HYDRAULIC DESIGN CRITERIA

SHEET 703-1

RIPRAP PROTECTION

TRAPEZOIDAL CHANNEL, 60 DEG-BEND

BOUNDARY SHEAR DISTRIBUTION

1. Riprap used to aid in the stabilization of natural streams and artificial channels is most commonly placed in the vicinity of bends. Procedures for estimating the required size of riprap in straight channels have been presented by the U. S. Army Engineer Waterways Experiment Station[1] and Office, Chief of Engineers.[2] No similar procedure has been developed for evaluating riprap size for channel bends. Hydraulic Design Chart 703-1 is based on laboratory tests at the Massachusetts Institute of Technology (MIT)[3] and should be useful for estimating relative boundary shear distribution in simple channel bends having trapezoidal cross sections, moderate side slopes, and approximately 60-deg deflection angles. It may also serve as a general guide for riprap gradation in natural channel bends of similar geometry. Shear distribution diagrams for other bend geometries and flow conditions have been published.[3,4]

2. Laboratory studies of boundary shear in open channel bends of trapezoidal cross section[3,5] indicate that the highest boundary shear caused by the bend geometry occurs immediately downstream from the bend and along the outside bank. Another area of high boundary shear is located at the inside of the bend. The relative boundary shear distribution in a simple bend with a rough boundary is given in Chart 703-1. The chart is based on fig. 21 of the MIT report.[3]

3. Experimental Data. Laboratory tests on smooth channel bends have been made at MIT,[3] at U. S. Bureau of Reclamation,[5] and at the University of Iowa.[6] In addition, limited tests on rough channel bends have been made at MIT. In the latter tests, the channel was roughened by fixing 0.18- by 0.10- by 0.10-in. parallelepipeds to the boundary in a random manner which resulted in an absolute roughness height of 0.10 in. The MIT test channel was 24 in. wide with 1 on 2 side slopes. The boundary shear distribution pattern has been generally found to be the same in all tests on simple curves having smooth and rough boundary conditions. However, the magnitude of the ratio of bend local boundary shear to the average boundary shear in the approach channel appears to be a function of the channel and bend geometry. Some work has also been done at MIT[3] on boundary shear distribution in double and reverse curve channels.

4. Application. Extensive variation in riprap gradation throughout a bend may not be practical or economical. However, increasing the 50 percent rock size and the thickness of the riprap blanket in areas of expected high boundary shear is recommended. Chart 703-1 can be used as a

guide for defining the location and extent of these areas in simple channel bends. The boundary shear ratios should be less than those shown in Chart 703-1 for bends with smaller deflection angles or with larger ratios of bend radius to water-surface width (r/w).

5. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Hydraulic Design of Rock Riprap, by F. B. Campbell. Miscellaneous Paper No. 2-777, Vicksburg, Miss., February 1966.

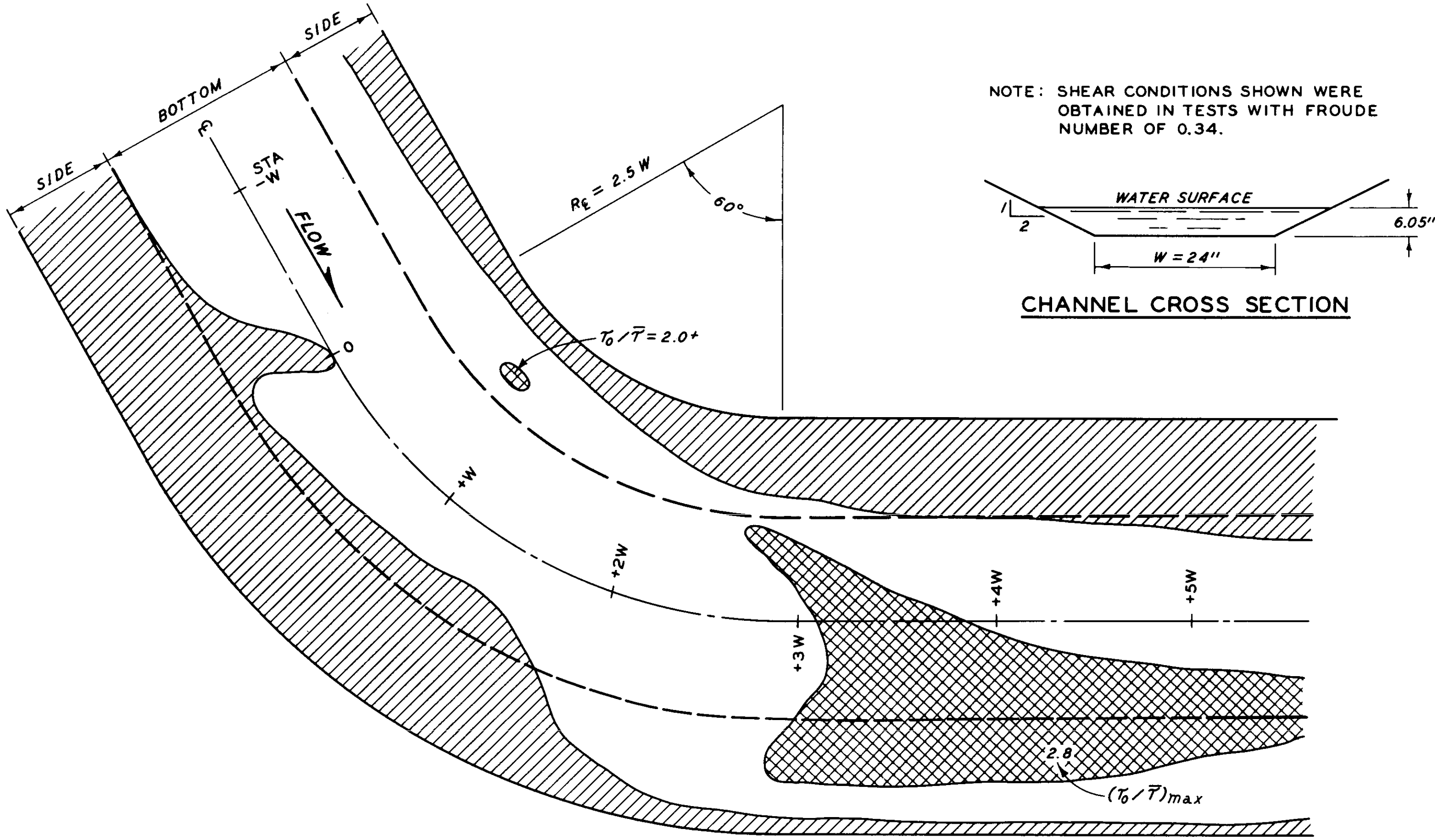
(2) U. S. Army Engineer, Office, Chief of Engineers, Stone Riprap Protection for Channels, by S. B. Powell.

(3) Ippen, A. T., and others, Stream Dynamics and Boundary Shear Distributions for Curved Trapezoidal Channels. Report No. 47, Hydrodynamics Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, January 1962.

(4) Ippen, A. T., and Drinker, P. A., "Boundary shear stress in trapezoidal channels." ASCE, Hydraulics Division, Journal, vol 88, HY 5, paper 3273 (September 1962), pp 143-179.

(5) U. S. Bureau of Reclamation, Progress Report No. 1--Boundary Shear Distribution Around a Curve in a Laboratory Canal, by E. R. Zeigler. Hydraulics Branch Report No. HYD 526, 26 June 1964.

(6) Yen, Ben-Chie, Characteristics of Subcritical Flow in a Meandering Channel. Institute of Hydraulic Research, University of Iowa, Iowa City, 1965.



NOTE: SHEAR CONDITIONS SHOWN WERE OBTAINED IN TESTS WITH FROUDE NUMBER OF 0.34.

CHANNEL CROSS SECTION

LEGEND

- $\tau_0 < \bar{\tau}$
- $\bar{\tau} < \tau_0 < 2.0\bar{\tau}$
- $2.0\bar{\tau} < \tau_0 < 2.8\bar{\tau}$

τ_0 = LOCAL BOUNDARY SHEAR STRESS
$\bar{\tau}$ = AVERAGE BOUNDARY SHEAR STRESS IN APPROACH FLOW.

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

TRAPEZOIDAL CHANNEL 60-DEG BEND

BOUNDARY SHEAR DISTRIBUTION

HYDRAULIC DESIGN CHART 703-1

WES 1-68

HYDRAULIC DESIGN CRITERIA

SHEET 704

ICE THRUST ON HYDRAULIC STRUCTURES

1. The expansion of an ice sheet as the result of a rise in air temperature can develop large thrusts against adjacent structures. The magnitude of this thrust is dependent upon the thickness of the ice sheet, the rate of air temperature rise, the amount of lateral restraint, and the extent of direct penetration of solar energy. Ice pressures from 3350 to 30,000 lb per lin ft[(1)] have been used for design purposes. EM 1110-2-2200[(3)] suggests a unit pressure of not more than 5000 lb per sq ft of contact area and indicates that ice thickness in the United States will not normally exceed 2 ft.

2. Although the work of Rose[(2)] stimulated a number of studies on ice pressure, the graphs proposed by him are of value for design purposes. These graphs are reproduced in HDC 704.

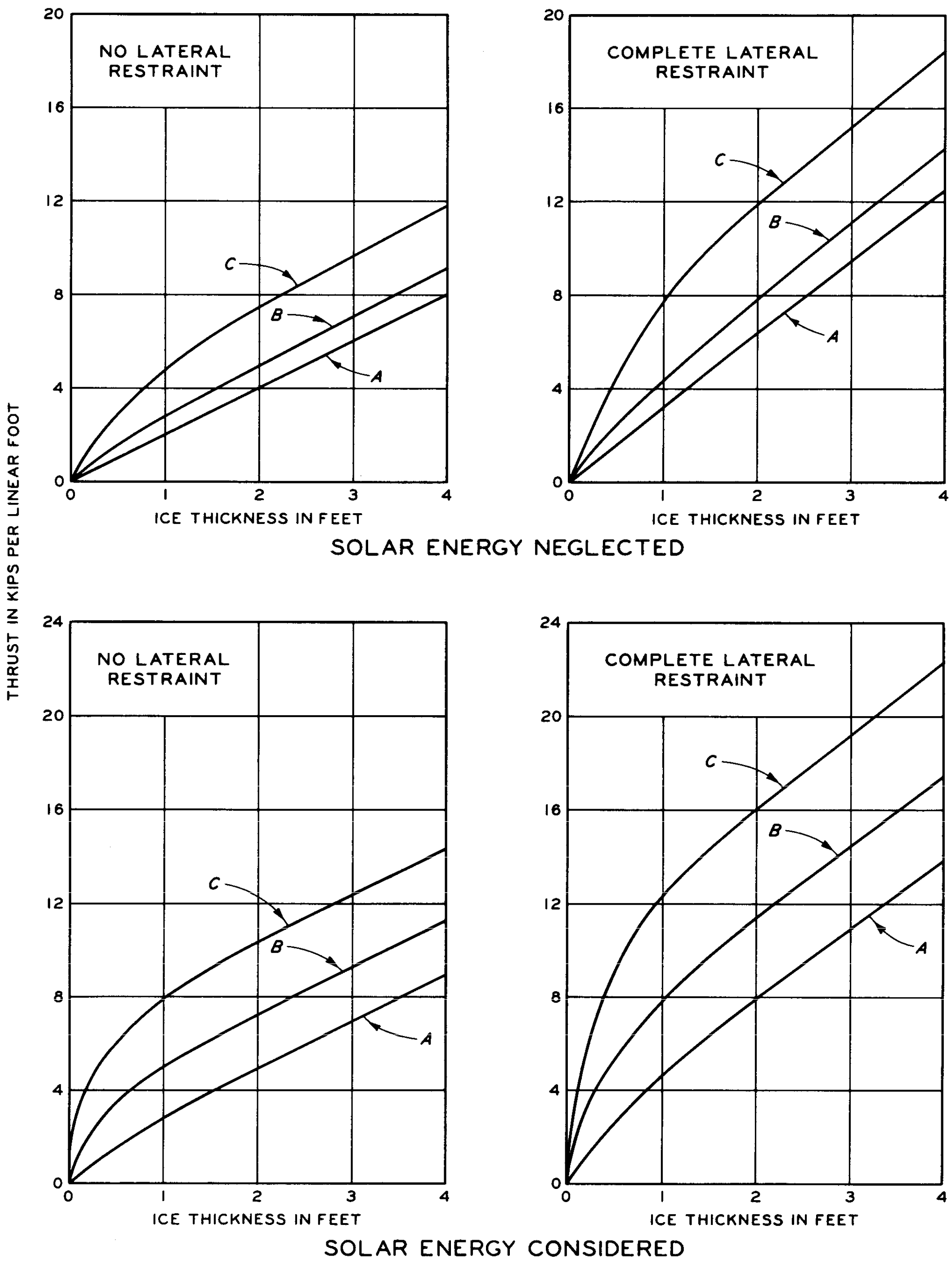
3. The ice thrust curves in HDC 704 are for ice thicknesses up to 4 ft and hourly air temperature rises of 5, 10, and 15°F. Separate curves are presented to show the effects of lateral restraint and solar radiation. The expected ice thicknesses, air temperature rise, and possible snow blanket thickness are dependent upon geographical location and elevation above sea level. In the region of chinook winds rapid air temperature rises can occur. The U. S. Weather Bureau has recorded a 49°F rise in two minutes at Spearfish, S. Dak. When the ice sheet is confined by steep banks close to the structure, spillway piers, or other vertical restrictions, the criteria for complete lateral restraint should be used. The direct effects of solar energy on the thrust are eliminated when the ice sheet is insulated by a blanket of snow only a few inches thick.

4. References.

(1) American Society of Civil Engineers, "Ice pressure against dams: A symposium." Transactions, American Society of Civil Engineers, vol 119 (1954), pp 1-42.

(2) Rose, E., "Thrust exerted by expanding ice sheet." Transactions, American Society of Civil Engineers, vol 112 (1947), pp 871-900.

(3) U. S. Army, Office, Chief of Engineers, Engineering and Design, Gravity Dam Design. EM 1110-2-2200, 25 September 1958.



LEGEND

A - AIR TEMPERATURE RISE OF 5°F PER HOUR
B - AIR TEMPERATURE RISE OF 10°F PER HOUR
C - AIR TEMPERATURE RISE OF 15°F PER HOUR

NOTE: CURVES BY ROSE, TRANSACTIONS, ASCE, VOL 112, 1947.

ICE THRUSTS ON HYDRAULIC STRUCTURES

HYDRAULIC DESIGN CHART 704

HYDRAULIC DESIGN CRITERIA

SHEET 711

LOW-MONOLITH DIVERSION

DISCHARGE COEFFICIENTS

1. Purpose. Several monoliths of the spillway section of a concrete gravity dam are occasionally left at a low elevation during spillway construction for diversion of floodflows. Information on the discharge characteristics of these monoliths is necessary for determining the number of monoliths required to allow floodflows to pass safely. HDC 711 should serve as a guide for selection of discharge coefficients for this purpose.

2. Free Overflow. The flow over low concrete monoliths is generally treated as flow over a broad-crested weir. The equation for free discharge is:

$$Q = C_f\ (L - 2\ KH)\ H^{3/2}$$

where C_f is an empirical coefficient, L is the length of opening transverse to the flow, H is the head on the weir, and K is an end contraction coefficient. The value of K is conventionally taken to be 0.10 for square-end contractions. The free-flow coefficient C_f varies with the ratio of head H to width B of the broad-crested weir in the direction of flow. HDC 711a shows the variation of C_f with H/B resulting from investigations summarized by Tracy.[1] Kindsvater[2] has recently shown the effect of boundary layer development on broad-crested-weir discharge. The rate of development is a function of the bottom roughness. However, present knowledge of this effect does not justify considering boundary layer development for diversion flow computations. The curve resulting from the classical experiments of Bazin[3] as shown by the solid curve in HDC 711a is recommended for general design purposes.

3. Submergence Effect. Discharge coefficients for broad-crested weirs are not usually affected until the depth of submergence is about 0.67 or more of the head on the weir. The phenomenon is commonly expressed in terms of the ratio of the coefficient of the submerged weir to that of the unsubmerged weir C_s/C_f as a function of the ratio of the tailwater depth on the weir to the head on the weir H_2/H_1. Available data indicate that sharp-crested-weir coefficients are more sensitive to submergence than broad-crested-weir coefficients.

4. Available data on the effects of submergence on discharge coefficients for both sharp- and broad-crested weirs[2,4,5,6] are summarized in HDC 711b. As far as is known, rectangular broad-crested weirs have not been subjected to submergence tests. A suggested design curve for submerged low monoliths is given in the chart.

5. Application. The suggested design curves given in HDC 711 should serve as guides for estimating diversion flows over low monoliths. In cases where the head-discharge relation may be critical, a more exact relation should be obtained by hydraulic model investigation. A model study of proposed low-monolith diversion schemes for Allatoona Dam[7] was made because of critical diversion requirements.

6. References.

(1) Tracy, H. J., Discharge Characteristics of Broad-Crested Weirs. U. S. Geological Survey Circular 397, 1957.

(2) Kindsvater, C. E., Discharge Characteristics of Embankment-Shaped Weir; Studies of Flow of Water Over Weirs and Dams. U. S. Geological Survey Water-Supply Paper 1617-A, 1964.

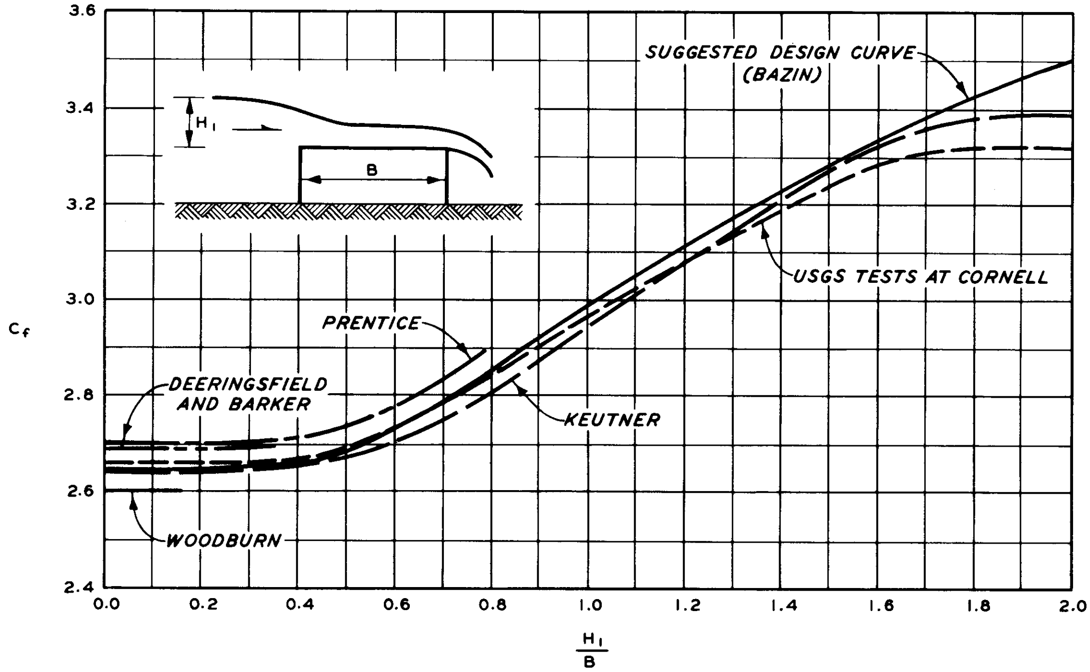
(3) Bazin, M. H., "Experiences nouvelles sur l'ecoulement en diversoir." Annales des Ponts et Chaussées, vol 7, Series 7, 1896.

(4) U. S. Geological Survey, Weir Experiments, Coefficients, and Formulas, by R. E. Horton. Water-Supply Paper No. 200, 1907, p 146.

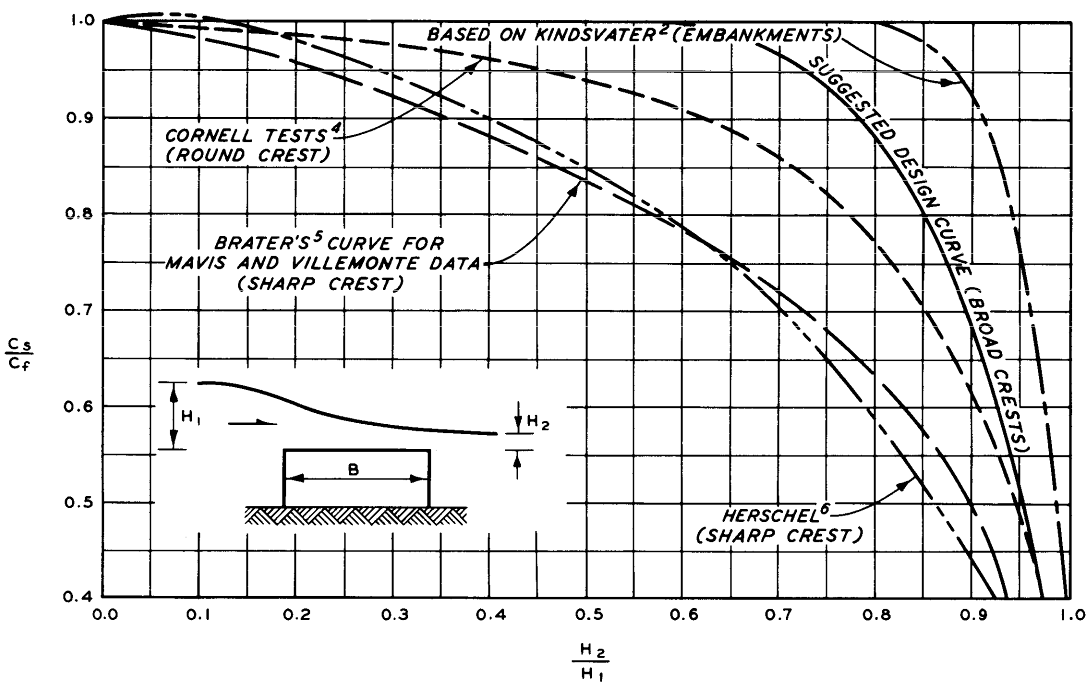
(5) King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, revised by E. F. Brater, 4th ed. McGraw-Hill Book Co., Inc., New York, N. Y., 1954, pp 4-18.

(6) King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, 3d ed. McGraw-Hill Book Co., Inc., New York, N. Y., 1939, p 99.

(7) U. S. Army Engineer Waterways Experiment Station, CE, Sluices and Diversion Scheme for Allatoona Dam, Etowah River, Georgia; Model Investigation. Technical Memorandum No. 214-2, Vicksburg, Miss., November 1948.



a. FREE FLOW



b. SUBMERGED FLOW

NOTE: C_f = FREE-FLOW COEFFICIENT
C_s = SUBMERGED-FLOW COEFFICIENT
NEGLIGIBLE VELOCITY OF APPROACH
RAISED NUMBERS ON SUBMERGED FLOW CHART ARE REFERENCE NUMBERS FROM TEXT.

LOW-MONOLITH DIVERSION
DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 711

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 1-66

HYDRAULIC DESIGN CRITERIA

SHEET 712-1

STONE STABILITY

VELOCITY VS STONE DIAMETER

1. Purpose. Hydraulic Design Chart 712-1 can be used as a guide for the selection of rock sizes for riprap for channel bottom and side slopes downstream from stilling basins and for rock sizes for river closures. Recommended stone gradation for stilling basin riprap is given in paragraph 6.

2. Background. In 1885 Wilfred Airy[1] showed that the capacity of a stream to move material along its bed by sliding is a function of the sixth power of the velocity of the water.[1] Henry Law applied this concept to the overturning of a cube,[2] and in 1896 Hooker[2] illustrated its application to spheres. In 1932 and 1936 Isbash published coefficients for the stability of rounded stones dropped in flowing water.[3,4] The design curves given in Chart 712-1 have been computed using Airy's law and the experimental coefficients for rounded stones published by Isbash.

3. Theory. According to Isbash the basic equation for the movement of stone in flowing water can be written as:

$$V = C \left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2} (D)^{1/2} \qquad (1)$$

where

V = velocity, fps

C = a coefficient

g = acceleration of gravity, ft/sec^2

γ_s = specific weight of stone, lb/ft^3

γ_w = specific weight of water, lb/ft^3

D = stone diameter, ft

The diameter of a spherical stone in terms of its weight W is

$$D = \left(\frac{6W}{\pi\gamma_s}\right)^{1/3} \qquad (2)$$

Substituting for D in equation 1 results in

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}\left(\frac{6W}{\pi\gamma_s}\right)^{1/6} \quad (3)$$

which describes Airy's law stated in paragraph 2.

4. Experimental Results. Experimental data on stone movement in flowing water from the early (1786) work of DuBuat[5] to the more recent Bonneville Hydraulic Laboratory tests[6] have been shown to confirm Airy's law and Isbash's stability coefficients.[7] The published experimental data are generally defined in terms of bottom velocities. However, some are in terms of average flow velocities and some are not specified. The Isbash coefficients are from tests with essentially no boundary layer development and the average flow velocities are representative of the velocity against stone. When the stone movement resulted by sliding, a coefficient of 0.86 was obtained. When movement was effected by rolling or overturning, a coefficient of 1.20 resulted. Extensive U. S. Army Engineer Waterways Experiment Station laboratory testing for the design of riprap below stilling basins indicates that the coefficient of 0.86 should be used with the average flow velocity over the end sill for sizing stilling basin riprap because of the excessively high turbulence level in the flow. For impact-type stilling basins, the Bureau of Reclamation[8] has adopted a riprap design curve based on field and laboratory experience and on a study by Mavis and Laushey.[9] The Bureau curve specifies rock weighing 165 lb/ft^3 and is very close to the Isbash curve for similar rock using a stability coefficient of 0.86.

5. Application. The curves given in Chart 712-1 are applicable to specific stone weights of 135 to 205 lb/ft^3. The use of the average flow velocity is desirable for conservative design. The solid-line curves are recommended for stilling basin riprap design and other high-level turbulence conditions. The dashed line curves are recommended for river closures and similar low-level turbulence conditions. Riprap bank and bed protection in natural and artificial flood-control channels should be designed in accordance with reference 10.

6. Stilling Basin Riprap.

a. Size. The W_{50} stone weight and the D_{50} stone diameter for establishing riprap size for stilling basins can be obtained using Chart 712-1 in the manner indicated by the heavy arrows thereon. The effect of specific weight of the rock on the required size is indicated by the vertical spread of the solid line curves.

b. Gradation. The following size criteria should serve as guidelines for stilling basin riprap gradation.

(1) The lower limit of W_{50} stone should not be less than the weight of stone determined using the appropriate "Stilling Basins" curve in Chart 712-1.

(2) The upper limit of W_{50} stone should not exceed the weight that can be obtained economically from the quarry or the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(3) The lower limit of W_{100} stone should not be less than two times the lower limit of W_{50} stone.

(4) The upper limit of W_{100} stone should not be more than five times the lower limit of W_{50} stone, nor exceed the size that can be obtained economically from the quarry, nor exceed the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(5) The lower limit of W_{15} stone should not be less than one-sixteenth the upper limit of W_{100} stone.

(6) The upper limit of W_{15} stone should be less than the upper limit of W_{50} stone as required to satisfy criteria for graded stone filters specified in EM 1110-2-1901.

(7) The bulk volume of stone lighter than the W_{15} stone should not exceed the volume of voids in the revetment without this lighter stone.

(8) W_0 to W_{25} stone may be used instead of W_{15} stone in criteria (5), (6), and (7) if desirable to better utilize available stone sizes.

c. Thickness. The thickness of the riprap protection should be $2D_{50\ max}$ or $1.5D_{100\ max}$, whichever results in the greater thickness.

d. Extent. Riprap protection should extend downstream to where nonerosive channel velocities are established and should be placed sufficiently high on the adjacent bank to provide protection from wave wash during maximum discharge. The required riprap thickness is determined by substituting values for these relations in equation 2.

7. References.

(1) Shelford, W., "On rivers flowing into tideless seas, illustrated by the river Tiber." Proceedings, Institute of Civil Engineers, vol 82 (1885).

(2) Hooker, E. H., "The suspension of solids in flowing water." Transactions, American Society of Civil Engineers, vol 36 (1896), pp 239-340.

(3) Isbash, S. V., Construction of Dams by Dumping Stones in Flowing

Water, Leningrad, 1932. Translated by A. Dorijikov, U. S. Army Engineer District, Eastport, CE, Maine, 1935.

(4) __________, "Construction of dams by depositing rock in running water." Transactions, Second Congress on Large Dams, vol 5 (1936), pp 123-136.

(5) DuBuat, P. L. G., Traite d'Hydraulique. Paris, France, 1786.

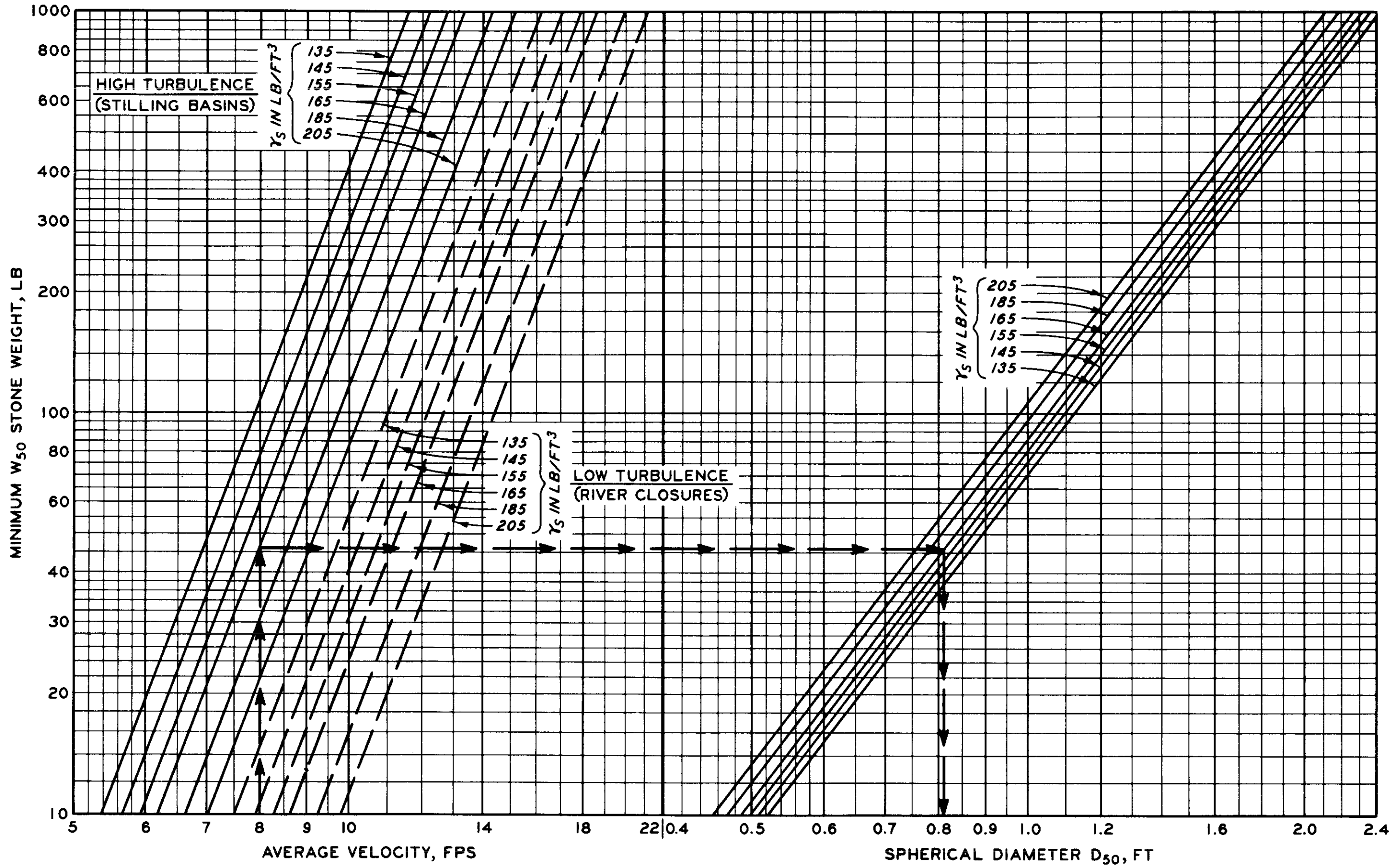
(6) U. S. Army Engineer District, Portland, CE, McNary Dam - Second Step Cofferdam Closure. Bonneville Hydraulic Laboratory Report No. 51-1, 1956.

(7) U. S. Army Engineer Waterways Experiment Station, CE, Velocity Forces on Submerged Rocks. Miscellaneous Paper No. 2-265, Vicksburg, Miss., April 1958.

(8) U. S. Bureau of Reclamation, Stilling Basin Performance; An Aid in Determining Riprap Sizes, by A. J. Peterka. Hydraulic Laboratory Report No. HYD-409, Denver, Colo., 1956.

(9) Mavis, F. T. and Laushey, L. M., "A reappraisal of the beginning of bed movement - competent velocity." Second Meeting, International Association for Hydraulic Structure Research, Stockholm, Sweden, 1948. See also Civil Engineering, vol 19 (January 1949), pp 38, 39, and 72.

(10) U. S. Army, Office, Chief of Engineers, Engineering and Design; Hydraulic Design of Flood Control Channels. EM 1110-2-1601, Washington, D. C., 1 July 1970.



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_S-\gamma_W}{\gamma_W}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50}=\left(\frac{6W_{50}}{\pi\gamma_S}\right)^{1/3}$$

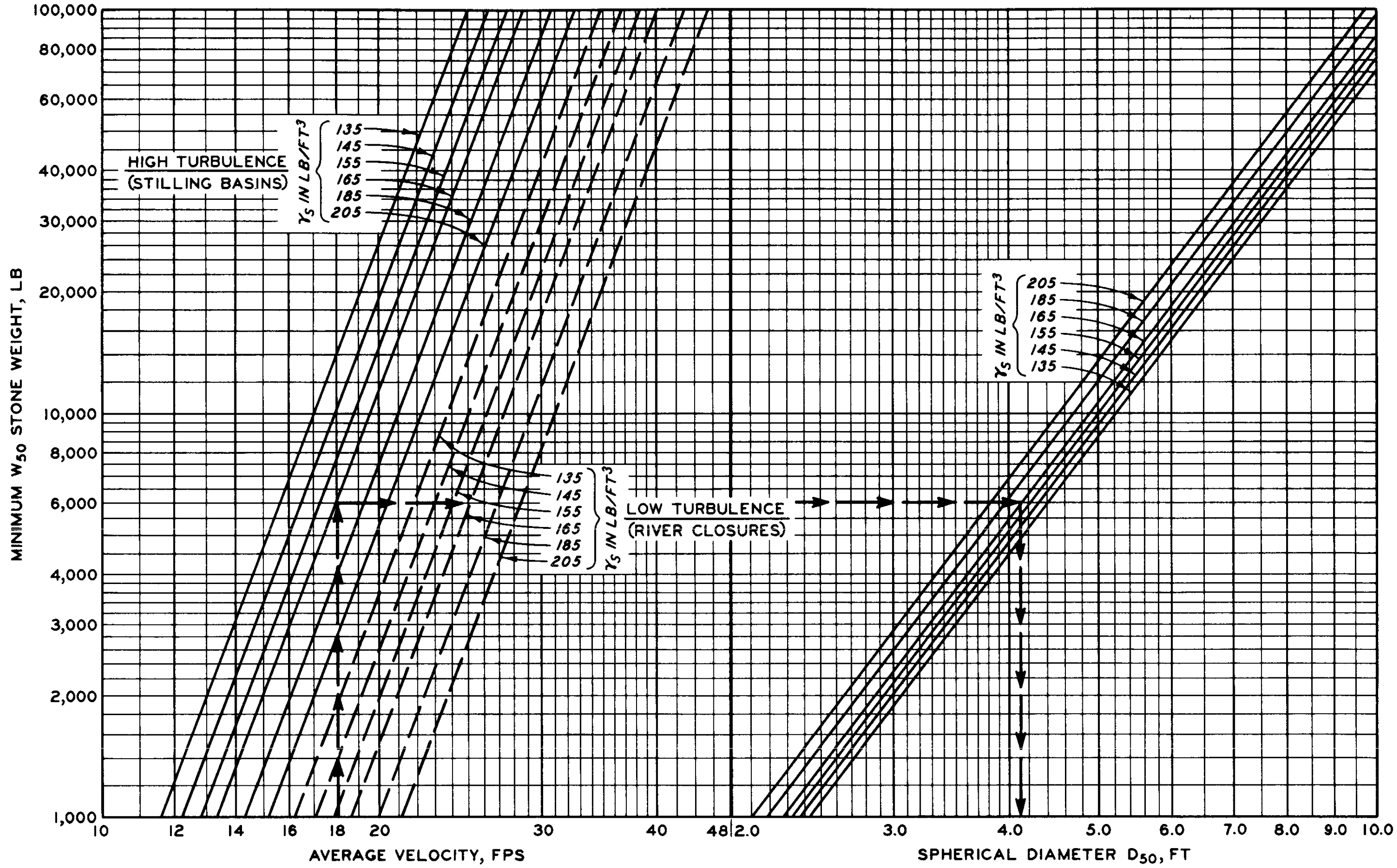
WHERE:
- V = VELOCITY, FPS
- γ_S = SPECIFIC STONE WEIGHT, LB/FT^3
- γ_W = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_s}\right)^{1/3}$$

WHERE:
- V = VELOCITY, FPS
- γ_s = SPECIFIC STONE WEIGHT, LB/FT^3
- γ_w = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 2 OF 2)

REV 8-58, 9-70

WES 6-57

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

HYDRAULIC DESIGN CRITERIA

SHEETS 722-1 TO 722-3

STORM DRAIN OUTLETS

FIXED ENERGY DISSIPATORS

1. Purpose. Storm drains frequently terminate in unstable channels and gullies. Under these conditions dissipation of the energy of the outflow is required to prevent serious erosion and potential undermining and subsequent failure of the storm drains. Adequate energy dissipation can be accomplished by extensive riprap protection[1,2] or by construction of specially designed fixed energy dissipators.[3,4,5,6]

2. Hydraulic Design Charts (HDC's) 722-1 to -3 present design criteria for three types of laboratory tested energy dissipators.[3] Each type has its advantages and limitations. Selection of the optimum type and size is dependent upon local tailwater conditions, maximum expected discharge, and economic considerations.

3. Stilling Wells. The stilling well energy dissipator shown in HDC 722-1 was developed at the U. S. Army Engineer Waterways Experiment Station (WES).[3] Energy dissipation in this stilling well is relatively independent of tailwater and is accomplished by flow expansion in the well, by impact of the fluid on the base and wall of the well, and by the change in momentum resulting from redirection of the flow to vertically upward. WES laboratory tests[3] indicated that the structure performs satisfactorily for flow-pipe diameter ratios ($Q/D_o^{2.5}$) up to 10 with a well-pipe diameter ratio of 5.

4. HDC 722-1 shows the relation between storm drain diameter, well diameter, and discharge. Designing for operation beyond the limits shown in HDC 722-1 is not recommended. Intermediate ratios of stilling well-drain pipe diameters within the limits shown in HDC 722-1 can be computed using the equation given in this chart.

5. Impact Energy Dissipators. The U. S. Bureau of Reclamation (USBR)[5] has developed an impact energy dissipator which is an effective stilling device even with deficient tailwater. The dimensions of this energy dissipator in terms of its width are shown in HDC 722-2. Energy dissipation in the basin is accomplished by the impact of the entering jet on the vertically hanging baffle and by the eddies that are formed following impact on the baffle.

6. HDC 722-2 shows the relation between storm drain diameters, basin width, and discharge. WES laboratory tests[3] showed that this structure properly designed performs satisfactorily for $Q/D_o^{2.5}$ ratios up to 21. Intermediate ratios of basin widths within the limits shown in HDC 722-2 can be computed using the equation given in this chart. Design for operation beyond these limits is not recommended. The WES

tests also showed that optimum energy dissipation for the design flow occurs with the tailwater midway up the hanging baffle. Excessive tailwater should be avoided as this causes flow over the top of the baffle.

7. Hydraulic Jump Energy Dissipators. The St. Anthony Falls Hydraulic Laboratory (SAFHL)[6] has developed the hydraulic jump energy dissipator shown in HDC 722-3. Design equations for dimensionalizing the structure in terms of the square of the Froude number of the flow entering the dissipator are also given in the chart. WES laboratory tests[3] showed that this type of stilling basin performs satisfactorily for ratios of $Q/D_o^{2.5}$ up to 9.5 with a basin width three times the storm drain diameter. WES tests were limited to basin widths of 1, 2, and 3 times the drain diameter with drops (drain invert to stilling basin) of 0.5 and 2 times the drain diameter. Parallel stilling basin walls were used for basin width-drain diameter ratios of 1 and 2. The transition wall flare was continued through the basin for $W = 3D_o$. Parallel basin sidewalls are generally recommended for best performance. Transition sidewall flare (1:D') during the WES tests was fixed at 1 on 8. The invert transition to the stilling basin should conform to the geometry of the trajectory of a flow not less than 1.25 times the drain outlet portal design velocity.

8. HDC 722-3 shows the relation between storm drain diameter and discharge for stilling basin widths up to 3 times the drain diameter which results in satisfactory performance. WES tests have been restricted to the limits shown in HDC 722-3, and the equation given in the chart can be used to compute intermediate basin width-drain diameter ratios within those limits. General WES model tests of outlet works indicate that this equation also applies to ratios greater than the maximum shown in the chart. However, outlet portal velocities exceeding 60 fps are not recommended for designs containing chute blocks. This chart does not reflect the outlet invert transition effects on basin performance. The design of the basin itself (HDC 722-3) is dependent upon the depth and velocity of the flow as it enters the basin. The values should be computed taking into account the drain outlet transition geometry.

9. Riprap Protection. Riprap protection in the immediate vicinity of the energy dissipator is recommended. Preliminary, unpublished WES test results[3] on riprap protection below energy dissipators indicates the following average diameter (D_{50}) stone size should result in adequate erosion protection.

$$D_{50} = D\left(\frac{V}{\sqrt{gD}}\right)^3$$

where

D_{50} = the minimum average size of stone, ft, whereby 50 percent by weight of the graded mixture is larger than D_{50} size

D = depth of flow in outlet channel, ft
V = average velocity in outlet channel, ft
g = gravitational acceleration, ft/sec^2

10. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets; Hydraulic Laboratory Model Investigation, by J. P. Bohan. Research Report H-70-2, Vicksburg, Miss., January 1970.

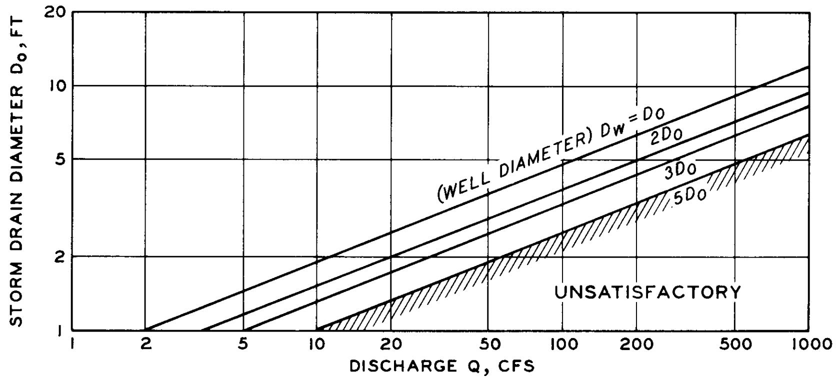
(2) ________, Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets, by B. P. Fletcher and J. L. Grace, Jr. Miscellaneous Paper H-72-5, Vicksburg, Miss., May 1972.

(3) ________, Evaluation of Three Energy Dissipators for Storm-Drain Outlets; Hydraulic Laboratory Investigation, by J. L. Grace, Jr., and G. A. Pickering. Research Report H-71-1, Vicksburg, Miss., April 1971.

(4) ________, Impact-Type Energy Dissipator for Storm-Drainage Outfalls Stilling Well Design; Hydraulic Model Investigation, by J. L. Grace, Jr. Technical Report No. 2-620, Vicksburg, Miss., March 1963.

(5) Beichley, G. L., Progress Report No. XIII - Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances - Section 14, Modification of Section 6 (Stilling Basin for Pipe or Open Channel Outlets - Basin VI. Report No HYD-572, Hydraulics Branch, Division of Research, U. S. Bureau of Reclamation, Denver, Colo., June 1969.

(6) Blaisdell, F. W., The SAF Stilling Basin. Agricultural Handbook No. 156, Agricultural Research Service and St. Anthony Falls Laboratory, University of Minnesota, Minneapolis, Minn., April 1959.



BASIC EQUATION

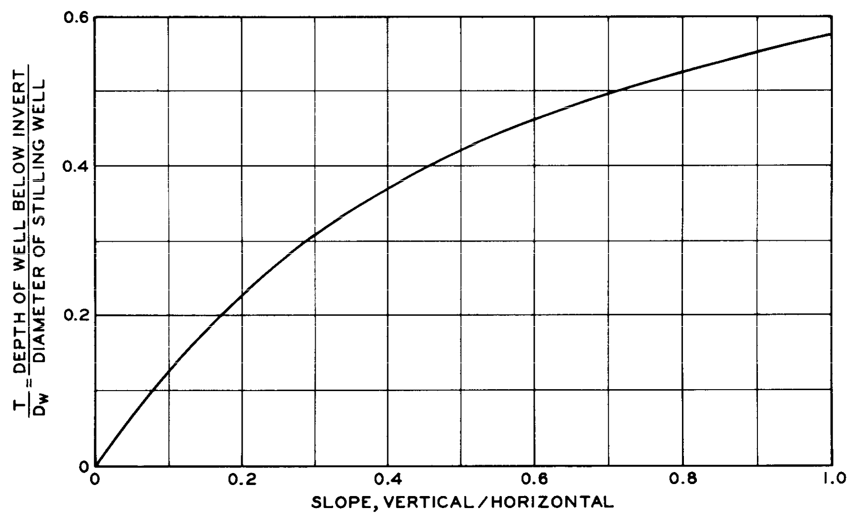
$$\frac{D_W}{D_O} = 0.53\left(\frac{Q}{D_O^{2.5}}\right) \text{ FOR } \frac{Q}{D_O^{2.5}} \leq 10$$

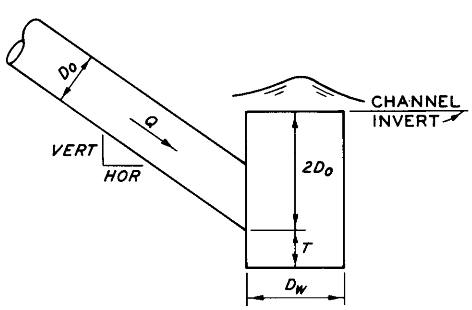
WHERE:

D_W = STILLING WELL DIAMETER, FT

D_O = DRAIN DIAMETER, FT

Q = DESIGN DISCHARGE, CFS



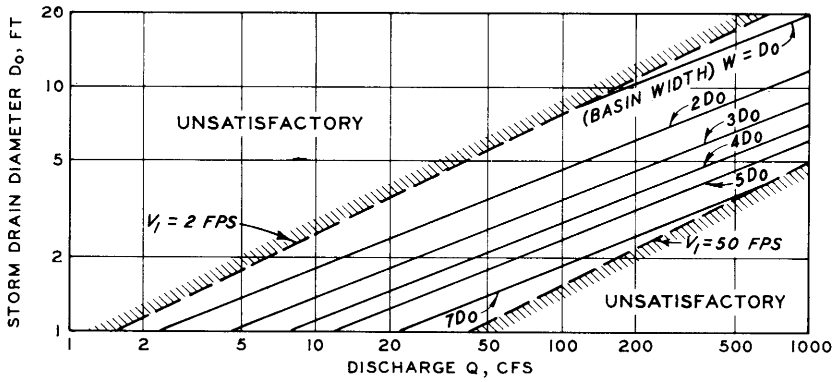


ELEVATION

STORM DRAIN OUTLETS
ENERGY DISSAPATORS
STILLING WELL

HYDRAULIC DESIGN CHART 722-1

WES 7-73

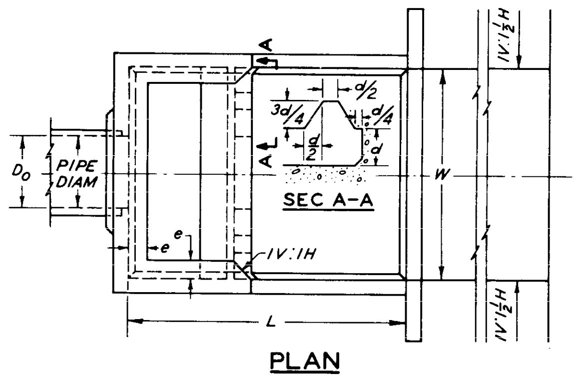


BASIC EQUATION

$$\frac{W}{D_0} = 1.3\left(\frac{Q}{D_0^{2.5}}\right) \quad \text{FOR} \quad \frac{Q}{D_0^{2.5}} \leq 21$$

WHERE:

W = BASIN WIDTH, FT
D_0 = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS
V_1 = PIPE VELOCITY, FPS



STILLING BASIN DESIGN

$H=\frac{3}{4}(W)$ $c=\frac{1}{2}(W)$

$L=\frac{4}{3}(W)$ $d=\frac{1}{6}(W)$

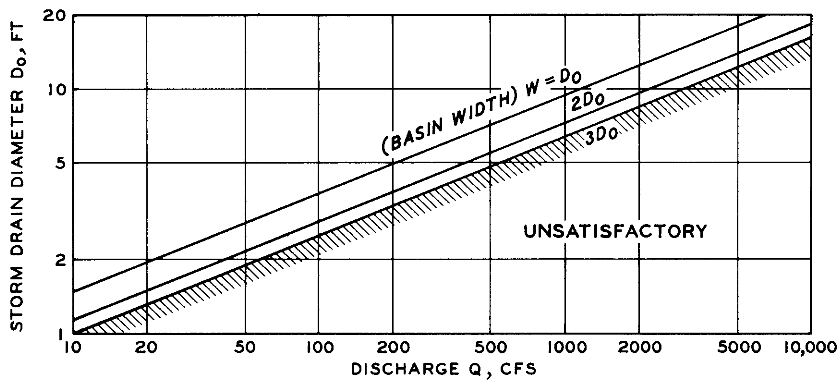
$a=\frac{1}{2}(W)$ $e=\frac{1}{12}(W)$

$b=\frac{3}{8}(W)$ $t=\frac{1}{12}(W)$, SUGGESTED MINIMUM

STORM DRAIN OUTLETS
ENERGY DISSAPATORS
IMPACT BASIN

HYDRAULIC DESIGN CHART 722-2

WES 7-73



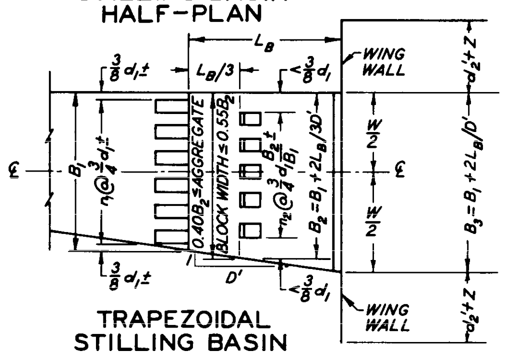
BASIC EQUATION

$$\frac{W}{D_o} = 0.3\left(\frac{Q}{D_o^{2.5}}\right) \text{ FOR } \frac{Q}{D_o^{2.5}} \leq 9.5$$

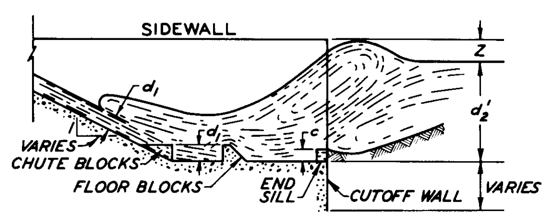
WHERE:

- W = END SILL LENGTH, FT
- D_o = DRAIN DIAMETER, FT
- Q = DESIGN DISCHARGE, CFS

RECTANGULAR
STILLING BASIN
HALF-PLAN



TRAPEZOIDAL
STILLING BASIN
HALF-PLAN



CENTER-LINE SECTION

DESIGN EQUATIONS

$$F = \frac{V_1^2}{g d_1} \quad (1)$$

$$d_2 = \frac{d_1}{2}\left(-1 + \sqrt{8F+1}\right) \quad (2)$$

F = 3 TO 30 $\quad d_2' = (1.10 - F/120)\, d_2 \quad$ (3a)

F = 30 TO 120 $\quad d_2' = 0.85 d_2 \quad$ (3b)

F = 120 TO 300 $\quad d_2' = (1.00 - F/800)\, d_2 \quad$ (3c)

$$L_B = \frac{4.5 d_2}{F^{0.38}} \quad (4)$$

$$Z = \frac{d_2}{3} \quad (5)$$

$$c = 0.07 d_2 \quad (6)$$

STORM DRAIN OUTLETS
ENERGY DISSAPATORS
STILLING BASIN

HYDRAULIC DESIGN CHART 722-3

WES 7-73

HYDRAULIC DESIGN CRITERIA

SHEET 722-4 TO 722-7

STORM DRAIN OUTLETS

RIPRAP ENERGY DISSIPATORS

1. Purpose. Criteria for the hydraulic design of fixed energy dissipating structures for storm drain outlets are presented in Hydraulic Design Charts (HDC's) 722-1 to 722-3. Under some conditions adequate energy dissipation can be accomplished more economically using riprap as an alternate to fixed structures. HDC's 722-4 to 722-5 present three basic riprap energy dissipator designs developed at WES.[1,2]

2. Scour Holes. Scour holes at storm drain exit portals effectively dissipate flow energy and reduce downstream erosion. However, uncontrolled scour holes can undermine the storm drain with subsequent structural failure. Basic laboratory tests were conducted at WES[1] during the period 1963-1969 to investigate scour hole development and erosion protection in cohesionless material downstream from storm drain exit portals. These tests showed that the length, width, depth, and volume of the scour hole could be related in terms of the storm drain diameter D_o in feet, the discharge Q in cfs, and the flow duration t in minutes. The tailwater depth TW in feet over the storm drain invert was also found to be important. The following set of design equations[2] describes the basic scour hole dimensions for two controlling tailwater conditions.

$$\frac{L_{sm}}{D_o} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{0.71}\left(t^{0.125}\right)\right] \tag{1}$$

$$\frac{D_{sm}}{D_o} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{0.375}\left(t^{0.10}\right)\right] \tag{2}$$

$$\frac{W_{sm}}{D_o} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{0.915}\left(t^{0.15}\right)\right] \tag{3}$$

$$\frac{V_s}{D_o^3} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{2}\left(t^{0.375}\right)\right] \tag{4}$$

where

L_{sm} = scour hole length, ft
D_{sm} = depth of maximum scour, ft
W_{sm} = half the width of the hole at the location of maximum scour, ft
V_s = volume of material removed from scour hole, ft^3

Empirically determined values of C in the equations above for the two controlling tailwater conditions are:

$\frac{TW}{D_o}$	Equation No. 1	2	3	4
>0.5	4.10	0.74	0.72	0.62
≤ 0.5	2.40	0.80	1.00	0.73

3. HDC 722-4 shows dimensionless scour hole profiles and cross sections for the two limiting tailwater conditions.

4. Horizontal Riprap Blanket. HDC 722-5 shows the recommended length L_{sp} and geometry of the horizontal riprap blanket protection required for satisfactory dissipation of the energy of the design outflow from a storm drain. (The required D_{50} riprap size can be estimated using HDC 722-7.)

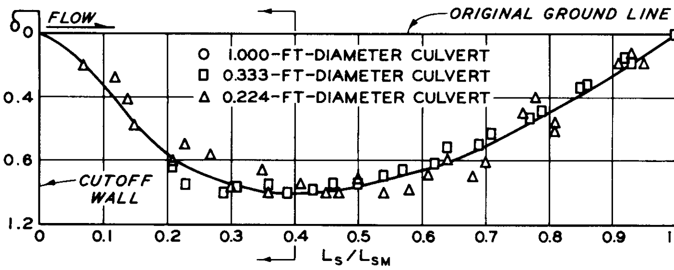
5. Preformed Scour Holes. Laboratory studies have shown that satisfactory energy dissipation of storm drain outflow occurs in riprap-lined, preformed scour holes of nominal size. HDC 722-6 shows the recommended design for preformed scour holes 0.5 and $1.0D_o$ deep. The D_{50} minimum stone size required for each scour hole depth can be estimated using HDC 722-7.

5. Application. Study of the basic test data indicates that the resulting design criteria are generally applicable to both circular and rectangular conduits flowing full or partly full. For rectangular conduits the conduit width is used in place of the diameter D_o of the circular conduits.

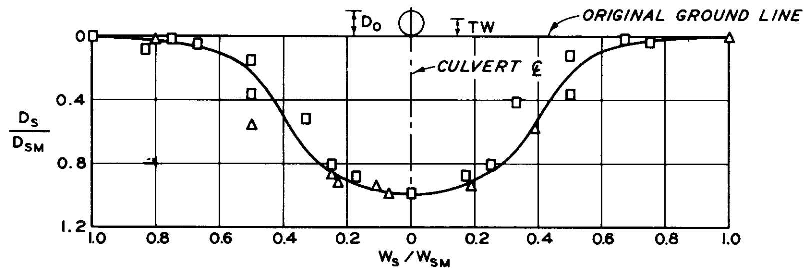
6. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets; Hydraulic Model Investigation, by J. P. Bohan. Research Report H-70-2, Vicksburg, Miss., January 1970.

(2) ________, Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets, by B. P. Fletcher and J. L. Grace, Jr., Miscellaneous Paper H-72-5, Vicksburg, Miss., May 1972.

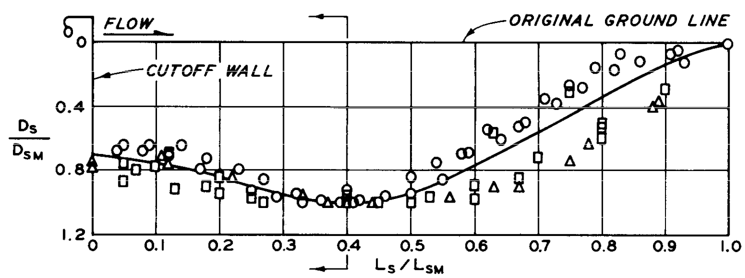


DIMENSIONLESS CENTER-LINE PROFILE

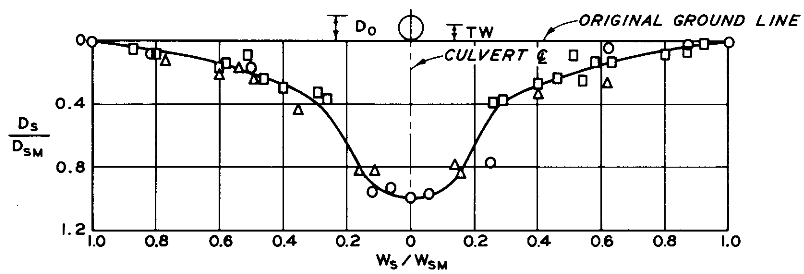


DIMENSIONLESS CROSS SECTION AT $0.4L_{SM}$

$TW > 0.5D_O$



DIMENSIONLESS CENTER-LINE PROFILE



DIMENSIONLESS CROSS SECTION AT $0.4L_{SM}$

$TW \leq 0.5D_O$

NOTE: L_S = DISTANCE FROM OUTLET TO D_S, FT
L_{SM} = DISTANCE FROM OUTLET TO END OF SCOUR, FT
W_S = DISTANCE (R & L) FROM ℄ TO D_S AT $0.4L_{SM}$, FT
W_{SM} = DISTANCE (R & L) FROM ℄ TO $0.0D_S$ AT $0.4L_{SM}$, FT
D_O = DIAMETER OR WIDTH OF STORM DRAIN, FT
TW = TAILWATER DEPTH ABOVE DRAIN INVERT, FT
D_S = DEPTH OF SCOUR, FT
D_{SM} = MAXIMUM SCOUR DEPTH, FT

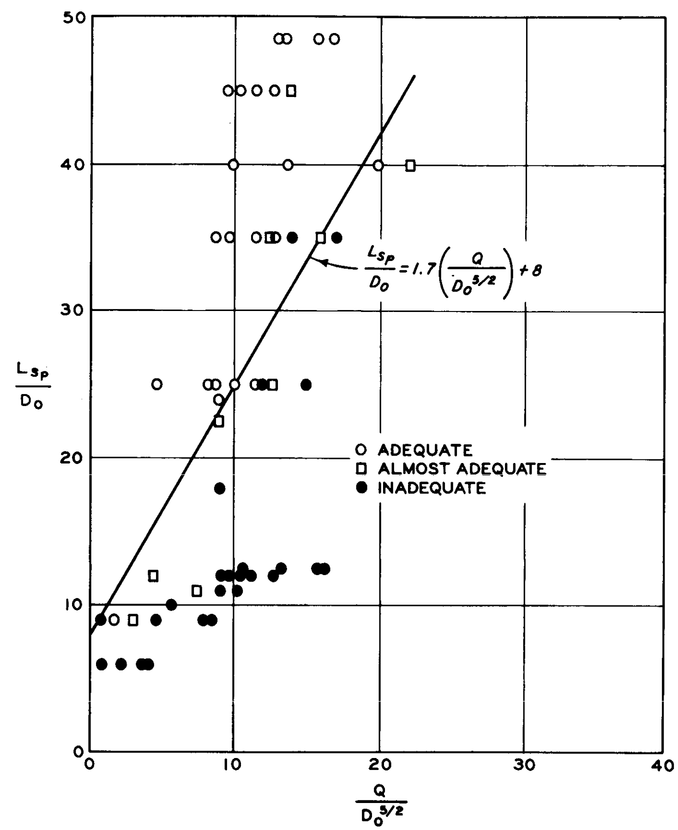
STORM DRAIN OUTLETS

SCOUR HOLE GEOMETRY

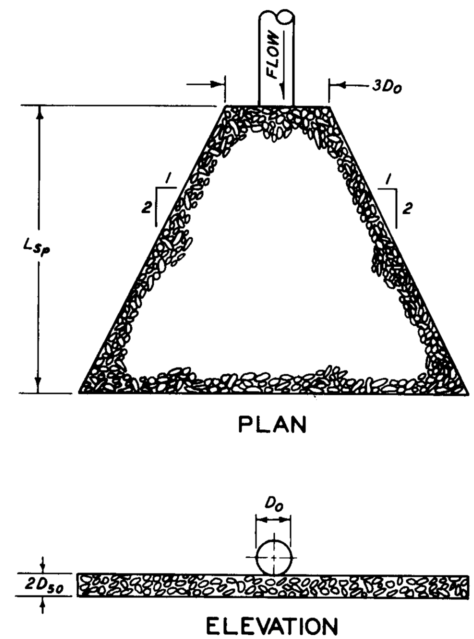
$TW > 0.5D_O$ AND $\leq 0.5D_O$

HYDRAULIC DESIGN CHART 722-4

WES 7-73



NOTE: D_o - DIAMETER OR WIDTH OF STORM DRAIN, FT
Q - STORM DRAIN DISCHARGE, CFS
L_{sp} - HORIZONTAL LENGTH OF BLANKET, FT

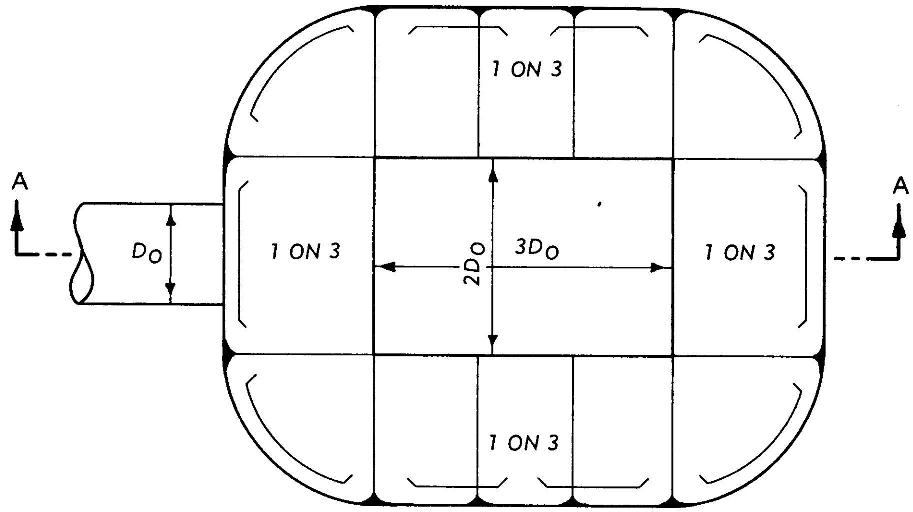


NOTE: D_{50} - MINIMUM AVERAGE SIZE OF STONE, FT

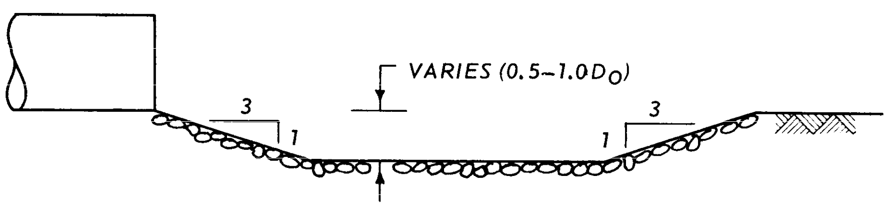
STORM DRAIN OUTLETS
RIPRAP ENERGY DISSAPATORS HORIZONTAL BLANKET LENGTH OF STONE PROTECTION

HYDRAULIC DESIGN CHART 722-5

WES 7-73



PLAN



SECTION A-A

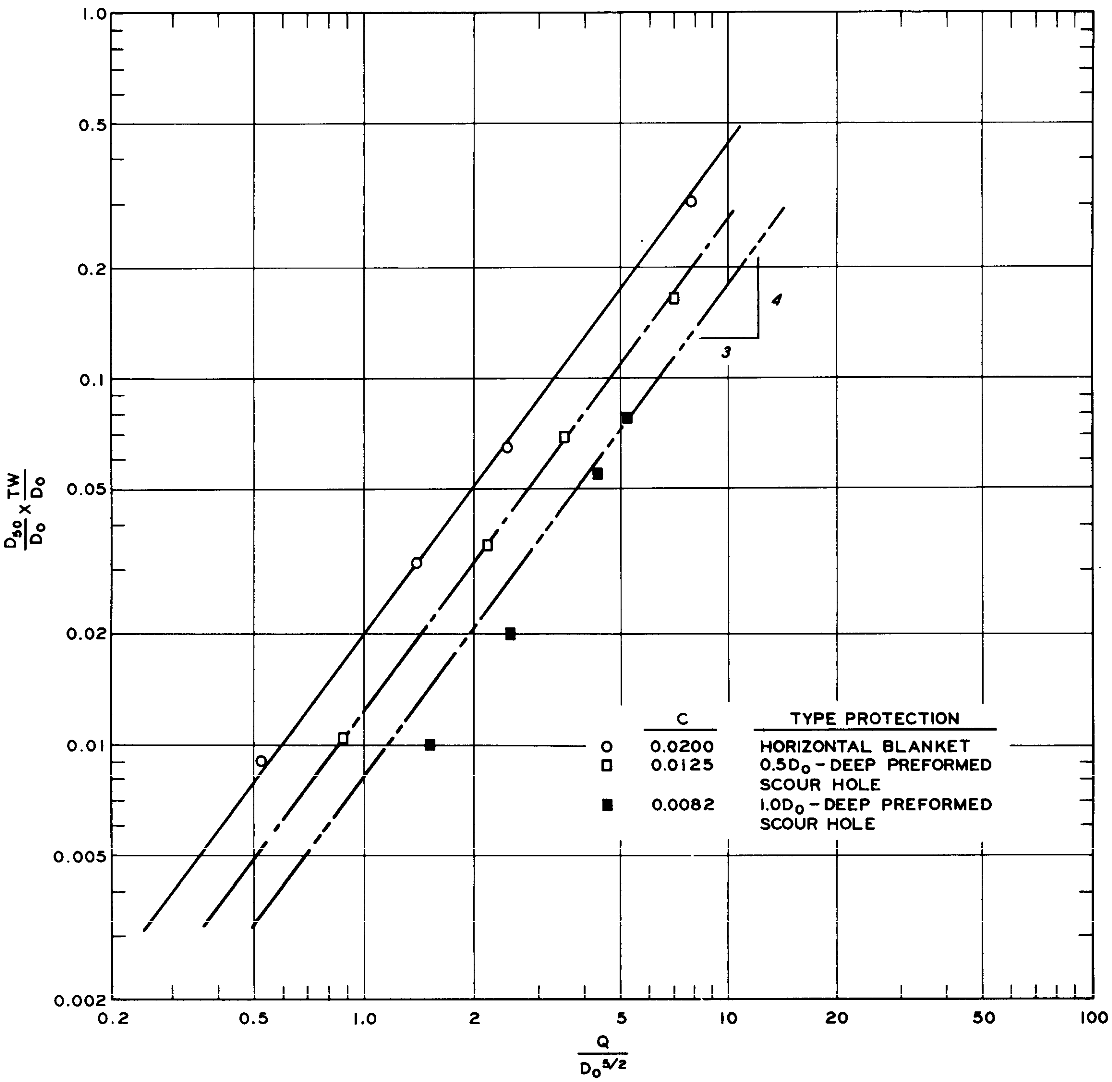
NOTE: D_O = DIAMETER OR WIDTH OF STORM DRAIN, FT

STORM DRAIN OUTLETS

RIPRAP ENERGY DISSAPATORS
PREFORMED SCOUR HOLE GEOMETRY

HYDRAULIC DESIGN CHART 722-6

WES 7-73



BASIC EQUATION

$$\frac{D_{50}}{D_o} = C \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}} \right)^{4/3}$$

WHERE:

D_{50} = MINIMUM AVERAGE SIZE OF STONE, FT

D_o = DIAMETER OR WIDTH OF STORM DRAIN, FT

Q = STORM DRAIN DISCHARGE, CFS

TW = TAILWATER DEPTH ABOVE DRAIN INVERT, FT

STORM DRAIN OUTLETS

RIPRAP ENERGY DISSAPATORS
D_{50} STONE SIZE

HYDRAULIC DESIGN CHART 722-7

WES 7-73

SHEET 733-1

SURGE TANKS

THIN PLATE ORIFICES

HEAD LOSSES

1. Thin plate orifices are often used in surge tank risers to restrict the flow during load-on and load-off operations. Computation of the head losses through these orifices is of interest in the design of surge tanks.

2. A number of experiments have been made on head losses through orifices in straight pipe. When an orifice is placed in a surge tank riser close to the penstock tee, the energy loss of flow entering or leaving the riser is affected by the orifice flow. Indri's[2] extensive study of orifices in branches has made available new data on head loss coefficients considered to be applicable to surge tank problems. The pipe used in this study was 9 cm (3.54 in.) in diameter. The orifice plates were located in the branches 125 mm (4.92 in.) from the center line of the main pipe. The test results indicate that the combined tee and orifice loss coefficients were independent of Reynolds number for $R_e > 3 \times 10^4$.

3. HDC 733-1 presents a head loss coefficient curve for thin plate orifices in tees. The head loss coefficient is based on the combined tee and orifice head loss. Indri's data shown in this chart indicate that a single curve is applicable to load on-load off turbine conditions. Also shown in this chart are head loss coefficient curves by Weisbach[3] and Marchetti[1] for thin plate orifices in straight pipe. These curves indicate that the location of the orifice with respect to other disturbances affects the head loss.

4. The data in HDC 733-1 are based on the equation:

$$H_L = K_o \frac{V^2}{2g}$$

where

H_L = head loss across the orifice or orifice and tee, ft
K_o = head loss coefficient
V = velocity in riser, ft per sec

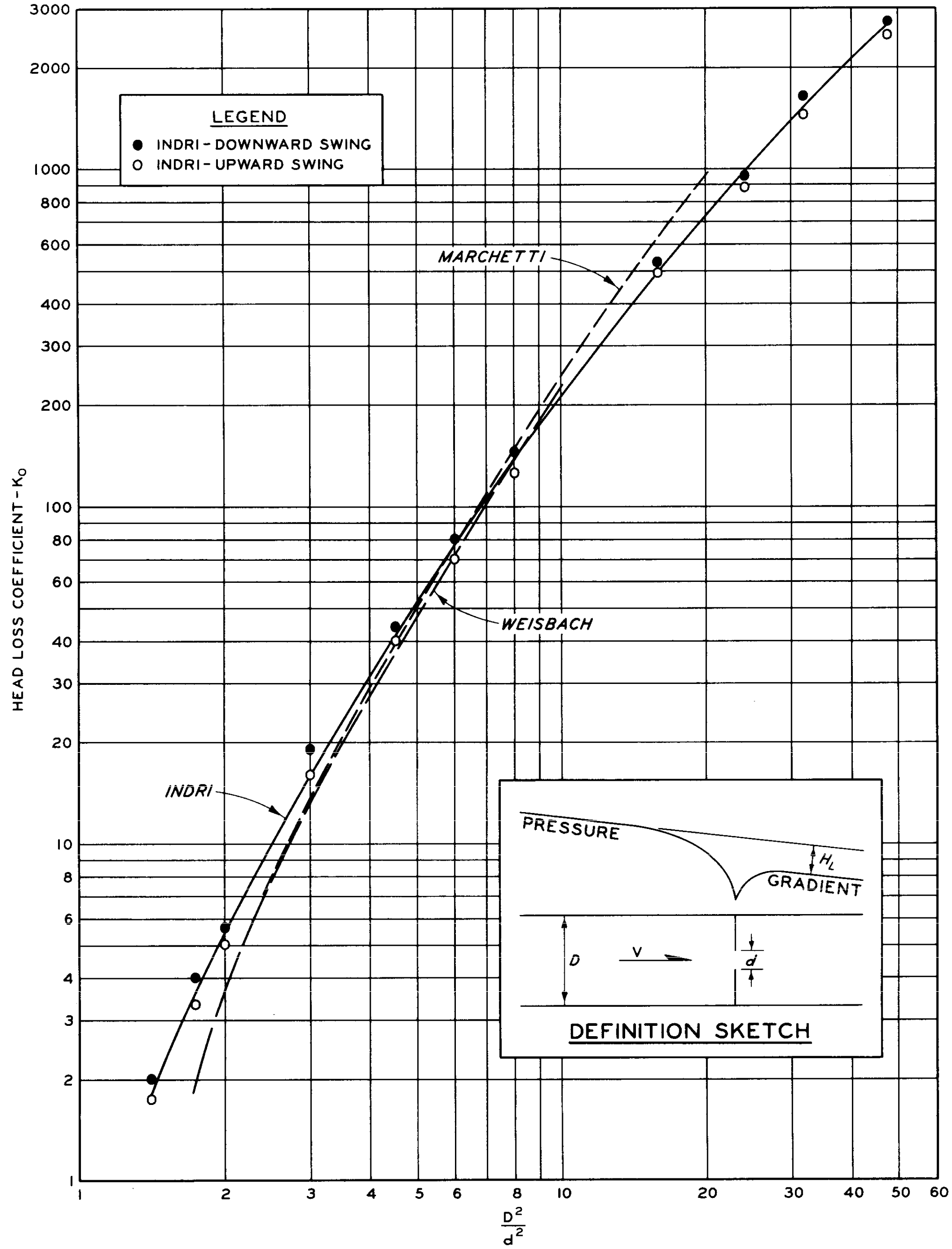
The head loss coefficient is plotted as a function of the ratio of the square of the riser diameter D to the square of the orifice diameter d . A sketch of an orifice in a straight pipe is included in the chart for purposes of defining the terms involved.

5. References.

(1) Caric, D. M., "Tehnicka hydraulika." Gradevenska, Knjiga, Belgrad (1952).

(2) Indri, E., "Richerche sperimentali su modelli di strozzature per pozzi piezometrici (Experimental research on models of constrictions for surge tanks)." L'Energia Elettrica, vol 34, No. 6 (June 1957), pp 554-569. Translation by Jan C. Van Tienhoven, for U. S. Army Engineer Waterways Experiment Station, CE, Translation No. 60-3, Vicksburg, Miss., April 1960.

(3) Weisbach, J., Untersuchungen in den Gebieten der Mechanik und Hydraulik. Leipzig, 1945.



EQUATION

$$H_L = K_O \frac{V^2}{2g}$$

WHERE:

H_L = HEAD LOSS ACROSS ORIFICE, FT

K_O = HEAD LOSS COEFFICIENT

V = VELOCITY IN PIPE, FT PER SEC

SURGE TANKS
THIN PLATE ORIFICES
HEAD LOSSES

HYDRAULIC DESIGN CHART 733-1

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 10-61