CW 002215
Permeability, cm/sec[4]	k fabric > k soil for all classes		ASTM D-4491
Ultraviolet Degradation at 150 hours	70% strength retained for all classes		ASTM D-4355

[1] Class A erosion control geotextiles are used where installation stresses are more severe than for Class B applications.

[2] Class B erosion control geotextiles are used in structures or under conditions where the fabric is protected by a sand cushion or by "zero drop height" placement of stone. Stone placement depth should be less than 3 feet and stone weights should not exceed 250 pounds.

[3] Values apply to both field and manufactured seams, if required.

[4] A nominal coefficient of permeability may be determined by multiplying permittivity value by nominal thickness. The k value of the fabric should be greater than the k value of the soil.

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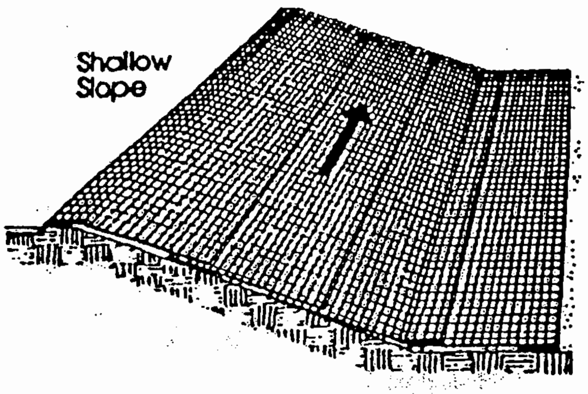
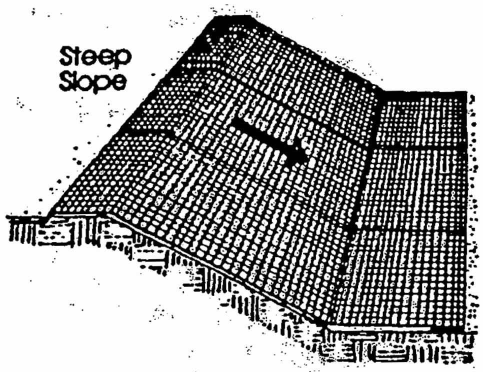


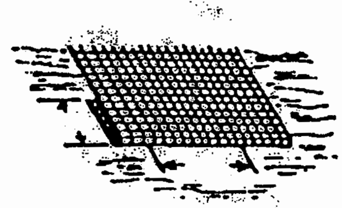
On shallow slopes, strips of netting may be applied across the slope.

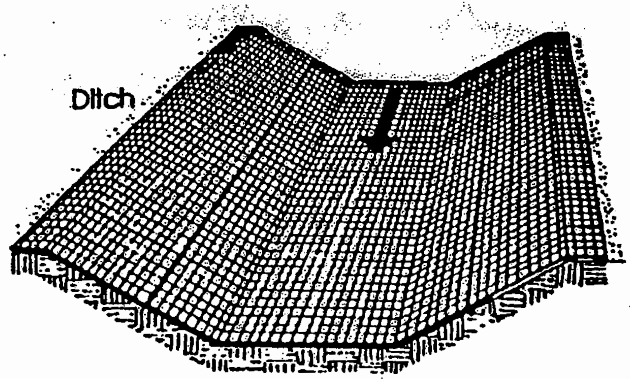
Where there is a berm at the top of the slope, bring the netting over the berm and anchor it behind the berm.



On steep slopes, apply strips of netting parallel to the direction of flow and anchor securely.

Bring netting down to a level area before terminating the installation. Turn the end under 6" and staple at 12" intervals.

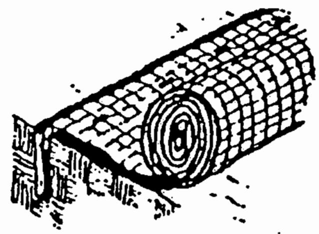




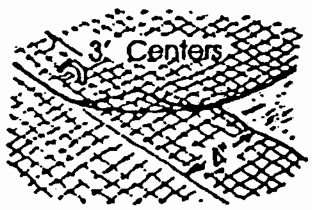
In ditches, apply netting parallel to the direction of flow. Use check slots every 15 feet. Do not join strips in the center of the ditch.

FILTER FABRIC PLACEMENT	FIGURE "J-17a"

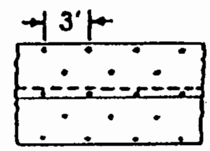
REPRODUCED FROM UD&FCD

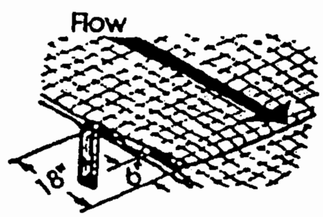


Anchor Slot: Bury the up-channel end of the net in a 6" deep trench. Tamp the soil firmly. Staple at 12" intervals across the net.

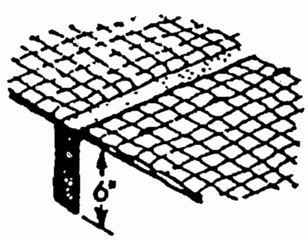


Overlap: Overlap edges of the strips at least 4". Staple every 3 feet down the center of the strip.

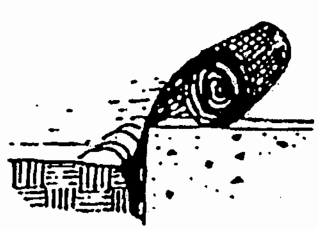




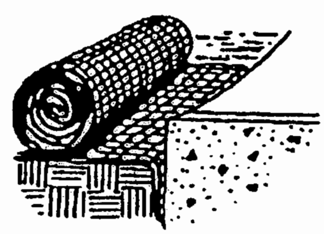
Joining Strips: Insert the new roll of net in a trench, as with the Anchor Slot. Overlap the up-channel end of the previous roll 18" and turn the end under 6". Staple the end of the previous roll just below the anchor slot and at the end at 12" intervals.



Check Slots: On erodible soils or steep slopes, check slots should be made every 15 feet. Insert a fold of the net into a 6" trench and tamp firmly. Staple at 12" intervals across the net. Lay the net smoothly on the surface of the soil - do not stretch the net, and do not allow wrinkles. (Not required for filter fabrics)

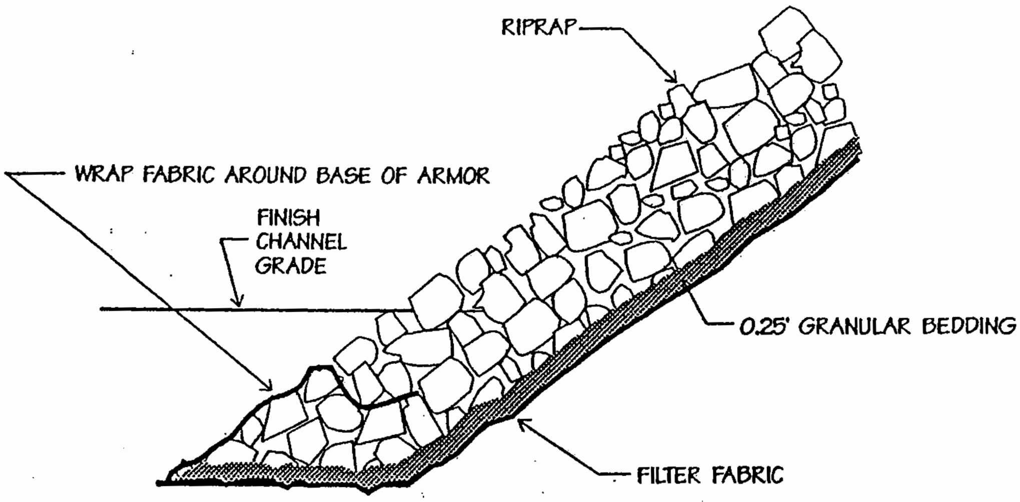


Anchoring Ends At Structures: Place the end of the net in a 6" slot on the up-channel side of the structure. Fill the trench and tamp firmly. Roll the net up the channel. Place staples at 12" intervals along the anchor end of the net.



FILTER FABRIC PLACEMENT	FIGURE "J-17b"

MODIFIED FROM FIGURE 19 IN HEC-11



FILTER FABRIC TOE WRAPPING	FIGURE "J-18"