HYDRAULIC DESIGN CRITERIA

SHEETS 111-16 TO 111-16/2

HIGH OVERFLOW DAMS

CREST PRESSURES

1. Hydraulic Design Charts 111-16 to 111-16/2 present plots of crest pressures for H/H_d values of 0.50, 1.00, 1.17, 1.33, and 1.50. The charts are based on recent ES 801 test data* for crests with and without piers: Chart 111-16 represents pressures on crests without piers; Chart 111-16/1, pressures midway between piers; and Chart 111-16/2, pressures adjacent to piers. Piezometers used for measuring the last condition were located immediately adjacent to the pier face.

2. The data shown are applicable to high overflow dams with standard crests. Data are plotted in terms of the dimensionless factors, pressure divided by design head (h_p/H_d) and horizontal distance divided by design head (X/H_d), to permit ready conversion to any selected design head. Pressures for intermediate head ratios can be obtained by plotting h_p/H_d versus H/H_d for a given X/H_d.

3. It is recommended that the spillway design head H_d be selected so that the minimum crest pressure for the maximum expected head H be no lower than -20 ft of water to ensure cavitation-free operation.

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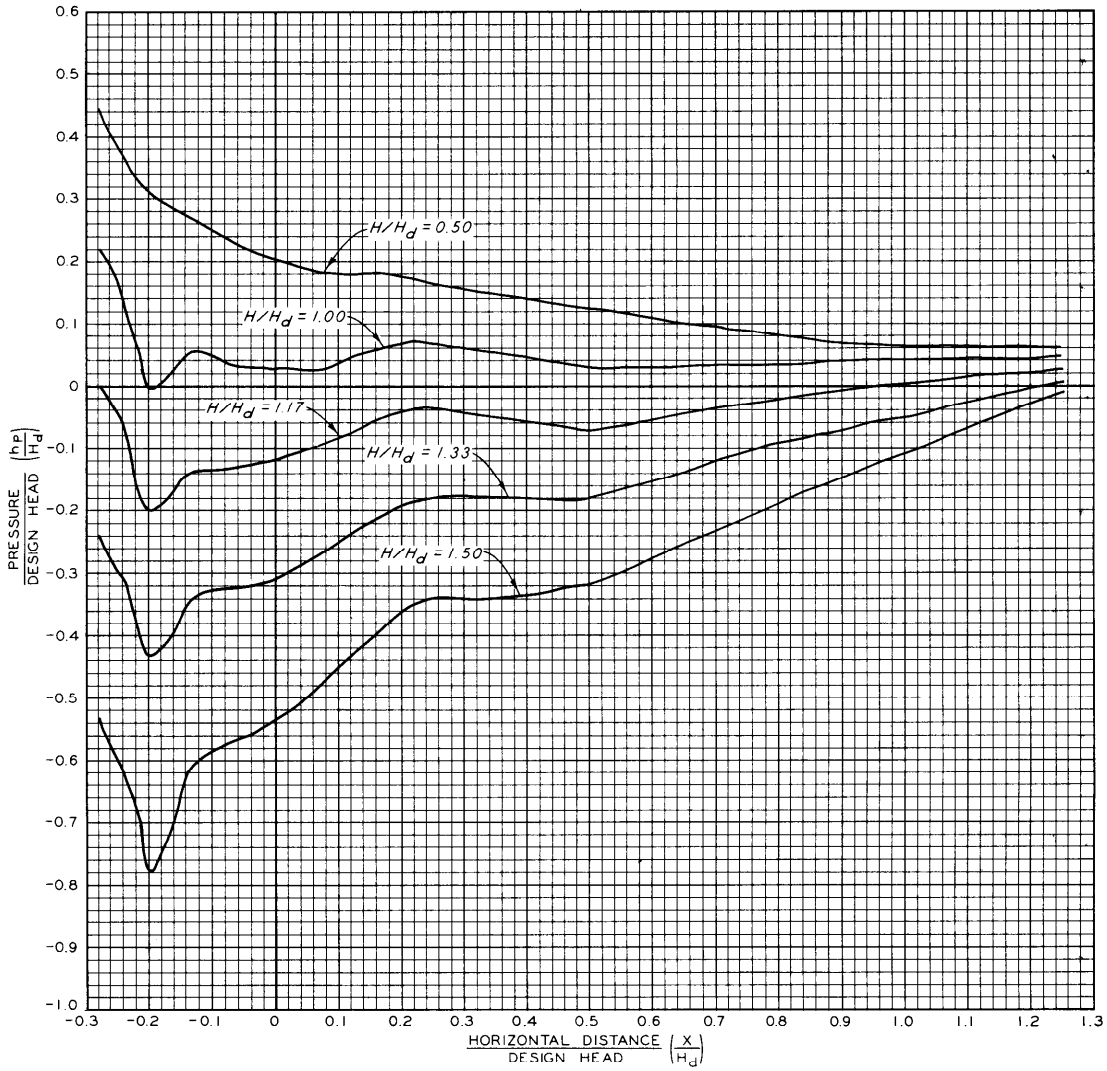
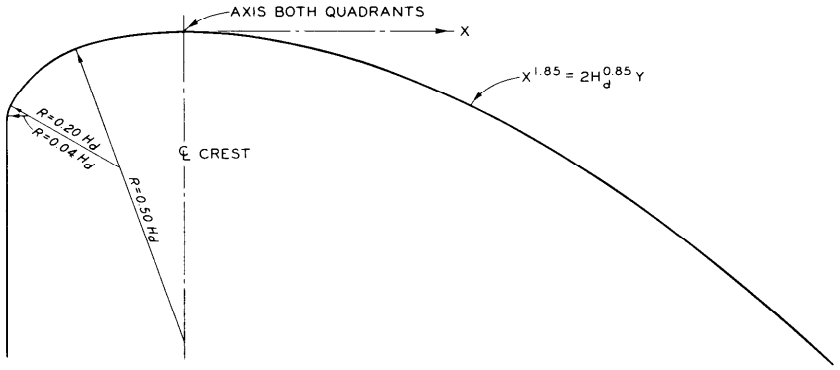
* U. S. Army Engineer Waterways Experiment Station, CE, Investigations of Various Shapes of the Upstream Quadrant of the Crest of a High Spillway; Hydraulic Laboratory Investigation, by E. S. Melsheimer and T. E. Murphy. Research Report H-70-1, Vicksburg, Miss., January 1970.





NOTE: DATA BASED ON ES801 TESTS.

HIGH OVERFLOW DAMS

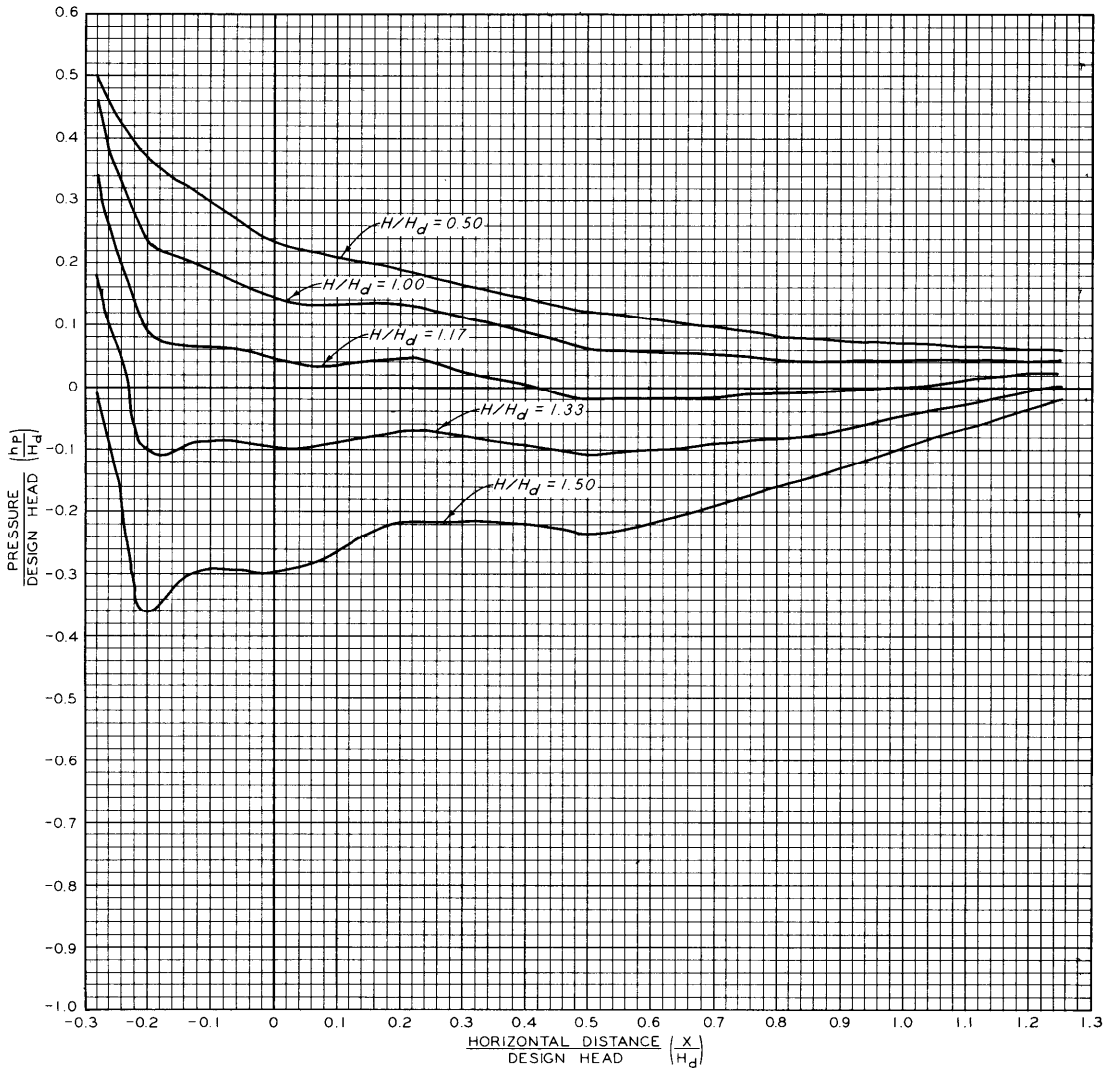
CREST PRESSURES
NO PIERS

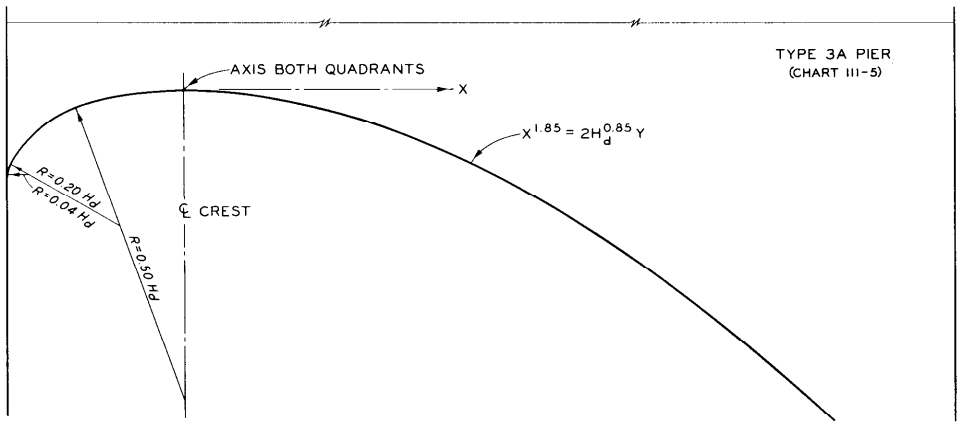
HYDRAULIC DESIGN CHART III-16

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

REV 3-55, 9-70

WES 9-54





NOTE: DATA BASED ON ES801 TESTS.

HIGH OVERFLOW DAMS

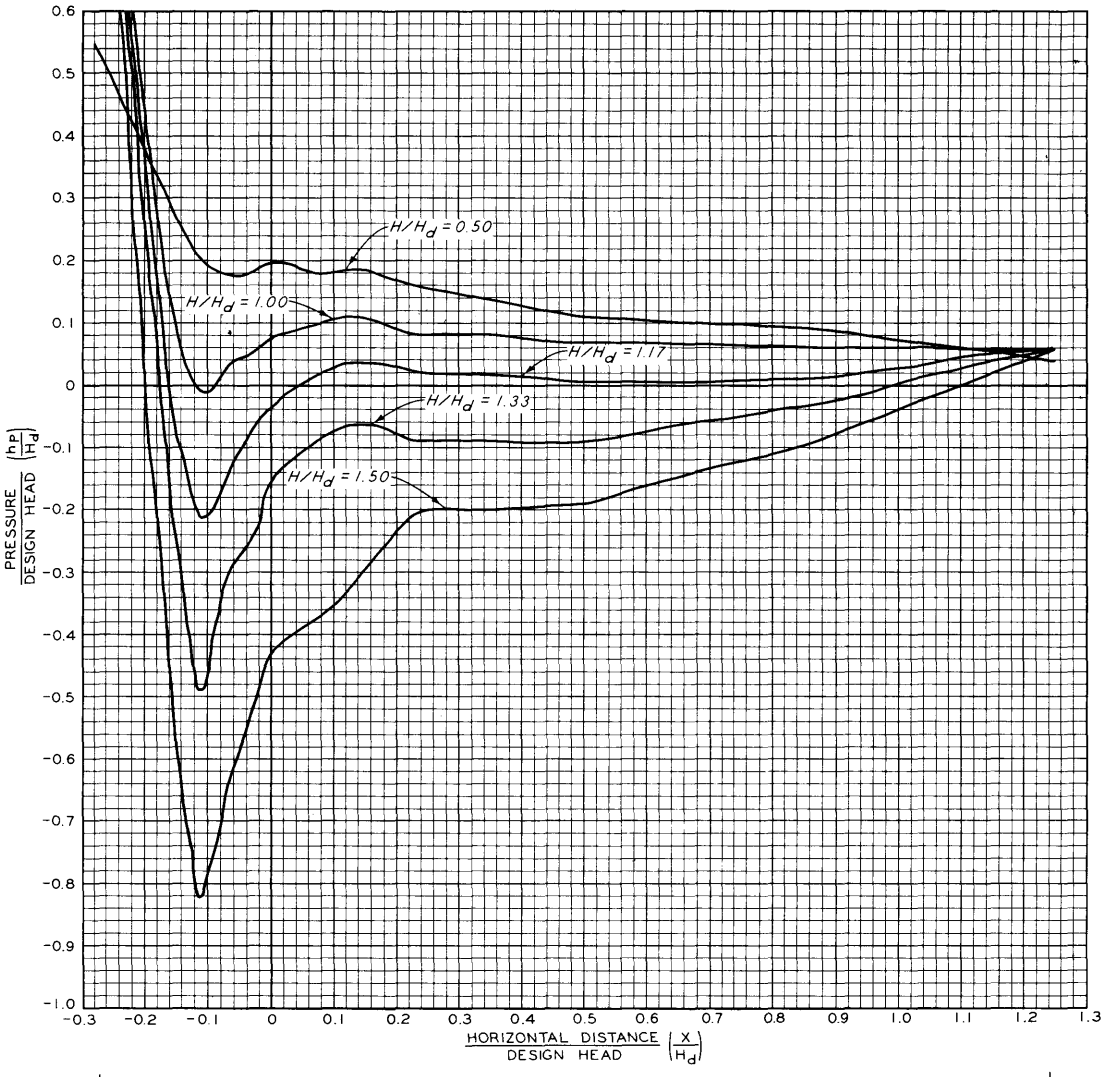
CREST PRESSURES
CENTER LINE OF PIER BAY

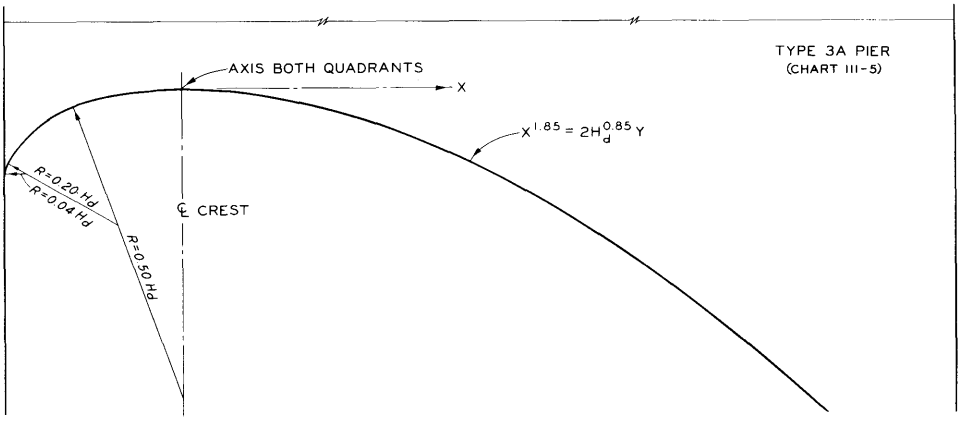
HYDRAULIC DESIGN CHART III-16/1

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

REV 9-70

WES 3-55





NOTE: DATA BASED ON ES801 TESTS.

HIGH OVERFLOW DAMS

CREST PRESSURES ALONG PIERS

HYDRAULIC DESIGN CHART III-16/2

REV 9-70

WES 3-55

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

HYDRAULIC DESIGN CRITERIA

SHEET 111-17

HIGH OVERFLOW DAMS

PRESSURE RESULTANTS

1. In certain stability problems it is desirable to determine the actual pressure forces acting on the upstream face of the dam rather than to assume straight-line pressure distribution near the crest. Hydraulic Design Chart 111-17 presents a plot of experimental data showing pressure distribution in terms of the design head. The data presented are based on CW 801 tests for crests without piers and on USBR tests[1] of pressures on a sharp-crested weir for $H/H_d = 1.00$. The location and magnitude (in terms of the design head) of three resultants based on integration of the pressure plot between the limits of $0 < y < 1.5\ H_d$ are also shown. The cross-hatched area (R_1) is a pressure reduction to be applied to the normally assumed pressure acting on the vertical face of the dam. Sufficient pressure data on the vertical face are not available to allow computation of the resultant (R_1) for H/H_d values of 0.50 and 1.33. R_2 is the vertical pressure resultant effected by flow over the curved surface of the crest. R_3 is the horizontal pressure resultant effected by the flow downstream from the spillway crest.

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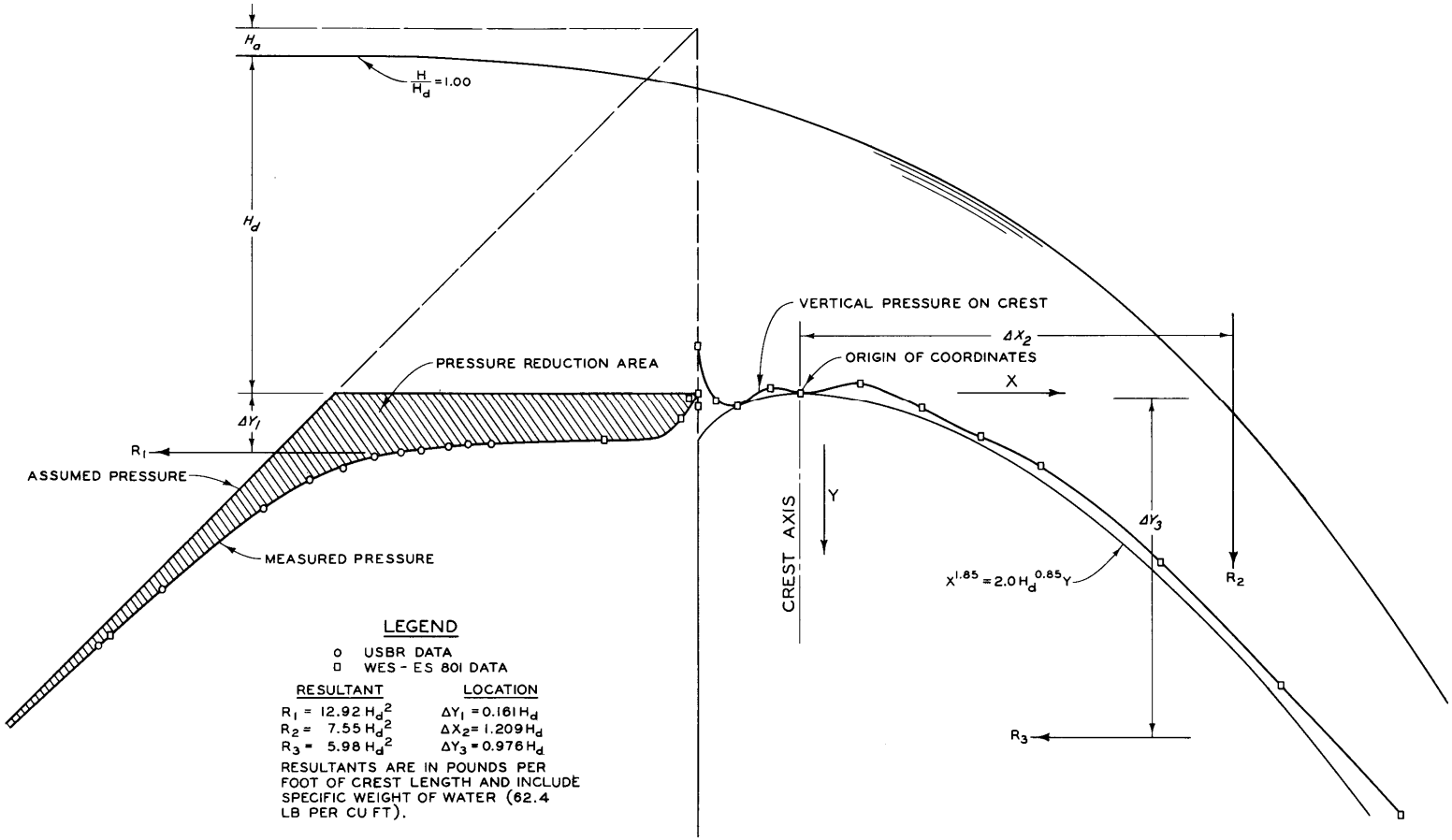
(1) "Studies of Crests for Overfall Dams, Boulder Canyon Project," Final Reports, Part VI, Bulletin 3, Bureau of Reclamation, 1948.



LEGEND

o USBR DATA
□ WES - ES 801 DATA

RESULTANT	LOCATION
$R_1 = 12.92 H_d^2$	$\Delta Y_1 = 0.161 H_d$
$R_2 = 7.55 H_d^2$	$\Delta X_2 = 1.209 H_d$
$R_3 = 5.98 H_d^2$	$\Delta Y_3 = 0.976 H_d$

RESULTANTS ARE IN POUNDS PER FOOT OF CREST LENGTH AND INCLUDE SPECIFIC WEIGHT OF WATER (62.4 LB PER CU FT).

NOTE: VERTICAL FACE OF DAM USED AS ZERO DATUM FOR HORIZONTAL PRESSURES UPSTREAM FROM CREST.
MASONRY LINE USED AS ZERO DATUM FOR HORIZONTAL PRESSURES DOWNSTREAM FROM CREST AND FOR VERTICAL PRESSURES.
VELOCITY OF APPROACH NEGLIGIBLE FOR HIGH OVERFLOW DAMS.

HIGH OVERFLOW DAMS

PRESSURE RESULTANTS
NO PIERS

HYDRAULIC DESIGN CHART III-17

REVISED 8-58 WES 9-54

HYDRAULIC DESIGN CRITERIA

SHEETS 111-18 TO 111-18/5

SPILLWAY ENERGY LOSSES

1. An estimate of the loss of energy on the downstream face of a high overflow spillway may be important in the design of energy-dissipating devices at the foot of the spillway. If the losses are substantial, their evaluation is desirable in order to design an economical stilling basin or to estimate the throw of a jet from a flip bucket. The problem is twofold: (a) determination of energy loss during development of the turbulent boundary layer, and (b) determination of energy loss in the fully developed turbulent flow. For a large head on the crest with the spillway design flow, usually only (a) need be considered. HDC 111-18 to 111-18/5 apply to the condition of turbulent boundary layer development.

2. Previous Design Criteria. It has been common practice to use the Manning equation or some other open-channel equation to determine spillway energy losses. Gumensky[1] based an analysis on the Manning equation and published a graph which has been widely used. Jansen[2] proposed an empirical equation based on Randolph's[3] observation on Madden Dam. Bradley and Peterka[4] published a graph which reflects spillway losses as indicated by tests on Shasta and Grand Coulee Dams. In general, the results obtained by these methods do not agree.

3. Boundary Layer Theory. The concepts of displacement thickness and momentum thickness of the boundary layer are discussed in modern fluid mechanics textbooks.[5] The concept of energy thickness, which is useful in the spillway energy-loss problem, has appeared in the literature only recently. Schlichting[6] makes reference to the use of the energy thickness by Wieghardt.[7] The decrease in energy flux in the boundary layer caused by friction is found by:

$$\frac{1}{2} \rho \int_0^\delta u \left(U^2 - u^2\right) dy \tag{1}$$

where δ is the boundary layer thickness as indicated in the definition sketch in HDC 111-18, u is the velocity at a distance y from the boundary, and U is the potential flow velocity. By definition, the energy thickness δ_3 is the thickness of a layer of fluid with velocity U which represents the loss of energy flux in the boundary layer:

$$\frac{1}{2} \rho U^3 \delta_3 = \frac{1}{2} \rho \int_0^\delta u \left(U^2 - u^2\right) dy \tag{2}$$

$$\delta_3 = \int_0^\delta \frac{u}{U} \left(1 - \frac{u^2}{U^2}\right) dy \tag{3}$$

If δ_3 can be estimated, the energy flux loss upstream from any point on the spillway face can be found from:

$$E_L = \frac{1}{2} \rho U^3 \delta_3 \qquad \text{(lb/sec)} \tag{4}$$

Division of equation 4 by $q\gamma$, where q is the unit discharge and γ is the specific weight, results in defining the energy loss in terms of feet of head:

$$H_L = \frac{\delta_3 U^3}{2gq} \qquad \text{(feet of head)} \tag{5}$$

where g is the acceleration due to gravity. The evaluation of δ_3 and U is discussed below.

4. Turbulent Boundary Layer Investigations. During spillway discharge, the turbulent boundary layer continues to develop until it reaches the free-water surface or until the flow enters the energy dissipator at the toe of the structure. Bauer[8,9] made extensive laboratory tests and correlated boundary layer thickness and development length (X) with surface roughness. His analysis included limited prototype data.[10] Keulegan[11] reanalyzed Bauer's data and proposed the equation

$$\frac{\delta}{X} = 0.96 \left(\frac{X}{k} \right)^{-1/4} \tag{6}$$

where X is the distance from an assumed origin and k is the absolute surface roughness height.

5. Spillway Energy Losses. A study of Bauer's[8,9] data, Keulegan's[11] reanalysis, additional prototype data,[12] and photographs* has been made by the Waterways Experiment Station[13,14] to develop design criteria for estimating spillway energy losses. This study resulted in the curve given in HDC 111-18 which is applicable for estimating the boundary layer thickness in flows over spillways. The equation of the curve is:

$$\frac{\delta}{L} = 0.08 \left(\frac{L}{k} \right)^{-0.233} \tag{7}$$

where L is the length along the spillway crest from the beginning of the crest curve (HDC 111-18). A roughness k value of 0.002 ft is recommended for concrete.

---

* Unpublished photographs by the U. S. Army Engineer District, Vicksburg, Miss., of flow over the spillway of Arkabutla Dam, Coldwater River, Mississippi.

6. The relations between boundary layer thickness δ, displacement thickness δ_1, and energy thickness δ_3, based on Bauer's data for screen roughness, have been found to be:

$$\delta_1 = 0.18\delta \tag{8}$$

$$\delta_3 = 0.22\delta \tag{9}$$

The use of these relations in conjunction with the potential flow depth and equations 5 and 7 are recommended for estimating spillway flow depths and energy losses. Modifications to equations 7, 8, and 9 may prove desirable as additional data become available.

7. Application.

a. Case 1. The boundary layer thickness δ, the flow depth d, and the spillway energy loss H_L for design heads of 20, 30, and 40 ft can be estimated directly from HDC 111-18/2 and 111-18/3 for the standard crest shape defined by HDC 111-1 to 111-2/1. The use of HDC 111-18/2 and 111-18/3 is applicable to spillways with tangent face slopes of 1:0.7 and a surface roughness k of 0.002 ft. HDC 111-18/4 illustrates the required computations. The computations indicate that the boundary layer has not reached the free-water surface. Therefore, no bulking of the flow is to be expected from air entrainment caused by turbulence generated at the spillway face.

b. Case 2. For the standard crest shapes with face slopes other than 1:0.7, HDC 111-1, 111-18, and 111-18/1 and equations 5, 8, and 9 should be used in the manner illustrated by the sample computation given in HDC 111-18/5. If the computed boundary layer thickness is indicated to become greater than the summation of the displacement thickness and the potential flow depth, the intersection of the free-water surface and the boundary layer, sometimes called the critical point, can be located. This can be done by plotting the flow depth d and boundary layer thickness δ as functions of the boundary length as shown in HDC 111-18/2. Air entrainment begins beyond the critical point, and the energy-loss mechanism is not well understood.

c. Case 3. For other than the standard crest shape, the curved crest length L_c is determined graphically or analytically. The computation procedure is then similar to that for Case 2.

8. References.

(1) Gumensky, D. B., "Air entrained in fast water affects design of training walls and stilling basins." Civil Engineering, vol 19 (December 1949), pp 831-833, 889.

(2) Jansen, R. B., "Flow characteristics on the ogee spillway." ASCE Hydraulics Division, Journal, vol 83 (December 1957), pp 1452-1 to 1452-11.

(3) Randolph, R. R., Jr., "Hydraulic tests on the spillway of the Madden Dam." Transactions, American Society of Civil Engineers, vol 103 (1938), pp 1080-1112.

(4) Bradley, J. N., and Peterka, A. J., "Hydraulic design of stilling basins." ASCE Hydraulics Division, Journal, vol 83, HY 5 (October 1957), pp 1401-1 to 1406-17.

(5) Rouse, Hunter, Advanced Mechanics of Fluids. John Wiley & Sons, Inc., New York, N. Y., 1959.

(6) Schlichting, H., Boundary Layer Theory. English translation by J. Kestin. McGraw-Hill Book Co., Inc., New York, N. Y., 1960.

(7) Wieghardt, K., "Ueber einen Energiesatz zur Berechnung laminarer Grenzschichten (Concerning an energy principle for calculation of laminar boundary layer)." Ingenieur-Archiv, vol 16 (1948), p 231.

(8) Bauer, W. J., The Development of the Turbulent Boundary Layer on Steep Slopes. A dissertation submitted to State University of Iowa, August 1951.

(9) _____, "Turbulent boundary layer on steep slopes." Transactions, American Society of Civil Engineers, vol 119 (1954), pp 1212-1233.

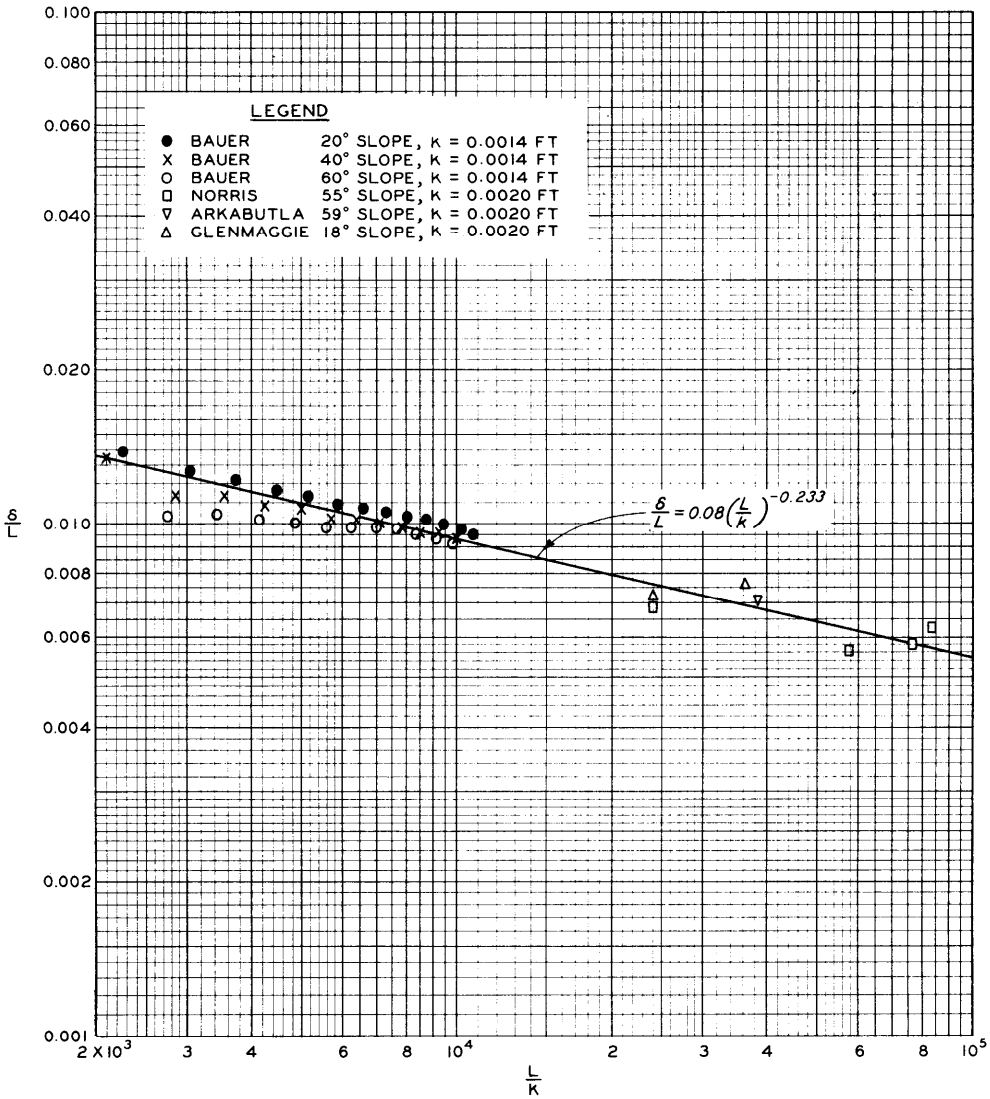
(10) Hickox, G. H., "Air entrainment on spillway faces." Civil Engineering, vol 15 (December 1945), pp 562-563.

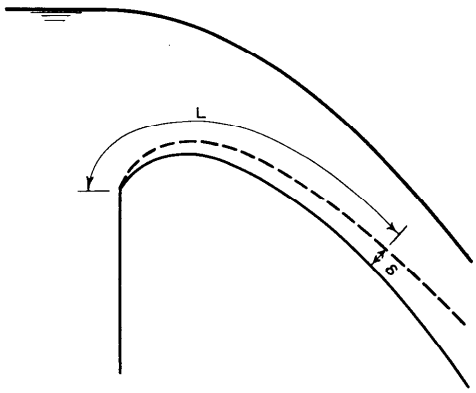
(11) U. S. Army Engineer Waterways Experiment Station, CE, Turbulent Boundary Layer Development on Spillways, by G. H. Keulegan. Miscellaneous Paper No. 2-587, Vicksburg, Miss., July 1963.

(12) Michels, V., and Lovely, M., "Some prototype observations of air entrainment flow." Proceedings, Minnesota International Hydraulics Convention (August 1953), pp 403-414.

(13) U. S. Army Engineer Waterways Experiment Station, CE, A Study of Spillway Energy Losses During Development of the Turbulent Boundary Layer. Miscellaneous Paper No. 2-642, Vicksburg, Miss., April 1964.

(14) Office, Chief of Engineers, Department of the Army, Engineering and Design; Hydraulic Design of Spillways. Engineer Manual EM 1110-2-1603, Washington, D. C., March 1965.





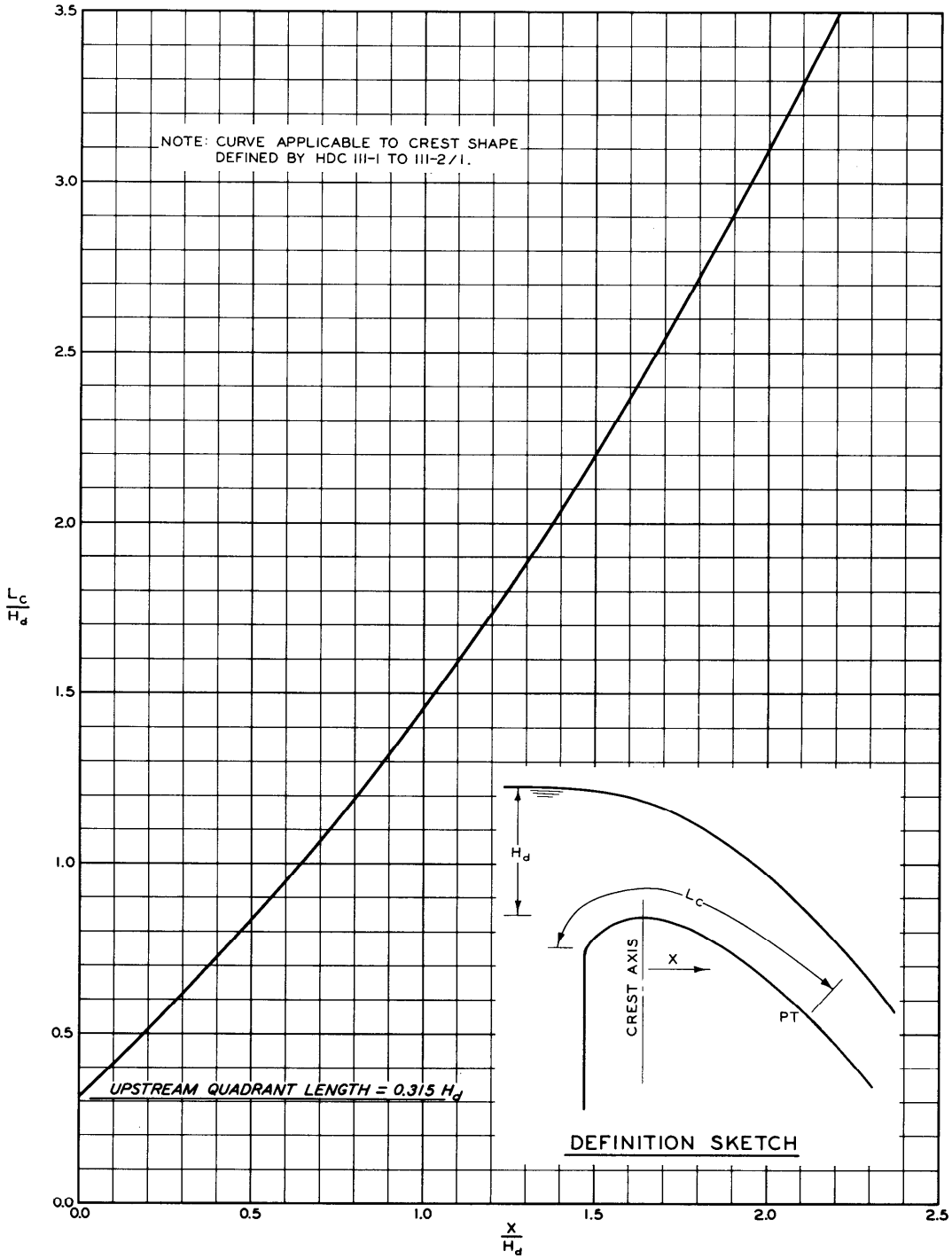
DEFINITION SKETCH

δ = BOUNDARY LAYER THICKNESS, FT
L = SURFACE LENGTH, FT
K = ABSOLUTE SURFACE ROUGHNESS HEIGHT, FT

HIGH OVERFLOW DAMS

SPILLWAY ENERGY LOSS
BOUNDARY LAYER DEVELOPMENT

HYDRAULIC DESIGN CHART III-18



NOTE: L_c = LENGTH ALONG CURVED CREST, FT
X = HORIZONTAL COORDINATE, FT
H_d = SPILLWAY DESIGN HEAD, FT

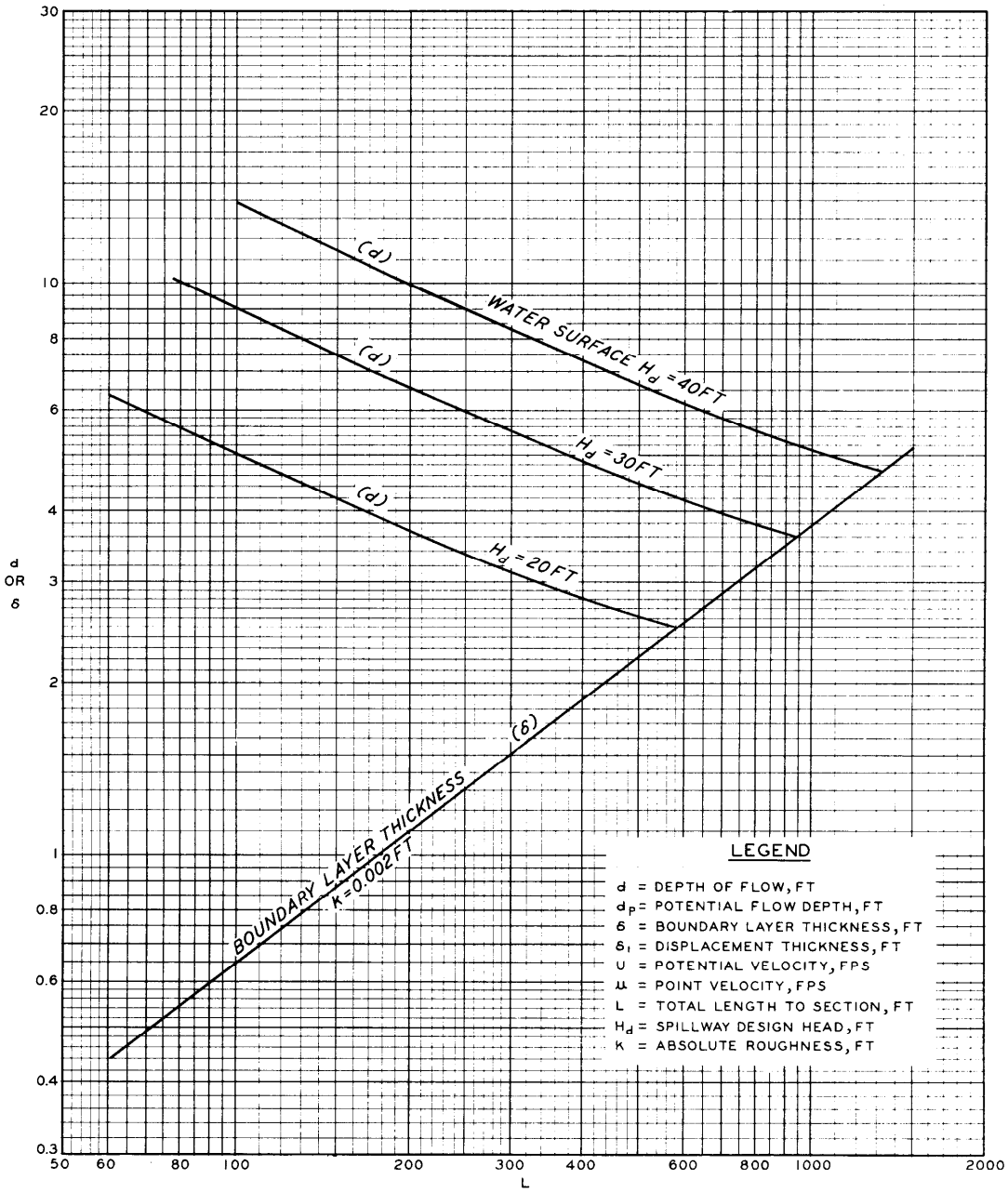
HIGH-OVERFLOW DAMS

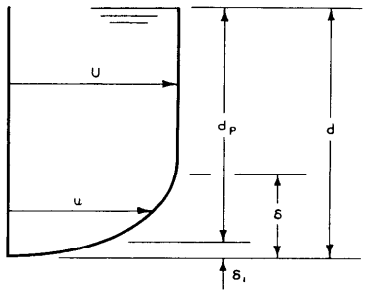
SPILLWAY ENERGY LOSS
STANDARD CREST LENGTH

HYDRAULIC DESIGN CHART III-18/1

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

WES 1-66





DEFINITION SKETCH

NOTE: CURVES APPLICABLE TO STANDARD SPILLWAY CREST (HDC III-1 TO III-2/1) WITH 1:0.7 TANGENT FACE SLOPE.

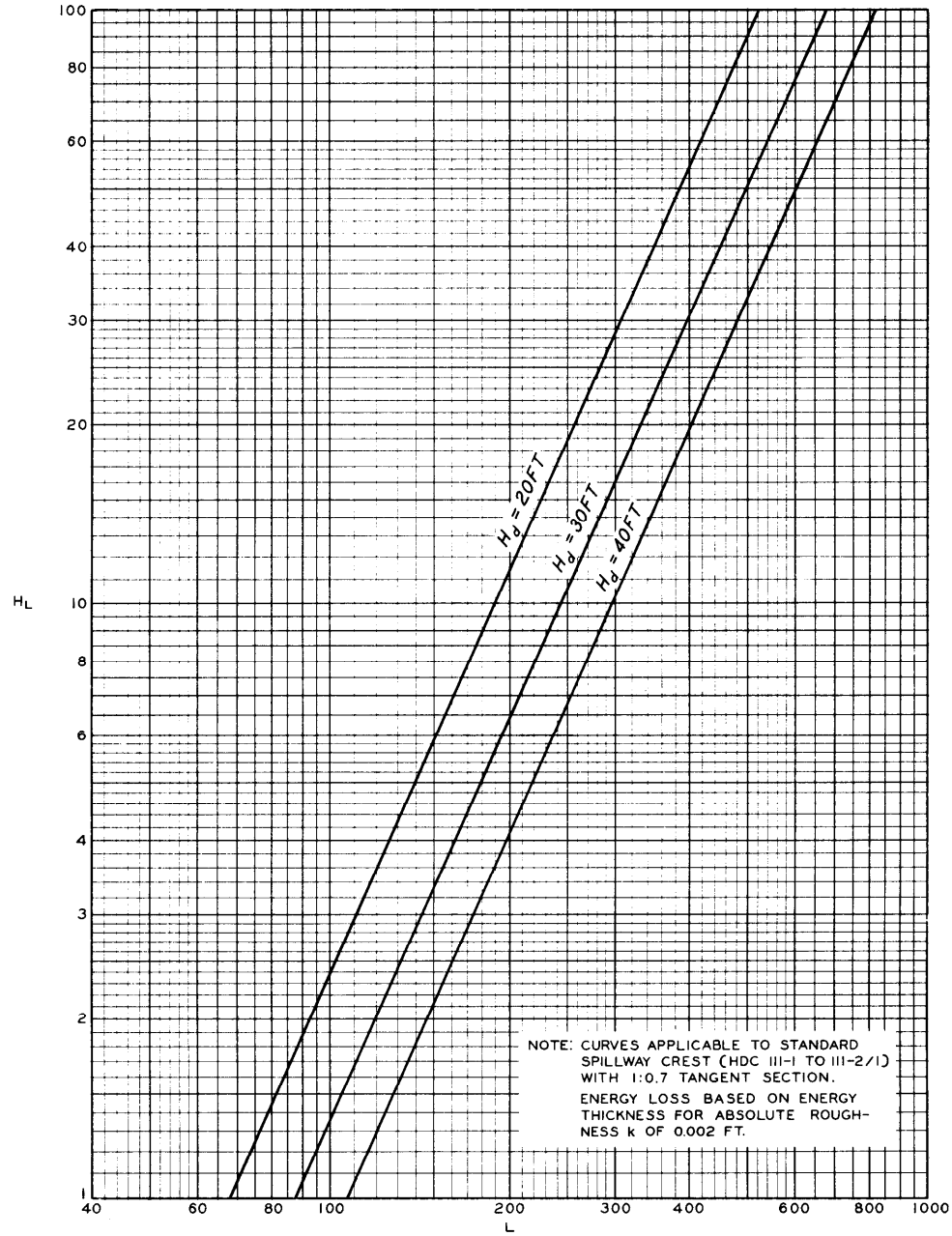
DEPTH (d) IS POTENTIAL FLOW DEPTH PLUS DISPLACEMENT THICKNESS.

HIGH-OVERFLOW DAMS

SPILLWAY ENERGY LOSS
STANDARD CREST
LOCATION OF CRITICAL POINT

HYDRAULIC DESIGN CHART III-18/2

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



LEGEND

H_L = ENERGY HEAD LOSS, FT
L = TOTAL LENGTH TO SECTION, FT
H_d = SPILLWAY DESIGN HEAD, FT

HIGH-OVERFLOW DAMS

SPILLWAY ENERGY LOSS
STANDARD CREST
FACE SLOPE 1:0.7

HYDRAULIC DESIGN CHART III-18/3

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

GIVEN:

$H_d = 30$ ft

$H = 250$ ft

$k = 0.002$ ft (Surface roughness)

Face slope = 1:0.7

COMPUTE:

1. Boundary Geometry

   a. Length of curved crest, L_c

   $$\frac{X_1}{H_d} = 1.67 \text{ (HDC 111-1)}$$

   $$\frac{L_c}{H_d} = 2.50 \text{ (HDC 111-18/1)}$$

   $$L_c = 2.50H_d = 75.0 \text{ ft}$$

   b. Length of tangent, L_T

   $$\frac{Y_1}{H_d} = 1.32 \text{ (HDC 111-1)}$$

   $$Y_1 = 1.32H_d = 39.6 \text{ ft}$$

   $$Y_2 - Y_1 = 250 - 39.6 = 210.4 \text{ ft}$$

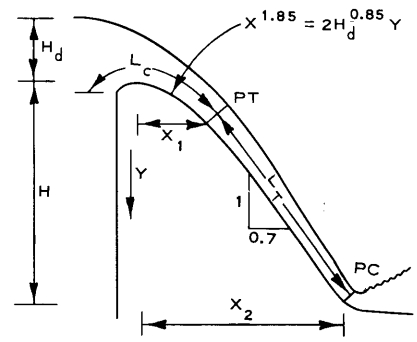
   $$X_2 - X_1 = \frac{0.7}{1.0}(Y_2 - Y_1) = 147.3 \text{ ft}$$

   $$L_T = \sqrt{(210.4)^2 + (147.3)^2}$$
   $$= 256.9 \text{ ft}$$

   c. Total crest length, L

   $$L = L_c + L_T$$
   $$= 75.0 + 256.9 = 331.9 \text{ ft}$$

Note: Computed H_b is satisfactory and no bulking of flow from air entrainment since $\delta < d$.



2. Hydraulic Computation

   a. Spillway energy loss, H_L

   For $L = 331.9$ ft and $H_d = 30$ ft

   $H_L = 20.0$ ft (HDC 111-18/3)

   b. Energy head entering stilling basin, H_b

   $$H_b = H + H_d - H_L$$
   $$= 250 + 30 - 20 = 260 \text{ ft}$$

   c. Depth of flow, d, and boundary layer thickness, δ, at PC of toe curve

   $d = 5.30$ ft (HDC 111-18/2)

   $\delta = 1.62$ ft (HDC 111-18/2)

HIGH OVERFLOW DAMS

SPILLWAY ENERGY LOSS

SAMPLE COMPUTATION

FACE SLOPE 1:0.7

HYDRAULIC DESIGN CHART 111-18/4

GIVEN:

$H_d = 30$ ft
$H = 350$ ft
$k = 0.002$ ft (Surface roughness)
Face slope = 1:0.78

COMPUTE ENERGY HEAD ENTERING STILLING BASIN:

1. Boundary Geometry

a. Length of curved crest, L_c

$$\frac{X_1}{H_d} = 1.47 \text{ (HDC 111-1)}$$

$$\frac{L_c}{H_d} = 2.15 \text{ (HDC 111-18/1)}$$

$$L_c = 2.15 H_d = 64.5 \text{ ft}$$

b. Length of tangent, L_T

$$\frac{Y_1}{H_d} = 1.04 \text{ (HDC 111-1)}$$

$$Y_1 = 1.04 H_d = 31.2 \text{ ft}$$

$$Y_2 - Y_1 = 350 - 31.2 = 318.8 \text{ ft}$$

$$\text{Tan } \alpha = \frac{1.0}{0.78} = 1.2821$$

$$\text{Sin } \alpha = 0.7885$$

$$L_T = \frac{Y_2 - Y_1}{\text{Sin } \alpha} = 404.4 \text{ ft}$$

c. Total crest length, L

$$L = L_c + L_T$$
$$= 64.5 + 404.4 = 468.9 \text{ ft}$$

2. Hydraulic computation

a. Boundary layer thickness, δ

$$\frac{L}{k} = \frac{468.9}{0.002} = 2.344 \times 10^5$$

$$\frac{\delta}{L} = 0.08 \left(\frac{L}{k}\right)^{-0.233} \text{ (HDC 111-18)}$$
$$= 0.08\ (0.0561)$$
$$= 0.00449$$
$$\delta = 2.10 \text{ ft}$$

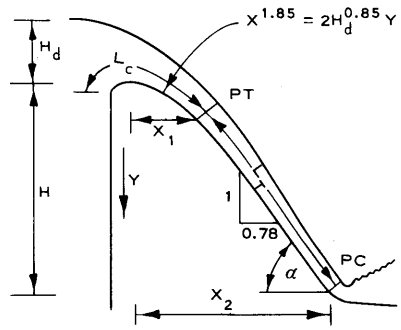
b. Energy thickness, δ_3

$$\delta_3 = 0.22\,\delta \text{ (Eq 9, sheets 111-18 to 111-18/5)}$$
$$= 0.462 \text{ ft}$$

c. Unit discharge, q

$$q = CH_d^{3/2}$$
$$C = 4.03 \text{ (HDC 111-3)}$$
$$q = 4.03\,(30)^{3/2} = 662 \text{ cfs}$$

Note: Computed H_b satisfactory and no bulking of flow from air entrainment since $\delta < d$.



d. Potential flow depth d_p and velocity U at PC of toe curve

$$H_T = d_p \cos \alpha + \frac{U^2}{2g}$$
$$\text{Cos } \alpha = 0.6150$$
$$H_T = H + H_d = 350 + 30 = 380 \text{ ft}$$

By trial

U	$\frac{U^2}{2g}$	$H_T - \frac{U^2}{2g}$	$d_p = \frac{H_T - \frac{U^2}{2g}}{0.6150}$	$q = Ud_p$
(ft)	(ft)	(ft)	(ft)	(cfs)
156.0	377.9	2.1	3.41	532 < 662
155.9	377.4	2.6	4.23	659 ≈ 662

e. Spillway energy loss

$$H_L = \frac{\delta_3\ U^3}{2g\,q} \text{ (Eq 5, sheets 111-18 to 111-18/5)}$$
$$= \frac{0.462\ (155.9)^3}{64.4\ (662)} = 41 \text{ ft}$$

f. Energy head entering stilling basin

$$H_b = H + H_d - H_L$$
$$= 350 + 30 - 41 = 339 \text{ ft}$$

g. Depth of flow, d, at PC of toe curve

$$d = d_p + \delta_1$$
$$\delta_1 = 0.18\,\delta \text{ (Eq 8, sheets 111-18 to 111-18/5)}$$
$$= 0.18\ (2.10) = 0.38 \text{ ft}$$
$$d = 4.23 + 0.38 = 4.61 \text{ ft}$$

HIGH OVERFLOW DAMS

SPILLWAY ENERGY LOSS
SAMPLE COMPUTATION
FACE SLOPE 1:0.78

HYDRAULIC DESIGN CHART 111-18/5

HYDRAULIC DESIGN CRITERIA

SHEETS 111-19 TO 111-19/2

HIGH OVERFLOW DAMS

SPILLWAY CREST WITH OFFSETS AND RISERS

CREST SHAPES

1. Purpose. Use of spillway crests with offsets and risers may effect appreciable economies in the construction of concrete gravity spillways provided the concrete mass eliminated from the standard crest shape defined in HDC 111-1 to 111-2/1 is not required for structural stability. The scheme has also been adopted for the high arch dam Monteynard.[1] HDC 111-19 to 111-19/2 should serve for developing crest shapes for preliminary economic studies. It is suggested that the final spillway shape be model-tested.

2. Background. A laboratory study of overhanging crests produced by an offset in a sharp-crested weir was reported by the U. S. Bureau of Reclamation (USBR).[2] A recent analysis of unpublished USBR test data was made by the Waterways Experiment Station (WES). In this study the dimensionless quantity of the ratio of the riser height to the head on the rounded crest M/H_d was selected as the basic shape variable. The offset dimension N does not appear to be very effective until the ratio M/H_d becomes very small. One limiting case is the offset weir with no riser ($M = 0$), which forms a 45-degree backward-sloping weir. The lower nappe of the flow over a 45-degree backward-sloping, sharp-crested weir may extend initially slightly upstream of the sharp crest. The other limiting case is a high spillway with no offset ($N = 0$) described in HDC 111-1 to 111-2/1. The test data selected for the WES study were from experiments with negligible velocity of approach, a condition representative of a high dam for which substantial savings in concrete would result from undercutting the upstream face. These data were for M/H_d test values of 0.240 to 0.396 having N/H_d values of 0.079 to 0.240. The resulting M/N values are 1.0 to 5.0. Sections having M/N values less than 0.5 are not recommended.[2] The results of this study are summarized in HDC 111-19 to 111-19/2 and generally define spillway crest geometry for riser-design head ratios of $0 < M/H_d < 1.0$.

3. Crest Location. The USBR lower-nappe coordinates are in terms of the head on the sharp-crested weir and have their origin at the weir. For design purposes, it is more convenient to have the coordinates in terms of the head H_d on the round crest with their origin at the crest apex. HDC 111-19 gives curves for estimating the distance X_e of the round crest from the sharp crest and the height Y_e of the round crest above the sharp crest. The curves are in terms of the design head H_d on the round crest and are plotted as a function of the ratio of the riser height to design head M/H_d. The values of X_e/H_d and Y_e/H_d were found to be 0.287 and 0.166, respectively, for the limiting case of $M/H_d = 0$. HDC 111-2/1

gives values of 0.270 and 0.126 for X_e/H_d and Y_e/H_d, respectively, for the case of $M/H_d \geq 1.0$. These limiting cases are plotted in HDC 111-19 and were used as guides in defining the general shape of the curves given in the chart.

4. Downstream Quadrant. The standard form of the equation for the downstream quadrant with the head on the round crest is:

$$\frac{Y}{H_d} = K\left(\frac{X}{H_d}\right)^n \tag{1}$$

Values of the constant K and the exponent n for various ratios of M/H_d were determined graphically and by electronic computer. Values of n and K resulting in best correlation with the basic data are plotted in graphs a and b in HDC 111-19/1. Data points appropriate for the limiting cases discussed in paragraph 3 are also shown. The vertical-face spillway defined in HDC 111-1 has n and K values of 1.85 and 0.50, respectively. It is reasoned that the values of n and K should approach these limits asymptotically as the riser height M becomes larger in relation to the design head H_d.

5. Upstream Quadrant. The standard form of the equation defined in HDC 111-1 to 111-2/1 for the upstream quadrant of the vertical-face weir was used as a basis for the case of weirs with offsets and risers:

$$\frac{Y_w}{H_w} = K_1\left(\frac{X}{H_w}\right)^{n_1} - K_2\left(\frac{X}{H_w}\right)^n \tag{2}$$

The subscript w refers to the sharp-crested weir. The conditions established for developing the exponents and constants of the equations included:

a. Curve slopes $\frac{dY_w}{dX_w}$ of zero at the round crest and of infinity at the sharp-crested riser for the selected values of X_e/H_d and Y_e/H_d.

b. For M/H_d values > 0.24, the exponent n_1 of the first term has the value of 0.625 developed for the vertical-face weir using relaxation data of McNown and others[3] discussed in HDC 111-1 to 111-2/1.

c. The exponent n of the second term is the same as that developed for the downstream quadrant.

d. The constants K_1 and K_2 developed to provide a reasonable fit to the experimental data.

6. Computed values of exponents and constants for equation 2 based on the selected USBR data are plotted in graphs a and c in HDC 111-19/1. The plotted points meet the conditions established in paragraph 5.

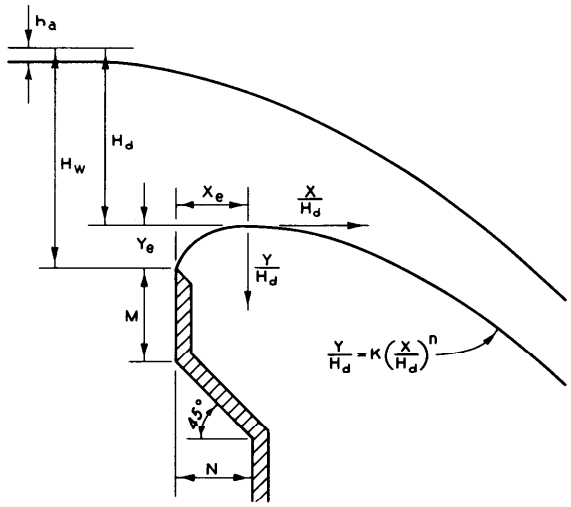
7. Application. The crest shape defined in HDC 111-1 to 111-2/1 should be applicable to overhanging spillways having riser heights $\geq 0.6H_d$. Use of the curves in HDC 111-19 and 111-19/1 is suggested for preliminary design purposes should there be design reasons for making the riser smaller than $0.6H_d$. The curves on these charts should be used in the manner indicated by the sample computation given in HDC 111-19/2. The final design should be model-tested. The use of a curvature of appreciable radius to connect the riser to the sloping overhang is recommended if model tests indicate pressure pulsations on the crest resulting from flow separations around the riser.

8. References.

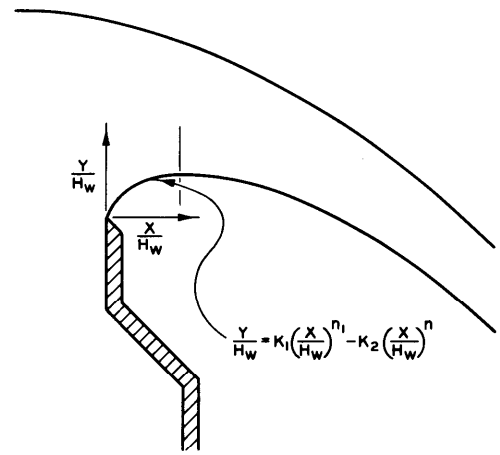
(1) Bowman, W. G., "French high arch dam is all-in-one (Monteynard Dam)." Engineering News-Record, vol 169 (25 October 1962), pp 30-37.

(2) U. S. Bureau of Reclamation, "Studies of crests for overfall dams." Boulder Canyon Project Final Reports, Part VI, Hydraulic Investigations, Bulletin 3, Denver, Colo. (1948).

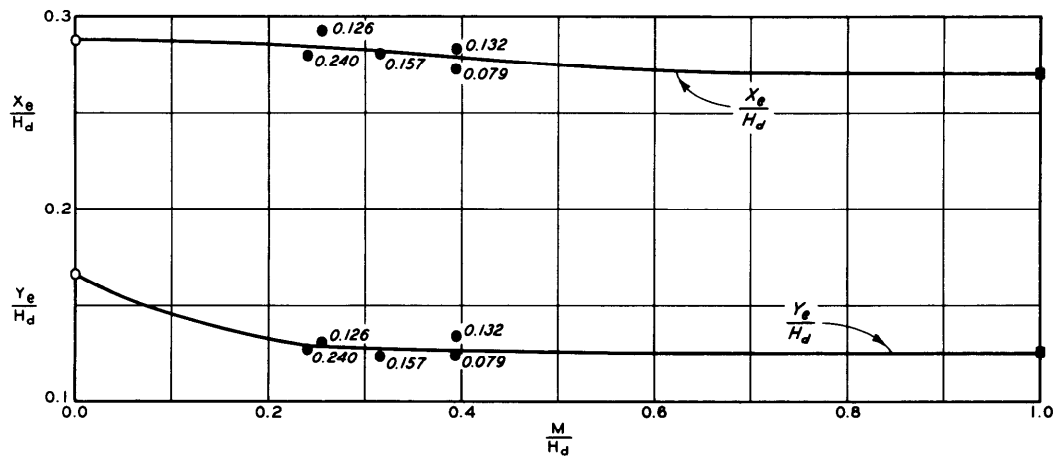
(3) McNown, J. S., Hsu, En-Yun, and Yih, Chia-Shun, "Applications of the relaxation technique in fluid mechanics." Transactions, American Society of Civil Engineers, vol 120 (1955), pp 650-669.



DOWNSTREAM QUADRANT



UPSTREAM QUADRANT



LEGEND

- ● BASED ON UNPUBLISHED USBR DATA
- O BASED ON PUBLISHED USBR DATA
- ■ HDC III-2/1

NOTE: NUMBERS ON GRAPH ARE VALUES OF N/H_d.
H_d IS DESIGN HEAD BASED ON LOWER SURFACE OF NAPPE FROM SHARP-CRESTED WEIR WITH NEGLIGIBLE VELOCITY OF APPROACH.

HIGH-OVERFLOW DAMS

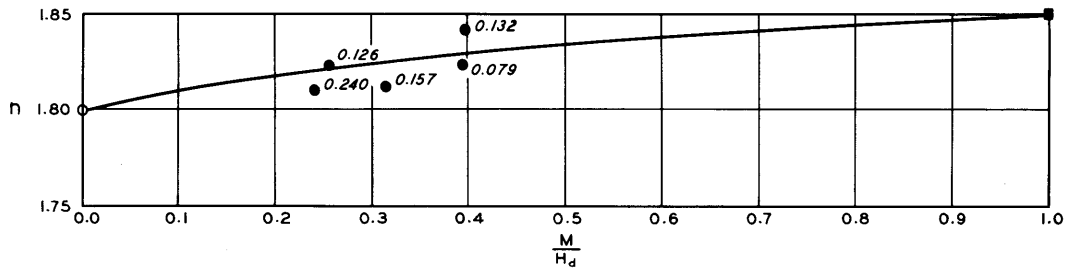
SPILLWAY CRESTS WITH OFFSET AND RISER

CREST LOCATION

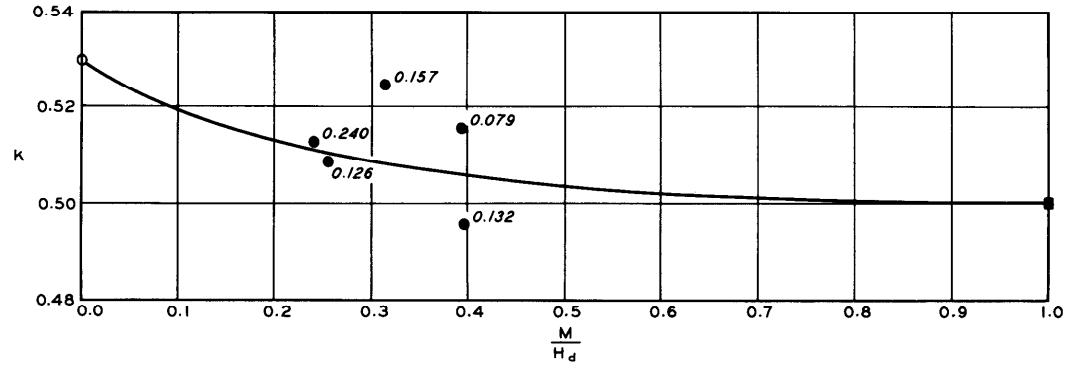
HYDRAULIC DESIGN CHART III-19

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

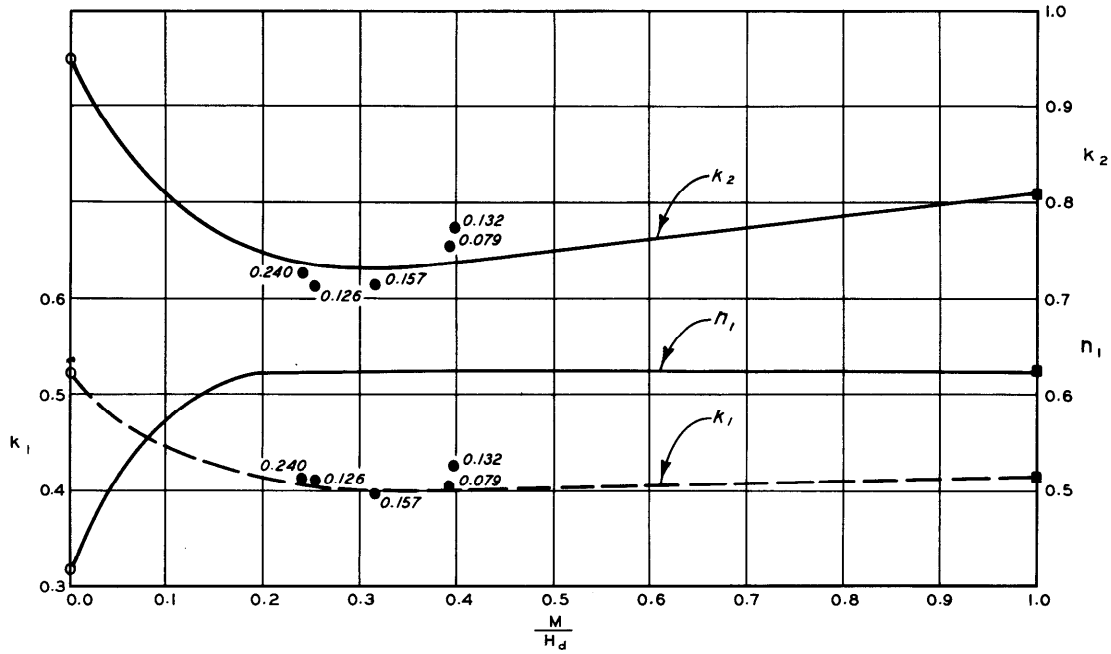
WES 1-66



a. EXPONENT n, DOWNSTREAM QUADRANT



b. COEFFICIENT K, DOWNSTREAM QUADRANT



c. COEFFICIENTS AND EXPONENTS, UPSTREAM QUADRANT

LEGEND

- ● BASED ON UNPUBLISHED USBR DATA
- O BASED ON PUBLISHED USBR DATA
- ■ HDC SHEETS III-1 TO III-2/1 AND HDC III-2

NOTE: NUMBERS ON GRAPHS ARE VALUES OF N/H_d. H_d IS DESIGN HEAD FOR NEGLIGIBLE VELOCITY OF APPROACH.

HIGH-OVERFLOW DAMS

SPILLWAY CRESTS WITH OFFSET AND RISER

CREST SHAPE

HYDRAULIC DESIGN CHART III-19/1

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

GIVEN:

$H_d = 40$ ft

$\frac{M}{H_d} = 0.30$ ft

$\frac{N}{H_d} = 0.25$ ft

Negligible velocity of approach

COMPUTE:

1. Downstream Quadrant Equation

$$\frac{Y}{H_d} = K\left(\frac{X}{H_d}\right)^n$$

$K = 0.508$, $n = 1.825$ (HDC 111-19/1)

$$\frac{Y}{H_d} = 0.508\left(\frac{X}{H_d}\right)^{1.825}$$

2. Upstream Quadrant Equation

$$\frac{Y}{H_w} = K_1\left(\frac{X}{H_w}\right)^{n_1} - K_2\left(\frac{X}{H_w}\right)^n$$

$$\left.\begin{array}{l} K_1 = 0.405,\ K_2 = 0.730 \\ n_1 = 0.625,\ \ n = 1.825 \end{array}\right\} \text{(HDC 111-19/1)}$$

$$\frac{Y}{H_w} = 0.405\left(\frac{X}{H_w}\right)^{0.625} - 0.730\left(\frac{X}{H_w}\right)^{1.825}$$

3. Check for Zero Slope at Crest

$\frac{X_e}{H_d} = 0.282$, $\frac{Y_e}{H_d} = 0.130$ (HDC 111-19)

$H_w = H_d + Y_e = H_d + 0.130 H_d = 1.130 H_d$

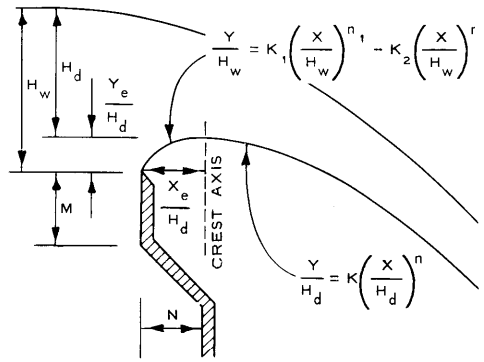
$\frac{X_e}{H_w} = \frac{X_e}{1.130 H_d} = \frac{0.282}{1.130} = 0.250$, $\frac{Y_e}{H_w} = \frac{Y_e}{1.130 H_d} = 0.115$

$$\frac{d\left(\frac{Y}{H_w}\right)}{d\left(\frac{X}{H_w}\right)} = K_1 n_1 \left(\frac{X}{H_w}\right)^{n_1 - 1} - K_2 n \left(\frac{X}{H_w}\right)^{n-1}$$

$$= \frac{0.405 \times 0.625}{(0.250)^{0.375}} - 0.730 \times 1.825\,(0.250)^{0.825}$$

$$= \frac{0.253}{0.594} - 1.331\,(0.319) = 0.002 \approx 0.000$$

Note: For final design greater accuracy of computations is recommended.



4. Solve for Values of K_1 and K_2 Giving Zero Slope at the Crest for $\frac{X_e}{H_w} = 0.250$

$$\frac{Y_e}{H_w} = K_1\left(\frac{X_e}{H_w}\right)^{n_1} - K_2\left(\frac{X_e}{H_w}\right)^n$$

$$0.115 = K_1\,(0.250)^{0.625} - K_2\,(0.250)^{1.825}$$

$$0.115 = 0.421K_1 - 0.080K_2 \qquad (1)$$

$$\frac{d\left(\frac{Y_e}{H_w}\right)}{d\left(\frac{X_e}{H_w}\right)} = 0.625K_1\,(0.250)^{-0.375}$$

$$- 1.825K_2\,(0.250)^{0.825} = 0$$

$$1.05K_1 - 0.583K_2 = 0$$

$$K_1 = 0.554K_2 \qquad (2)$$

Substitute Equation 2 into Equation 1

$0.115 = 0.421 \times 0.554K_2 - 0.080K_2$

$K_2 = 0.752$, $K_1 = 0.416$

Upstream Quadrant Equation

$$\frac{Y}{H_w} = 0.416\left(\frac{X}{H_w}\right)^{0.625} - 0.752\left(\frac{X}{H_w}\right)^{1.825}$$

HIGH-OVERFLOW DAMS

SPILLWAY CRESTS WITH OFFSET AND RISER CREST GEOMETRY SAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 111-19/2

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

HYDRAULIC DESIGN CRITERIA

SHEETS 111-20 TO 111-20/1

ELLIPTICAL CREST SPILLWAY

COORDINATES

1. Previous Crest Shapes.

a. Downstream Quadrant. The U. S. Bureau of Reclamation (USBR) (reference 6) conducted extensive experiments on the shape of the falling nappe over a sharp-crested weir. Using these data, the U. S. Army Corps of Engineers recommended that the 1.85 power of X be used to define the downstream quadrant for a high overflow spillway with negligible approach velocity (see Chart 111-2). Again, based on USBR data (references 6 and 7), Charts 111-7 through 111-10 present coordinates from the best fit of the general equation: $X^n = KH_d^{n-1}Y$ for 3V on 3H, 3V on 2H, and 3V on 1H upstream faces. For low ogee spillway crests (with 45-deg upstream face only) Charts 122-3 to 122-3/5 present plots of X, Y, n, and K for values of the ratio of velocity head divided by design head h_a/H_d of 0.08 and 0.12 where X = horizontal coordinate positive to the right; Y = vertical coordinate positive downward; n = variable, however usually set equal to 1.85; and K = variable dependent on approach depth.

b. Upstream Quadrant. It has long been known that, for curving flow of the type encountered in overflow spillways, conditions at any point in the flow are dependent upon influences directly upstream. Early attempts to fit circular arcs to the profile of the lower nappe of flow over a sharp-crested weir produced surface discontinuities at the weir crest. This problem was partially remedied through the use of a combination of relaxation techniques and data fitting (references 1 and 2). Another method, the fitting of a third, short-radius arc tangent to the vertical face and the intermediate-radius arc, was model tested at the U. S. Army Engineer Waterways Experiment Station (WES) (reference 4) and is incorporated in Chart 111-2/1. No additional criteria were available for high overflow spillways with other than a vertical upstream face. Curves and factors for a 45-deg upstream face low ogee crest are given in Charts 122-3/1, 122-3/4, and 122-3/5.

2. Elliptical Upstream Quadrant Model Tests. In the late 1960's, WES conducted a study to compare performance of four commonly used upstream quadrant design procedures (reference 4). Of the four tested, the short-radius arc method (Chart 111-2/1) and an elliptical formula of the following form

$$\frac{X^2}{\left(AH_d\right)^2} + \frac{Y^2}{\left(BH_d\right)^2} = 1 \qquad (1)$$

where A and B are the major and minor axes, respectively, of the ellipse, appeared to yield the most acceptable results. In 1973 WES published results (reference 5) of preliminary studies done to verify a design procedure incorporating an elliptical upstream quadrant developed from the USBR data of reference 6. The procedure was verified for high spillways during these tests. A comprehensive test program with a wide range of approach velocities, upstream face slopes, and head ratios was conducted at WES from 1977 to 1982 (reference 3). For these later experiments P/H_d, where P is the approach depth, ranged from 0.25 to 2.0, H_e/H_d ranged from 0.4 to 1.5, and upstream face slopes ranged from vertical to 2V on 3H.

3. Spillway Crest Coordinates: Downstream Quadrant. For high spillways where the velocity of approach can be considered negligible ($h_a/H_d < 0.06$), the downstream quadrant equation is that given is Chart 111-2. As the depth of approach decreases, approach velocities increase and the spillway should become flatter to match the partially suppressed vertical contraction of the nappe. Data for sharp-crested weirs were found to be closely fit by maintaining the form of the equation ($X^n = KH_d^{n-1}Y$) with $n = 1.85$ and varying K with approach depth (reference 5). The K value can be determined from Chart 111-20.

4. Point of Tangency: Downstream Quadrant. Chart 111-1 is a plot of coordinates for the tangent points X/H_d and Y/H_d versus slope function α (1/slope) for $K = 2.0$. Coordinates of the downstream tangent points, X/H_d and Y/H_d, for other K values can be determined from Chart 111-1 values by multiplying those values by $(K/2)^{1/0.85}$. Alternately the coordinates of the downstream tangent point can be determined directly from

$$\frac{X}{H_d} = \left(\frac{K}{1.85\alpha}\right)^{1/0.85} \qquad (2)$$

and

$$\frac{Y}{H_d} = \frac{1}{K}\left(\frac{X}{H_d}\right)^{1.85} \tag{3}$$

or

$$\frac{Y}{H_d} = \frac{1}{K}\left(\frac{K}{1.85\alpha}\right)^{1.85/0.85} \tag{4}$$

5. Spillway Crest Coordinates: Upstream Quadrant. Model studies indicated that the quadrant of an ellipse in which the axes systematically varied with depth of approach would fit the measured data except that the ellipse quadrants would extend upstream of the position of the sharp-crested weir used to generate the nappe form to become tangent to the vertical (reference 5). This extension is more pronounced for lower values of P/H_d . The general equation for the elliptical upstream quadrant is given by

$$\frac{X^2}{A^2} + \frac{(B - Y)^2}{B^2} = 1 \tag{5}$$

where the origin of the coordinates has been translated to the crest apex and the positive y-direction is downward (see definition sketch on Chart 111-20). Solving for Y , equation 5 becomes

$$Y = B\left[1 - \left(1 - \frac{X^2}{A^2}\right)^{1/2}\right] \tag{6}$$

Graphs to determine A and B , normalized by the design head H_d , for various ratios of approach depth to design head P/H_d are given in Chart 111-20. For $P/H_d \geq 2.0$, A and B become constant with values of $0.28H_d$ and $0.164H_d$, respectively.

6. Point of Tangency: Upstream Quadrant. If a sloping upstream face is desirable (normally $P/H_d < 1.0$), then the elliptical upstream quadrant is designed for a vertical face as discussed above and the sloping upstream face is attached tangent to the ellipse. The coordinates of the upstream point of tangency (PT) are determined by differentiating equation 6 with respect to X and setting it equal to the upstream face slope F_s and solving for the X coordinate, i.e.

$$X = -\frac{A^2F_s}{\left(A^2F_s^2 + B^2\right)^{1/2}} \tag{7}$$

where F_s is equal to the slope $\Delta Y/\Delta X$ of the upstream face. Substituting for X in equation 6 and solving for the Y coordinate gives

$$Y = B - \frac{B^2}{\left(A^2F_s^2 + B^2\right)^{1/2}} \tag{8}$$

Chart 111-20/1 illustrates the computational procedure for determining the coordinates of a spillway with an elliptical upstream quadrant.

7. References.

(1) McNown, J. S., Hsu, En-Yun, and Yih, Chia-Shun, "Applications of the relaxation technique in fluid mechanics," Transactions, American Society of Civil Engineers, vol 120 (1955), pp 650-669.

(2) Office, Chief of Engineers, Department of the Army, Engineering and Design; Hydraulic Design of Spillways, Engineer Manual EM 1110-2-1603, Washington, D. C., March 1965.

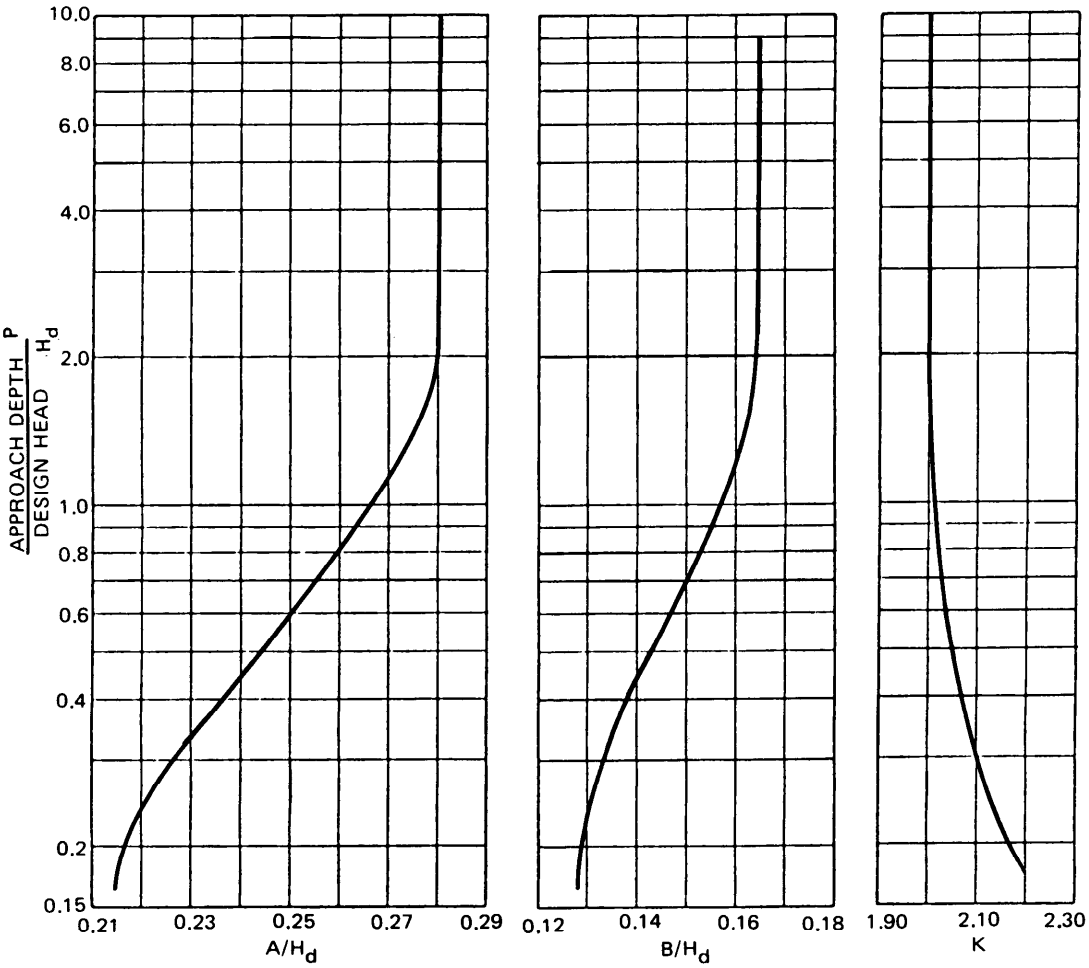
(3) U. S. Army Engineer Waterways Experiment Station, CE, General Spillway Investigation; Hydraulic Model Investigation, by S. T. Maynord, Technical Report HL-85-1, Vicksburg, Miss., March 1985.

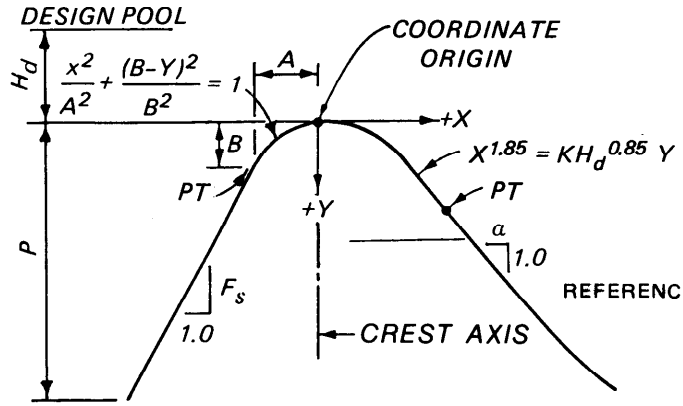
(4) _____, Investigations of Various Shapes of the Upstream Quadrant of the Crest of a High Spillway; Hydraulic Laboratory Investigation, by E. S. Melsheimer and T. E. Murphy. Research Report H-70-1, Vicksburg, Miss., January 1970.

(5) _____, Spillway Crest Design, by T. E. Murphy. Miscellaneous Paper No. H-73-5, Vicksburg, Miss., December 1973.

(6) U. S. Bureau of Reclamation, U. S. Department of the Interior, Boulder Canyon Project, Hydraulic Investigations; Studies of Crests for Overfall Dams, Part VI, Bulletin 3, Denver, Colo., 1948.

(7) U. S. Bureau of Reclamation, U. S. Department of the Interior, Design of Small Dams, Washington, D. C., 1973.





REFERENCE: WES, SPILLWAY CREST DESIGN, BY T.E. MURPHY, MISCELLANEOUS PAPER H-73-5, DECEMBER 1973 (REFERENCE 5).

DEFINITION SKETCH

H_d = TOTAL HEAD

F_s = UPSTREAM FACE SLOPE

ELLIPTICAL CREST SPILLWAY COORDINATES COORDINATE COEFFICIENTS

HYDRAULIC DESIGN CHART 111-20

U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

JOB GS 801 PROJECT JOHN DOE SPILLWAY SUBJECT ELLIPTICAL DESIGN

COMPUTATION SPILLWAY DESIGN

COMPUTED BY AJR DATE 10/26/82 CHECKED BY BJB DATE 11/20/82

GIVEN:

APPROACH DEPTH P = 15 FT

DESIGN HEAD H_d = 25 FT

4V ON 1H UPSTREAM FACE

2V ON 3H DOWNSTREAM FACE

REQUIRED:

SPILLWAY SHAPE WITH ELLIPTICAL UPSTREAM QUADRANT

COMPUTE:

1. COORDINATE COEFFICIENTS

$P/H_d = 15/25 = 0.6$

FROM CHART 111-20 $A/H_d = 0.25$ $B/H_d = 0.146$ $K = 2.04$

$A = 6.25$ $B = 3.65$

$F_s = V/H = 4/1 = 4.0$

$\alpha = H/V = 3/2 = 1.5$

2. TANGENT POINTS

UPSTREAM:

$$X = -\frac{A^2 F_s}{(A^2 F_s^2 + B^2)^{1/2}} = -\frac{(6.25)^2(4.0)}{[(6.25)^2(4)^2 + (3.65)^2]^{1/2}} = -6.184 \text{ FT}$$

$$Y = B - \frac{B^2}{(A^2 F_s^2 + B^2)^{1/2}} = 3.65 - \frac{(3.65)^2}{[(6.25)^2(4)^2 + (3.65)^2]^{1/2}} = 3.123 \text{ FT}$$

DOWNSTREAM:

$$X = \left(\frac{K}{1.85\alpha}\right)^{1/0.85} H_d = \left(\frac{2.04}{(1.85)(1.5)}\right)^{1/0.85} (25) = 17.41 \text{ FT}$$

$$Y = \frac{1}{K}\left(\frac{X}{H_d}\right)^{1.85} = \frac{1}{2.04}\left(\frac{17.41}{25}\right)^{1.85} = 6.27 \text{ FT}$$

3. EQUATIONS:

UPSTREAM:

$$Y = B\left[1 - \left(1 - \frac{X^2}{A^2}\right)^{1/2}\right] = 3.65\left[1 - \left(1 - \frac{X^2}{6.25^2}\right)^{1/2}\right]$$

DOWNSTREAM:

$$Y = \frac{X^{1.85}}{K H_d^{0.85}} = \frac{X^{1.85}}{(2.04)(25)^{0.85}} = \frac{X^{1.85}}{31.469}$$

NOTE: ATTACH UP- AND DOWNSTREAM FACES AT TANGENT POINTS. Y POSITIVE DOWNWARD.

ELLIPTICAL CREST SPILLWAY
COORDINATES
CREST DESIGN

HYDRAULIC DESIGN CHART 111-20/1

HYDRAULIC DESIGN CRITERIA

SHEETS 111-21 TO 111-21/1

ELLIPTICAL CREST SPILLWAY

DISCHARGE COEFFICIENTS

1. General. Discharge over an uncontrolled spillway crest is computed using the equation

$$Q = CL\,H_e^{3/2}$$

where

Q = total discharge, cfs

C = discharge coefficient (Chart 111-21 and 111-21/1)

L = effective crest length, ft (Hydraulic Design Sheet 111-3/1)

H_e = energy head on crest, ft

2. Test Data. Tests were conducted at the U. S. Army Engineer Waterways Experiment Station (WES) in 1970 (reference 1) and 1977-1980 (reference 2). The later set of tests was conducted in a 2.5-ft-wide flume with a design head of 0.8 ft. Because of possible Reynolds number effects, measurements were not made for H_e less than about 0.3 ft ($H_e/H_d = 0.4$, where H_d is the design energy head on crest in feet). Instead, curves were extrapolated to the theoretical value of about $C = 3.09$, which is the coefficient for critical flow over a broad-crested weir (reference 3). For $P/H_d = 0.25$, where P is the crest height above approach channel invert in feet, measurements were difficult above $H_e/H_d = 1.0$ due to upstream turbulence associated with a Froude number near 1.0. The kinetic energy correction factor α was assumed to be 1.0 for all tests.

3. Two separate plots of the three variable groupings involved are provided for ease in interpolation. The curve marked 3.4 on the C vs H_e/H_d plot in Chart 111-21 is identical with the curve for high overflow dams presented in Chart 111-3. It can be considered to be the limit above which the relative spillway height has little or no influence.

4. Application. Although there is a general tendency for flatter upstream slopes to result in lower discharge coefficients, elliptical crest data for all upstream slopes fall into two distinct envelopes: slopes less than 1V on 1H and slopes greater than 1V on 1H. See

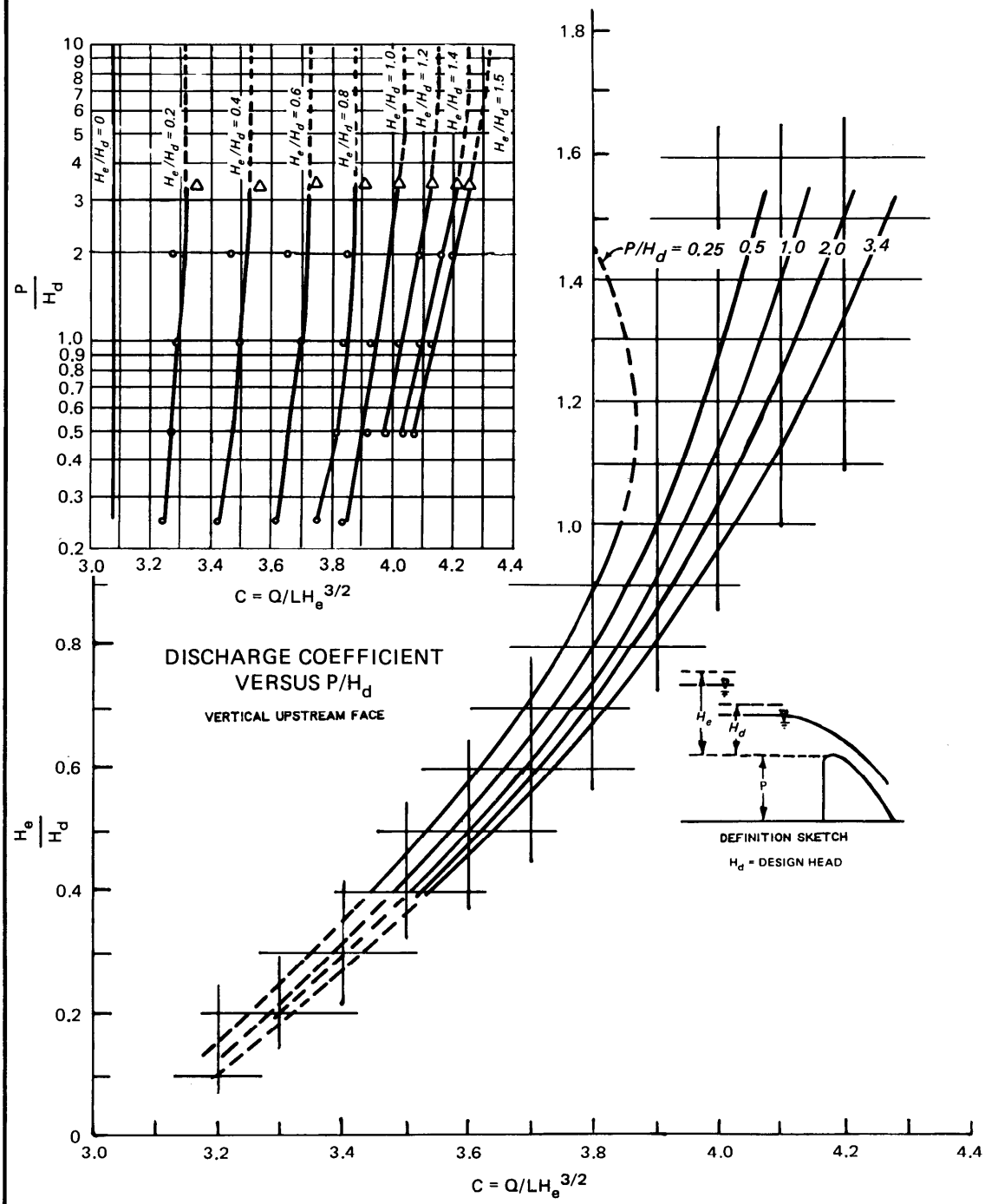
Sheets 111-20 to 111-20/1 for a discussion of slopes used. It is recommended that the two families of curves shown in Charts 111-21 and 111-21/1 be used directly for slopes greater or less than 1V on 1H, respectively. Charts 111-21 and 111-21/1 should also be used to develop rating curves in lieu of Chart 111-3/3 which was found not to fit experimental data for elliptical crest spillways. The concept of underdesigning for $H_e > H_d$ is also applicable to elliptical crest spillways since discharge coefficients increase beyond $H_e/H_d = 1.0$, especially for values of P/H_d above 1.0.

5. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Investigations of Various Shapes of the Upstream Quadrant of the Crest of a High Spillway; Hydraulic Laboratory Investigation, by E. S. Melsheimer and T. E. Murphy, Research Report H-70-1, Vicksburg, Miss., January 1970.

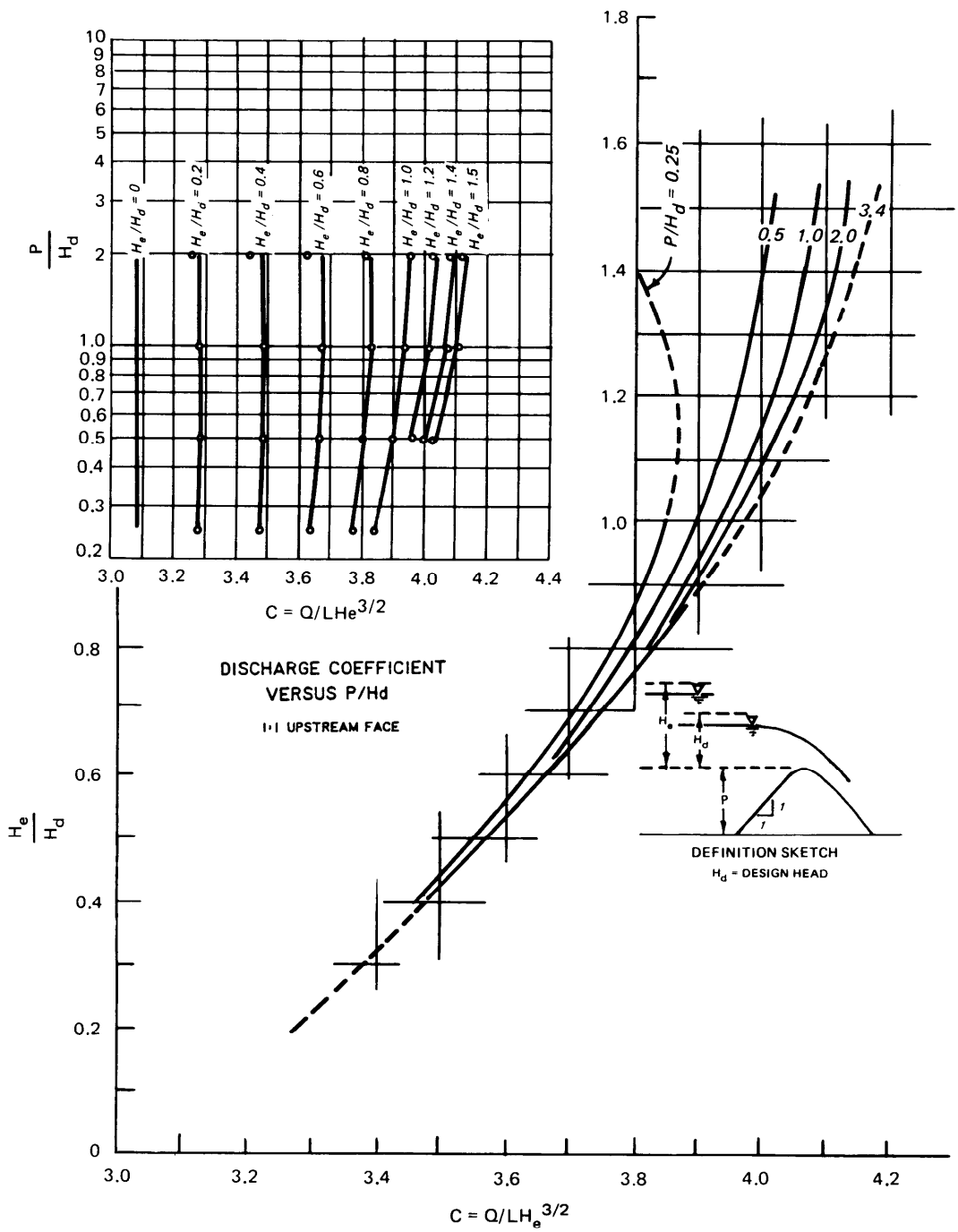
(2) _____, General Spillway Investigation; Hydraulic Model Investigation, by S. T. Maynord, Technical Report HL-85-1, Vicksburg, Miss., March 1985.

(3) Brater, E. F., and King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, 6th ed., McGraw-Hill, New York, N. Y., 1976.



REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2).

ELLIPTICAL CREST SPILLWAY
DISCHARGE COEFFICIENTS
VERTICAL UPSTREAM FACE
HYDRAULIC DESIGN CHART III-21



REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2).

ELLIPTICAL CREST SPILLWAY
DISCHARGE COEFFICIENTS
1:1 UPSTREAM FACE
HYDRAULIC DESIGN CHART III-21/1

GATED ELLIPTICAL CREST SPILLWAY

PIER CONTRACTION COEFFICIENTS

1. Previous Criteria. See Charts 111-5 and 111-6.

2. General. See Sheets 111-20 to 111-20/1 for a description of model studies conducted at the U. S. Army Engineer Waterways Experiment Station (WES) to test elliptical crest spillway design concepts.* Two Type 3 piers were located as shown in Chart 111-22. See Chart 111-5 for a description of Type 3 pier geometry. For a vertical upstream face, the pier nose was located in the same plane as the face of the spillway, and this same horizontal distance was maintained upstream of the axis of the spillway crest for a 1V on 1H face. Measurements were taken for $P/H_d = 0.25$, 0.5 , and 1.0 over a range of H_e/H_d values where P is the crest height above approach channel in feet, H_d is the design energy head on crest in feet, and H_e is the energy head on crest in feet.

3. Application. Chart 111-22 provides an estimate of pier contraction coefficients for an elliptical crest spillway. Curves are drawn for clarity purposes and should be used with caution since coefficients depend greatly on approach conditions. Further data may be necessary to confirm trends and values indicated by this figure.

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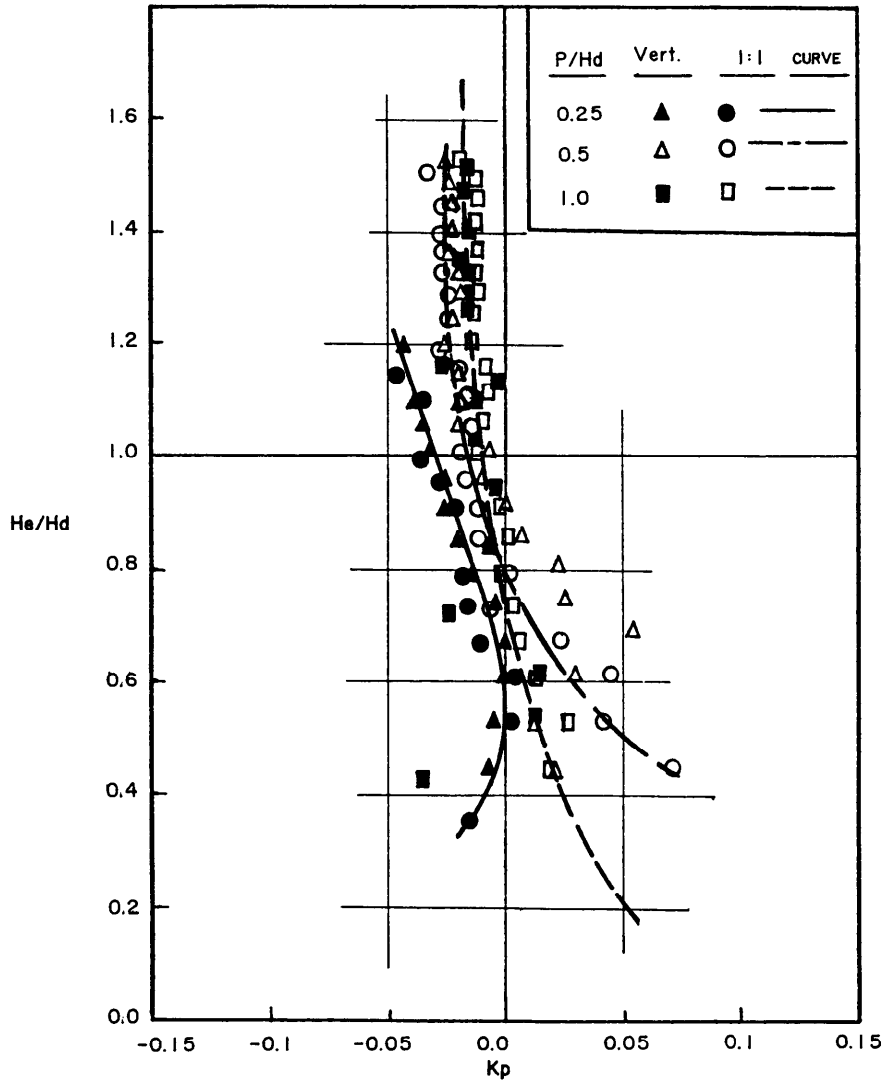
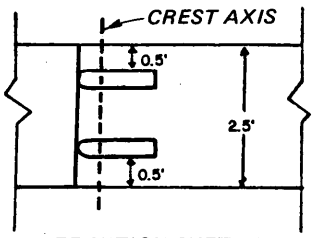
* U. S. Army Engineer Waterways Experiment Station, CE, General Spillway Investigation; Hydraulic Model Investigation, by S. T. Maynord, Technical Report HL-85-1, Vicksburg, Miss., March 1985.





DEFINITION SKETCH
TYPE 3 PIERS (CHART III-5)

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985

ELLIPTICAL CREST SPILLWAY PIER CONTRACTION COEFFICIENTS

HYDRAULIC DESIGN CHART 111-22

HYDRAULIC DESIGN CRITERIA

SHEETS 111-23 TO 111-23/3

ELLIPTICAL CREST SPILLWAY

WATER SURFACE PROFILES

1. General. See Sheets 111-11 to 111-14/1 and 122-3/9 to 122-3/10 for discussion of previous criteria and other available information.

2. Model Tests. See Sheets 111-20 to 111-20/1 for general model test details (reference 2). Tests were conducted for both gated and ungated crests for P/H_d values of 0.25, 0.5, and 1.0 and H_e/H_d values of 0.5, 1.0, and 1.5, where P is the crest height above approach channel in feet, H_d is the design energy head on crest in feet, and H_e is the energy head on crest in feet.

3. Chart 111-23 is applicable for ungated elliptical spillway crest design. Charts 111-23/1 through 111-23/3 depict upper nappe profiles along piers and in gate bay center lines for the three different P/H_d values. See Chart 111-22 for a description of pier placement.

4. Application. Upper nappe profiles were found not to vary significantly with change in upstream face slope over the range of slopes and P/H_d values tested. For values of P/H_d other than those given, P/H_d may be plotted versus Y/H_d for constant X/H_d as demonstrated by Rouse (reference 1). Ungated flow values of X/H_d to about 0.8 and central values of X/H_d for gated flow may be interpolated linearly without significant error.

5. References.

(1) Rouse, H., ed., Engineering Hydraulics, John Wiley and Sons, Inc., New York, N. Y., 1951, p 530.

(2) U. S. Army Engineer Waterways Experiment Station, CE, General Spillway Investigation; Hydraulic Model Investigation, by S. T. Maynord, Technical Report HL-85-1, Vicksburg, Miss., March 1985.

$\frac{P}{H_d} = 0.25$

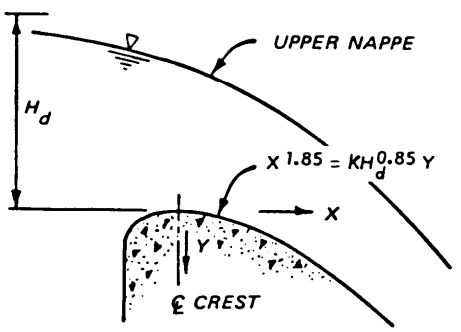
$\frac{H_e}{H_d}$	0.5	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
-1.0	-0.452	-0.768
-0.8	-0.452	-0.759
-0.6	-0.446	-0.750
-0.4	-0.435	-0.735
-0.2	-0.414	-0.712
0.0	-0.378	-0.678
0.2	-0.319	-0.629
0.4	-0.233	-0.550
0.6	-0.120	-0.453
0.8	0.020	-0.331
1.0	0.188	-0.172
1.2	0.375	0.008
1.4	0.578	0.212

$\frac{P}{H_d} = 0.5$

$\frac{H_e}{H_d}$	0.5	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
-1.0	-0.467	-0.849
-0.8	-0.463	-0.840
-0.6	-0.452	-0.822
-0.4	-0.436	-0.796
-0.2	-0.409	-0.764
0.0	-0.365	-0.714
0.2	-0.297	-0.647
0.4	-0.199	-0.557
0.6	-0.076	-0.449
0.8	0.071	-0.307
1.0	0.244	-0.140
1.2	0.445	0.059
1.4	0.661	0.278

$\frac{P}{H_d} = 1.0$

$\frac{H_e}{H_d}$	0.5	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
-1.0	-0.479	-0.897
-0.8	-0.472	-0.879
-0.6	-0.462	-0.857
-0.4	-0.445	-0.829
-0.2	-0.419	-0.792
0.0	-0.377	-0.742
0.2	-0.318	-0.677
0.4	-0.219	-0.579
0.6	-0.102	-0.465
0.8	0.041	-0.328
1.0	0.218	-0.160
1.2	0.412	0.033
1.4	0.629	0.243



SEE CHART 111-20 FOR VALUES OF K.

DEFINITION SKETCH

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

ELLIPTICAL CREST SPILLWAY

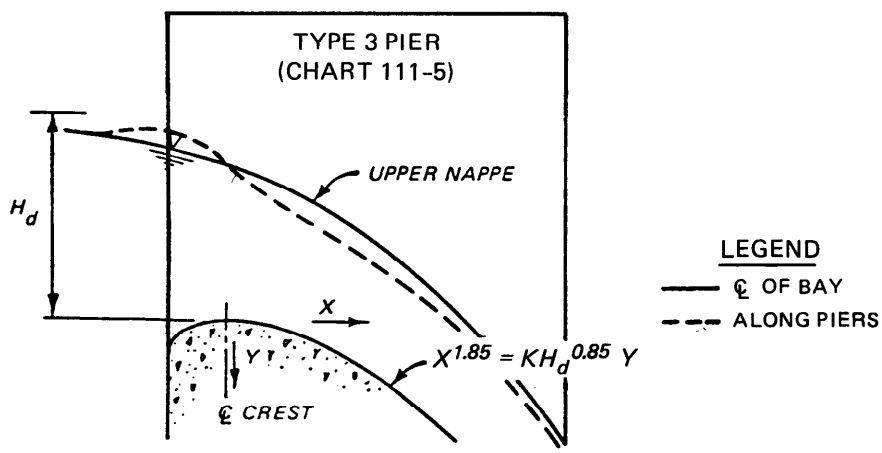
WATER SURFACE PROFILES
UNCONTROLLED CREST
HYDRAULIC DESIGN CHART 111-23

CENTER LINE OF GATE BAY

$\frac{H_e}{H_d}$	0.5	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
-1.0	-0.469	-0.850
-0.8	-0.469	-0.848
-0.6	-0.464	-0.839
-0.4	-0.454	-0.823
-0.2	-0.438	-0.796
0.0	-0.405	-0.758
0.2	-0.358	-0.715
0.4	-0.260	-0.640
0.6	-0.151	-0.553
0.8	-0.018	-0.448
1.0	0.135	-0.303
1.2	0.315	-0.135
1.4	0.528	+0.045

ALONG PIERS

$\frac{H_e}{H_d}$	0.5	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
-1.0	-0.469	-0.838
-0.8	-0.469	-0.835
-0.6	-0.466	-0.833
-0.4	-0.469	-0.835
-0.2	-0.488	-0.894
0.0	-0.414	-0.900
0.2	-0.286	-0.756
0.4	-0.175	-0.615
0.6	-0.066	-0.471
0.8	+0.061	-0.311
1.0	+0.209	-0.139
1.2	+0.378	+0.044
1.4	+0.577	+0.250



DEFINITION SKETCH

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

WATER SURFACE PROFILES
CONTROLLED CREST
$P/H_d = 0.25$

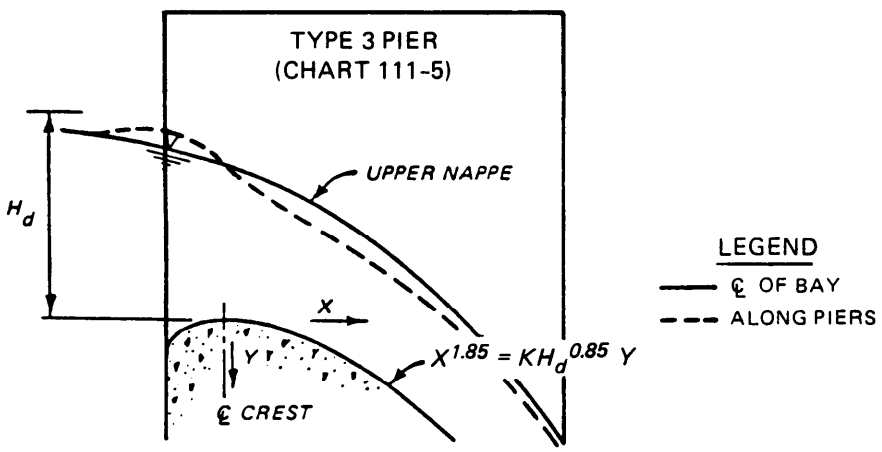
HYDRAULIC DESIGN CHART 111-23/1

CENTER LINE OF GATE BAY

$\frac{H_e}{H_d}$	0.5	1.0	1.5
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.483	-0.894	-1.254
-0.8	-0.479	-0.886	-1.244
-0.6	-0.471	-0.871	-1.229
-0.4	-0.454	-0.851	-1.208
-0.2	-0.429	-0.824	-1.183
0.0	-0.388	-0.783	-1.149
0.2	-0.329	-0.728	-1.099
0.4	-0.241	-0.655	-1.034
0.6	-0.123	-0.570	-0.951
0.8	+0.019	-0.458	-0.856
1.0	+0.198	-0.300	-0.753
1.2	+0.394	-0.104	-0.631
1.4	+0.613	+0.119	-0.426

ALONG PIERS

$\frac{H_e}{H_d}$	0.5	1.0	1.5
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.483	-0.889	-1.257
-0.8	-0.481	-0.880	-1.248
-0.6	-0.477	-0.869	-1.233
-0.4	-0.480	-0.880	-1.243
-0.2	-0.467	-0.917	-1.338
-0.15	-0.450	-0.910	-1.373
0.0	-0.356	-0.825	-1.324
0.2	-0.252	-0.677	-1.176
0.4	-0.159	-0.541	-1.029
0.6	-0.055	-0.414	-0.885
0.8	0.081	-0.258	-0.735
1.0	0.256	-0.089	-0.566
1.2	0.477	0.105	-0.383
1.4	0.672	0.319	-0.188



DEFINITION SKETCH

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

ELLIPTICAL CREST SPILLWAY
WATER SURFACE PROFILES
CONTROLLED CREST
$P/H_d = 0.5$

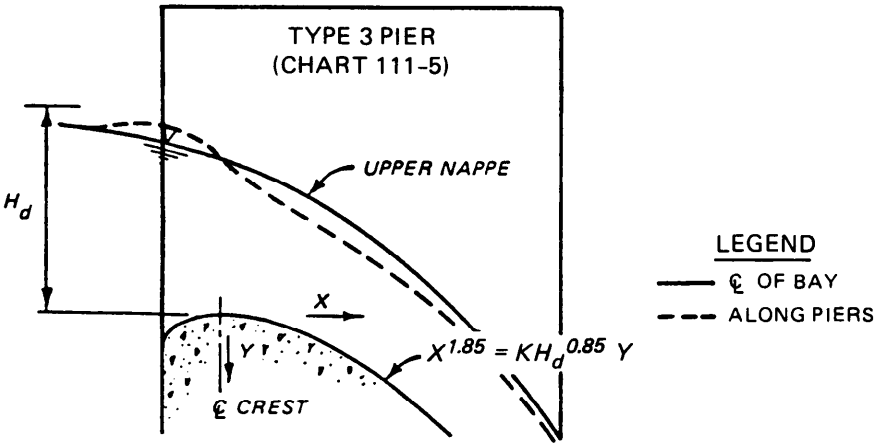
HYDRAULIC DESIGN CHART 111-23/2

CENTER LINE OF GATE BAY

$\frac{H_e}{H_d}$	0.5	1.0	1.5
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.494	-0.939	-1.311
-0.8	-0.488	-0.925	-1.300
-0.6	-0.483	-0.913	-1.275
-0.4	-0.476	-0.888	-1.248
-0.2	-0.445	-0.855	-1.210
0.0	-0.403	-0.808	-1.162
0.2	-0.335	-0.743	-1.102
0.4	-0.240	-0.666	-1.029
0.6	-0.116	-0.573	-0.938
0.8	+0.029	-0.454	-0.833
1.0	0.201	-0.291	-0.707
1.2	0.403	-0.086	-0.562
1.4	0.626	+0.150	-0.395

ALONG PIERS

$\frac{H_e}{H_d}$	0.5	1.0	1.5
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.489	-0.933	-1.311
-0.8	-0.483	-0.925	-1.300
-0.6	-0.475	-0.918	-1.280
-0.4	-0.488	-0.931	-1.313
-0.2	-0.463	-0.935	-1.375
-0.15	-0.438	-0.915	-1.383
0.0	-0.369	-0.829	-1.315
0.2	-0.264	-0.695	-1.171
0.4	-0.170	-0.571	-1.023
0.6	-0.063	-0.441	-0.882
0.8	0.069	-0.298	-0.730
1.0	0.234	-0.128	-0.555
1.2	0.431	0.065	-0.362
1.4	0.651	0.278	-0.140



DEFINITION SKETCH

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

ELLIPTICAL CREST SPILLWAY
WATER SURFACE PROFILES
CONTROLLED CREST
$P/H_d = 1.0$

HYDRAULIC DESIGN CHART 111-23/3

HYDRAULIC DESIGN CRITERIA

SHEETS 111-24 TO 111-24/10

ELLIPTICAL CREST SPILLWAY

SPILLWAY CREST PRESSURES

1. Hydraulic Design Charts. Charts 111-24 to 111-24/8 present plots of crest pressures for H_e/H_d values of 0.50, 1.00, 1.17, 1.33, and 1.50 and P/H_d values of 3.4, 1.0, 0.5, and 0.25 for crests with and without piers, where P is the crest height above approach channel in feet, H_d is the design energy head on crest in feet, and H_e is the energy head on crest in feet. Piers for $P/H_d = 3.4$ were Type 3A (see Chart 111-5) with the test arrangement depicted in the insets of Charts 111-24/7 and 111-24/8 (reference 3). Piers for all other P/H_d values were Type 3 with the same test configuration except flume width equalled 2.5 ft (reference 2).

2. Application. These charts apply to spillways with and without piers over the given range of P/H_d values which are designed in accordance with references 2 and 4 and Chart 111-20. Pressures for intermediate head ratios can be obtained by plotting H/H_d versus H_e/H_d for a given X/H_d .

3. It is recommended that spillway design head H_d be selected so that minimum crest pressure for the maximum expected head is less than -20 feet of water to ensure cavitation-free operation and avoid possible pulsating and inefficient spillway operation (reference 1) (see Chart 111-25). Chart 111-24/9 provides a suggested minimum allowable pressure design curve for an elliptical spillway crest without piers. It was constructed using maximum negative pressures for P/H_d values of 0.5, 1.0, and 3.4 for vertical and 1V on 1H upstream faces. Chart 111-24/10 provides a data summary curve for the case of an elliptical spillway crest with piers designed as described in paragraph 1 above. Note that maximum negative pressures along the pier control the design head limit in all spillway crest designs with piers.

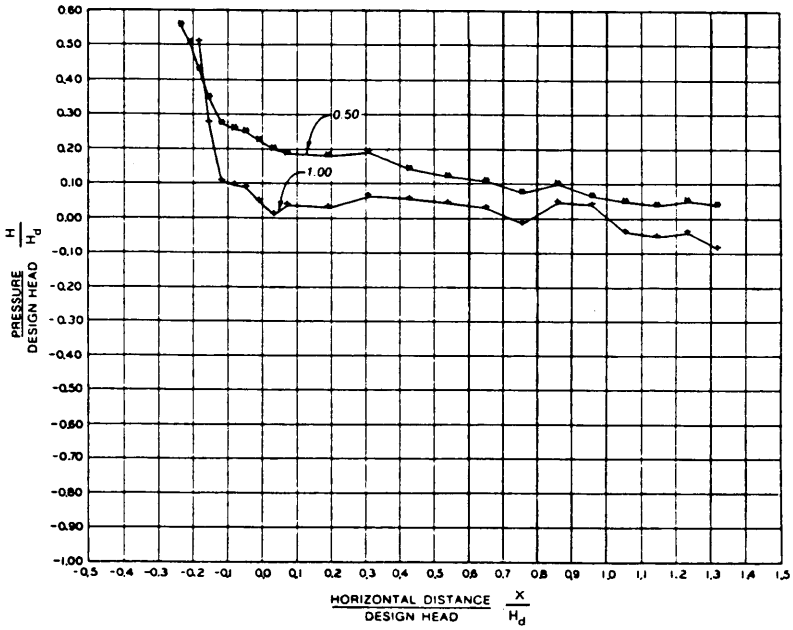
4. References.

(1) Bauer, W., and Beck, E., Handbook of Applied Hydraulics, McGraw-Hill, 1969, Section 20.

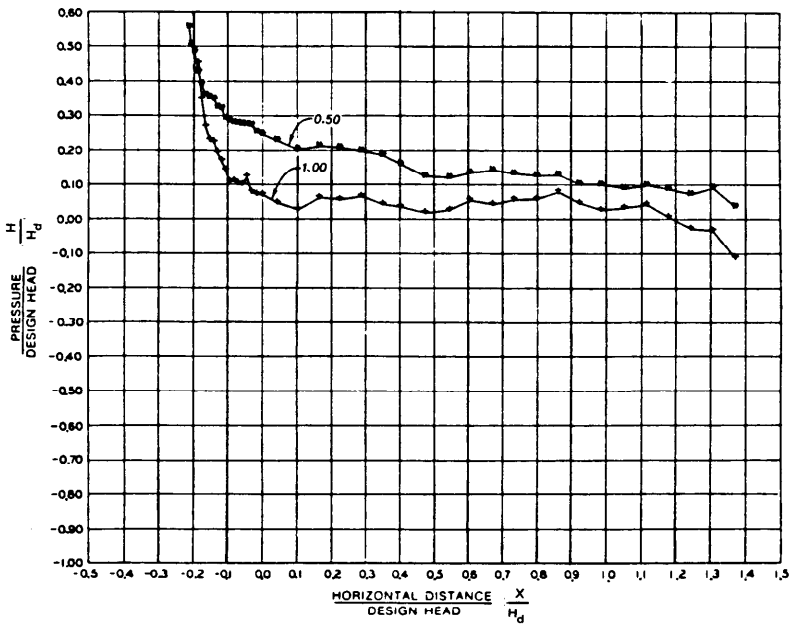
(2) U. S. Army Engineer Waterways Experiment Station, CE, General Spillway Investigation; Hydraulic Model Investigation, by S. T. Maynord, Technical Report HL-85-1, Vicksburg, Miss., March 1985.

(3) U. S. Army Engineer Waterways Experiment Station, CE, Investigations of Various Shapes of the Upstream Quadrant of the Crest of a High Spillway; Hydraulic Laboratory Investigation, by E. S. Melsheimer and T. E. Murphy, Research Report H-70-1, Vicksburg, Miss., January 1970.

(4) _____, Spillway Crest Design, by T. E. Murphy, Miscellaneous Paper H-73-5, Vicksburg, Miss., December 1973.



IV ON IH UPSTREAM FACE



VERTICAL UPSTREAM FACE

NOTE NUMBERED CURVES REFER TO VALUES OF H_e/H_d

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

ELLIPTICAL CREST SPILLWAY

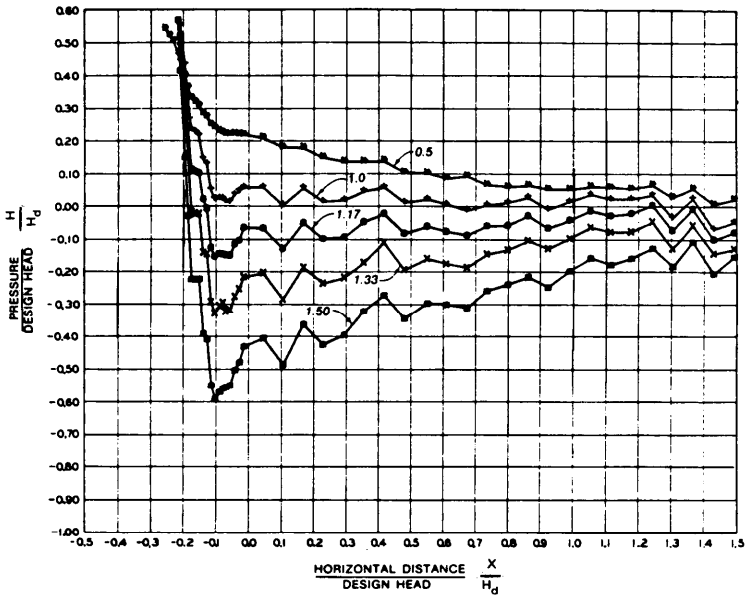
SPILLWAY CREST PRESSURES

NO PIERS

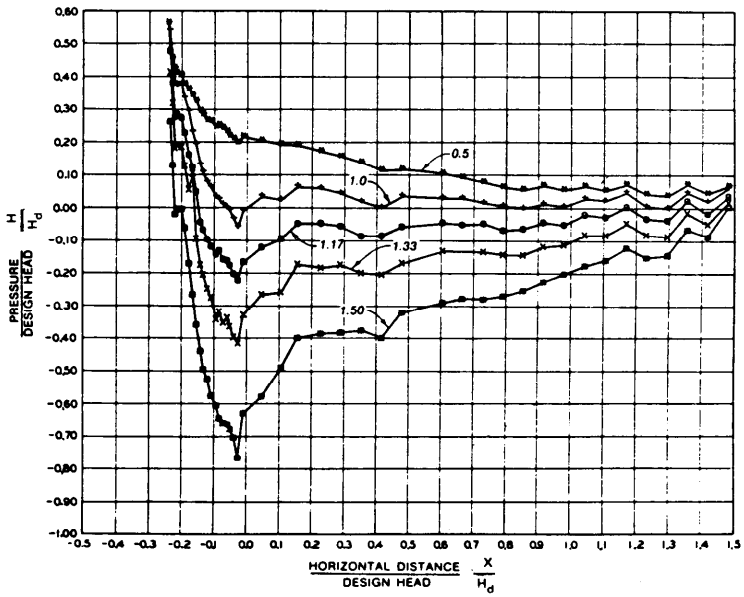
$P/H_d = 0.25$

HYDRAULIC DESIGN CHART 111-24

PREPARED BY U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI WES 11-87



1V ON 1H UPSTREAM FACE



VERTICAL UPSTREAM FACE

NOTE: NUMBERED CURVES REFER TO VALUES OF H_e/H_d

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

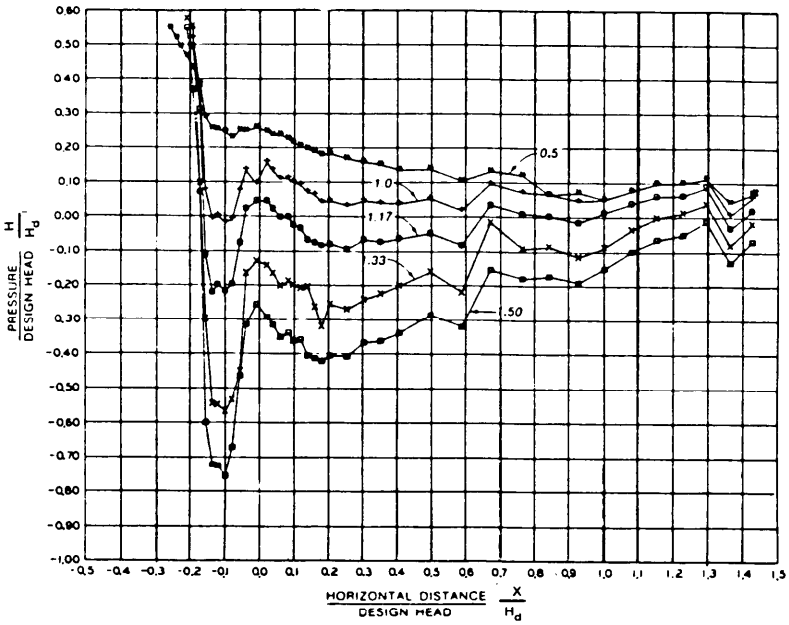
ELLIPTICAL CREST SPILLWAY

SPILLWAY CREST PRESSURES

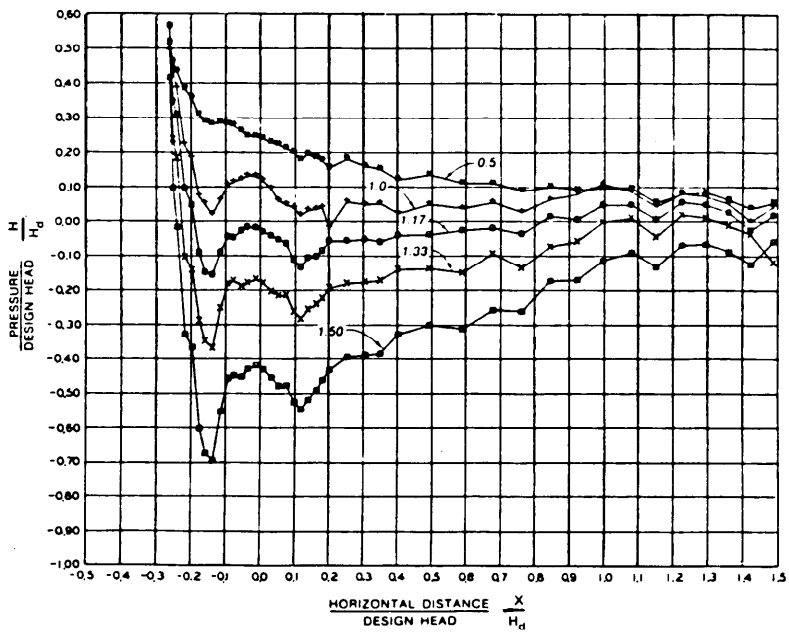
NO PIERS

$P/H_d = 0.5$

HYDRAULIC DESIGN CHART 111-24/1



IV ON IH UPSTREAM FACE



VERTICAL UPSTREAM FACE

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

NOTE: NUMBERED CURVES REFER TO VALUES OF H_e/H_d

ELLIPTICAL CREST SPILLWAY

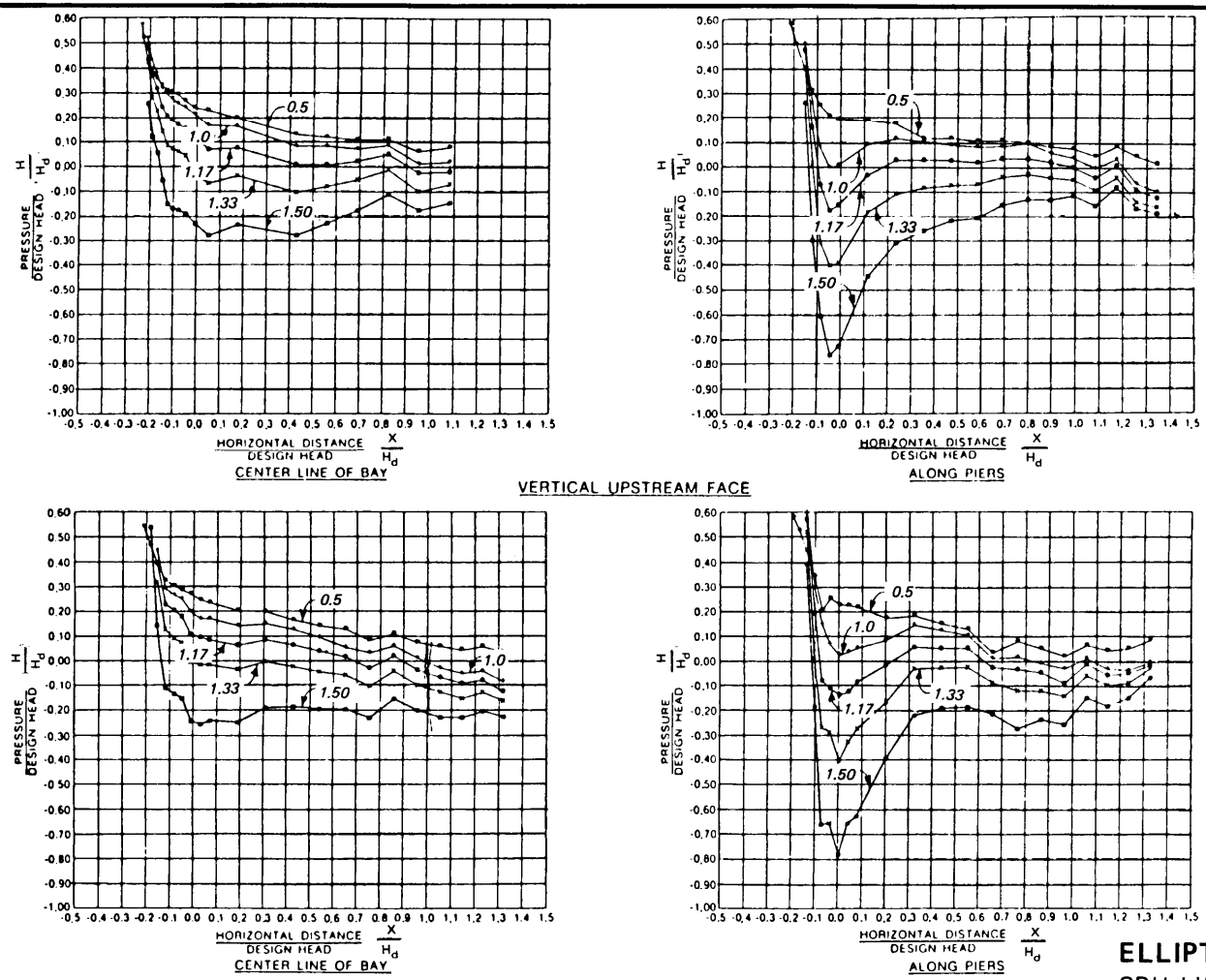
SPILLWAY CREST PRESSURES

NO PIERS

$P/H_d = 1.0$

HYDRAULIC DESIGN CHART 111-24/2

PREPARED BY U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI WES 11-87



VERTICAL UPSTREAM FACE

IV. IH UPSTREAM FACE

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2).

NOTE NUMBERED CURVES REFER TO VALUES OF H_e/H_d.

ELLIPTICAL CREST SPILLWAY

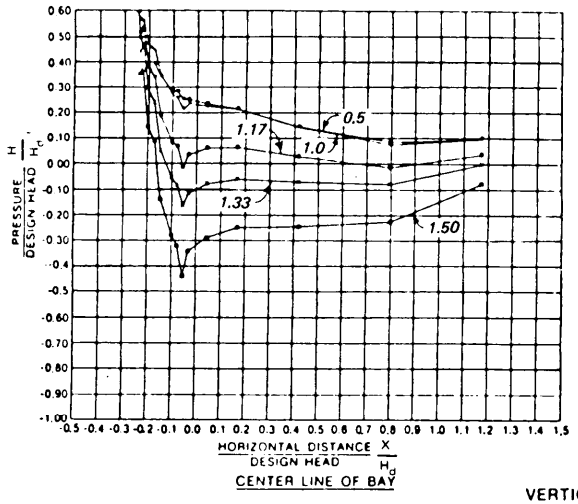
SPILLWAY CREST PRESSURES WITH PIERS

$P/H_d = 0.25$

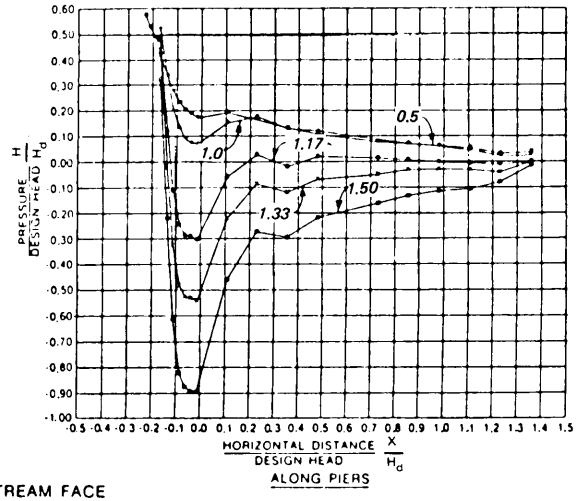
HYDRAULIC DESIGN CHART 111-24/4

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

WES 11-87

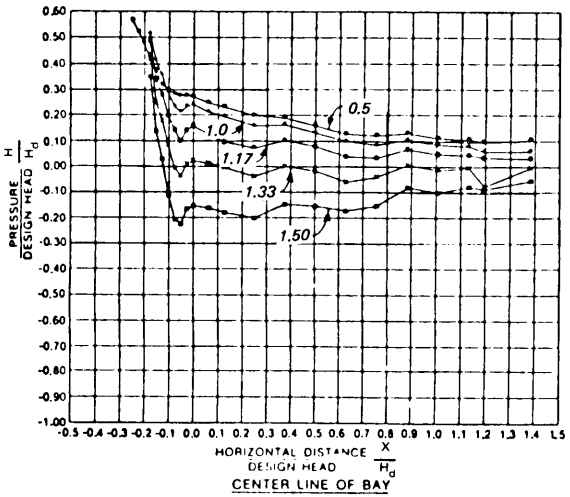


CENTER LINE OF BAY

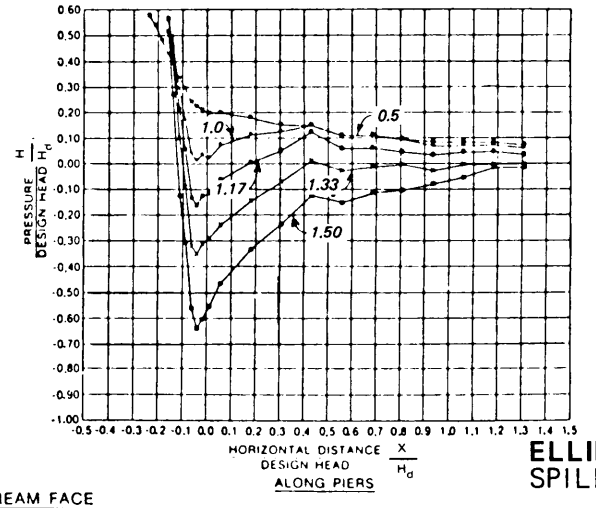


ALONG PIERS

VERTICAL UPSTREAM FACE



CENTER LINE OF BAY



ALONG PIERS

IV: IH UPSTREAM FACE

NOTE NUMBERED CURVES REFER TO VALUES OF H_e/H_d

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

ELLIPTICAL CREST SPILLWAY

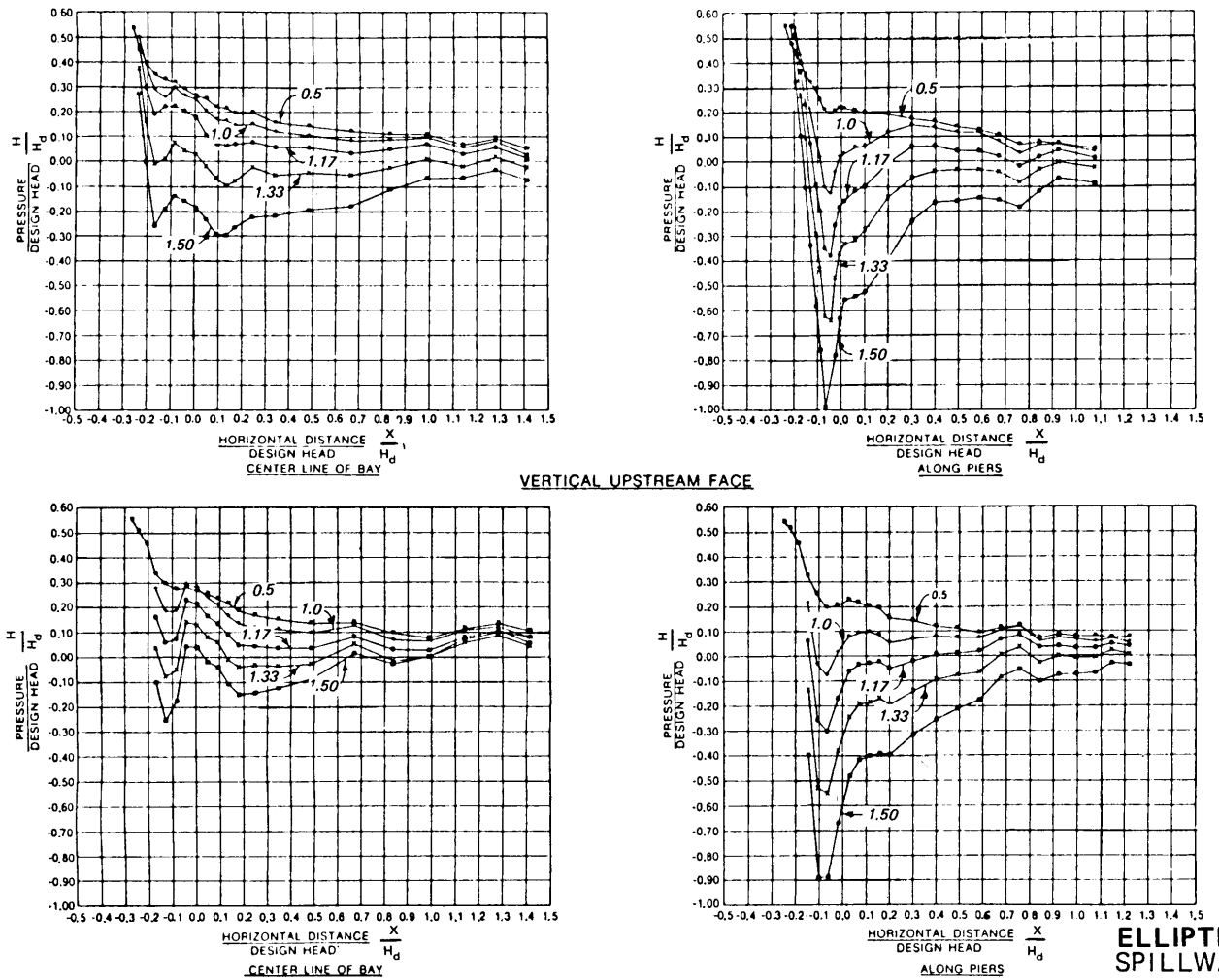
SPILLWAY CREST PRESSURES WITH PIERS

$P/H_d = 0.5$

HYDRAULIC DESIGN CHART 111-24/5

WES 11-87

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



VERTICAL UPSTREAM FACE

IV: IH UPSTREAM FACE

NOTE: NUMBERED CURVES REFER TO VALUES OF H_d/H_d

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2).

ELLIPTICAL CREST SPILLWAY

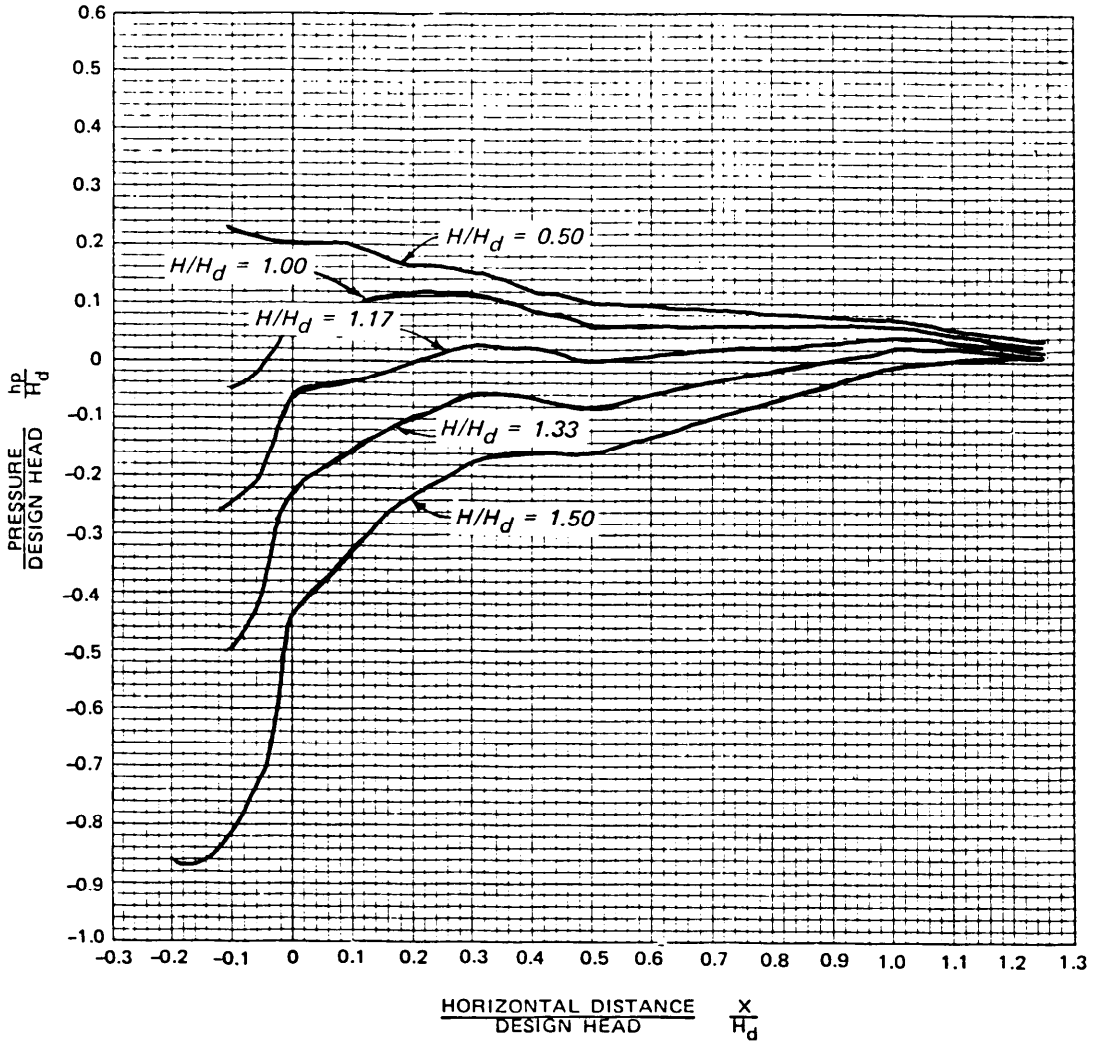
SPILLWAY CREST PRESSURES WITH PIERS

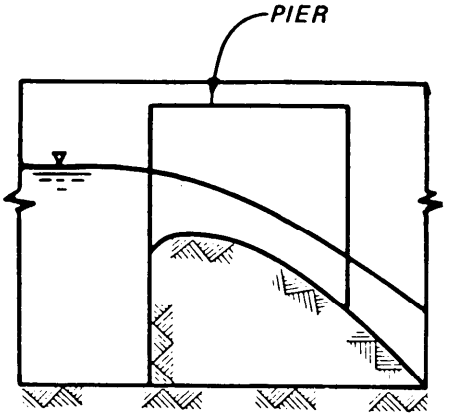
$P/H_d = 1.00$

HYDRAULIC DESIGN CHART 111-24/6

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

WES 11-87





ELEVATION VIEW

REFERENCE: WES, INVESTIGATIONS OF VARIOUS SHAPES OF THE UPSTREAM QUADRANT OF THE CREST OF A HIGH SPILLWAY; HYDRAULIC LABORATORY INVESTIGATION, BY E.S. MELSHEIMER AND T.E. MURPHY, RESEARCH REPORT H-70-1, JANUARY 1970 (REFERENCE 3).

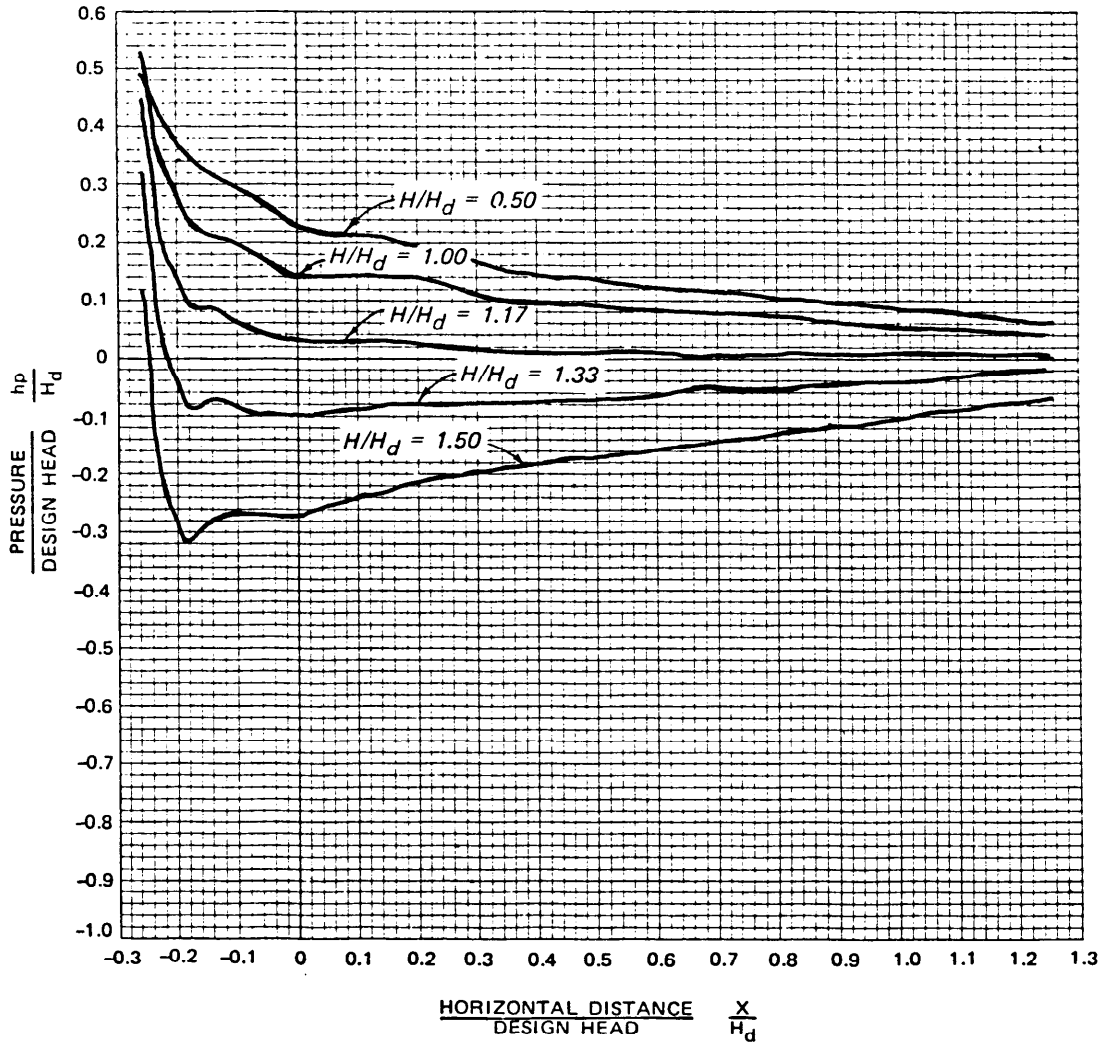
ELLIPTICAL CREST SPILLWAY

SPILLWAY CREST PRESSURES ALONG PIER

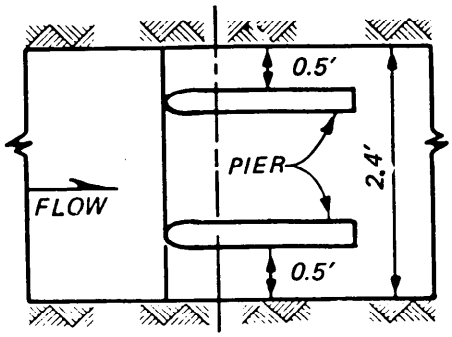
$P/H_d = 3.4$

VERTICAL UPSTREAM FACE

HYDRAULIC DESIGN CHART 111-24/7



REFERENCE: WES, INVESTIGATIONS OF VARIOUS SHAPES OF THE UPSTREAM QUADRANT OF THE CREST OF A HIGH SPILLWAY; HYDRAULIC LABORATORY INVESTIGATION, BY E.S. MELSHEIMER AND T.E. MURPHY, RESEARCH REPORT H-70-1, JANUARY 1970 (REFERENCE 3).



PLAN VIEW

ELLIPTICAL CREST SPILLWAY

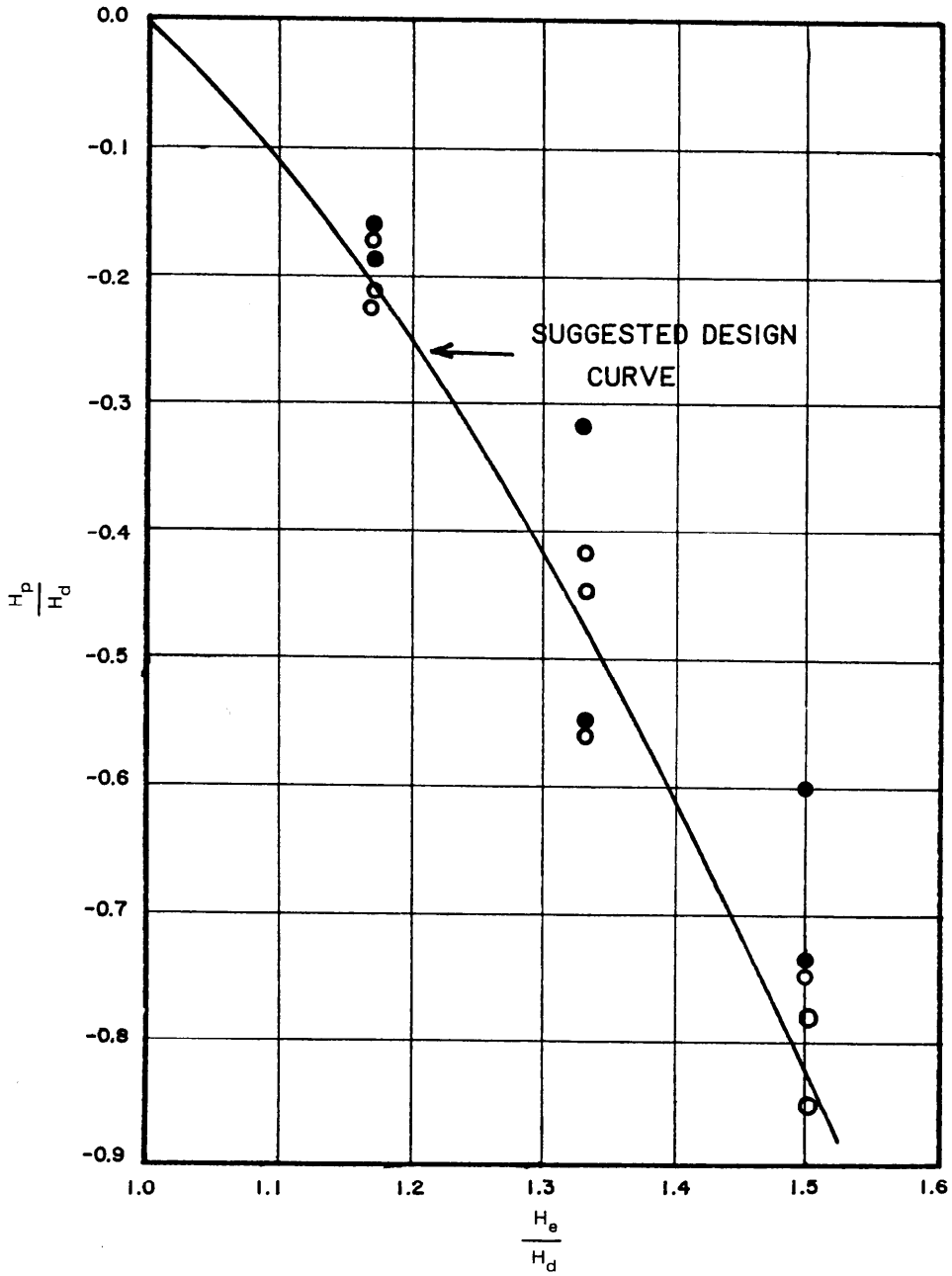
SPILLWAY CREST PRESSURES

CENTER LINE OF GATE BAY

$P/H_d = 3.4$

VERTICAL UPSTREAM FACE

HYDRAULIC DESIGN CHART 111-24/8



LEGEND

○ VERTICAL UPSTREAM FACE

● 1V ON 1H UPSTREAM FACE

H_p = PRESSURE HEAD ON CREST, FT

H_d = DESIGN HEAD, FT

H_e = ENERGY HEAD, FT

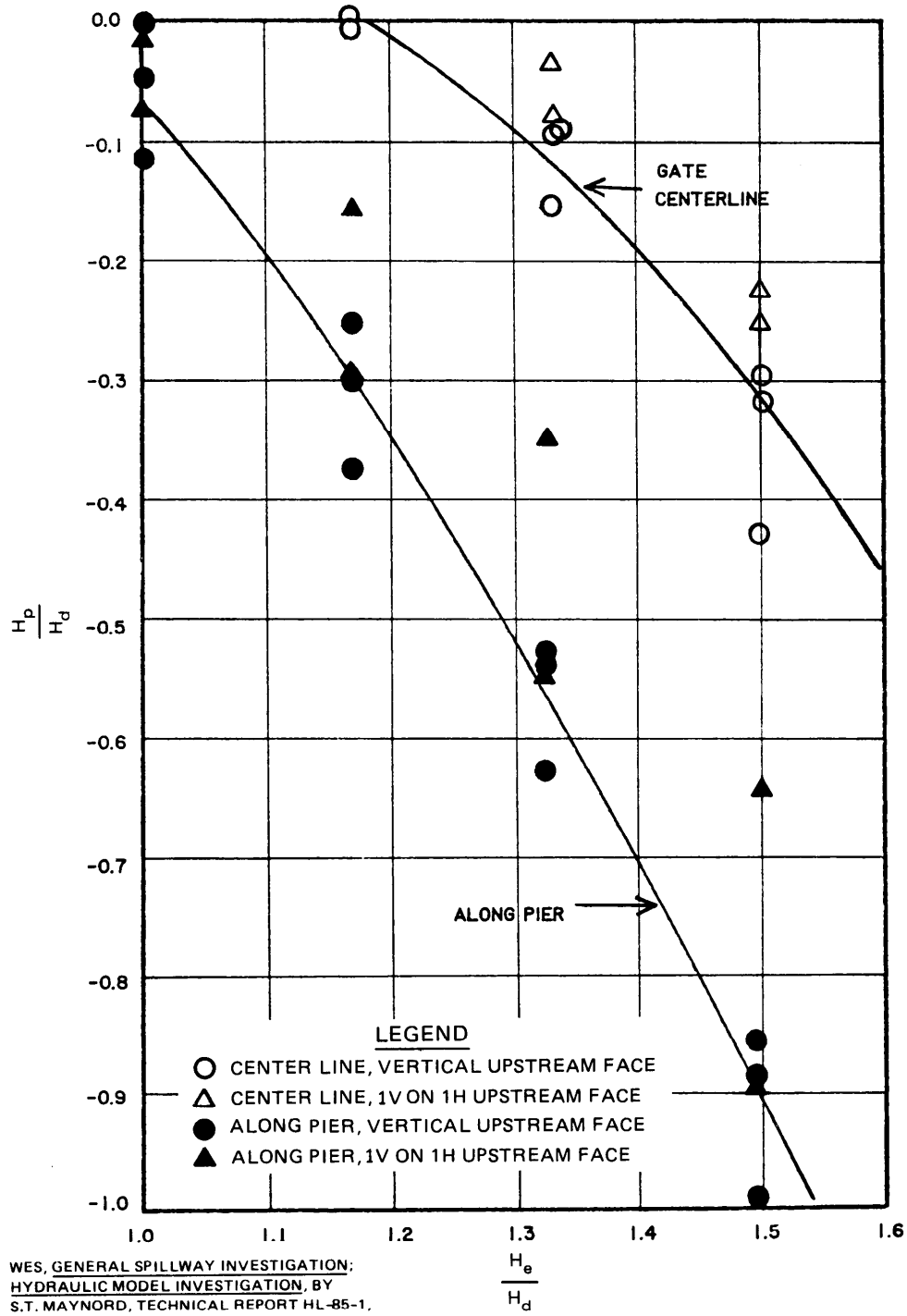
P = AVERAGE APPROACH DEPTH, FT

REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

ELLIPTICAL CREST SPILLWAY MAXIMUM NEGATIVE PRESSURE VS H_e/H_d UNCONTROLLED CREST ($P/H_d \geq 0.5$)

HYDRAULIC DESIGN CHART III - 24/9

PREPARED BY U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI WES 11-87



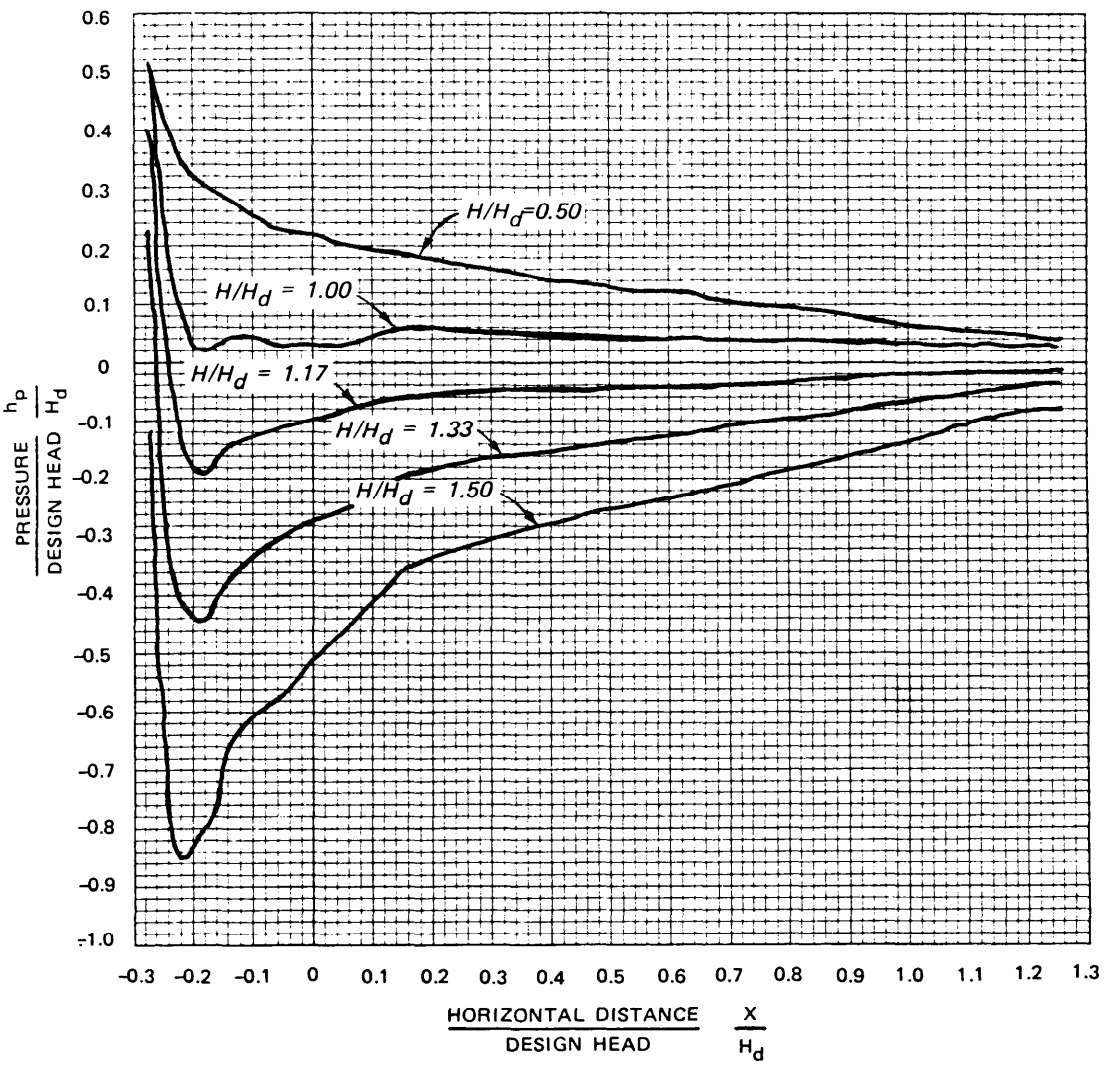
REFERENCE: WES, GENERAL SPILLWAY INVESTIGATION; HYDRAULIC MODEL INVESTIGATION, BY S.T. MAYNORD, TECHNICAL REPORT HL-85-1, MARCH 1985 (REFERENCE 2)

H_p = PRESSURE HEAD ON CREST, FT
H_d = DESIGN HEAD, FT
H_e = ENERGY HEAD, FT
P = AVERAGE APPROACH DEPTH, FT

ELLIPTICAL CREST SPILLWAY
MAXIMUM NEGATIVE PRESSURE VS H_e/H_d
WITH PIERS ($P/H_d > 0.5$)

HYDRAULIC DESIGN CHART 111-24/10

PREPARED BY U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI WES 11-87



REFERENCE: WES, INVESTIGATIONS OF VARIOUS SHAPES OF THE UPSTREAM QUADRANT OF THE CREST OF A HIGH SPILLWAY; HYDRAULIC LABORATORY INVESTIGATION, BY E.S. MELSHEIMER AND T.E. MURPHY, RESEARCH REPORT H-70-1, JANUARY 1970 (REFERENCE 3).

ELLIPTICAL CREST SPILLWAY
SPILLWAY CREST PRESSURES
WITHOUT PIERS
P/H_d = 3.4
VERTICAL UPSTREAM FACE
HYDRAULIC DESIGN CHART 111-24/3

PREPARED BY U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI WES 11-87

HYDRAULIC DESIGN CRITERIA

SHEETS 111-25 TO 111-25/1

ELLIPTICAL CREST SPILLWAY

CAVITATION SAFETY CURVES

1. General. Sheets 111-20 through 111-24/10 describe various design aspects of elliptical crest spillways. For a description of model test data to support these sheets, see reference 2 and Sheets 111-20 to 111-20/1.

2. Cavitation Safety Curves. Charts 111-24/9 and 111-24/10 present plots of the function $H_p/H_d = f(H_e/H_d)$ where H_p is the pressure head, H_d is the design total head on crest, and H_e is the actual total head on crest. This formulation is versatile and can be used for any value of the pressure head H_p. However, vacuum tank observations by Abecasis (reference 1) indicated that cavitation on a spillway crest would be incipient at an average pressure of about -25 feet. For safety purposes, it is recommended that spillway crests be designed so that the maximum expected head will result in a pressure no lower than about -20 feet. Chart 111-25 is taken from the data of Maynord (reference 2) for $H_p = -20$ feet and a crest without piers. Curves of -25 and -15 feet are also presented. The -25-foot curve corresponds to the lower portion of Abecasis' measured envelope of values for incipient cavitation. Chart 111-25/1 presents the same curves for a crest with piers located as described in Charts 111-24 to 111-24/10. These pressures were measured at a point 0.02 foot from the pier edge in a model with a design head H_d of 0.8 foot. The two -20-foot curves can be approximated by the following equations:

(without piers) $$H_d = 0.277(H_e)^{1.2585} \quad (1)$$

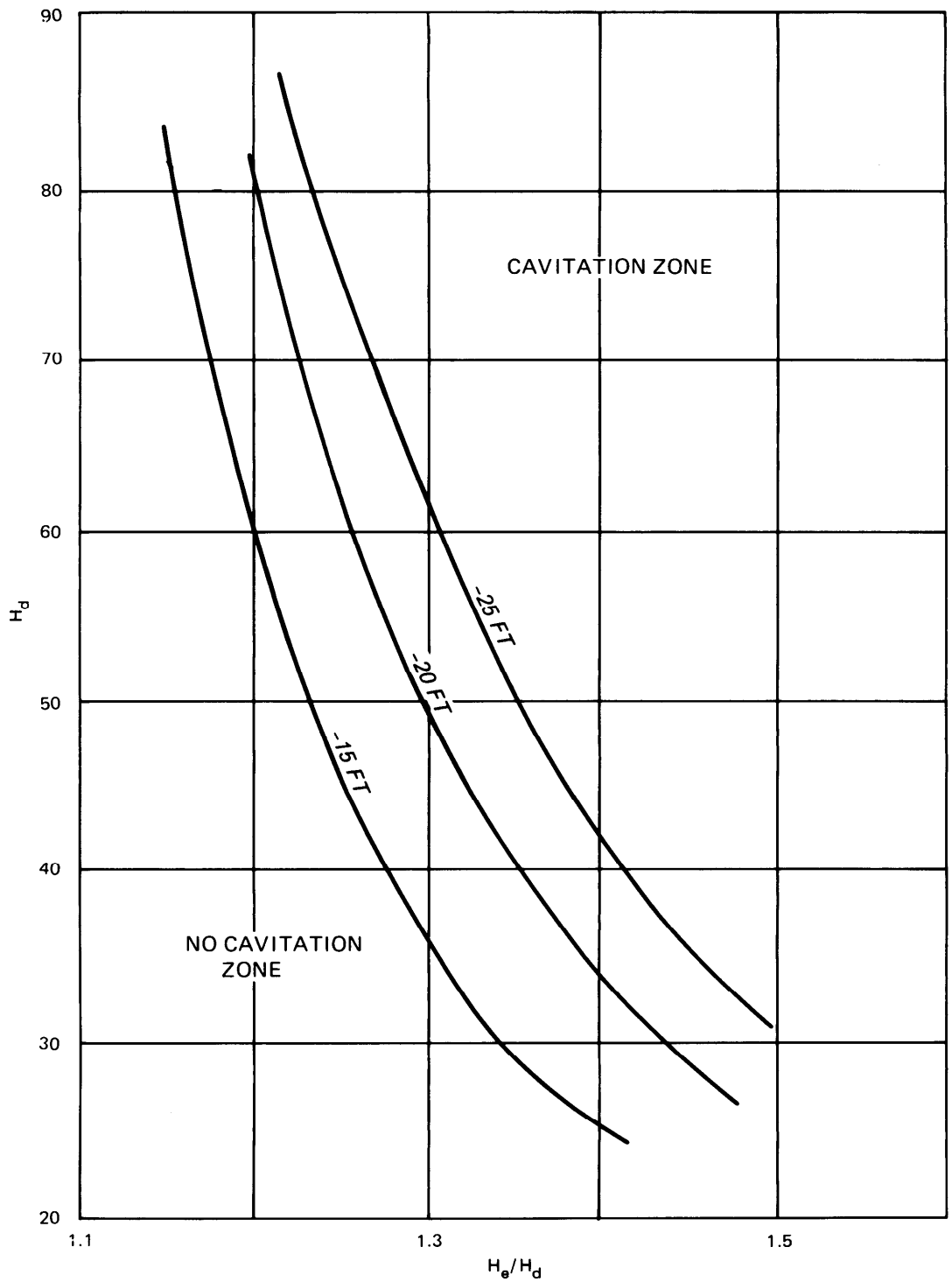
(with piers) $$H_d = 0.309(H_e)^{1.2186} \quad (2)$$

3. Design Examples. Depending on the available data, economics, and other design restrictions, there are many different approaches to the proper sizing of a spillway for preliminary design.

4. References.

(1) Abecasis, F. M., "Discussion of 'Designing spillway crests for high-head operation,'" by J. J. Cassidy. Journal of the Hydraulics Division, American Society of Civil Engineers, vol 96, No. HY12, December, 1970.

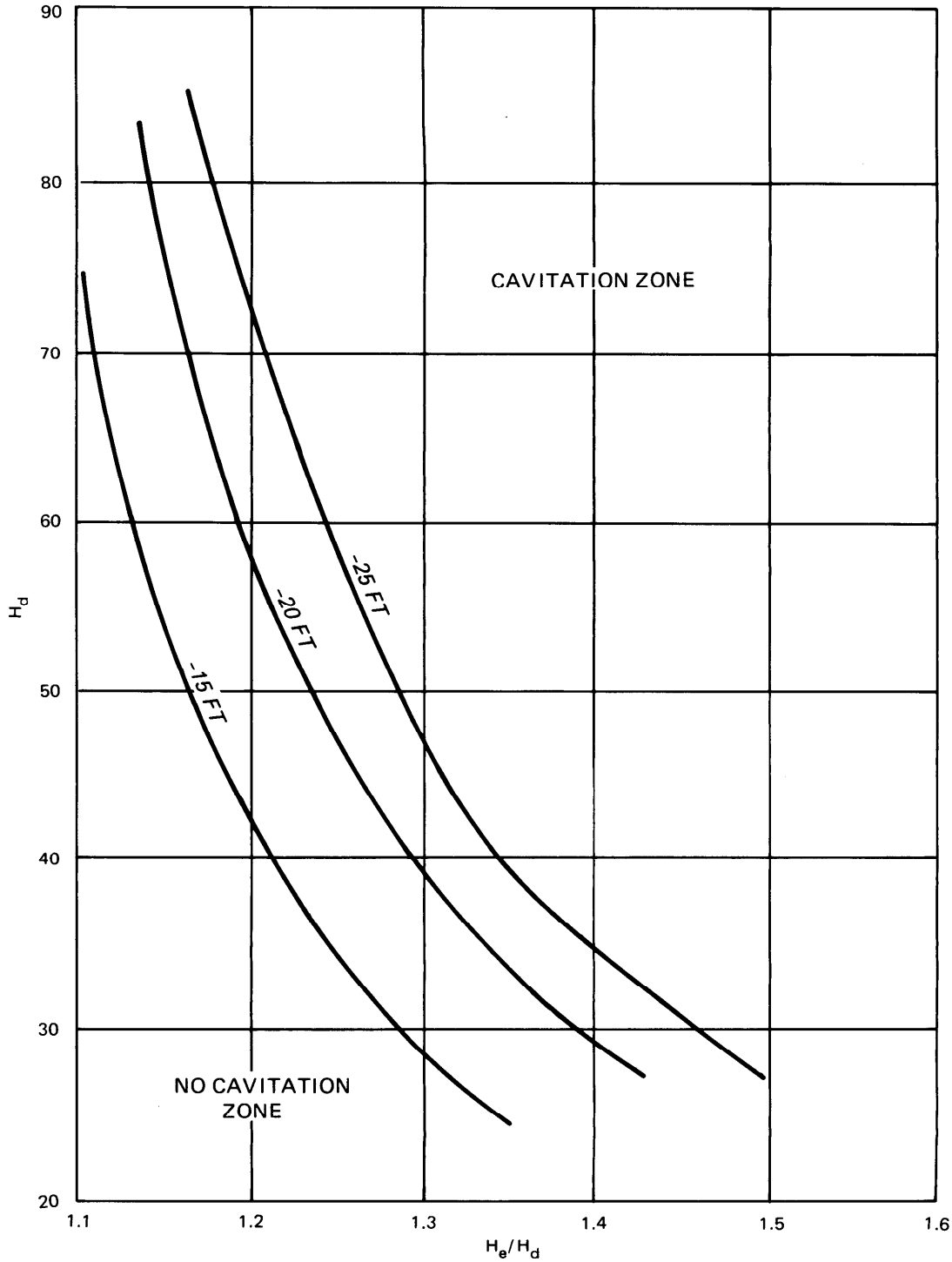
(2) U. S. Army Engineer Waterways Experiment Station, CE, General Spillway Investigation; Hydraulic Model Investigation, by S. T. Maynord, Technical Report HL-85-1, Vicksburg, Miss., March 1985.



NOTE: H_d = DESIGN TOTAL HEAD, FT
H_e = ACTUAL TOTAL HEAD, FT

ELLIPTICAL CREST SPILLWAY
CAVITATION SAFETY CURVES
NO PIERS
HYDRAULIC DESIGN CHART III-25

PREPARED BY U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI WES 11-87



NOTE: H_d = DESIGN TOTAL HEAD, FT
H_e = ACTUAL TOTAL HEAD, FT

ELLIPTICAL CREST SPILLWAY
CAVITATION SAFETY CURVES
WITH PIERS

HYDRAULIC DESIGN CHART III-25/1

PREPARED BY U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI WES 11-87

HYDRAULIC DESIGN CRITERIA

SHEETS 112-1 AND 112-2

SPILLWAY STILLING BASINS

HYDRAULIC JUMP

1. General. The principle of conservation of linear momentum results in the classical hydraulic jump equation

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}}$$

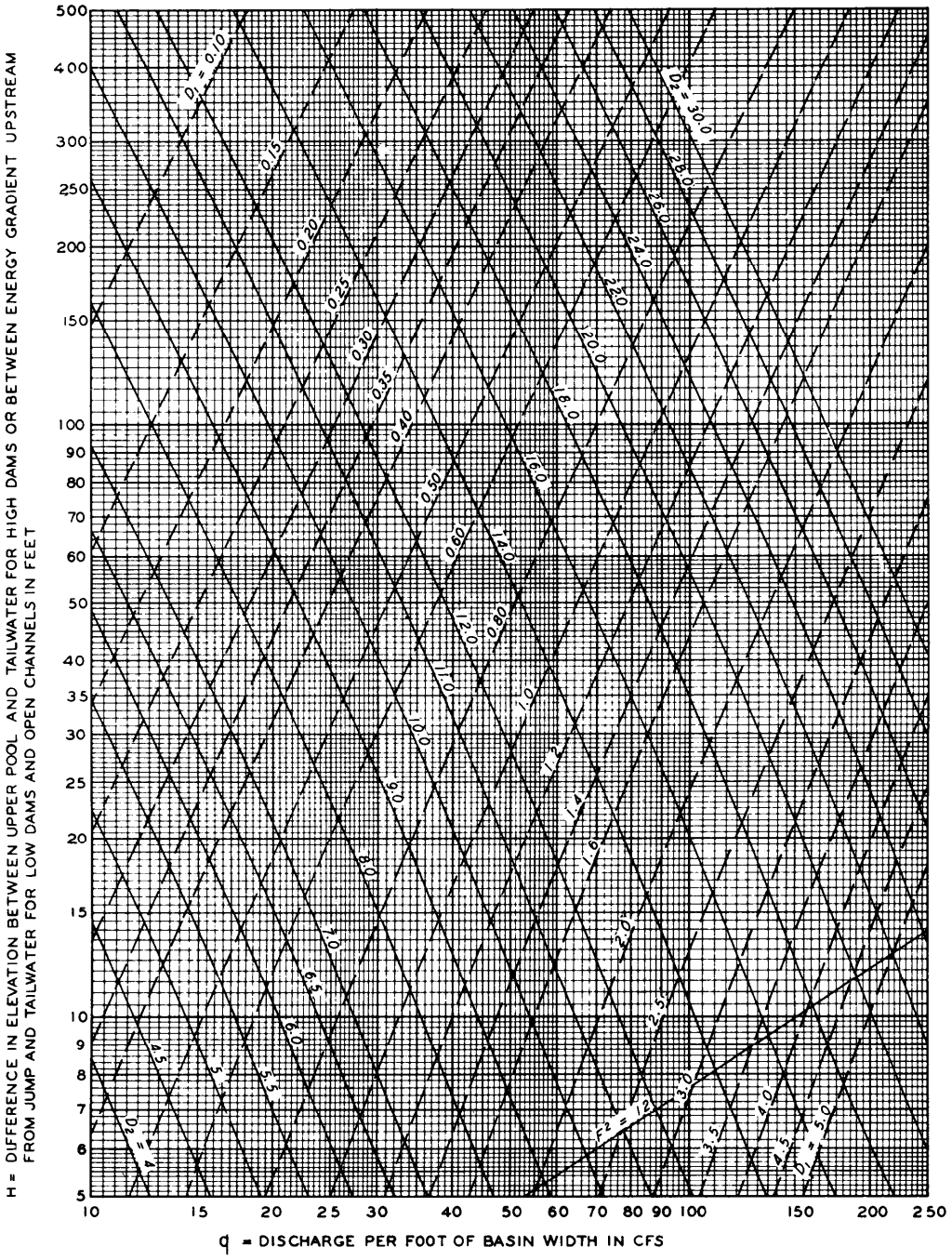
Tables for the evaluation of D_2 may be found in the Corps of Engineers Hydraulic Tables, 2d Edition, 1944.

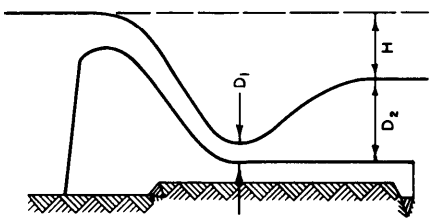
2. Spillway Stilling Basins. The purpose of Hydraulic Design Charts 112-1 and 112-2 is the determination of the elevation of the apron or stilling basin floor when headwater and tailwater elevations are known for a given discharge. The form of the graphs was devised by Irwin.* On each chart families of D_1 and D_2 curves are shown for various discharges per foot of basin width and heads. Chart 112-1 covers a head range of 5 to 500 ft and a discharge range of 10 to 250 cfs. Chart 112-2 covers a similar range of heads but has a discharge range of 100 to 2500 cfs. The head (H) as defined by the sketch at the foot of each chart is the difference between the headwater and tailwater elevations. In the development of these charts friction losses were neglected. Recent experiments at State University of Iowa** for the Waterways Experiment Station indicate that friction losses in accelerating flow down the face of a spillway may be considerably less than the normal friction loss in well-developed turbulent flow. Two lines depicting the Froude number squared ($F^2 = 3$ and $F^2 = 12$) are shown to define the jump characteristics. The line $F^2 = 3$ marks the boundary between jumps of the undular and shock types. Model studies indicate that a strong hydraulic jump forms in the region above the line $F^2 = 12$.

---

* R. L. Irwin, "Diagram for hydraulic jump," Civil Engineering (June 1942), p 335.

** W. J. Bauer, "The Development of the Turbulent Boundary Layer on Steep Slopes," Dissertation, University of Iowa, Iowa City, August 1951.



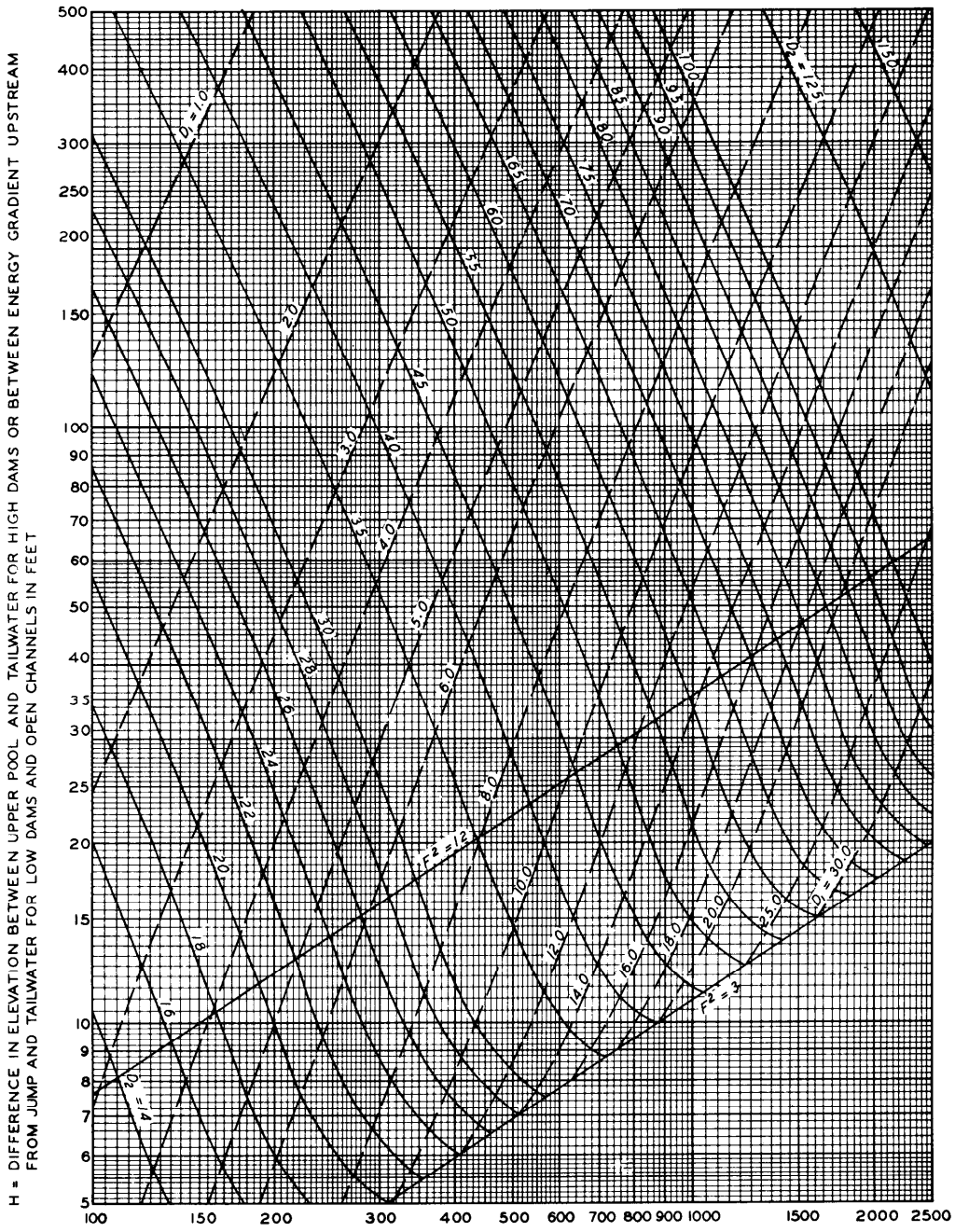


FORMULA: $$H = \frac{2q^2}{gD_2^2\left[\sqrt{\frac{8q^2}{gD_2^3}+1}-1\right]^2} + \frac{D_2}{2}\left[\sqrt{\frac{8q^2}{gD_2^3}+1}-1\right] - D_2$$

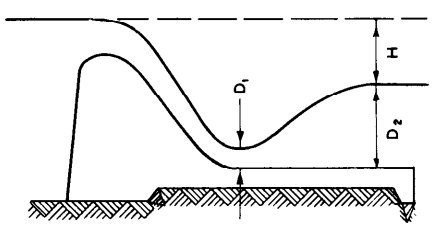
SPILLWAY STILLING BASINS
HYDRAULIC JUMP
$10 < q < 250$

HYDRAULIC DESIGN CHART 112-1

WES 4-1-53



FORMULA: $$H = \frac{2q^2}{gD_2^2\left[\sqrt{\frac{8q^2}{gD_2^3}+1}-1\right]^2} + \frac{D_2}{2}\left[\sqrt{\frac{8q^2}{gD_2^3}+1}-1\right] - D_2$$



SPILLWAY STILLING BASINS

HYDRAULIC JUMP

$100 < q < 2500$

HYDRAULIC DESIGN CHART 112-2

WES 4-1-53

HYDRAULIC DESIGN CRITERIA

SHEET 112-2/1

SPILLWAY STILLING BASINS

HYDRAULIC JUMP

VELOCITY DISTRIBUTION

1. The balance of pressure-plus-momentum upstream and downstream from the jump is the basis for the theoretical equation of the hydraulic jump. This equation is given in HDC 112-3. When an end sill or baffle piers are added there is an additional force in the upstream direction. U. S. Army Corps of Engineers EM 1110-2-1603[8] suggests that a satisfactory jump will occur for $0.9d_2$ by the use of an end sill and baffles although more shallow basins have been demonstrated to perform satisfactorily in the laboratory.

2. If the drag force of baffles is to be estimated, the effective velocity against the baffle and the drag coefficient must be known. The drag coefficient of the isolated cube referred to in HDC 712-1 has been estimated to be 1.33 based on State University of Iowa air tunnel tests.[1] The drag coefficient of a single cube in open channel flow has been computed to be about 1.5.[7] Tests at Massachusetts Institute of Technology[3] indicate that the drag coefficient is about 0.7 for a single row of stepped piers and about 0.4 for a double row of Bluestone Dam type baffle piers.[6]

3. HDC 112-2/1 can be used as a guide in selecting baffle pier height and location as well as in estimating the velocity in the vicinity of the baffles. HDC 112-2/1 presents vertical velocity distribution curves in the hydraulic jump at X/d_2 stations of 1, 2, and 3 from the toe of the jump. The curves result from experimental data by Rouse[4] and Mahoning Dam model study tests.[5] Curves for Froude numbers of entering flow of 2, 3, 4, 6, 8, and 10 are given. The local velocity V is expressed in terms of the entering velocity V_1. The distance y above the floor is in terms of the depth downstream from the jump d_2.

4. The force exerted on the baffle piers and the resulting reduction in total downstream pressure and momentum can be estimated by use of HDC 112-2/1 and an estimated drag coefficient. Available data indicate that the pier spacing as well as pier size and geometry has an effect upon the drag coefficient. The designer is also sometimes concerned with the force exerted on baffle piers by logs and other debris passing through the basin.

5. References.

(1) Chien, Ning, Fing, Yin, Wang, Huang-Ju, and Siao, Tien-To, Wind Tunnel Studies of Pressure Distribution on Elementary Building Forms. Iowa

Institute of Hydraulic Research, for Office of Naval Research, 1951.

(2) Harleman, D. R. F., "Effect of baffle piers on stilling basin performance." Journal, Boston Society of Civil Engineers, vol 42, No. 2 (April 1955), pp 84-99.

(3) Newman, J. B., III, and La Boon, F. A., Effects of Baffle Piers on the Hydraulic Jump. M.S. thesis, Massachusetts Institute of Technology, 1953.

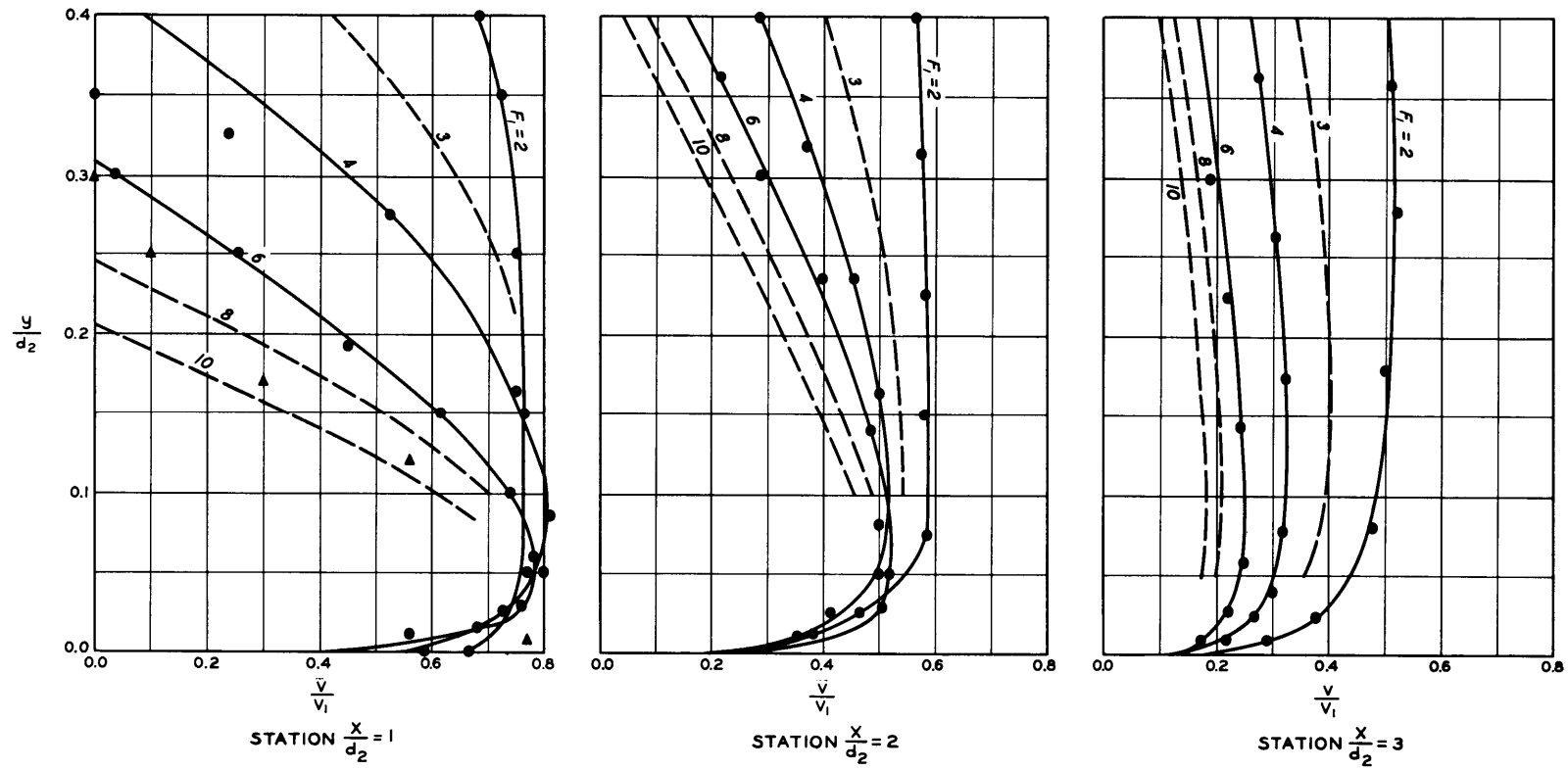
(4) Rouse, H., Siao, Tien-To, and Nagaratnam, S., "Turbulence characteristics of the hydraulic jump." Transactions, American Society of Civil Engineers, vol 124 (1959), pp 926-966.

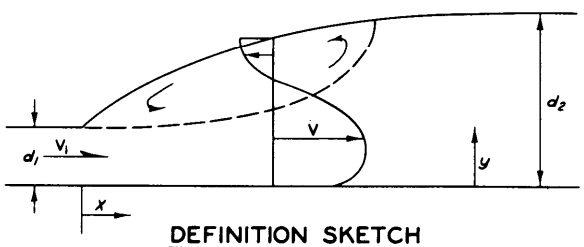
(5) U. S. Army Engineer District, Pittsburgh, CE, Report on Hydraulic Model Studies on the Spillway and Outlet Works of Mahoning Dam, on Mahoning Creek, near Punxsutawney, Pennsylvania. Case Institute of Technology, May 1938.

(6) U. S. Army Engineer Waterways Experiment Station, CE, A Laboratory Development of Cavitation-free Baffle Piers, Bluestone Dam, New River, West Virginia. Technical Memorandum No. 2-243, Vicksburg, Miss., March 1948.

(7) _____, unpublished data. 1957.

(8) U. S. Army, Office, Chief of Engineers, Hydraulic Design, Spillways. EM 1110-2-1603, March 1953.





DEFINITION SKETCH

LEGEND

SYMBOL	SOURCE	FROUDE NO. (F_1)
●——	ROUSE, SIAO, NAGARATNAM	2, 4, 6
▲	MAHONING MODEL	9
— — —	INTERPOLATED	3, 8, 10

SPILLWAY STILLING BASINS

HYDRAULIC JUMP VELOCITY DISTRIBUTION

HYDRAULIC DESIGN CHART 112-2/1

CHART 112-2/1

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

WES 10-61

HYDRAULIC DESIGN CRITERIA

SHEETS 112-3 TO 112-5

SPILLWAY STILLING BASINS

SEQUENT DEPTH CURVES FOR RECTANGULAR CHANNEL

1. The conventional hydraulic jump equation is based on the principles of conservation of momentum and continuity of flow. The equation is

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2\,V_1^{\,2}\,D_1}{g} + \frac{D_1^{\,2}}{4}}$$

where D_1 and D_2 are sequent depths upstream and downstream, respectively, from the jump. V_1 and V_2 are the corresponding sequent velocities.

2. Tables have been published by King(3) and the Corps of Engineers(6) that give the D_2 value when the D_1 and V_1 values are known.

3. A log-log graph, devised by E. W. Lane(5) and published by the National Resources Committee(4), is given as Chart 112-3. It may be used to determine D_2 and V_2 when D_1 and V_1 are known.

4. A graph on Cartesian coordinates was devised by Douma(2) and republished by Abbett(1). This graph gives the solution for D_2 when D_1 and V_1 are known. The graph was prepared for a range of values of $10 < V_1 < 100$ in Chart 112-4 and of $6 < V_1 < 40$ in Chart 112-5.

5. List of References.

(1) Abbett, R. W., American Civil Engineering Practice. Vol II, John Wiley & Sons., Inc., New York, N. Y., sec 17, p 56.

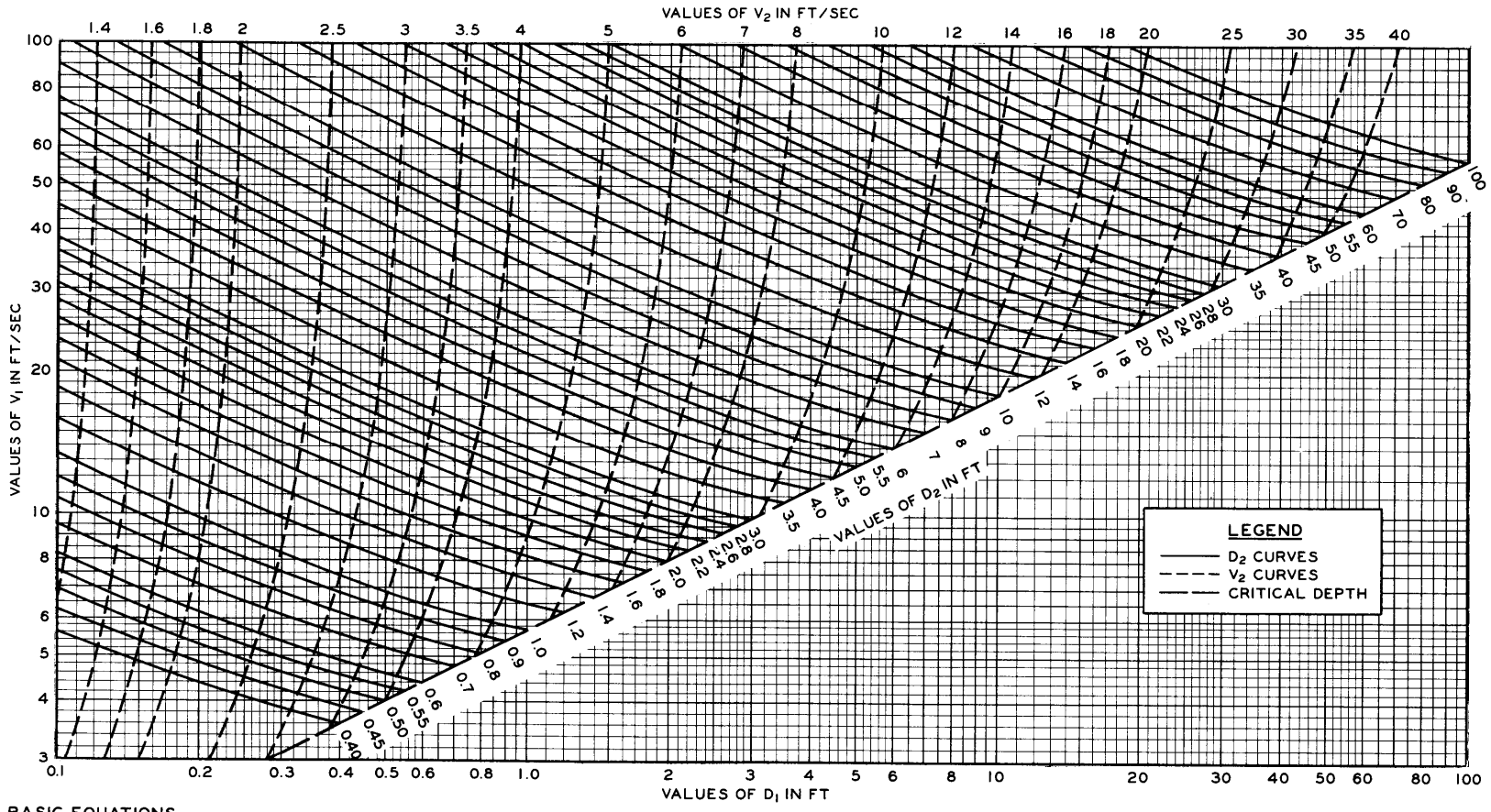
(2) Douma, J. H., Hydraulic Model Studies of the Wickiup Outlet Works Stilling Basin, Deschutes Project, Oregon. U. S. Bureau of Reclamation, Memorandum to Chief Designing Engineer, Denver, Colo., 30 June 1939, Appendix 1, fig. 17. (Available on loan only)

(3) King, H. W., Handbook of Hydraulics. 3d ed., McGraw-Hill Book Co., Inc., New York, N. Y., 1939, table 133, p. 444-445.

(4) National Resources Committee, Low Dams. Washington, D. C., 1938, p. 105.

(5) U. S. Bureau of Reclamation, Drawing No. X-D-931. 14 October 1933.

(6) War Department, Corps of Engineers, Hydraulic Tables. 2d ed., U. S. G. P. O., Washington, D. C., 1944, table 3, p. 16-56.



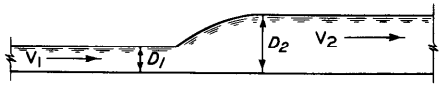
BASIC EQUATIONS

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}}$$

$$V_2 = \frac{V_1 D_1}{D_2}$$

WHERE:

D_1 = DEPTH ABOVE THE JUMP, FT
D_2 = DEPTH BELOW THE JUMP, FT
V_1 = VELOCITY ABOVE THE JUMP, FT/SEC
V_2 = VELOCITY BELOW THE JUMP, FT/SEC
g = GRAVITY CONSTANT, FT/SEC/SEC



SPILLWAY STILLING BASIN

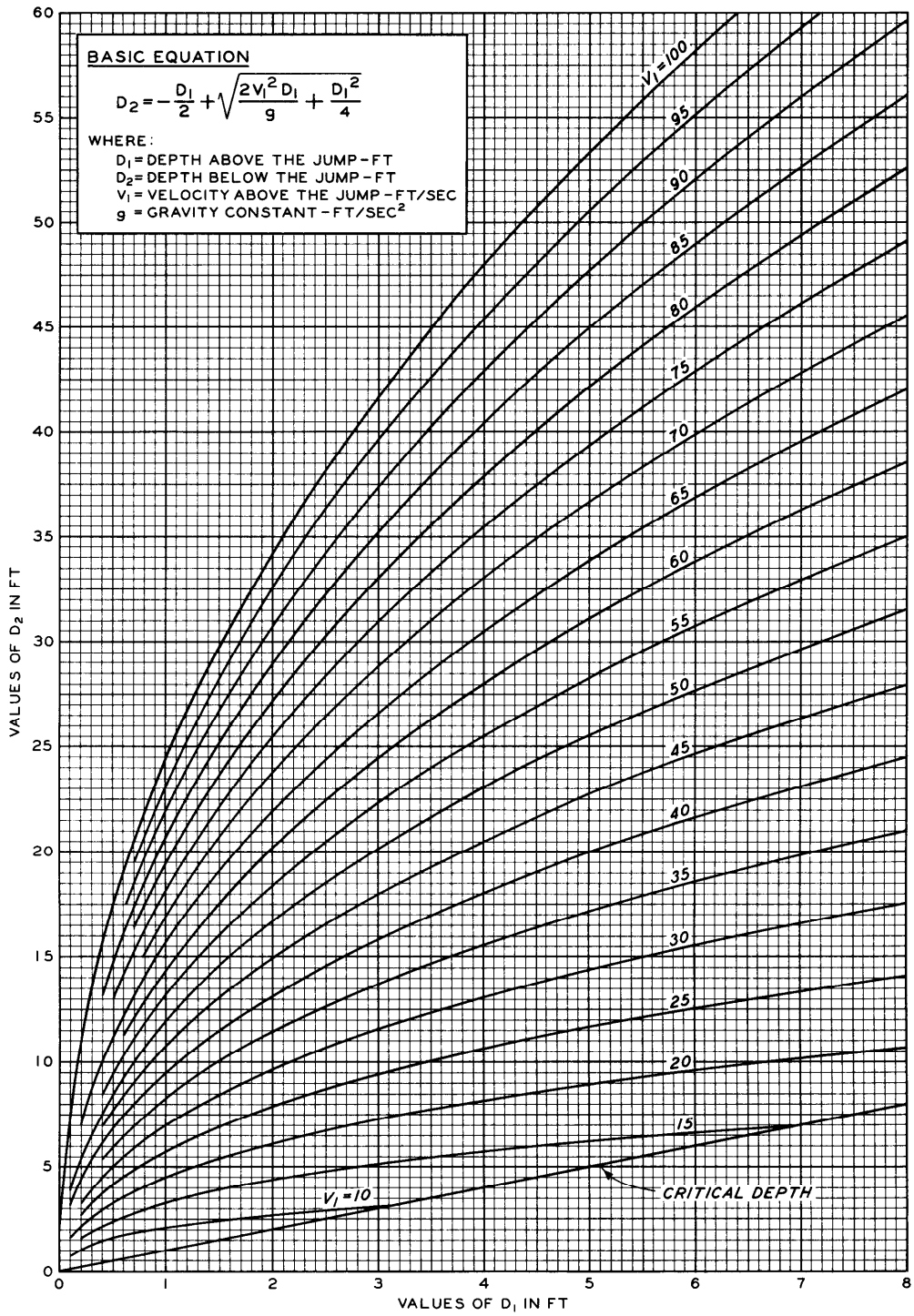
SEQUENT DEPTH CURVES
RECTANGULAR CHANNELS
$3 < V_1 < 100$

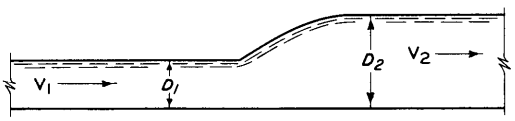
HYDRAULIC DESIGN CHART 112-3

REV 1-64

WES 8-58

CHART 112-3



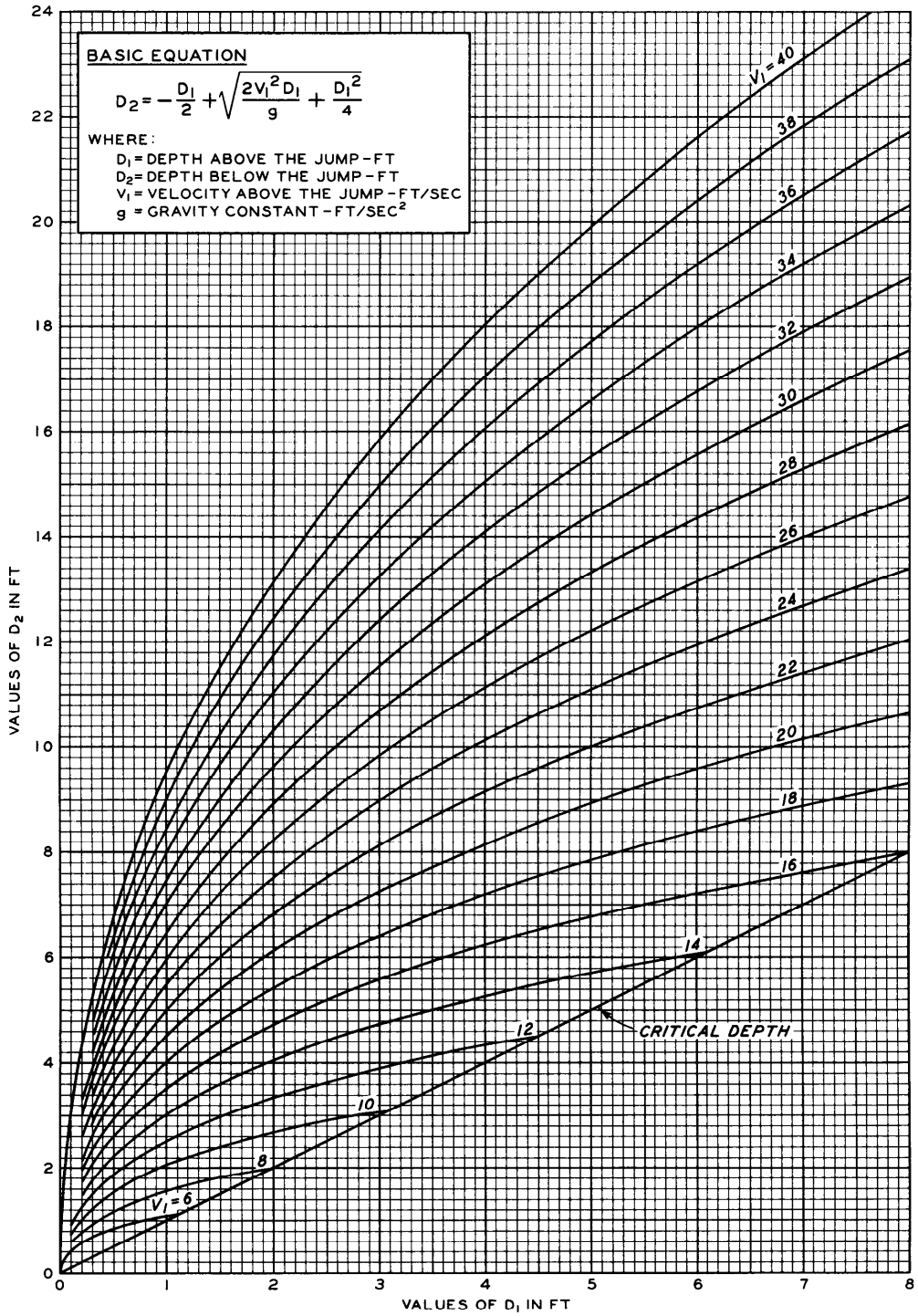


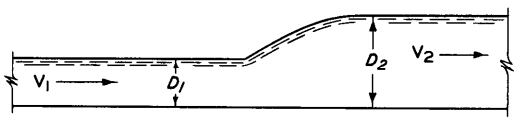
SPILLWAY STILLING BASIN

SEQUENT DEPTH CURVES
RECTANGULAR CHANNELS
$10 < V_1 < 100$

HYDRAULIC DESIGN CHART 112-4

WES 8-58





SPILLWAY STILLING BASIN

SEQUENT DEPTH CURVES
RECTANGULAR CHANNELS
$6 < V_1 < 40$

HYDRAULIC DESIGN CHART 112-5

WES 8-58

SPILLWAY STILLING BASINS

END SILL

TAILWATER REDUCTION

1. An end sill is commonly used as the terminal wall of a stilling basin. Where a tailwater deficiency prevents satisfactory hydraulic jump performance and accompanying energy dissipation, the stilling basin floor is set lower than the riverbed and an end sill forms a step, or rise, to the elevation of the bed of the channel. Hydraulic Design Chart 112-5/1 can be used to determine the relation between the Froude number of the entering jet, the sill height, and the downstream depth required for stabilizing the hydraulic jump when baffle piers are not used.

2. The effects of end sill height upon the reduction of flow depth downstream from a sill have been investigated experimentally by Forster and Skrinde.* Chart 112-5/1 reproduces the data and curves published by Forster and Skrinde. The ratio of the depth d_3 over the end sill to the entering depth d_1 is plotted against the Froude number F_1 of the entering flow. The curves represent various ratios of sill height to the upstream depth h/d_1 for basin lengths of $5(h + d_3)$. The dashed line labeled $h/d_1 = 0$ is the theoretical hydraulic jump curve for sequent depths.

3. The design criteria above apply to a stilling basin that requires a vertical end sill or downstream channel invert sufficiently high to produce the tailwater required for formation of the hydraulic jump. The end sill may act as a critical-depth control, and flow into the downstream channel may be very turbulent with supercritical velocities. Excessive wave action, surges, and supercritical velocities may require that increased protection be provided for 100 ft or more downstream of the stilling basin to prevent channel erosion. Careful consideration should be given to the need for increased riprap protection downstream of the stilling basin. In extreme cases lowering of the basin elevation and use of a standard-type stilling basin may be more economical than extensive riprap protection.

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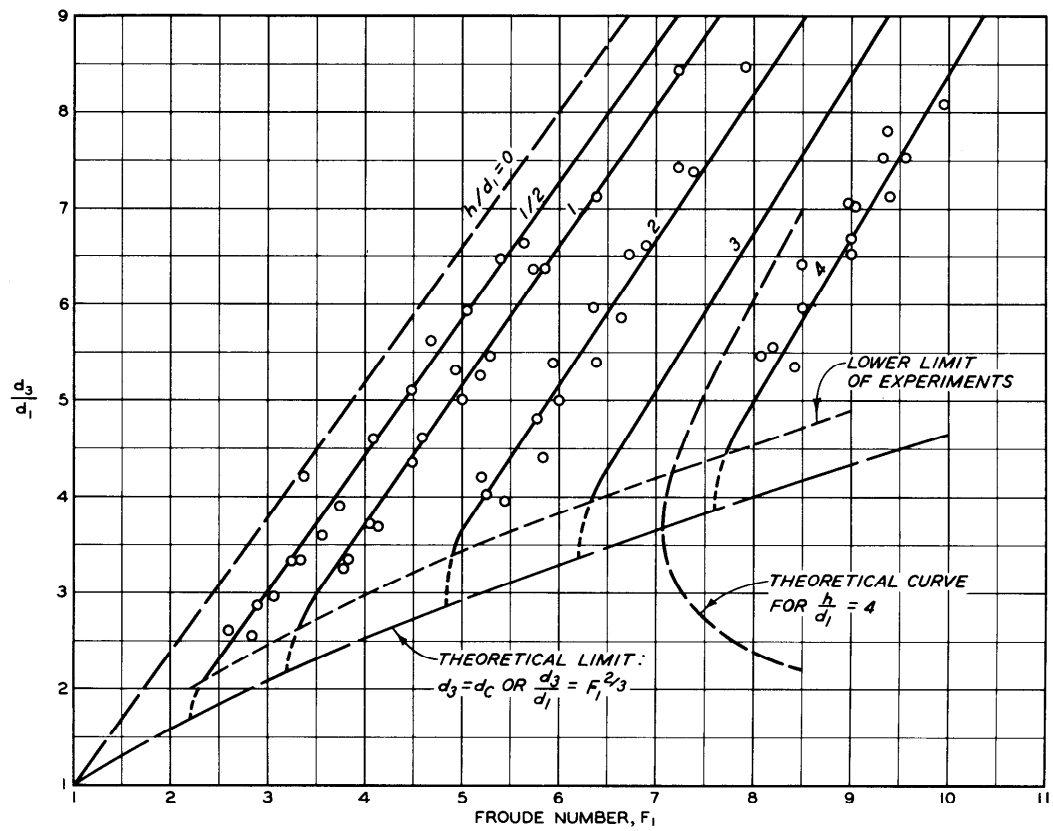
* J. W. Forster and R. A. Skrinde, "Control of the hydraulic jump by sills." Transactions, American Society of Civil Engineers, vol 115, paper 2415 (1950), pp 973-987.

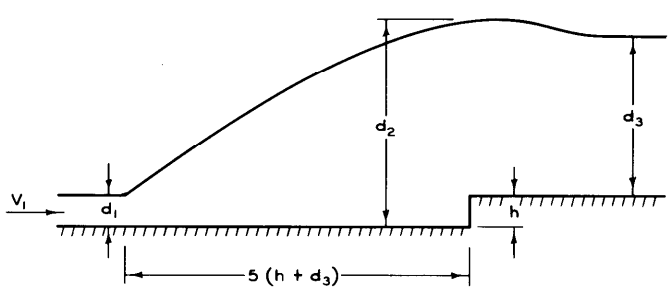


LEGEND

$F_1 = \frac{V_1}{\sqrt{g\,d_1}}$

d_c = CRITICAL DEPTH

NOTE: CURVES AND DATA BY FORSTER AND SKRINDE.



DEFINITION SKETCH

SPILLWAY STILLING BASINS

END SILL
TAILWATER REDUCTION

HYDRAULIC DESIGN CHART 112-5/1

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

HYDRAULIC DESIGN CRITERIA

SHEETS 112-6 to 112-6/2

HIGH OVERFLOW DAMS

BUCKET-TYPE ENERGY DISSIPATOR

1. Bucket-type energy dissipators are used where excessive tailwater depths prevent adequate energy dissipation by means of a hydraulic jump on a horizontal stilling basin floor. HDC 112-6 and 112-6/1 can be used to estimate probable roller and surge heights for preliminary design purposes.

2. The design curves shown in the charts were developed by McPherson and Karr[1] from extensive laboratory tests wherein the discharge was distributed uniformly over the bucket. The test data have been omitted from the charts in the interest of clarity. The data points shown are from Waterways Experiment Station (WES) studies[2-6] and are in reasonably good agreement with McPherson's and Karr's curves for q parameters $\leq 26 \times 10^{-3}$. The agreement is less satisfactory for higher q parameters. The charts are therefore not considered applicable for q parameter values $> 26 \times 10^{-3}$, and the final design for large structures should be developed by hydraulic model study.

3. The streambed was generally at the same elevation as the bucket invert in the McPherson and Karr tests. In the WES tests, the streambed elevation varied from bucket-lip elevation to below the bucket-invert elevation. However, it is believed that the channel-bed elevation has negligible effect on roller and surge heights.

4. The discharge parameter of the design curves can be related to the Froude number of the entering jet in the following manner:

$$q = V\,d \quad \text{and} \quad V = F\sqrt{gd}$$

Then

$$q = F\,d^{3/2}\,g^{1/2}$$

and

$$\frac{q}{g^{1/2}\,h_1^{3/2}} = \frac{F}{(h_1/d)^{3/2}}$$

where F = Froude number of entering jet

q = discharge per ft of bucket width, cfs

h_1 = available energy head (pool to bucket invert), ft

d = depth of flow entering bucket, ft

V = velocity of flow entering bucket, fps

5. HDC 112-6/2 illustrates application of HDC 112-6 and 112-6/1 for the preliminary design of bucket-type energy dissipators. The sample computation is for a specific spillway discharge. The full range of spillway flows should be investigated.

6. The WES model data shown in HDC 112-6 indicate that good energy dissipation is obtained when the bucket roller depth h_b is between 75 and 90 percent of the tailwater depth h_2. For this condition, the surge height is 105 to 130 percent of the tailwater depth.

7. References.

(1) McPherson, M. B., and Karr, M. H., "A study of bucket-type energy dissipator characteristics." ASCE, Hydraulics Division, Journal, vol 83, Paper 1266, No. HY 3 (June 1957); vol 83, Paper 1348, No. HY 4 (August 1957), Corrections, pp 57-64; vol 84, Paper 1832, No. HY 5 (October 1958), Closure, pp 41-48.

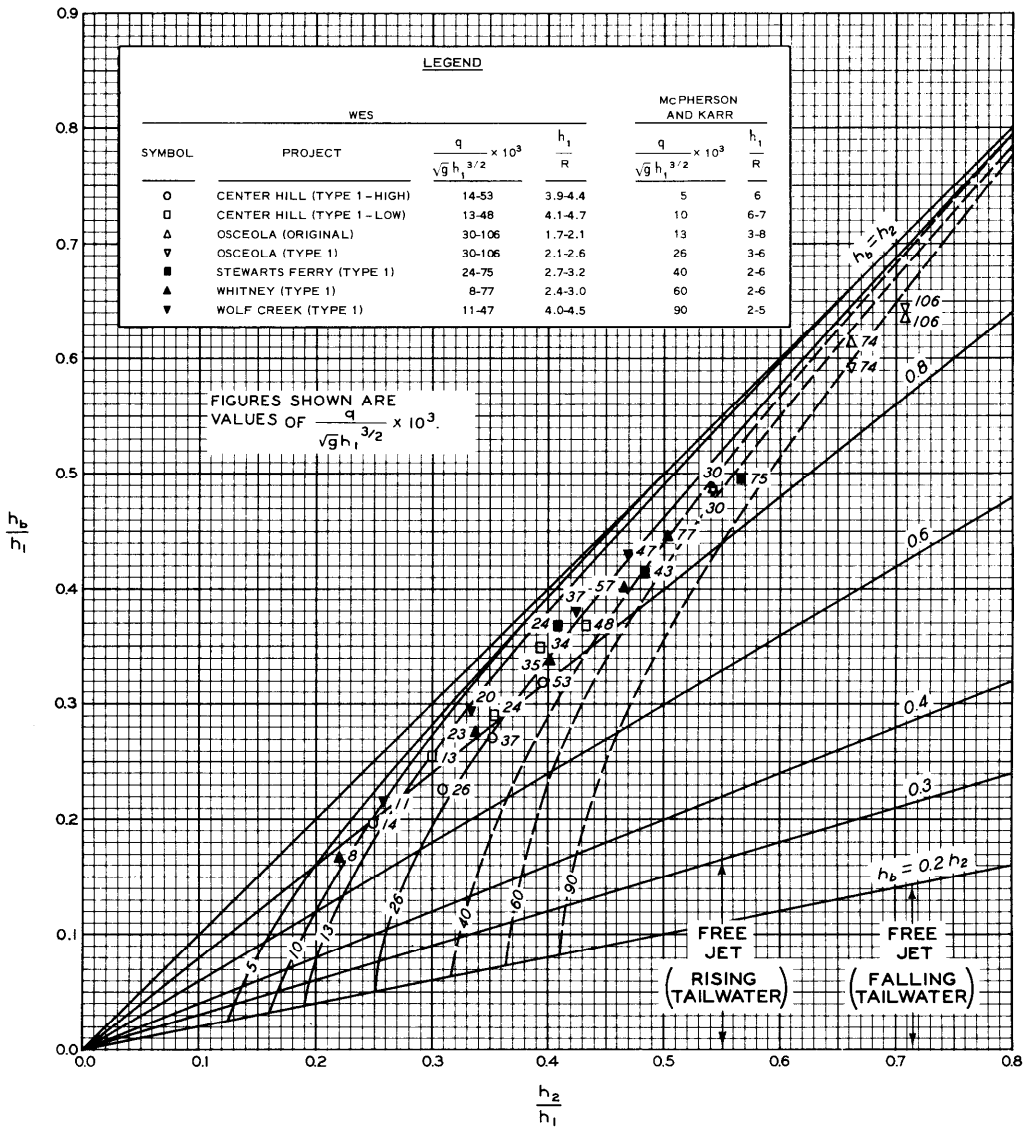
(2) U. S. Army Engineer Waterways Experiment Station, CE, Model Studies of Spillway and Regulating Sluices for Wolf Creek Dam, Cumberland River, Kentucky. Technical Memorandum No. 201-1, Vicksburg, Miss., January 1944.

(3) _____, Model Studies of Spillway and Bucket for Center Hill Dam, Caney Fork River, Tennessee. Technical Memorandum No. 202-1, Vicksburg, Miss., August 1946.

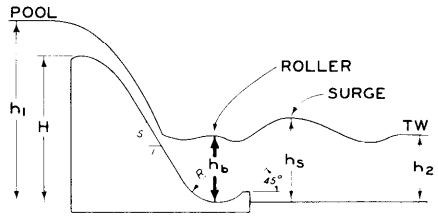
(4) _____, Model Study of Spillway and Bucket, Stewarts Ferry Dam, Stones River, Tennessee. Technical Memorandum No. 2-239, Vicksburg, Miss., September 1947.

(5) _____, Spillway for Osceola Dam, Osage River, Missouri; Model Investigation. Technical Memorandum No. 2-261, Vicksburg, Miss., October 1948.

(6) _____, Spillway Design for Whitney Dam, Brazos River, Texas; Model Investigation. Technical Memorandum No. 2-263, Vicksburg, Miss., October 1948.



NOTE: DESIGN CURVES SHOWN DEVELOPED BY McPHERSON AND KARR FROM EXPERIMENTAL DATA. THESE DATA OMITTED TO SIMPLIFY CHART.



DEFINITION SKETCH

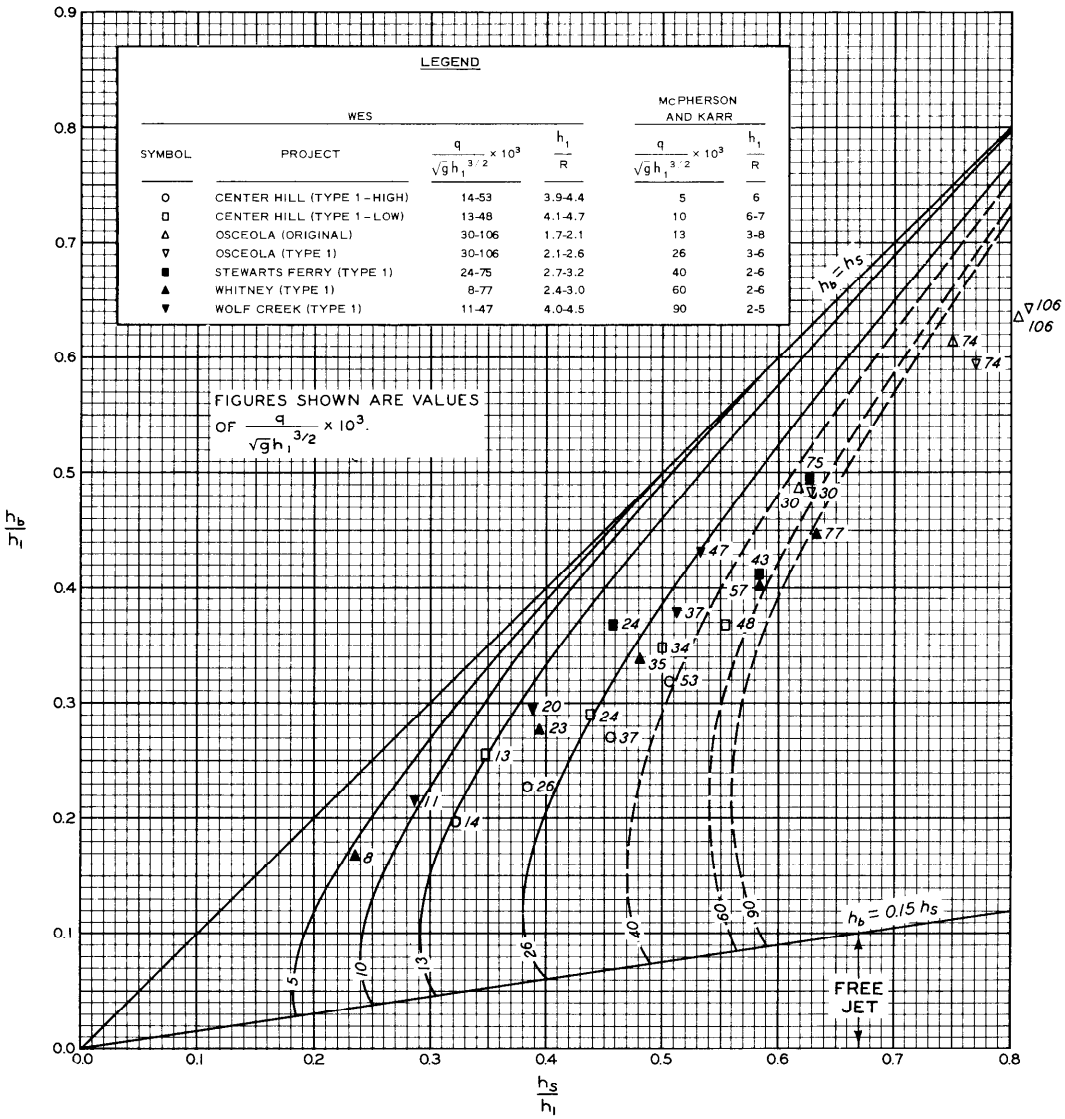
RANGES OF VARIABLES

	WES	McPHERSON AND KARR
SPILLWAY SLOPE	1.41-1.67:1	1:1
LIP ANGLE	45°	45°
H/h_1	0.68-0.93	>0.75
h_1/R	SEE LEGEND	
$q/\sqrt{g}\,h_1^{3/2} \times 10^3$	SEE LEGEND	

HIGH OVERFLOW DAMS

BUCKET-TYPE ENERGY DISSIPATOR ROLLER DEPTH

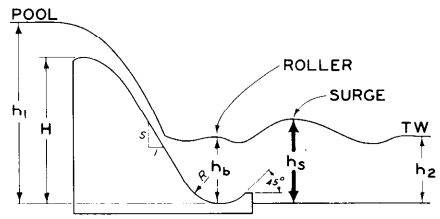
HYDRAULIC DESIGN CHART 112-6



NOTE: DESIGN CURVES SHOWN DEVELOPED BY McPHERSON AND KARR FROM EXPERIMENTAL DATA. THESE DATA OMITTED TO SIMPLIFY CHART.

RANGES OF VARIABLES

	WES	McPHERSON AND KARR
SPILLWAY SLOPE	1.41-1.67:1	1:1
LIP ANGLE	45°	45°
H/h_1	0.68-0.93	>0.75
h_1/R	SEE LEGEND	
$q/\sqrt{g}h_1^{3/2} \times 10^3$	SEE LEGEND	



DEFINITION SKETCH

HIGH OVERFLOW DAMS

BUCKET-TYPE ENERGY DISSIPATOR SURGE HEIGHT

HYDRAULIC DESIGN CHART 112-6/1

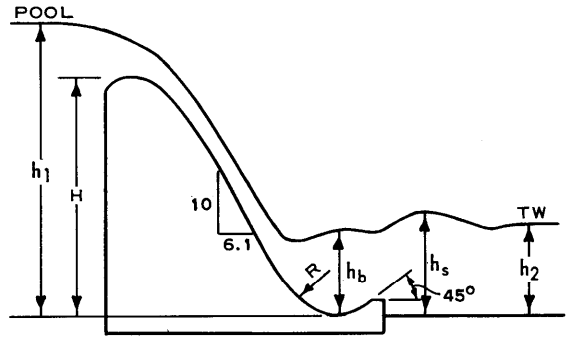
COMPUTATION FOR PRELIMINARY DESIGN

GIVEN:

Discharge (q) per ft of basin width = 450 cfs
Pool elevation = 765 ft
Tailwater elevation = 650 ft
Radius of bucket = 50 ft
Slope of bucket lip = 45 deg
Spillway slope = 10:6.1
Ratio of H to h_1 >0.75

ASSUME:

Bucket invert elevation = 550 ft



COMPUTE:

1. h_1 = pool elevation − bucket invert elevation
   = 765 − 550 = 215 ft
2. h_2 = tailwater elevation − bucket invert elevation
   = 650 − 550 = 100 ft
3. h_2/h_1 = 100/215 = 0.465
4. Discharge parameter

$$\frac{q}{\sqrt{g}\, h_1^{3/2}} \times 10^3 = \frac{450 \times 10^3}{\sqrt{32.2}\,(215)^{3/2}}$$

$$= \frac{450 \times 10^3}{5.68 \times 3153}$$

$$= 25$$

5. From Chart 112-6 read h_b/h_1 = 0.42 for h_2/h_1 = 0.465 and

$$\frac{q}{\sqrt{g}\, h_1^{3/2}} \times 10^3 = 25$$

6. $h_b = 0.42 \times h_1$
   $= 0.42 \times 215 = 90$ ft
7. From Chart 112-6/1 read h_s/h_1 = 0.52 for h_b/h_1 = 0.42 and

$$\frac{q}{\sqrt{g}\, h_1^{3/2}} \times 10^3 = 25$$

8. $h_s = 0.52 \times h_1$
   $= 0.52 \times 215 = 112$ ft
9. Compute h_b and h_s for full range of spillway flows.
10. Determine maximum elevation of bucket roller and surge.

Note: Good energy dissipation is indicated if the roller height (h_b) is between 75 and 90 percent of the tailwater depth (h_2).

HIGH OVERFLOW DAMS
BUCKET-TYPE ENERGY DISSIPATOR
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 112-6/2

HIGH OVERFLOW DAMS

ENERGY DISSIPATORS

FLIP BUCKET AND TOE CURVE PRESSURES

1. Hydraulic forces acting on high, overflow spillway flip buckets and toe curves are of interest in the structural design of these devices. Theoretical studies and model and prototype data indicate that the bottom pressures change continuously throughout the curve and are influenced by the curve radius, the total head, and the unit discharge. It has also been shown that the pressures immediately upstream and, in the case of the toe curve, downstream of the curve are influenced by the curvature. Rouse(3) illustrates the application of the flow net solution to this problem.

2. Flip Buckets. Approximate techniques, including use of the centrifugal force equation(2) and a vortex analogy,(1,2) have been suggested for computing the pressures on spillway flip buckets. A recent WES study(7) using a theoretical approach similar to vortex analogy suggested by Douma(2) indicated that, for relatively high dams, bucket pressures could be expressed as:

$$\frac{h_p}{H_T} = f\left(\frac{q}{R\sqrt{2g\ H_T}}\ ,\ \frac{\alpha}{\alpha_T}\right)$$

where

h_p = pressure head against boundary, ft

H_T = total head (point to energy gradient), ft

q = unit discharge, cfs

R = radius of curve, ft

g = acceleration due to gravity, ft per sec per sec

α = angle of rotation from beginning of curve, degrees

α_T = total deflection angle, degrees

The term α/α_T defines the relative position along the curve.

3. HDC 112-7 presents dimensionless flip bucket pressure curves based on Pine Flat(4) and Hartwell(5) model spillway data analyzed in

accordance with the preceding expression. Spillway energy losses were assumed negligible in the analysis. The plotted data for α/α_T from 0.25 to 0.75 indicate that the pressure in the middle portion of the bucket is nearly constant. Therefore, the pressure distribution through the bucket can be adequately defined by the four curves shown in the chart. The curve for $\alpha/\alpha_T = 1.0$ coincides with the axis of the chart ordinates since the pressure on the lower nappe of the jet leaving the bucket is atmospheric. The Pine Flat(6) prototype data plotted on the chart indicate satisfactory correlation with the model data. Allowances were made for spillway energy losses in computing the total head H_T for the prototype data.

4. Toe Curves. The available data(8) from model tests conducted under the Corps of Engineers Engineering Studies Item 801 which are plotted in HDC 112-7 indicate that for a high dam the flip bucket pressure distribution curves generally apply to spillway toe curves. However, the data show that the pressure at the end of the toe curve approximates that at the beginning of the curve if the toe curve is not submerged.

5. HDC 112-7 can be used to estimate the pressure distribution on spillway flip buckets or toe curves associated with high overflow dams. However, for design purposes, allowance for spillway energy losses should be included in the computations of H_T required for use of the chart. The user is cautioned that the curves are not applicable to toe curves affected by submergence.

6. References.

(1) Balloffet, A., "Pressures on spillway flip buckets." Journal of the Hydraulics Division, American Society of Civil Engineers (September 1961), pp 87-96.

(2) Douma, J. H., discussion of paper, "Design of side walls in chutes and spillways," by D. B. Gumensky. Transactions, American Society of Civil Engineers, vol 119 (1954), pp 364-367.

(3) Rouse, H., Engineering Hydraulics. John Wiley and Sons, Inc., New York, N. Y., 1950, p 47.

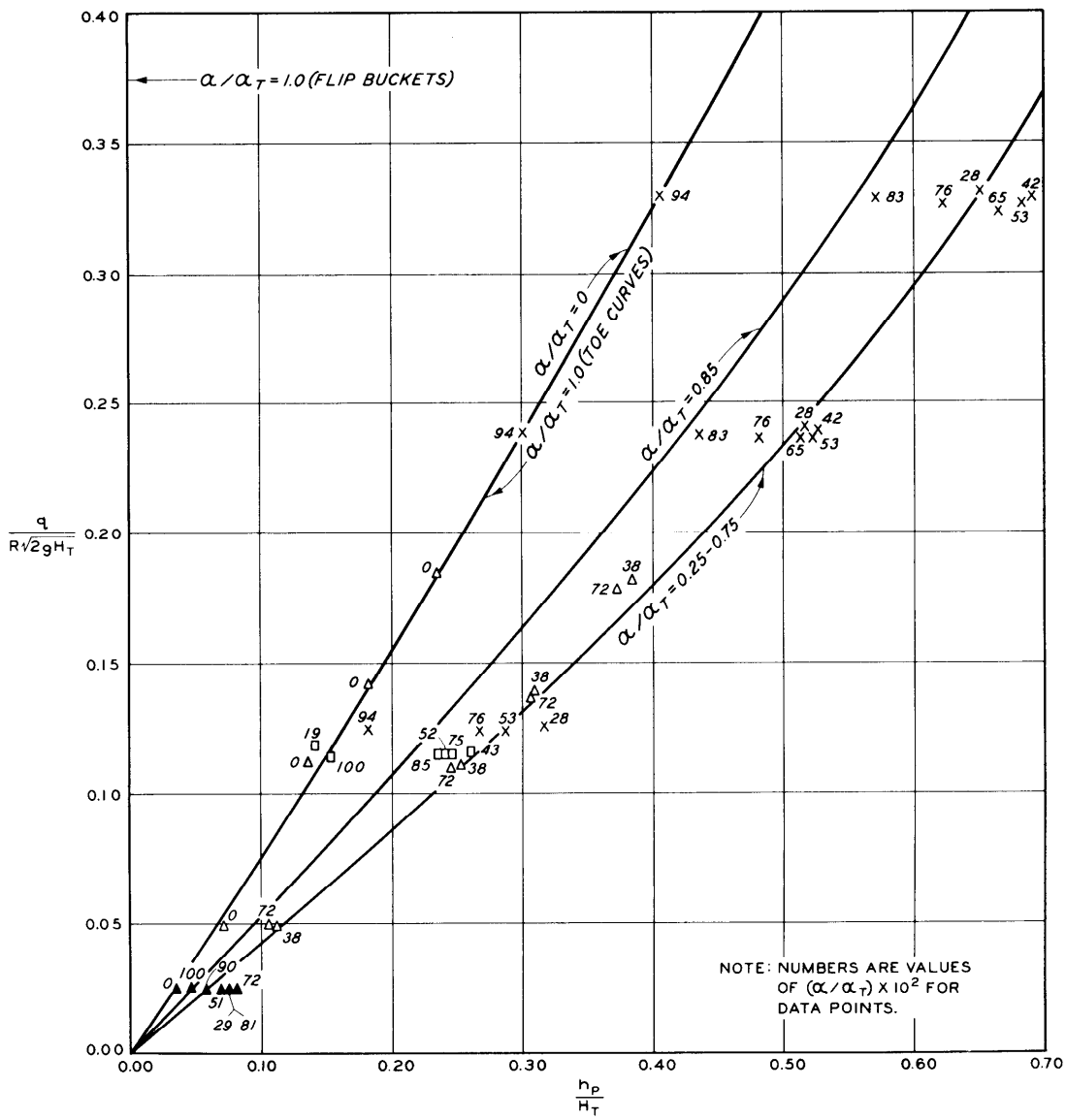
(4) U. S. Army Engineer Waterways Experiment Station, CE, Spillway and Conduits for Pine Flat Dam, Kings River, California; Hydraulic Model Investigation. Technical Memorandum No. 2-375, Vicksburg, Miss., December 1953.

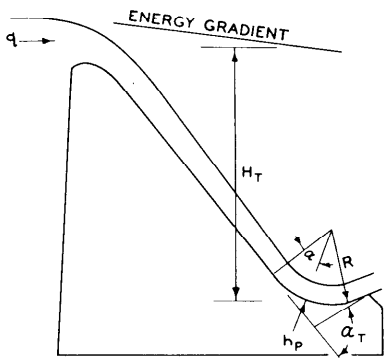
(5) _____, Sluice Outlet Portal and Spillway Flip Bucket, Hartwell Dam, Savannah River, Georgia; Hydraulic Model Investigation. Technical Memorandum No. 2-393, Vicksburg, Miss., August 1954.

(6) _____, Prototype Tests of Spillway Crest and Flip Bucket, Pine Flat Dam, Kings River, California. Technical Report No. 2-511, Vicksburg, Miss., June 1959.

(7) U. S. Army Engineer Waterways Experiment Station, CE, An Investigation of Spillway Bucket and Toe Curve Pressures. Miscellaneous Paper No. 2-625, Vicksburg, Miss., February 1964.

(8) _____, unpublished ES 801 data.





DEFINITION SKETCH

LEGEND

- Δ PINE FLAT MODEL
- X HARTWELL MODEL
- ▲ PINE FLAT PROTOTYPE
- □ ES 801 (TOE CURVE)

HIGH OVERFLOW DAMS

ENERGY DISSIPATORS

FLIP BUCKET AND TOE CURVE PRESSURES

HYDRAULIC DESIGN CHART 112-7

REV 1-64

WES 10-61

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

HIGH OVERFLOW DAMS

ENERGY DISSIPATORS

FLIP BUCKET THROW DISTANCE

1. For economy, flip bucket or ski-jump energy dissipators are sometimes used when spray from the jet can be tolerated and erosion by the plunging jet will not be a problem. Flip buckets have caused trouble in climates where spray from the jet resulted in icing of nearby roadways or electrical equipment. The major amount of energy dissipation occurs in the region where the jet plunges into the tailwater; a minor amount occurs as the jet frays after leaving the bucket.

2. Factors affecting the horizontal throw distance from the bucket lip to point of jet impact are the initial velocity of the jet, the bucket lip angle, and the difference in elevation between the lip and the tailwater.

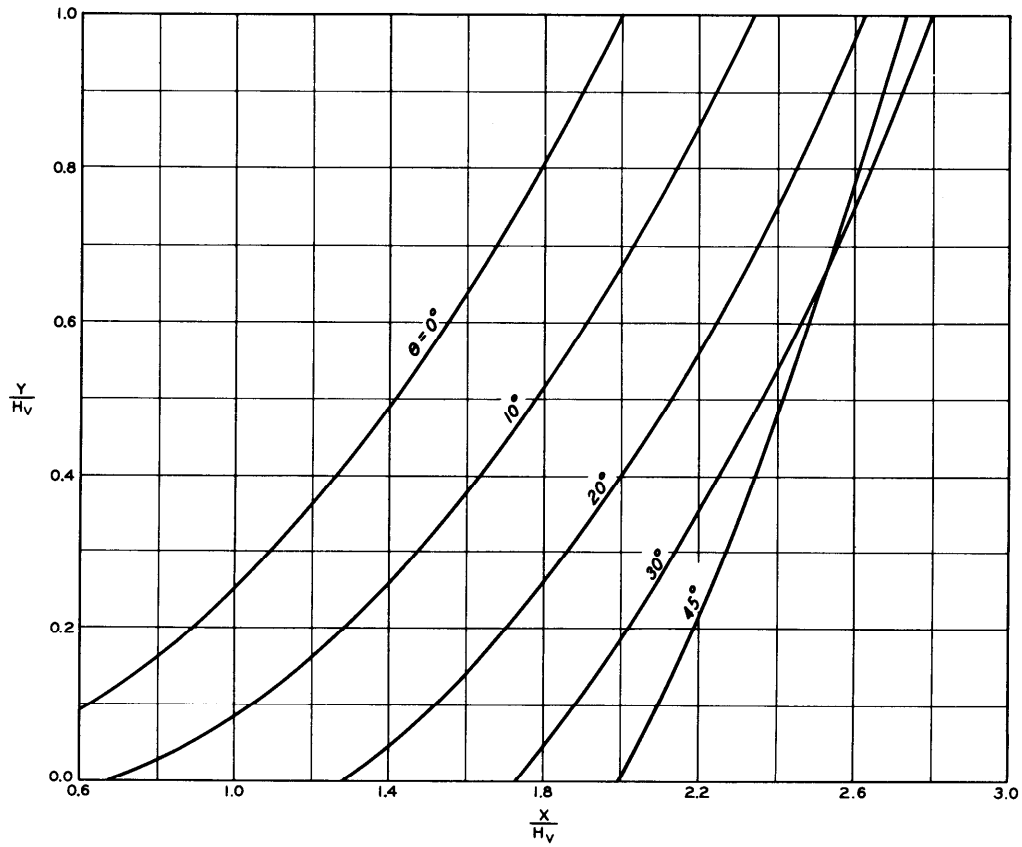
3. HDC 112-8 presents throw-distance curves for lip angles of 0 to 45 degrees. The horizontal throw distance X and the vertical drop from the bucket lip to tailwater Y are expressed in terms of the jet velocity head H_v. The following expression based on the theoretical equations for trajectories was used for developing the curves:

$$X/H_v = \sin 2\theta + 2 \cos \theta \sqrt{\sin^2 \theta + Y/H_v}$$

where

X = throw distance, ft
Y = vertical drop from lip to tailwater surface, ft
H_v = velocity head of jet at bucket lip, ft
θ = bucket lip angle, deg

4. HDC 112-8 is a guide for judging the point of impact of the jet. The throw distance may be substantially less than indicated, depending upon spillway energy losses. Prototype measurements of spillway energy losses are needed to permit a comparison of theoretical and actual throw distances.

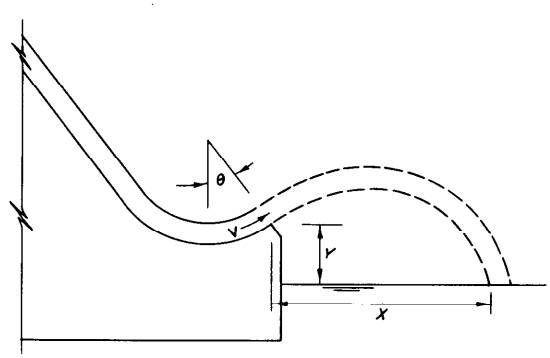


EQUATION

$$\frac{X}{H_V} = \sin 2\theta + 2\cos\theta\sqrt{\sin^2\theta + \frac{Y}{H_V}}$$

WHERE:

- X = THROW DISTANCE, FT
- θ = BUCKET LIP ANGLE
- H_V = VELOCITY HEAD AT BUCKET LIP, FT
- Y = VERTICAL DROP FROM LIP TO TAILWATER SURFACE, FT



DEFINITION SKETCH

HIGH OVERFLOW DAMS

ENERGY DISSIPATORS
FLIP BUCKET THROW DISTANCE

HYDRAULIC DESIGN CHART 112-8

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