HYDRAULIC DESIGN CRITERIA

SHEETS 111-1 to 111-2/1

OVERFLOW SPILLWAY CREST

1. Previous Crest Shapes. Some early crest shapes were based on a simple parabola designed to fit the trajectory of the falling nappe. Bazin's experiments of the 19th century were the basis of many early designs. The Bureau of Reclamation conducted extensive experiments on the shape of the nappe over a sharp-crested weir (reference 2). Numerous crests have been designed using the coordinates of the lower surface of the nappe for the shape of the crest, without resort to an equation. The Huntington District has used an equation involving the 1.82 power of X and the Nashville District has used the 1.88 power of X.

2. Standard Shape, Downstream Quadrant. A comparison of the Bureau of Reclamation data with those of other experimenters was made by the Office, Chief of Engineers. On the basis of this study, Circular Letter No. 3281 was issued on 2 September 1944, suggesting the use of the 1.85 power of X. This equation is given in Hydraulic Design Charts 111-1 and 111-2 and was adopted to define the downstream quadrant shape.

3. Point of Tangency. The slope function graph of the tangents X and Y to the downstream quadrant is shown in Chart 111-1 to facilitate the location of the point of tangency α. Although it is realized that the tangent point will often be determined analytically for the final design, this graph should be of value in the preliminary layouts in connection with stability analyses and cost estimates. The downstream tangent points can be computed from

$$\frac{X}{H_d} = 1.096 \left(\frac{1}{\alpha}\right)^{1.176} \tag{1}$$

and

$$\frac{Y}{H_d} = 0.592 \left(\frac{1}{\alpha}\right)^{2.176} \tag{2}$$

where H_d is the design head.

4. Standard Shape, Upstream Quadrant. The upstream quadrant shape of circular arcs originally defined in Chart 111-1, dated 4-1-52 (revised 8-60), resulted in a surface discontinuity at the vertical spillway face. A third, short-radius arc ($R = 0.04H_d$) incorporated in this design has been model tested (reference 1) and found to result in

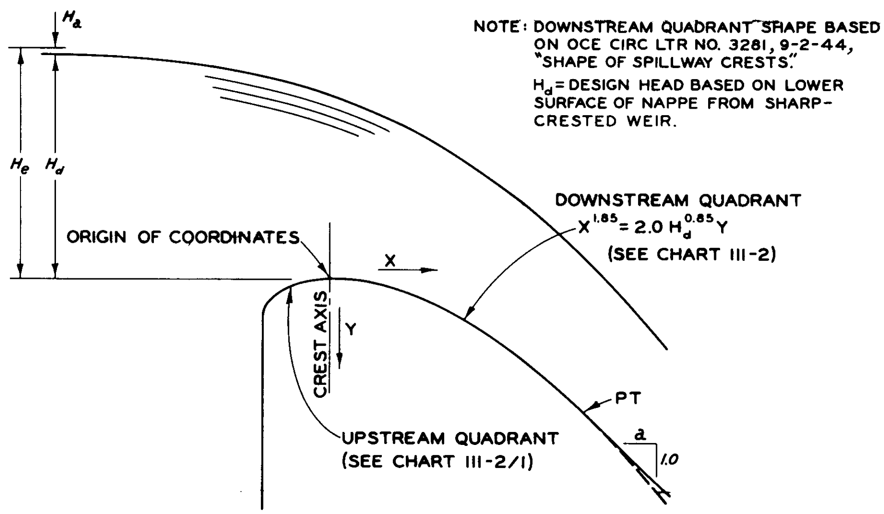
improved pressure conditions and discharge coefficients for heads exceeding the design head. Chart 111-2/1 (revised 9-70) presents this upstream crest quadrant design. A table of coordinates in terms of X/H_d and Y/H_d is included as Chart 111-2 for design convenience.

5. Recent model studies have verified the elliptical upstream quadrant design also presented in reference 1. This method, depicted in Hydraulic Design Charts 111-20 through 111-25/1, should be used for future spillway design. The Standard Shape Criteria will be retained for reference purposes.

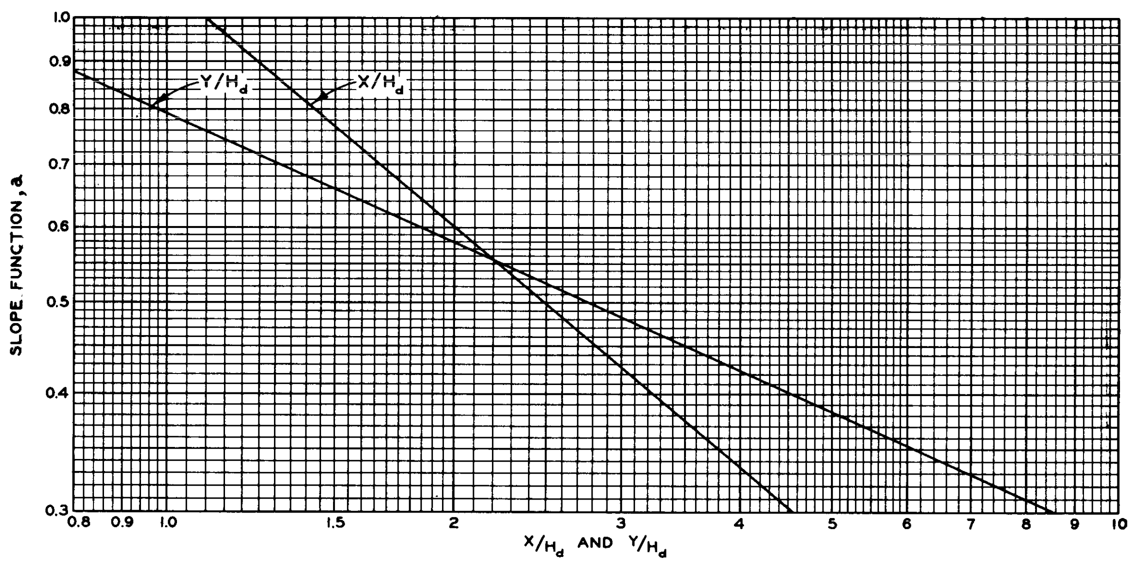
6. References.

(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) 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.



NOTE: DOWNSTREAM QUADRANT SHAPE BASED ON OCE CIRC LTR NO. 3281, 9-2-44, "SHAPE OF SPILLWAY CRESTS."
H_d = DESIGN HEAD BASED ON LOWER SURFACE OF NAPPE FROM SHARP-CRESTED WEIR.



NOTE: COORDINATES OF TANGENT POINT FOR PRELIMINARY LAYOUTS AND ESTIMATES.

OVERFLOW SPILLWAY CREST
TANGENT ORDINATES

HYDRAULIC DESIGN CHART 111-1

X	$X^{1.85}$	X	$X^{1.85}$	H_d	$2H_d^{0.85}$	H_d	$2H_d^{0.85}$	H_d	$2H_d^{0.85}$
0.10	0.0141	6	27.515	1	2.000	26	31.896	51	56.554
.15	.0299	7	36.596	2	3.605	27	32.937	52	57.495
.20	.0509	8	46.851	3	5.088	28	33.971	53	58.434
.25	.0769	9	58.257	4	6.498	29	35.000	54	59.370
.30	.1078	10	70.795	5	7.855	30	36.024	55	60.303
.35	.1434	12	99.194	6	9.172	31	37.041	56	61.234
.40	.1836	14	131.928	7	10.460	32	38.054	57	62.162
.45	.2283	16	168.897	8	11.713	33	39.063	58	63.088
.50	.2774	18	210.017	9	12.946	34	40.066	59	64.011
.60	.3887	20	255.215	10	14.159	35	41.067	60	64.932
.70	.5169	25	385.646	11	15.354	36	42.062	61	65.851
.80	.6618	30	540.349	12	16.532	37	43.053	62	66.767
.90	.8229	35	718.664	13	17.696	38	44.040	63	67.681
1.00	1.000	40	920.049	14	18.847	39	45.023	64	68.594
1.20	1.401	45	1144.045	15	19.985	40	46.002	65	69.503
1.40	1.864	50	1390.255	16	21.112	41	46.978	66	70.411
1.60	2.386	55	1658.330	17	22.229	42	47.950	67	71.317
1.80	2.967	60	1947.959	18	23.335	43	48.919	68	72.221
2.00	3.605	65	2258.863	19	24.433	44	49.884	69	73.123
2.50	5.447	70	2590.785	20	25.521	45	50.846	70	74.022
3.00	7.633	75	2943.496	21	26.602	46	51.807	71	74.920
3.50	10.151	80	3316.779	22	27.674	47	52.761	72	75.816
4.00	12.996	90	4124.285	23	28.741	48	53.714	73	76.710
4.50	16.160	100	5011.872	24	29.799	49	54.663	74	77.603
5.00	19.638			25	30.852	50	55.610	75	78.493

OVERFLOW SPILLWAY CREST EQUATIONS

$$X^{1.85} = 2H_d^{0.85}Y, \quad Y = \frac{X^{1.85}}{2H_d^{0.85}}; \text{ WHERE } H_d = \text{DESIGN HEAD}$$

NOTE: SEE CHART 111-2/1 FOR UPSTREAM QUADRANT COORDINATES.

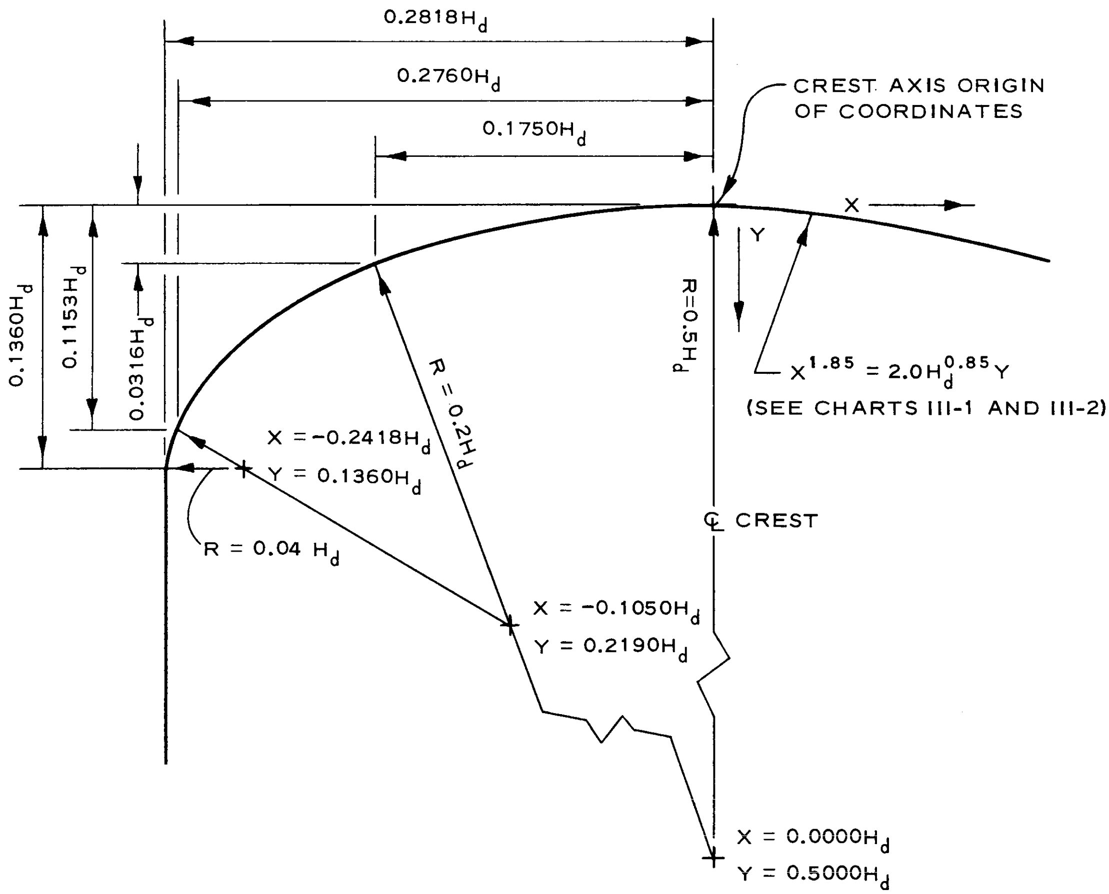
OVERFLOW SPILLWAY CREST
DOWNSTREAM QUADRANT
TABLE OF FUNCTIONS

HYDRAULIC DESIGN CHART 111-2

REV 8-60

WES 4-52

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



COORDINATES FOR UPSTREAM QUADRANT

$\frac{X}{H_d}$	$\frac{Y}{H_d}$	$\frac{X}{H_d}$	$\frac{Y}{H_d}$
-0.0000	0.0000	-0.2200	0.0553
-0.0500	0.0025	-0.2400	0.0714
-0.1000	0.0101	-0.2600	0.0926
-0.1500	0.0230	-0.2760	0.1153
-0.1750	0.0316	-0.2780	0.1190
-0.2000	0.0430	-0.2800	0.1241
		-0.2818	0.1360

NOTE: H_d = DESIGN HEAD BASED ON LOWER SURFACE OF NAPPE FROM SHARP-CRESTED WEIR WITH NEGLIGIBLE VELOCITY OF APPROACH AND CREST AT $X = -0.2818H_d$, $Y = 0.1259H_d$.

OVERFLOW SPILLWAY CREST
UPSTREAM QUADRANT

HYDRAULIC DESIGN CHART 111-2/1

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

REV 9-70, 6-71

HYDRAULIC DESIGN CRITERIA

SHEET 111-3

SPILLWAY CREST

DISCHARGE COEFFICIENT

HIGH OVERFLOW DAMS

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

$$Q = CLH_e^{3/2}$$

where

Q = total discharge, cfs

C = discharge coefficient (Hydraulic Design Chart 111-3)

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

H_e = energy head on crest, ft

2. Design Criteria. Early studies of the discharge coefficient C used the relation of C to the ratio H_e/H_d. These studies indicated that C ranged from 3.90 to 4.10 at design head and decreased to 3.10 at zero head. An approximation of the upper value can be derived by transferring the sharp-crested weir coefficient to a rounded weir crest that fits the lower nappe. The head on the rounded crest is known to be 0.888 times the head on the sharp crest. Using a discharge coefficient of 3.33 for a sharp-crested weir and assuming the velocity of the approach flow to be negligible, the coefficient for design head is derived as 3.93. The lower limit of $C = 3.10$ closely approximates the theoretical broad-crested weir coefficient of 3.087. The theory, which is based on critical depth in rectangular channels, is given by King.[1] Friction can be expected to reduce the coefficient at low heads. New, smooth concrete crests should have a high coefficient at low heads compared to crests that have been roughened by weathering or other causes.

3. Test Data. The curve in Chart 111-3 is based primarily on data obtained from model tests conducted under Corps of Engineers Engineering Studies Item 801, General Spillway Investigation, at the U. S. Army Engineer Waterways Experiment Station (WES). Only those tests in which a deep approach channel and negligible velocity of approach existed were used in developing the curve. The plotted points from ES 801 are the basis for the curve above the H_e/H_d ratio of 0.4. Prototype test results are plotted for the low head range, and that portion of the curve is based on

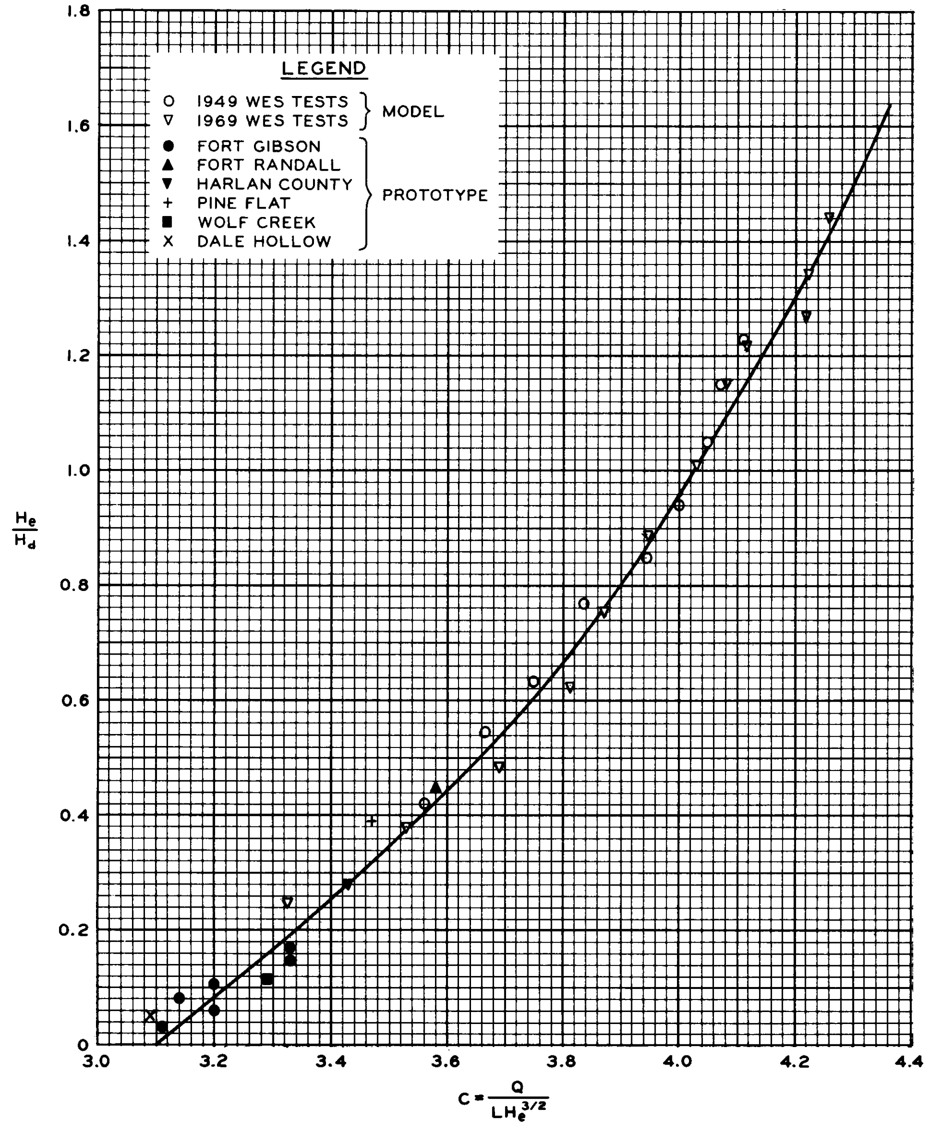
the field tests indicated in the legend. More prototype observations are needed for the newer design shapes that approximate the spillway crest defined in Hydraulic Design Criteria 111-1 and 111-2/1.

4. The open-circle data points are from tests on the originally published crest shape (Charts 111-1 and 111-2, dated 4-1-52). The open-triangle points are from recent laboratory tests[2] in which a third short-radius curve ($R = 0.04H_d$, Chart 111-2/1) was added to the upstream quadrant shape to eliminate the surface discontinuity in the original design where the curve intersected the vertical face of the spillway.

5. References.

(1) King, H. W., Handbook of Hydraulics; For the Solution of Hydraulic Problems, 3d ed. (1939), pp 379-380 and 4th ed. (1954, revised by E. F. Brater), pp 8-8 and 8-9, McGraw-Hill, New York.

(2) 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: H_e = ENERGY HEAD ON CREST, FT
H_d = DESIGN HEAD ON CREST, FT
C = DISCHARGE COEFFICIENT
Q = DISCHARGE, CFS
L = NET LENGTH OF CREST, FT

SPILLWAY CREST DISCHARGE COEFFICIENT

HIGH OVERFLOW DAMS

HYDRAULIC DESIGN CHART 111-3

HYDRAULIC DESIGN CRITERIA

SHEET 111-3/1

OVERFLOW SPILLWAY CREST WITH ADJACENT CONCRETE SECTIONS

ABUTMENT CONTRACTION COEFFICIENT

1. The effective length L of a spillway crest used in uncontrolled-spillway discharge computations is expressed by the equation:

$$L = L' - 2\,(NK_p + K_a)\,H_e$$

where

L' = net length of crest, ft

N = number of piers

K_p = pier contraction coefficient

K_a = abutment contraction coefficient

H_e = energy head on crest, ft

2. HDC 111-3/1 presents a suggested abutment contraction coefficient design curve for high overflow spillways with adjacent concrete sections. Discharge and pier contraction coefficients from appropriate HDC charts were used with model discharge data to compute the plotted abutment contraction coefficients. These abutment contraction coefficients include the weir end contraction and the effect of approach flow angularity, if any, on all elements of the spillway. The coefficient K_a is plotted in terms of the ratio of the energy head on the spillway H_e to the abutment radius R . Pertinent information concerning each project is tabulated in HDC 111-3/1. An abutment contraction coefficient of 0.1 is suggested for design purposes for spillways with adjacent concrete nonoverflow sections when the approach flow is normal to the spillway crest. Higher coefficients should be assumed for projects involving extreme angularity of approach flow. It is also suggested that the maximum design head-abutment radius ratio be limited to 5.0.

3. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Model Studies of Spillway and Bucket for Center Hill Dam, Caney Fork River, Tennessee. Technical Memorandum No. 202-1, Vicksburg, Miss., August 1946.

(2) __________, Spillway for Philpott Dam, Smith River, Virginia; Model Investigation. Technical Memorandum No. 2-321, Vicksburg, Miss., December 1950.

(3) __________, Spillway and Conduits for Pine Flat Dam, Kings River,

California; Hydraulic Model Investigation. Technical Memorandum No. 2-375, Vicksburg, Miss., December 1953.

(4) U. S. Army Engineer Waterways Experiment Station, CE, Folsom Dam Spillway, Uncontrolled. (Unpublished data.)

(5) ________, General Spillway Tests (ES 801). (Unpublished data.)

SHEET 111-3/2

OVERFLOW SPILLWAY CREST WITH ADJACENT EMBANKMENT SECTIONS

ABUTMENT CONTRACTION COEFFICIENT

1. The effective length of a spillway crest used in uncontrolled-spillway discharge computations is expressed by the equation given in HDC Sheet 111-3/1.

2. HDC 111-3/2 presents a suggested abutment contraction coefficient design curve for spillways with adjacent earth embankment sections. Discharge and pier contraction coefficients from appropriate HDC charts were used with model discharge data to compute the plotted abutment contraction coefficients. These abutment contraction coefficients include the contractive effects of the upstream rounding of the embankments, the weir end contraction, and the effects of approach flow angularity on all elements of the spillway. The coefficient K_a is plotted in terms of the ratio of the energy head on the spillway to the spillway design head. This parameter is believed to be more representative of the composite abutment contraction effects than the energy head-abutment radius ratio used on the original HDC 111-3/2 dated August 1960. Pertinent information concerning each project is tabulated in the chart.

3. An abutment contraction coefficient of 0.2 is suggested for design purposes for spillways with adjacent nonoverflow earth embankments. Higher coefficients should be assumed for projects involving extreme angularity of approach flow. An abutment contraction coefficient of 0.74 was measured during the model study[4] of John Redmond Dam spillway for a design head of 41 ft and a weir height of 6.0 ft.

4. References.

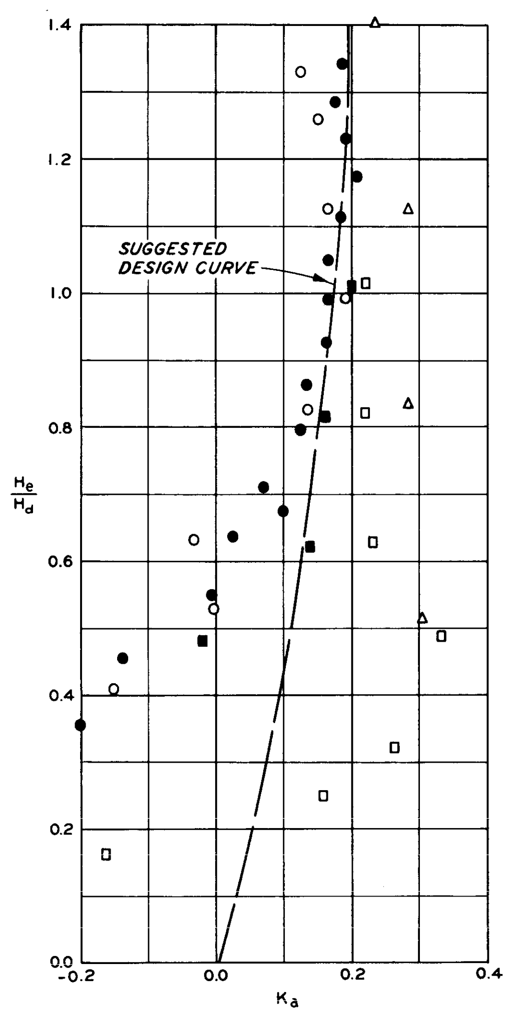
(1) U. S. Army Engineer District, Portland, CE, Spillway for Dorena Dam, Row River, Oregon; Hydraulic Model Investigation. Bonneville Hydraulic Laboratory Report No. 27-1, Bonneville, Oreg., May 1953.

(2) U. S. Army Engineer Waterways Experiment Station, CE, Walter F. George Lock and Dam, Chattahoochee River, Alabama and Georgia; Hydraulic Model Investigation. Technical Report No. 2-519, Vicksburg, Miss., August 1959.

(3) ________, Carlyle Dam, Kaskaskia River, Illinois; Hydraulic Model Investigation. Technical Report No. 2-568, Vicksburg, Miss., June 1961.

(4) ________, Spillway for John Redmond Dam, Grand (Neosho) River, Kansas; Hydraulic Model Investigation. Technical Report No. 2-611, Vicksburg, Miss., November 1962.

(5) ________, Red Rock Dam Model Tests. (Unpublished data.)



BASIC EQUATION

$$Q = C\left[L' - 2(NK_p + K_a)H_e\right]H_e^{3/2}$$

WHERE:

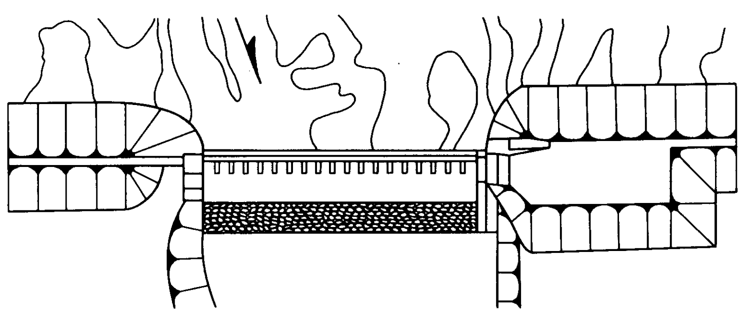
Q = DISCHARGE, CFS
C = DISCHARGE COEFFICIENT
L' = NET LENGTH OF CREST, FT
N = NUMBER OF PIERS
K_p = PIER CONTRACTION COEFFICIENT
K_a = ABUTMENT CONTRACTION COEFFICIENT
H_e = ENERGY HEAD ON CREST, FT

LEGEND

SYMBOL	PROJECT	R	W/L	W/H
□	DORENA	2	5.60	10.7
■	DORENA	4	5.60	10.7
○	RED ROCK*	7.8	3.42	16.5
●	CARLYLE*	9	8.44	75.5
△	WALTER F. GEORGE*	4	5.44	55.3

*GATED SPILLWAY WITH PIERS

NOTE: R = RADIUS OF ABUTMENT, FT
W = WIDTH OF APPROACH REPRODUCED IN MODEL, FT
L = GROSS WIDTH OF SPILLWAY, FT
H = DEPTH OF APPROACH IN MODEL, FT
H_d = DESIGN HEAD ON CREST, FT



OVERFLOW SPILLWAY CREST WITH ADJACENT EMBANKMENT SECTIONS

ABUTMENT CONTRACTION COEFFICIENT

HYDRAULIC DESIGN CHART 111-3/2

HYDRAULIC DESIGN CRITERIA

SHEET 111-3/3

OVERFLOW SPILLWAYS

STAGE-DISCHARGE RELATION

UNCONTROLLED FLOW

1. Purpose. Hydraulic Design Chart 111-3/3 provides a method for developing an uncontrolled spillway flow rating curve when the spillway discharge and head for any unsubmerged, uncontrolled flow are known or can be computed. It also provides a means of optimizing spillway design through extensive use of the sharp-crested weir data published by the USBR.[1] Its use is limited to unsubmerged flow conditions.

2. Theory. When flow over a spillway is controlled only by the head on the spillway, the relation between the head and discharge can be expressed by the following equation:

$$Q = CLH_e^{3/2} \qquad (1)$$

where

Q = spillway discharge, cfs

C = total flow coefficient combining the approach channel and crest shape, abutment, and pier effects

L = net spillway length, ft

H_e = energy head on the spillway, ft

3. A comparable equation for the spillway design flow is

$$Q_d = C_d L H_d^{3/2} \qquad (2)$$

where the subscript d refers to the spillway design flow. Division of equation 1 by equation 2 results in the equation

$$\frac{Q}{Q_d} = u\left(\frac{H_e}{H_d}\right)^{3/2} \qquad (3)$$

where $u = \frac{C}{C_d}$. Model data for ten widely varying spillway designs have been analyzed in accordance with the parameters of equation 3. The results are shown in Chart 111-3/3. The spillway design features for each project are tabulated below.

Project	Ref No.	Upstream Face Slope	Abutment Condition	No. of Bays	Approach Depth (H_d/P)	Downstream Quad Shape
Proctor*	2	3V on 2H	None	2	2.00	$X^{1.81}$
Pine Flat	3	Vertical	Concrete	6	0.11	$X^{1.85}$
Gavins Point	4	3V on 2H	Earth	14	1.64	$X^{1.78}$
Two-Dimensional	5	Vertical	None	--	0.3-0.4	$X^{1.85}$
Oakley	6	3V on 3H	Earth	4	2.2	$X^{1.747}$
Hugo	7	Vertical	Earth	6	4.3	$X^{1.85}$
Fort Randall	8	Vertical	Earth	21	1.7	$X^{1.85}$
John Redmond	9	Vertical	Earth	14	8.2	$X^{1.776}$
Kaysinger Bluff**	10	1V on 0.5H	Earth and concrete	4	0.74	$X^{1.825}$
Clarence Cannon†	11	Vertical	Concrete	4	0.40	$X^{1.85}$

* Section model (1 full and 2 half bays).
** Includes effects of 90-deg approach channel bend.
† Includes effects of adjacent outlet works and water quality weir.

4. The plotted points on Chart 111-3/3 can be expressed by the general equation:

$$\frac{Q}{Q_d} = u\left(\frac{H_e}{H_d}\right)^n \tag{4}$$

For $Q/Q_d = 1.0$ and $H_e/H_d = 1.0$, the value of u is 1.0 and the value of n has been graphically determined to be 1.60. Therefore, the equation fitting all the data within experimental accuracy limits is

$$Q = Q_d\left(\frac{H_e}{H_d}\right)^{1.60} \tag{5}$$

5. Application.

a. Design. The spillway design flow Q_d is computed using appropriate coefficients and equations given in Charts 111-3 to 111-3/2, 111-5, 111-6, 122-1 to 122-2, and 122-4 for the spillway design head H_d . Equation 5 is then solved for the desired ratios of H_e/H_d using the computed Q_d value. In a graphical solution, values of Q/Q_d are read from the chart for the selected values of H_e/H_d . These discharge ratios are then multiplied by Q_d to obtain the required Q values.

b. Operation. The theoretical or model rating curve of a spillway can be checked for the full range of spillway heads provided one accurate prototype discharge and corresponding spillway head measurement are available. Equation 5 is solved for the design discharge Q_d using the measured discharge Q , and the ratio of the measured head H to the design H_d . The derived value of Q_d is then used in equation 5 with selected values of H/H_d to obtain the required discharge quantities. If preferred, a graphical solution similar to that described in 5a above can be used.

6. References.

(1) U. S. Bureau of Reclamation, Studies of Crests for Overfall Dams; Hydraulic Investigations. Bulletin 3, Part VI, Boulder Canyon Project Final Reports, Denver, Colo., 1948.

(2) U. S. Army Engineer Waterways Experiment Station, CE, Spillway for Proctor Dam, Leon River, Texas; Hydraulic Model Investigation. Technical Report No. 2-645, Vicksburg, Miss., March 1964.

(3) __________, Spillway and Conduits for Pine Flat Dam, Kings River, California; Hydraulic Model Investigation. Technical Memorandum No. 2-375, Vicksburg, Miss., December 1953.

(4) __________, Spillway for Gavins Point Dam, Missouri River, Nebraska; Hydraulic Model Investigation. Technical Memorandum No. 2-404, Vicksburg, Miss., May 1955.

(5) __________, Investigation 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.

(6) __________, Spillway for Oakley Dam, Sangamon River, Illinois; Hydraulic Model Investigation, by E. S. Melsheimer. Technical Report H-70-13, Vicksburg, Miss., November 1970.

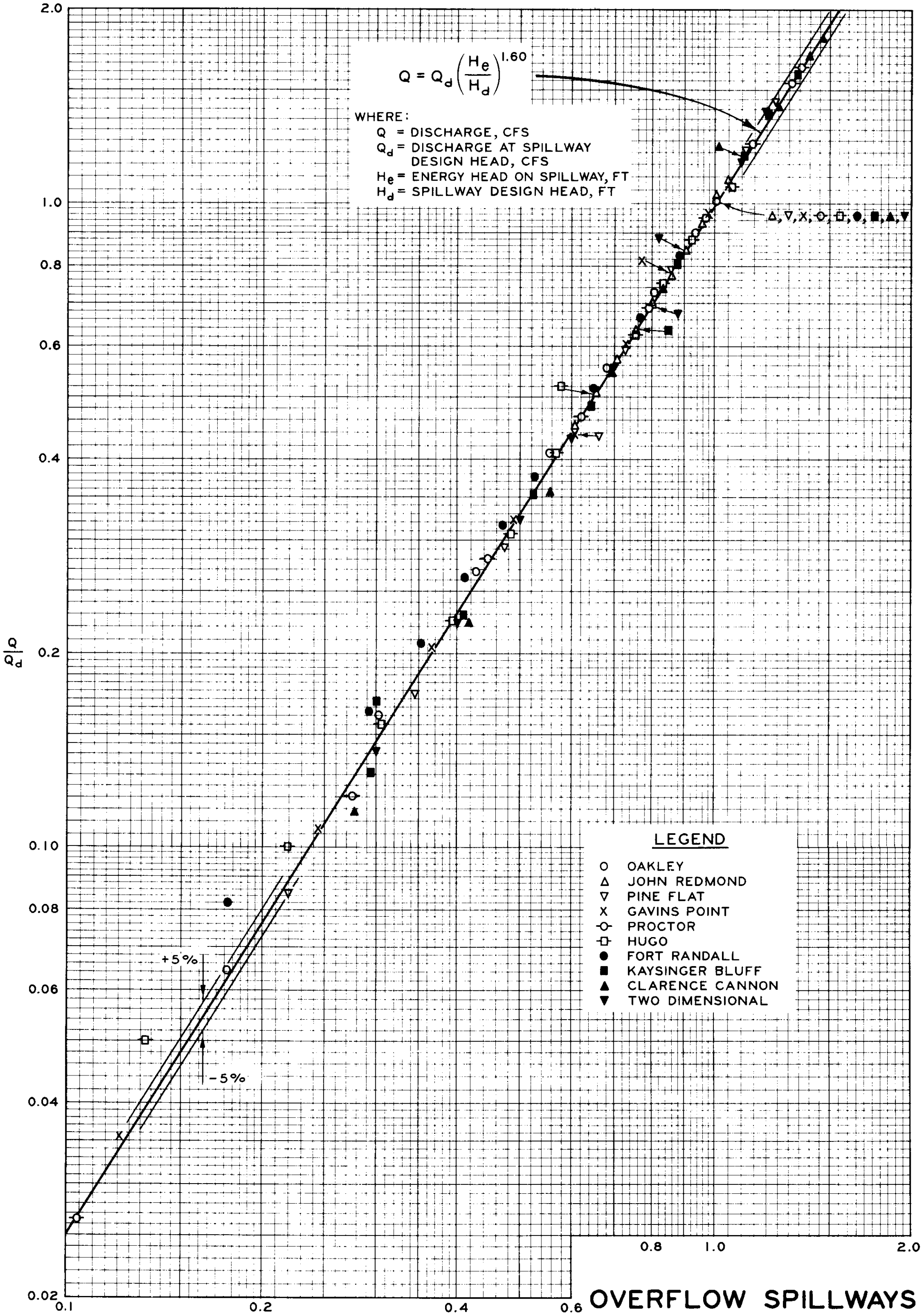
(7) __________, Spillway for Hugo Dam, Kiamichi River, Oklahoma; Hydraulic Model Investigation, by B. P. Fletcher and J. L. Grace, Jr., Technical Report H-69-15, Vicksburg, Miss., November 1969.

(8) __________, Spillway and Outlet Works, Fort Randall Dam, Missouri River, South Dakota; Hydraulic Model Investigation. Technical Report No. 2-528, Vicksburg, Miss., October 1959.

(9) U. S. Army Engineer Waterways Experiment Station, CE, Spillway for John Redmond Dam, Grand (Neosho) River, Kansas; Hydraulic Model Investigation. Technical Report No. 2-611, Vicksburg, Miss., November 1962.

(10) __________, Spillway for Kaysinger Bluff Dam, Osage River, Missouri; Hydraulic Model Investigation. Technical Report No. 2-809, Vicksburg, Miss., January 1968.

(11) __________, Spillway for Clarence Cannon Reservoir, Salt River, Missouri; Hydraulic Model Investigation, by B. P. Fletcher. Technical Report H-71-7, Vicksburg, Miss., October 1971.



OVERFLOW SPILLWAYS
STAGE-DISCHARGE RELATION UNSUBMERGED CRESTS

HYDRAULIC DESIGN CHART 111-3/3

WES 2-72

HYDRAULIC DESIGN CRITERIA

SHEETS 111-4 TO 111-4/2

SUBMERGED CREST COEFFICIENTS

OVERFLOW DAMS

1. Background. A number of important experiments on submerged sharp-crested weirs were made in the 19th century. The submerged weir coefficients based on Herschel's analysis and republished in King's *Handbook of Hydraulics* have been widely used. Coefficients for the more modern shapes of submerged, round-crested weirs have been determined by various experiments. However, the results have not been widely publicized and are not generally available to the design engineer. The experiments of Cox (reference 4) were published, and the extensive test program of the U. S. Bureau of Reclamation was reported by Bradley (references 2 and 3).

2. Bureau of Reclamation Tests. The form of plotting of the variables used on Hydraulic Design Chart 111-4 was devised by Bradley. The family of curves shows various reductions in percent from the coefficient for free or unsubmerged flow as presented on Hydraulic Design Charts 111-21, 111-21/1, and 122-1. The general pattern of the curves shows that for low ratios of total drop from upper pool to apron floor divided by head on the crest $(h_d + d)/H_e$, the flow is supercritical and the decrease in coefficient is principally affected by this ratio. The cross section B-B in the upper right-hand corner of Chart 111-4 shows the variations of $(h_d + d)/H_e$ at h_d/H_e of 0.78. For large values of $(h_d + d)/H_e$, the decrease in coefficient is principally affected by the ratio h_d/H_e. For values of h_d/H_e less than 0.10, the jet is on the surface and no jump occurs. The cross section A-A shows the variations of h_d/H_e at $(h_d + d)/H_e$ of 5.0.

3. Current Analysis. The experimental observations of the Bureau of Reclamation were plotted on the same graph with those of other experimenters (references 1, 5-7). The combined data produced 201 experimental observations. The complete plot of the test points nearly obliterates the coefficient reduction curves and for this reason was omitted from the chart. The current analysis results in some deviation from the Bradley curves which were omitted from the chart for the sake of clarity and utility.

4. Application. The curves shown on Chart 111-4 were based on three different test conditions of the individual experimenters as follows: the approach and apron floors at the same constant elevation, both floors at the same elevation but varied with respect to the crest,

and the approach floor elevation held constant with the apron elevation varied. The decrease in the coefficient was based on the unsubmerged coefficient for the condition tested. The general agreement of the percentage decreases in coefficients from the five independent studies indicated that these values may be used in combination with Chart 111-21 or 111-21/1 to determine coefficients for higher velocities of approach.

5. Discharge Rating Curves. Chart 111-4/1 can be used in the determination of the limits of the effects of submergence. It is taken from data on typical ogee spillways (reference 8). In these tests, the P/H_d ratio varied from 0.12 to 1.67 and the exit channel elevations varied from 0.55 to $1.67H_d$ below the spillway crest. For set discharges the tailwater was slowly raised from zero submergence effect to 100 percent submergence. A comparison of these data with those of Koloseus (reference 5) confirms their wide range of applicability. Chart 111-4/2 illustrates the use of these submergence charts in the construction of a discharge rating curve.

6. References.

(1) Barshany, M., "Pressure Distribution on Downstream Face of a Submerged Weir," Thesis, State University of Iowa, Iowa City, Iowa, June 1950.

(2) Bradley, J. N., "Studies of Flow Characteristics, Discharge and Pressures Relative to Submerged Dams," Hydraulic Laboratory Report No. 182, Bureau of Reclamation, Denver, Colo., 1945.

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

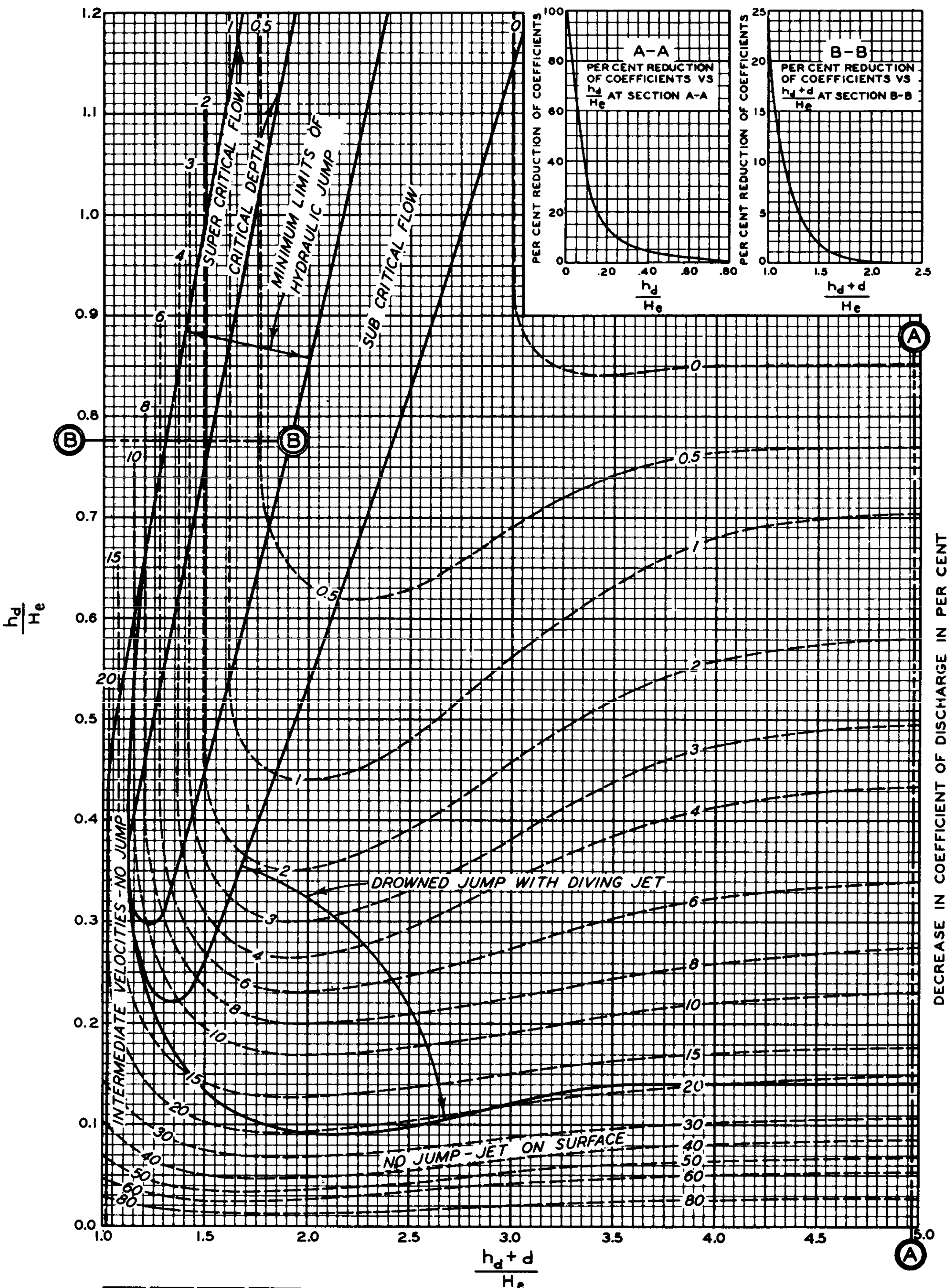
(4) Cox, G. N., "The Submerged Weir as a Measuring Device," University of Wisconsin, Engineering Experiment Station, Bulletin No. 67, Madison, Wis., 1928.

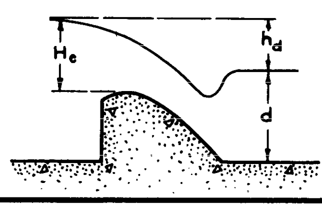
(5) Koloseus, H. J., "Discharge Characteristics of Submerged Spillways," Colorado Agricultural and Mechanical College, Fort Collins, Colo., Dec. 1951.

(6) U. S. Army Engineer Waterways Experiment Station, CE, "Morgantown Spillway, Special Tests," Vicksburg, Miss. (unpublished).

(7) __________, "Spillway and Lock Approach, Jim Woodruff Dam, Apalachicola River, Florida; Model Investigation," Technical Memorandum No. 2-340, Vicksburg, Miss., May 1952.

(8) __________, "Typical Spillway Structure for Central and Southern Florida Water-Control Project," Technical Report No. 2-633, Vicksburg, Miss., September, 1963.



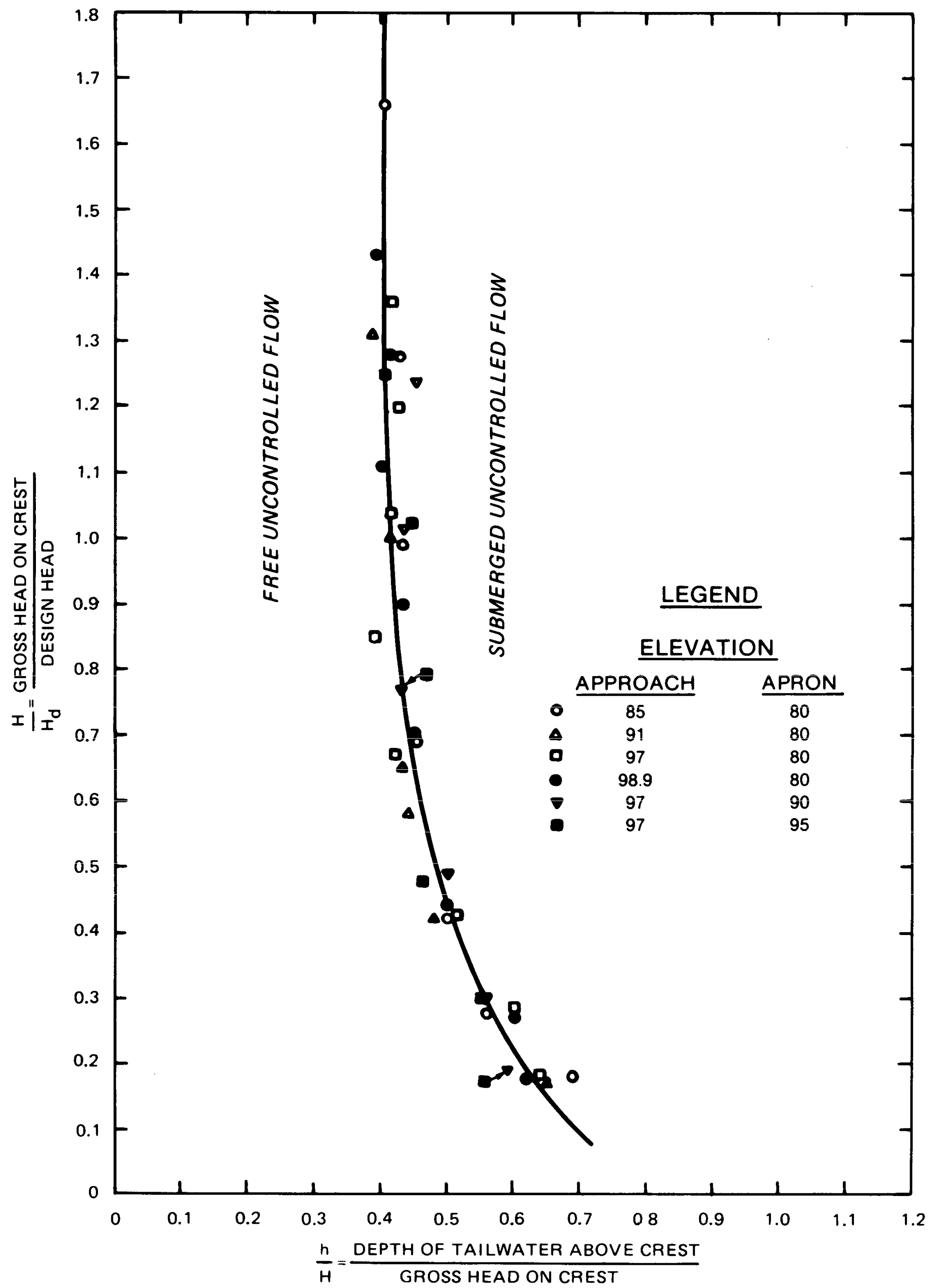


SUBMERGED CREST COEFFICIENTS

OVERFLOW CRESTS

HYDRAULIC DESIGN CHART 111-4

WES 4-1-53



SUBMERGED CREST COEFFICIENTS
UNCONTROLLED FLOW REGIMES
HYDRAULIC DESIGN CHART III-4/1

U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB GS 801 **PROJECT** BURLINGTON **SUBJECT** SUBMERGED FLOW

COMPUTATION DISCHARGE RATING CURVE

COMPUTED BY JVM **DATE** 3-14-83 **CHECKED BY** AJR **DATE** 10-3-83

1. MAX POOL EL 1,625 FT, CREST EL = 1,600 FT, H_d = 25 FT, L = 129 FT, P = 9 FT

   USE: 1,605, 1,610, 1,615, 1,620, 1,622.5, & 1,625 FT AS RANGE OF HEADWATER ELEVATIONS.

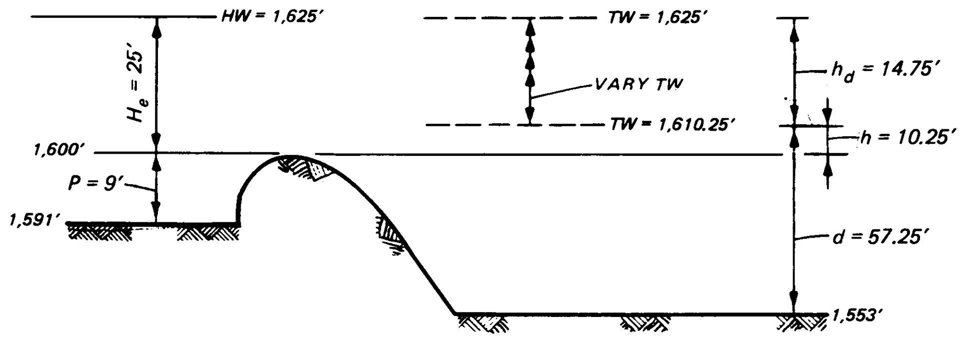
2. FOR HW = 1,625 FT, $P/H_d = 0.36$, $H_e/H_d = 1.0$, C = 3.88 (HDC 111-21)

   FROM HDC 111-4/1 (RANGE OF SUBMERGED FLOWS)

   $H_e/H_d = 1.0$ $h = 0.41\ H_e$ $h = 10.25'$

   Q (FREE FLOW) $= (3.88)(129)(25)^{3/2}$ $= 62{,}565$ CFS

   LOWER LIMIT = 1,600 + 10.25 = 1,610.25'
   UPPER LIMIT = 1,625'



3. FOR TW = 1,610.25 $h_d = 14.75'$ $H_e = 25'$ % REDUCTION FROM HDC 111-4 = 0.82%

   d = 57.25'

   $$\frac{h_d}{H_e} = 1 - \frac{10.25}{25.0} = 0.59'$$

   $$Q = (3.85)(129)(25)^{3/2} = 62{,}052 \text{ CFS}$$

   $$C_s = C\left(1.0 - \frac{\%}{100}\right) = 3.88\left(1.0 - \frac{0.82}{100}\right) = 3.85$$

   $$\frac{d + h_d}{H_e} = \frac{57.25 + 14.75}{25} = 2.88$$

   h = 10.25'

TW	h_d	d	h_d/H_e	$\frac{d+h_d}{H_e}$	%	C_s	Q
1,610.25	14.75	57.25	0.59	2.88	0.82	3.85	62,050
1,615	10	62	0.40	2.88	2.5	3.78	61,000
1,620	5	67	0.20	2.88	9.3	3.52	56,750
1,624	1	71	0.04	2.88	60.0	1.55	25,030

4. REPEAT FOR RANGE OF HW'S AND PLOT Q VS. TW FOR HW
5. REPLOT IN TRANSPOSED FORMAT: HW VS. TW FOR A GIVEN Q

SUBMERGED CREST COEFFICIENTS
TAILWATER EFFECT
EXAMPLE CALCULATION
HYDRAULIC DESIGN CHART 111-4/2

HYDRAULIC DESIGN CRITERIA

SHEETS 111-5 AND 111-6

GATED OVERFLOW SPILLWAYS

PIER CONTRACTION COEFFICIENTS

1. General. The basic equation for computing flow over a spillway crest is given in Hydraulic Design Criteria (HDC) Sheet 111-3. The crest length L to be used in the equation is defined in paragraph 1 of Sheet 111-3/1. The length L includes both abutment and pier effects which result in a reduction of the net crest length. The net crest length is the gross spillway width less the combined thicknesses of the crest piers.

2. Abutment effects are described in Sheets 111-3/1 and 111-3/2, and design criteria are given in Hydraulic Design Charts 111-3/1 and 111-3/2. Design criteria for the effects of piers are given in HDC 111-5, 111-6, and 122-2. Both abutment and pier effects increase proportionally with skewness of flow and with velocity of approach. In such cases, increasing the contraction coefficients is recommended.

3. When spillways are operated with one or more bays closed, the piers adjacent to these bays produce abutment-type effects and result in greater flow contractions than when the flow is evenly divided around the piers. HDC 111-3/1 should be used to estimate contraction coefficients when piers function essentially as abutments because of closed bays.

4. Previous Criteria. Many gated spillways have been designed according to Creager and Justin[1] who recommended a coefficient K_p of 0.1 for thick, blunt piers to 0.04 for thin or pointed piers. The research of Escande and Sabathe[2] indicated that the coefficient can be smaller than 0.04, and their observations were verified in general by the Center Hill spillway model tested at the U. S. Army Engineer Waterways Experiment Station (WES). The data for types 1, 2, 3, and 4 piers presented in the accompanying charts are based on results of tests conducted in 1949 at the WES under Corps of Engineers Engineering Studies Item 801, General Spillway Investigation. The type 3A pier coefficient curve results from 1969 studies.[3]

5. 1949 Tests. The 1949 tests were made with a standard spillway crest shape essentially as defined in Chart 111-2/1, but without the short radius of $0.04\ H_d$. A model design head of 0.75 ft was used. The spillway crest shape was installed in a glass flume having parallel sidewalls to eliminate the effects of end or abutment contraction. The piers were 0.2 ft thick and the clear distance between piers was 1.0 ft. Thus the prototype design head would be 30 ft, the pier thickness 8 ft, and the clear span 40 ft if a 1:40-scale ratio was adopted. Tests for other ratios of pier thickness to gate width were not made. Type 4 pier

(Chart 111-5) resulted in the most favorable pier contraction coefficient curve. However, type 4 pier and type 1 pier are the least desirable from the standpoint of development of negative pressures and are not recommended for high heads unless a thorough investigation of pressure conditions is made.

6. 1969 Tests.[3] The 1969 tests were similar to the 1949 tests except the upstream quadrant of the spillway terminated with the short radius ($R = 0.04\ H_d$) shown in Chart 111-2/1 to eliminate a surface discontinuity at the spillway vertical face. A design head of 1.0 ft was used. Tests were limited to type 3A pier shape.

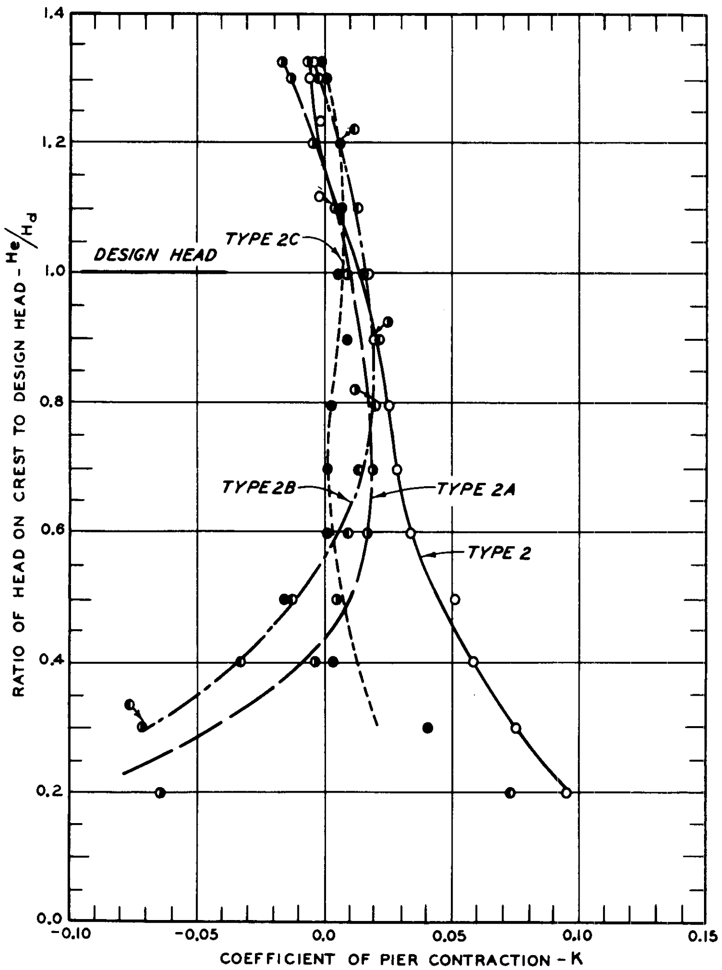
7. Application. Chart 111-5 gives the contraction coefficients for five different pier-nose shapes plotted against the ratio of H_e/H_d . In each case the pier nose was at the plane of the upstream face of the spillway. Types 2, 3, and 3A are recommended for general use with high heads. Chart 111-6 represents tests on pier type 2 nose shape located at variable distances upstream from the crest. Similar data are not available for the other three nose shapes. The noses of pier types 2A, 2B, and 2C were $0.133\ H_d$, $0.267\ H_d$, and $0.533\ H_d$ upstream from the spillway face, respectively. The data apply to the condition of adjacent gates being open.

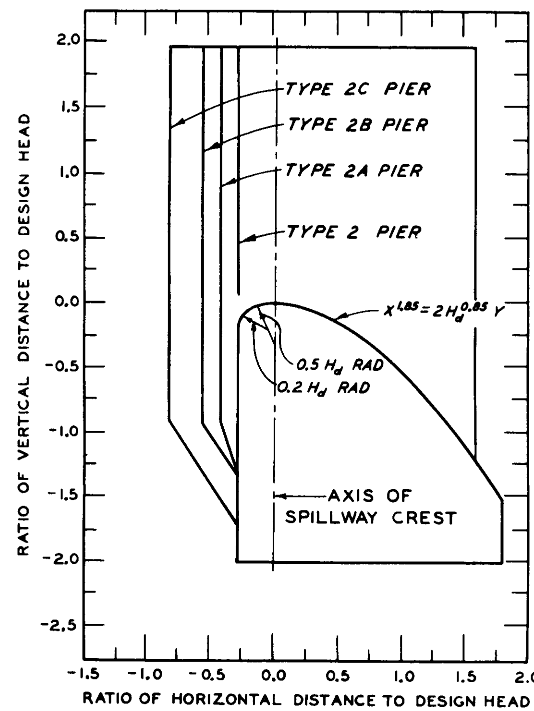
8. References.

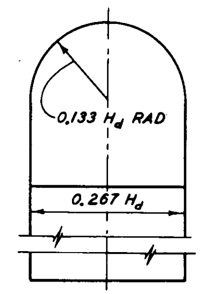
(1) Creager, W. P. and Justin, J. D., Hydroelectric Handbook, 1st ed., John Wiley & Sons, New York, 1927, p 132.

(2) Escande, L. and Sabathe, G., "On the use of aerodynamic profiles for piers of overfall weirs, movable dams and bridges." Revue General de l'Hydraulique (July-August 1936).

(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.







TYPE 2 PIER

HIGH GATED OVERFLOW CRESTS
PIER CONTRACTION COEFFICIENTS
EFFECT OF PIER LENGTH

HYDRAULIC DESIGN CHART 111-6

WES 4-1-53

HYDRAULIC DESIGN CRITERIA

SHEETS 111-7 to 111-10

OVERFLOW SPILLWAY CRESTS WITH SLOPING UPSTREAM FACES

1. Charts 111-7 to 111-10 supplement Chart 111-1. These charts present suggested shapes for design of spillway crests with sloping upstream faces and negligible approach velocity. They are based on U. S. Bureau of Reclamation (USBR) data (references 1 and 2). These charts remain in the Hydraulic Design Criteria (HDC) to serve as a design reference for spillways which were designed using this guidance. For current design purposes these charts have been superseded by Chart 111-20.

2. Crest Shapes.

a. 3V on 1H and 3V on 2H Upstream Face Slopes. The crest shapes presented in Charts 111-7 and 111-8 apply to spillways with upstream face slopes of 3V on 1H and 3V on 2H, respectively. Equations for the downstream face of the spillway result from the best fit of the general equation $X^n = KH_d^{n-1}Y$ to the experimental data published by USBR (reference 1). Each chart contains tables of functions necessary for solution of the equations. The shape of the crest upstream from the axis results from fitting circular arcs to the experimental data.

b. 3V on 3H Upstream Face Slope. The downstream quadrant crest shape presented in Chart 111-9 applies to spillways with 3V on 3H upstream face slopes. The equation for the downstream shape is based on curves published by USBR (reference 2) and reproduced in Chart 122-3/1. The published curves have been confirmed by an independent U. S. Army Engineer Waterways Experiment Station (WES) study of the USBR data for weirs sloping 45 deg downstream. Tables of functions necessary for solution of the equation are included in Chart 111-9. A tabulation of the slope of the downstream crest shape is given to aid in locating the beginning of the toe curve or the sloping tangent face. A WES study to determine equations for upstream quadrant shapes of spillways having 45-deg sloping upstream faces is summarized in paragraph 5 of the HDC Sheets 122-3 to 122-3/5, and the general results are presented in Chart 122-3/4. The upstream quadrant coordinates listed below are based on this study and have their origins at the apex of the spillway. The X and Y coordinates are considered positive to the right and downward, respectively.

Upstream Quadrant Coordinates			
X/H_d	Y/H_d	X/H_d	Y/H_d
-0.000	0.0000	-0.150	0.0239
-0.020	0.0004	-0.155	0.0257
-0.040	0.0016	-0.160	0.0275
-0.060	0.0036	-0.165	0.0293
-0.080	0.0065	-0.170	0.0313
-0.100	0.0103	-0.175	0.0333
-0.110	0.0125	-0.180	0.0354
-0.120	0.0150	-0.185	0.0376
-0.130	0.0177	-0.190	0.0399
-0.140	0.0207	-0.195	0.0424
-0.145	0.0223	-0.200	0.0450

3. Intermediate Shapes. Chart 111-10 shows computed values of n and K for the equation $X^n = KH_d^{n-1}Y$ for the vertical and sloping weirs shown in Charts 111-1 and 111-7 to 111-9. Values of n and K which apply to intermediate slopes can be approximated using the curves shown in Chart 111-10. Upstream quadrant data for intermediate slopes are not presently available. However, the circular arcs given in Charts 111-7 and 111-8 and the coordinates given in the above tabulation should serve for interpolating the approximate shape for design purposes.

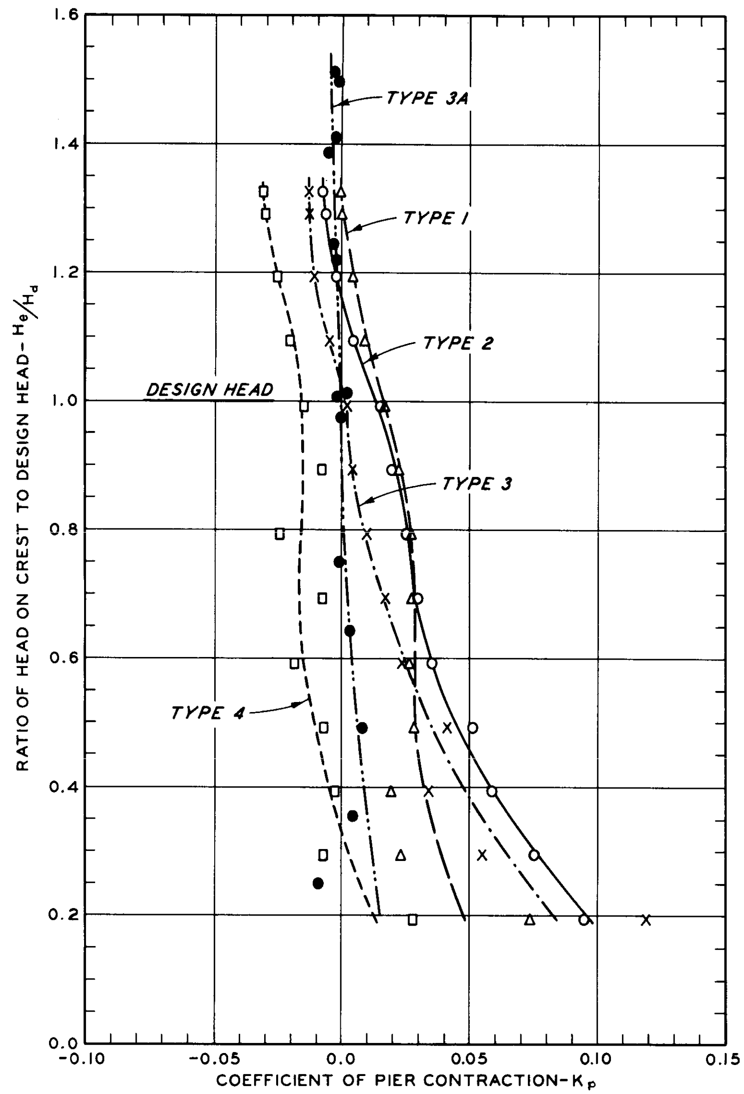
4. The crest shapes defined in Charts 111-7 to 111-10 have not been studied in a model to determine pressure and discharge coefficients.

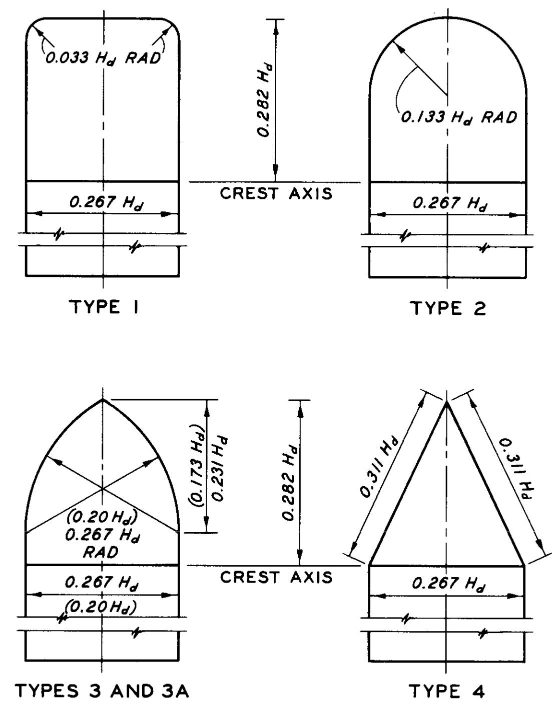
5. Application. The shapes shown are intended for use with overflow dams with negligible velocity of approach flows. Model tests of spillways with vertical faces have shown that pressures and discharge coefficients are approximately the same for approach depth-design head ratios P/H_d greater than 1. It is therefore suggested that the shapes in Charts 111-7 to 111-9 be used in this range, unless in a particular case it is especially desired to keep pressures as close to atmospheric pressure as possible. In the latter case, Charts 111-20 and 111-24 through 111-25/1 should be used as appropriate.

6. References.

(1) 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.

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





PIER NOSE SHAPES

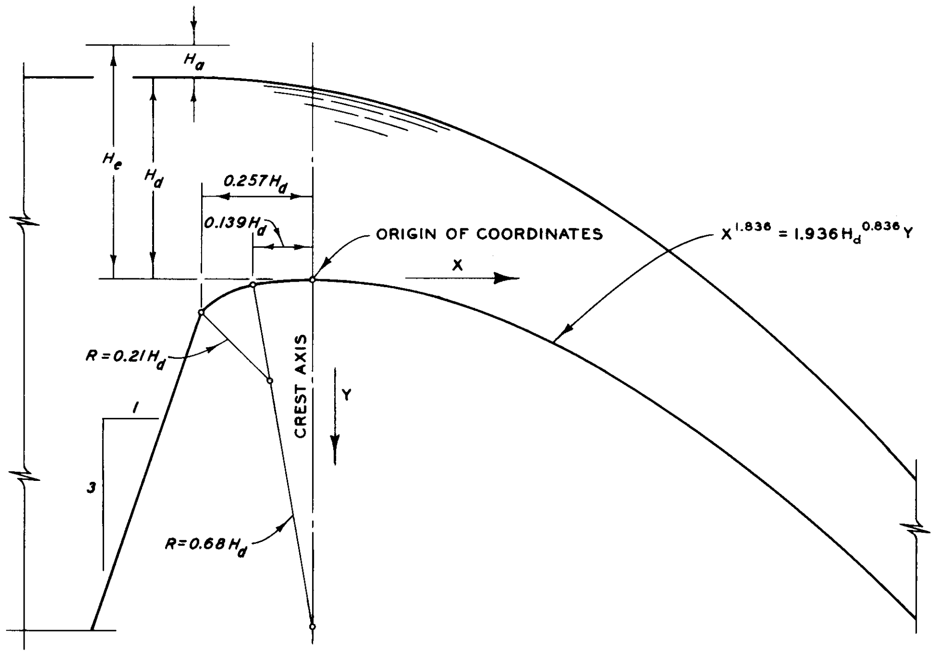
NOTE: PIER NOSE LOCATED IN SAME PLANE AS UPSTREAM FACE OF SPILLWAY.
DIMENSIONS IN PARENTHESES ARE FOR TYPE 3A.

HIGH GATED OVERFLOW CRESTS
PIER CONTRACTION COEFFICIENTS
EFFECT OF NOSE SHAPE
HYDRAULIC DESIGN CHART 111-5

REV 9-70

WES 4-53

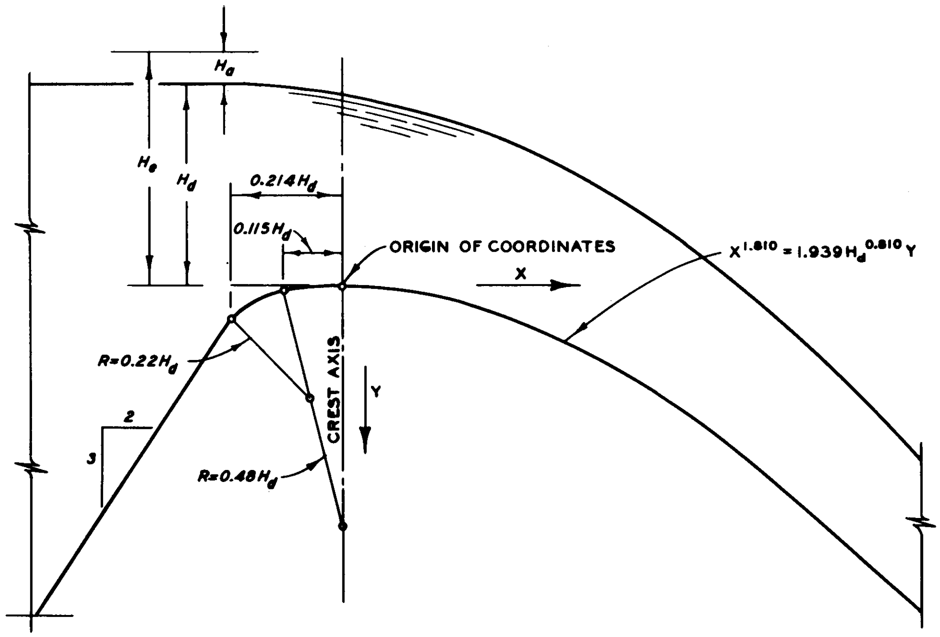
PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI



X	$X^{1.836}$	X	$X^{1.836}$	H_d	1.936 x $H_d^{0.836}$	H_d	1.936 x $H_d^{0.836}$	H_d	1.936 x $H_d^{0.836}$
.10	.0146	6	26.834	1	1.936	26	29.500	51	51.812
.15	.0307	7	35.612	2	3.456	27	30.446	52	52.660
.20	.0521	8	45.507	3	4.851	28	31.386	53	53.506
.25	.0785	9	56.492	4	6.169	29	32.320	54	54.348
.30	.1097	10	68.549	5	7.434	30	33.249	55	55.188
.35	.1455	12	95.803	6	8.659	31	34.173	56	56.026
.40	.1859	14	127.143	7	9.849	32	35.092	57	56.861
.45	.2308	16	162.467	8	11.013	33	36.007	58	57.694
.50	.2801	18	201.688	9	12.152	34	36.917	59	58.525
.60	.3915	20	244.732	10	13.271	35	37.822	60	59.353
.70	.5195	25	368.653	11	14.372	36	38.724	61	60.178
.80	.6639	30	515.221	12	15.456	37	39.621	62	61.002
.90	.8241	35	683.768	13	16.526	38	40.514	63	61.824
1.00	1.000	40	873.740	14	17.582	39	41.403	64	62.643
1.20	1.398	45	1084.673	15	18.626	40	42.289	65	63.460
1.40	1.855	50	1316.161	16	19.659	41	43.171	66	64.275
1.60	2.370	55	1567.855	17	20.681	42	44.050	67	65.089
1.80	2.942	60	1839.441	18	21.693	43	44.925	68	65.899
2.00	3.570	65	2130.631	19	22.696	44	45.797	69	66.709
2.50	5.378	70	2441.180	20	23.690	45	46.665	70	67.516
3.00	7.517	75	2770.847	21	24.676	46	47.530	71	68.321
3.50	9.975	80	3119.415	22	25.655	47	48.393	72	69.125
4.00	12.746	90	3872.480	23	26.626	48	49.252	73	69.927
4.50	15.823	100	4698.941	24	27.591	49	50.108	74	70.727
5.00	19.200			25	28.548	50	50.962	75	71.525

NOTE: EQUATION BASED ON DATA FOR NEGLIGIBLE VELOCITY OF APPROACH PUBLISHED IN BULLETIN 3, BOULDER CANYON REPORT, USBR, 1948.

OVERFLOW SPILLWAY CREST
3 ON 1 UPSTREAM FACE
HYDRAULIC DESIGN CHART 111-7

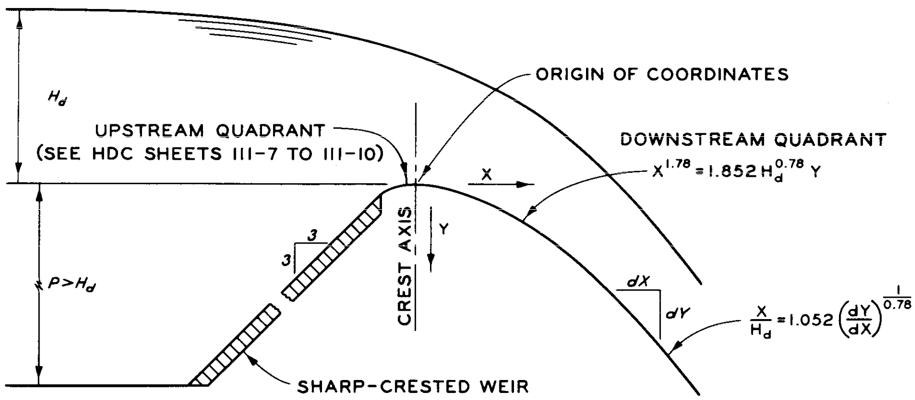


X	$X^{1.810}$	X	$X^{1.810}$	H_d	1.939 x $H_d^{0.810}$	H_d	1.939 x $H_d^{0.810}$	H_d	1.939 x $H_d^{0.810}$
.10	.0155	6	25.613	1	1.939	26	27.146	51	46.850
.15	.0323	7	33.855	2	3.399	27	27.989	52	47.593
.20	.0543	8	43.111	3	4.721	28	28.825	53	48.333
.25	.0813	9	53.355	4	5.960	29	29.657	54	49.070
.30	.1131	10	64.565	5	7.141	30	30.482	55	49.805
.35	.1495	12	89.809	6	8.277	31	31.303	56	50.537
.40	.1904	14	118.711	7	9.378	32	32.118	57	51.267
.45	.2357	16	151.167	8	10.449	33	32.929	58	51.994
.50	.2852	18	187.087	9	11.495	34	33.735	59	52.719
.60	.3967	20	226.394	10	12.519	35	34.536	60	53.442
.70	.5244	25	339.056	11	13.524	36	35.333	61	54.162
.80	.6677	30	471.617	12	14.512	37	36.126	62	54.880
.90	.8264	35	623.395	13	15.484	38	36.915	63	55.596
1.00	1.000	40	793.832	14	16.442	39	37.700	64	56.310
1.20	1.391	45	982.459	15	17.386	40	38.481	65	57.021
1.40	1.839	50	1188.874	16	18.320	41	39.258	66	57.731
1.60	2.341	55	1412.721	17	19.242	42	40.032	67	58.438
1.80	2.898	60	1653.689	18	20.153	43	40.804	68	59.144
2.00	3.506	65	1911.496	19	21.056	44	41.569	69	59.847
2.50	5.251	70	2185.885	20	21.949	45	42.333	70	60.549
3.00	7.304	75	2476.629	21	22.834	46	43.093	71	61.249
3.50	9.655	80	2783.511	22	23.710	47	43.851	72	61.947
4.00	12.295	90	3444.918	23	24.580	48	44.605	73	62.643
4.50	15.217	100	4168.694	24	25.442	49	45.356	74	63.337
5.00	18.413			25	26.297	50	46.105	75	64.029

NOTE: EQUATION BASED ON DATA FOR NEGLIGIBLE VELOCITY OF APPROACH PUBLISHED IN BULLETIN 3, BOULDER CANYON REPORT USBR, 1948.

OVERFLOW SPILLWAY CREST
3-ON-2 UPSTREAM FACE

HYDRAULIC DESIGN CHART 111-8



NOTE: EQUATION BASED ON USBR CURVES, HDC 122-3/1, $h_a/H_d = 0.00$.

DOWNSTREAM QUADRANT DATA

X	$X^{1.78}$	X	$X^{1.78}$	H_d	$1.852 \times H_d^{0.78}$	H_d	$1.852 \times H_d^{0.78}$
0.10	0.0165	6	24.272	1	1.852	26	23.512
0.15	0.0341	7	31.936	2	3.180	27	24.214
0.20	0.0569	8	40.504	3	4.363	28	24.911
0.25	0.0847	9	49.952	4	5.460	29	25.602
0.30	0.1172	10	60.256	5	6.498	30	26.288
0.35	0.1543	12	83.357	6	7.491	31	26.969
0.40	0.1957	14	109.675	7	8.449	32	27.645
0.45	0.2413	16	139.102	8	9.376	33	28.317
0.50	0.2911	18	171.548	9	10.278	34	28.984
0.60	0.4028	20	206.935	10	11.159	35	29.647
0.70	0.5299	25	307.846	11	12.020	36	30.306
0.80	0.6722	30	425.869	12	12.864	37	30.960
0.90	0.8289	35	560.326	13	13.692	38	31.611
1.00	1.0000	40	710.668	14	14.507	39	32.258
1.20	1.383	45	876.432	15	15.309	40	32.901
1.40	1.820	50	1057.223	16	16.100	41	33.541
1.60	2.309	55	1252.696	17	16.879	42	34.178
1.80	2.847	60	1462.545	18	17.649	43	34.811
2.00	3.434	65	1686.498	19	18.409	44	35.440
2.50	5.109	70	1924.308	20	19.161	45	36.067
3.00	7.068	75	2175.750	21	19.904	46	36.691
3.50	9.300	80	2440.620	22	20.639	47	37.311
4.00	11.794	90	3009.897	23	21.368	48	37.929
4.50	14.546	100	3630.780	24	22.089	49	38.544
5.00	17.546			25	22.803	50	39.156

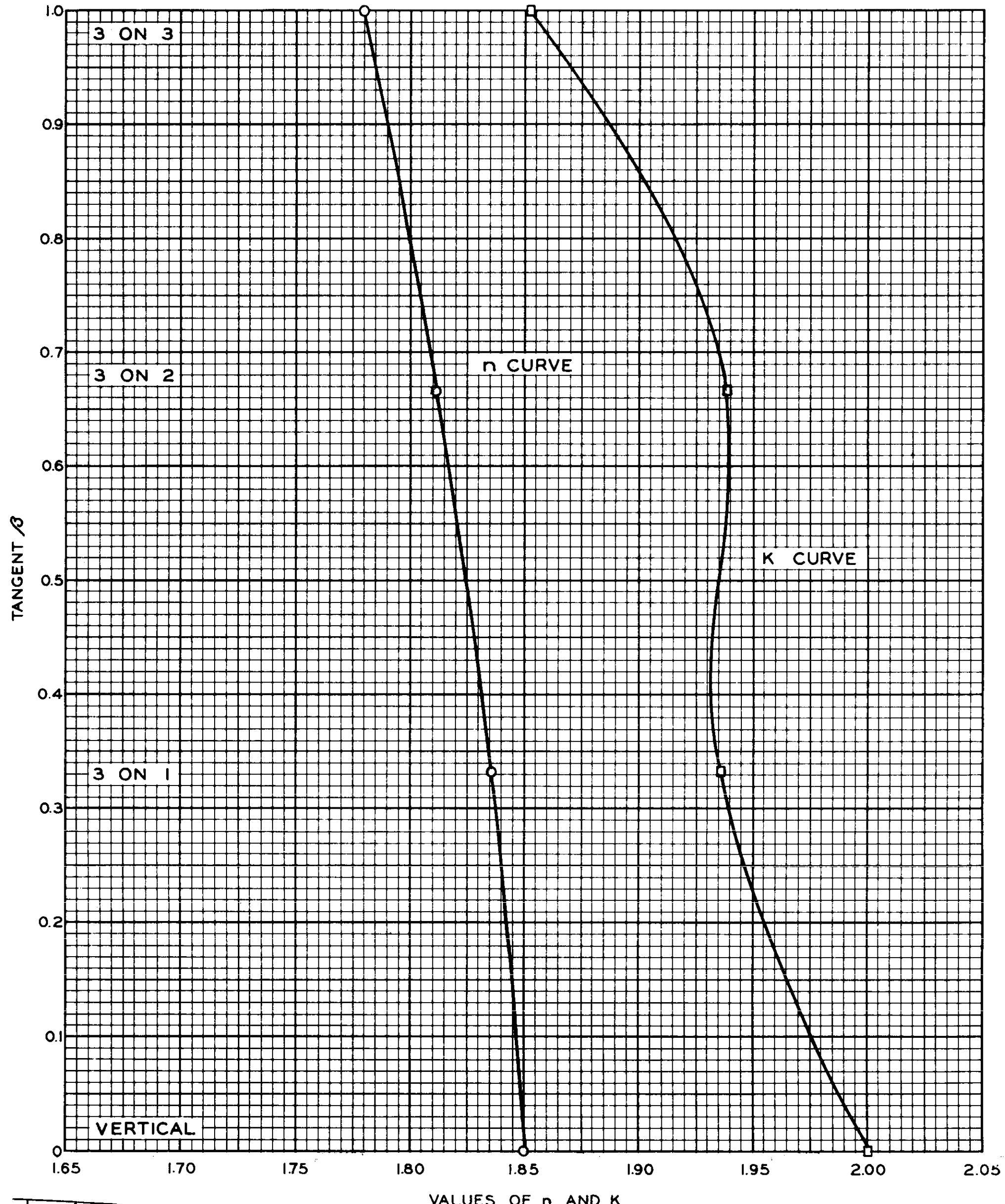
SLOPE DATA

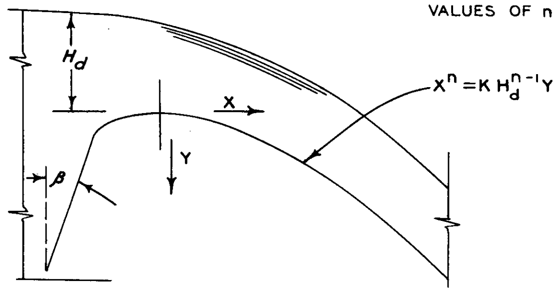
$\frac{dY}{dX}$	$\frac{X}{H_d}$
1.00	1.052
1.05	1.120
1.10	1.189
1.15	1.258
1.20	1.329
1.25	1.400
1.30	1.473
1.35	1.546
1.40	1.619
1.45	1.694
1.50	1.769
1.55	1.845
1.60	1.922
1.65	1.999
1.70	2.077
1.75	2.156
1.80	2.235

OVERFLOW SPILLWAY CREST
3-ON-3 UPSTREAM FACE

HYDRAULIC DESIGN CHART 111-9

REV 1-64 WES 2-54





NEGLIGIBLE VELOCITY OF APPROACH FLOW

OVERFLOW SPILLWAY CREST

n AND K CURVES

HYDRAULIC DESIGN CHART 111-10

REVISED 1-64 WES 2-54

HYDRAULIC DESIGN CRITERIA

SHEETS 111-11 TO 111-14/1

OVERFLOW SPILLWAY CRESTS

UPPER NAPPE PROFILES

1. The shapes of upper nappe profiles for overflow spillway crests are used in the design of spillway abutment walls and in the selection of trunnion elevations for tainter gates. Hydraulic Design Charts 111-11 to 111-14/1 give nappe profile data for spillways without and with abutments and piers and for a sampling of irregular approach flow conditions.

2. Charts 111-11 to 111-12/1 present tabulated coordinates of upper nappe profiles in terms of the design head (H_d) for gated and ungated spillways with head ratios (H/H_d) of 0.50, 1.00, and 1.33. Chart 111-11 is applicable to standard spillway crests of high overflow dams without piers or abutment effects. Charts 111-12 and 12/1 are applicable to center bays of gated spillways without abutment effects. Profiles for intermediate head ratios may be obtained by plotting Y/H_d vs H/H_d for a given X/H_d. The profile coordinates are based on investigations for negligible velocity of approach conducted under CW 801, "General Spillway Tests."

3. Charts 111-13 and 13/1 present graphically upper nappe profiles for three gate bays adjacent to abutments and show the abutment effects on the nappe profiles, based on Pine Flat Dam model test data.(1) The observations were not in sufficient detail to justify definition of the shapes by tabular coordinates.

4. Charts 111-14 and 14/1 present graphically upper nappe profiles along left and right abutments for varying abutment radii and the effects of approach channel irregularities near the abutment (i.e., a water-quality control tower, a power house, and a relatively shallow and irregular approach channel). Chart 111-14 is based on Rowlesburg Dam(2) model test data and Chart 111-14/1 is based on Alum Creek Dam(3) and Clarence Cannon Dam(4) model test data. The observations were not in sufficient detail to justify definition of the shapes by tabular coordinates.

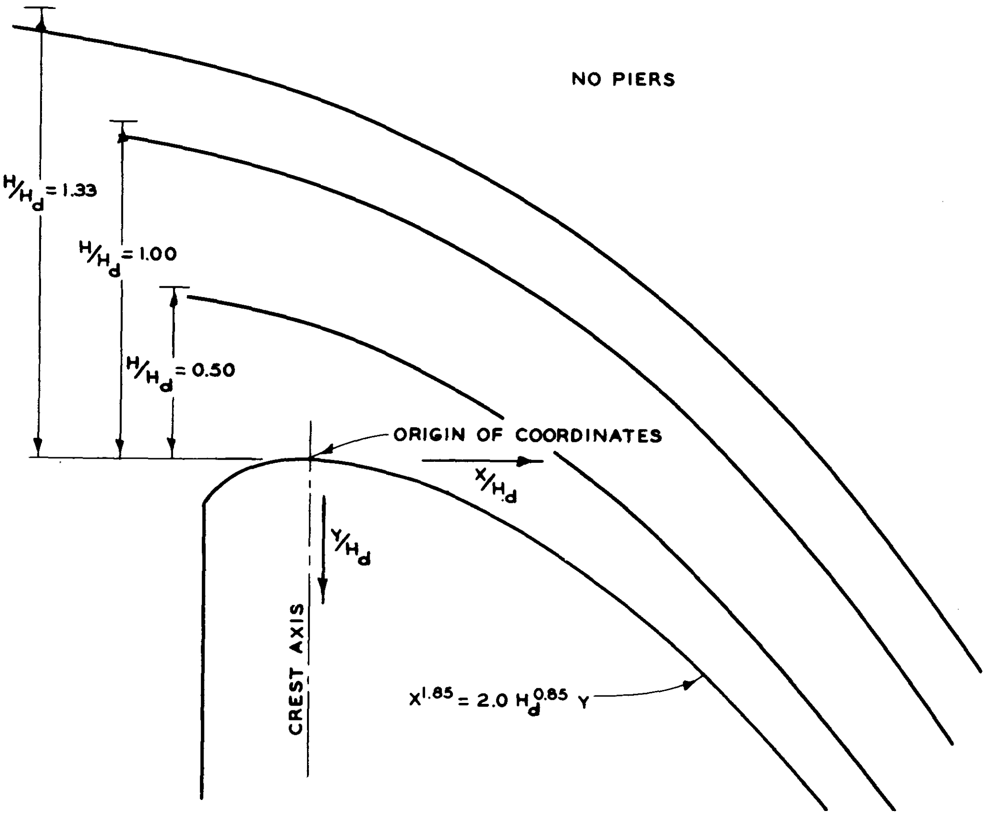
5. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Spillway and Conduits for Pine Flat Dam, Kings River, California, Technical Manual No. 2-375, Vicksburg, Miss., December 1953.

(2) U. S. Army Engineer Waterways Experiment Station , CE, Spillway and Outlet Works, Rowlesburg Dam, Cheat River, West Virginia; Hydraulic Model Investigation, by J. H. Ables, Jr. and M. B. Boyd. Technical Report H-70-7, Vicksburg, Miss., June 1970.

(3) U. S. Army Engineer Waterways Experiment Station, CE, Spillway for Alum Creek Dam, Alum Creek, Ohio; Hydraulic Model Investigation, by G. A. Pickering. Technical Report H-70-4, Vicksburg, Miss., April 1970.

(4) U. S. Army Engineer Waterways Experiment Staiton, CE, Spillway for Clarence Cannon Reservoir, Salt River, Missouri; Hydraulic Model Investigation, by B. P. Fletcher. Technical Report H-71-7, Vicksburg, Miss., October 1971.



COORDINATES FOR UPPER NAPPE WITH NO PIERS*

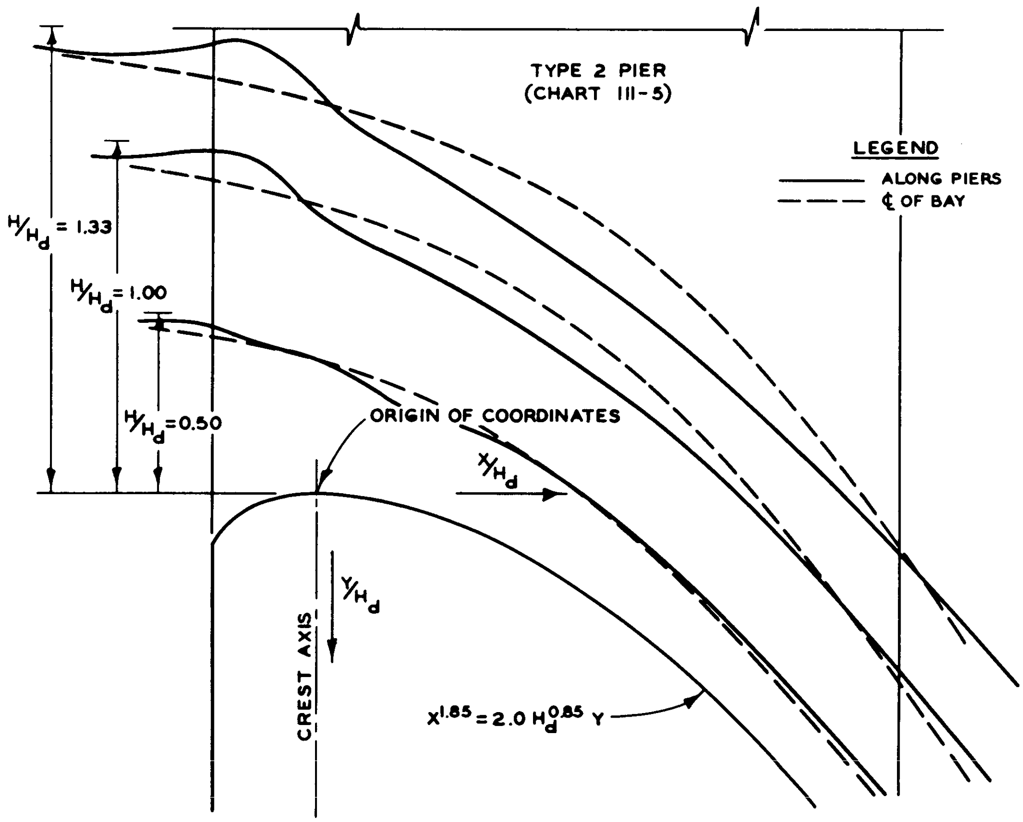
$H/H_d = 0.50$		$H/H_d = 1.00$		$H/H_d = 1.33$	
X/H_d	Y/H_d	X/H_d	Y/H_d	X/H_d	Y/H_d
-1.0	-0.490	-1.0	-0.933	-1.0	-1.210
-0.8	-0.484	-0.8	-0.915	-0.8	-1.185
-0.6	-0.475	-0.6	-0.893	-0.6	-1.151
-0.4	-0.460	-0.4	-0.865	-0.4	-1.110
-0.2	-0.425	-0.2	-0.821	-0.2	-1.060
0.0	-0.371	0.0	-0.755	0.0	-1.000
0.2	-0.300	0.2	-0.681	0.2	-0.919
0.4	-0.200	0.4	-0.586	0.4	-0.821
0.6	-0.075	0.6	-0.465	0.6	-0.705
0.8	0.075	0.8	-0.320	0.8	-0.569
1.0	0.258	1.0	-0.145	1.0	-0.411
1.2	0.470	1.2	0.055	1.2	-0.220
1.4	0.705	1.4	0.294	1.4	-0.002
1.6	0.972	1.6	0.563	1.6	0.243
1.8	1.269	1.8	0.857	1.8	0.531

*BASED ON ES 801 TESTS FOR NEGLIGIBLE VELOCITY OF APPROACH

OVERFLOW SPILLWAY CREST

UPPER NAPPE PROFILES WITHOUT PIERS

HYDRAULIC DESIGN CHART 111-11



COORDINATES FOR UPPER NAPPE ALONG PIERS*

H/H_d = 0.50 X/H_d	Y/H_d	H/H_d = 1.00 X/H_d	Y/H_d	H/H_d = 1.33 X/H_d	Y/H_d
-1.0	-0.495	-1.0	-0.950	-1.0	-1.235
-0.8	-0.492	-0.8	-0.940	-0.8	-1.221
-0.6	-0.490	-0.6	-0.929	-0.6	-1.209
-0.4	-0.482	-0.4	-0.930	-0.4	-1.218
-0.2	-0.440	-0.2	-0.925	-0.2	-1.244
0.0	-0.383	0.0	-0.779	0.0	-1.103
0.2	-0.265	0.2	-0.651	0.2	-0.950
0.4	-0.185	0.4	-0.545	0.4	-0.821
0.6	-0.076	0.6	-0.425	0.6	-0.689
0.8	0.060	0.8	-0.285	0.8	-0.549
1.0	0.240	1.0	-0.121	1.0	-0.389
1.2	0.445	1.2	0.067	1.2	-0.215
1.4	0.675	1.4	0.286	1.4	0.011
1.6	0.925	1.6	0.521	1.6	0.208
1.8	1.177	1.8	0.779	1.8	0.438

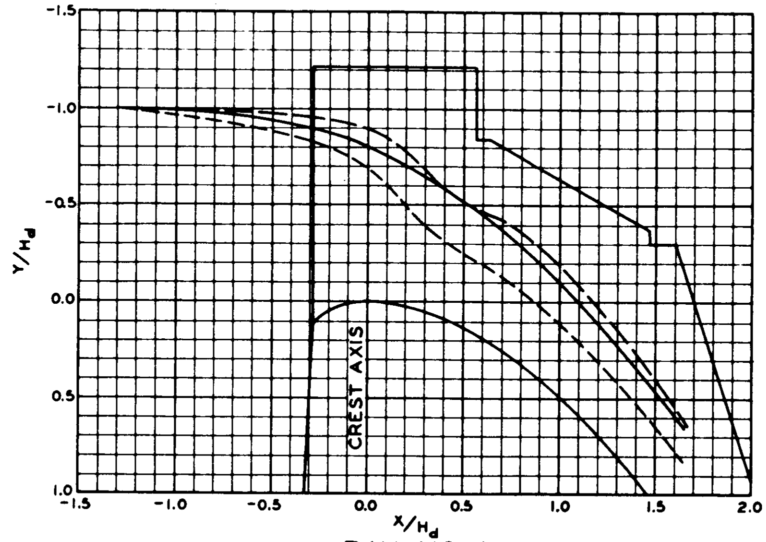
*BASED ON ES 801 TESTS FOR NEGLIGIBLE VELOCITY OF APPROACH

OVERFLOW SPILLWAY CREST

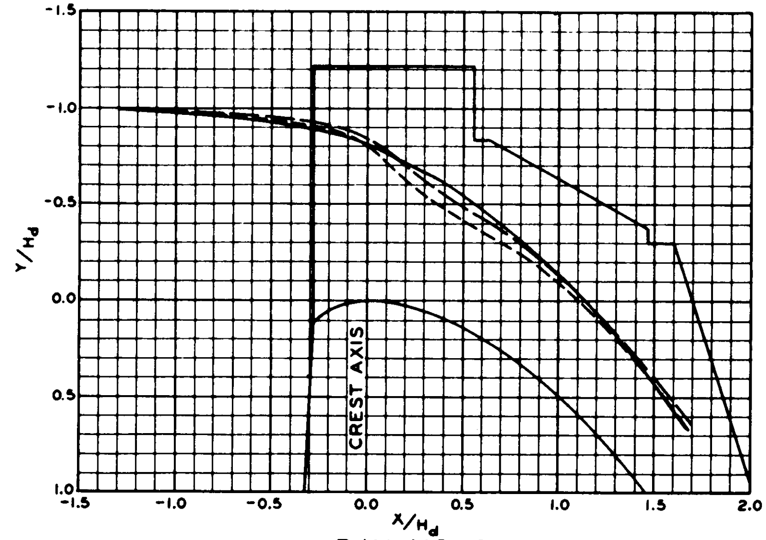
UPPER NAPPE PROFILES ALONG PIERS

HYDRAULIC DESIGN CHART 111-12/1

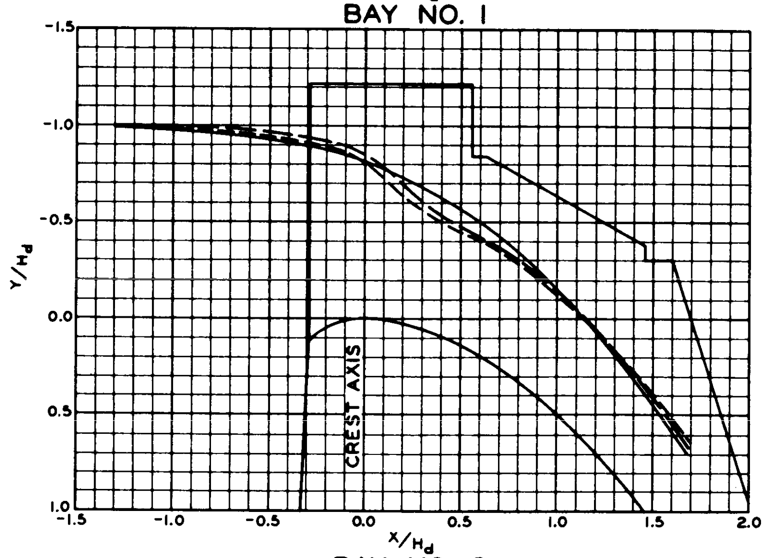
WES 9-54, REV 9-75



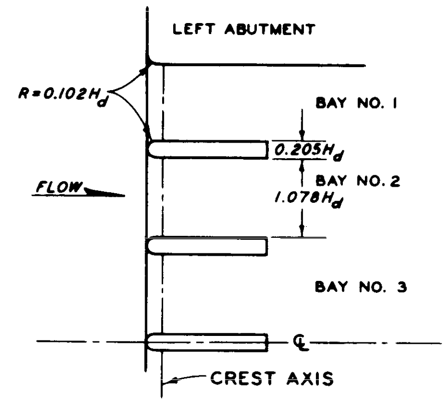
BAY NO. 1



BAY NO. 2



BAY NO. 3



LEGEND

- ------ LEFT SIDE OF BAY
- ——— ℄ OF BAY
- —-—- RIGHT SIDE OF BAY

PROFILES ARE BASED ON PINE FLAT MODEL TESTS FOR SPILLWAY HAVING PIERS SIMILAR TO TYPE 2 (HYDRAULIC DESIGN CHART 111-5).

$H/H_d = 1.00$

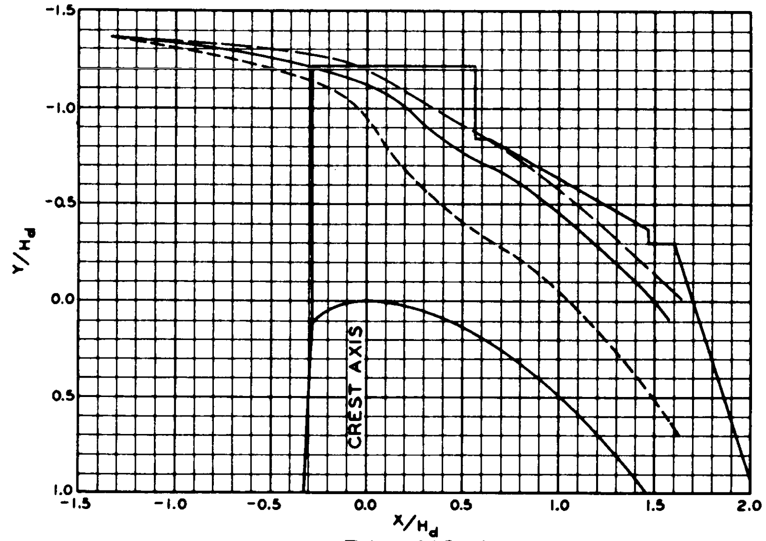
OVERFLOW SPILLWAY CREST

UPPER NAPPE PROFILES

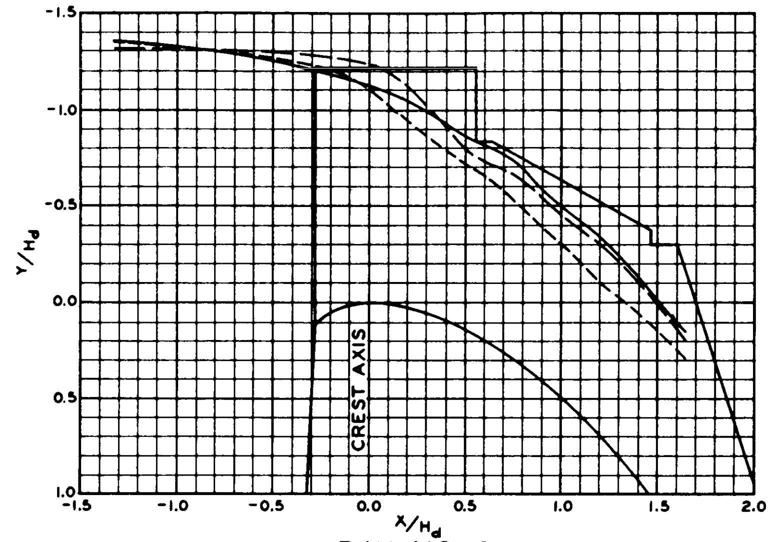
ABUTMENT EFFECTS

HYDRAULIC DESIGN CHART 111-13

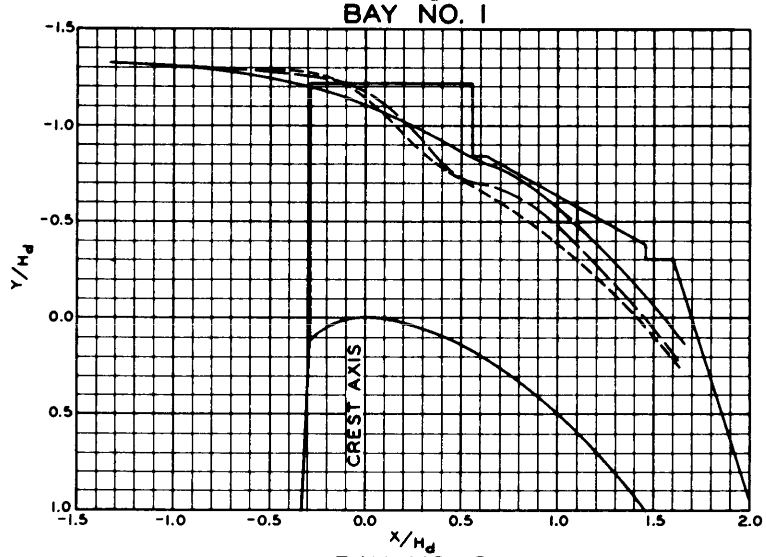
WES 9-54, REV 9-75



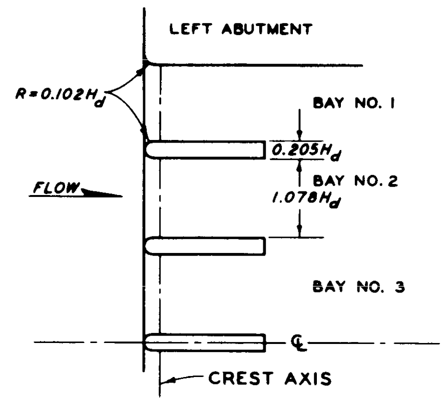
BAY NO. 1



BAY NO. 2



BAY NO. 3



LEGEND

- ------ LEFT SIDE OF BAY
- —— ℄ OF BAY
- — — RIGHT SIDE OF BAY

PROFILES ARE BASED ON PINE FLAT MODEL TESTS FOR SPILLWAY HAVING PIERS SIMILAR TO TYPE 2 (HYDRAULIC DESIGN CHART 111-5).

$H/H_d = 1.35$

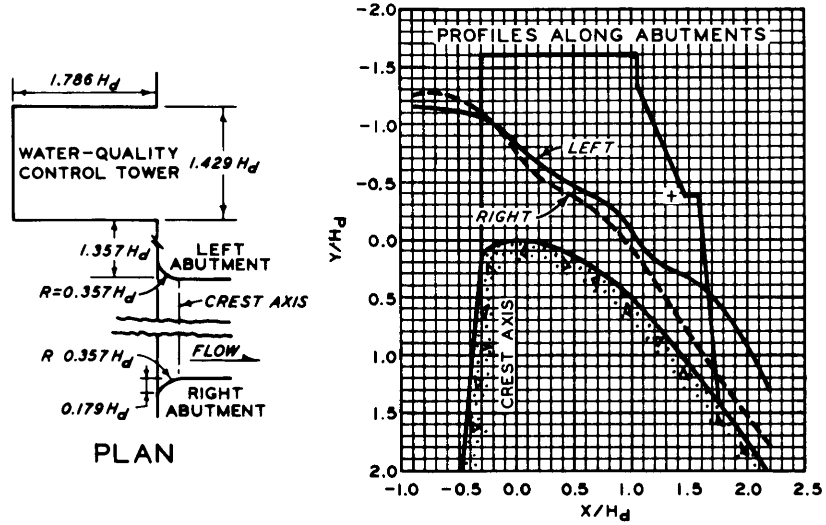
OVERFLOW SPILLWAY CREST

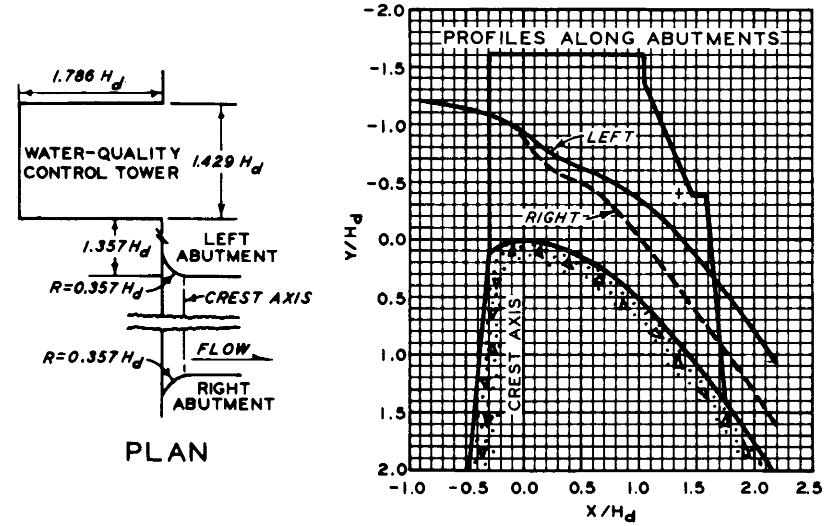
UPPER NAPPE PROFILES

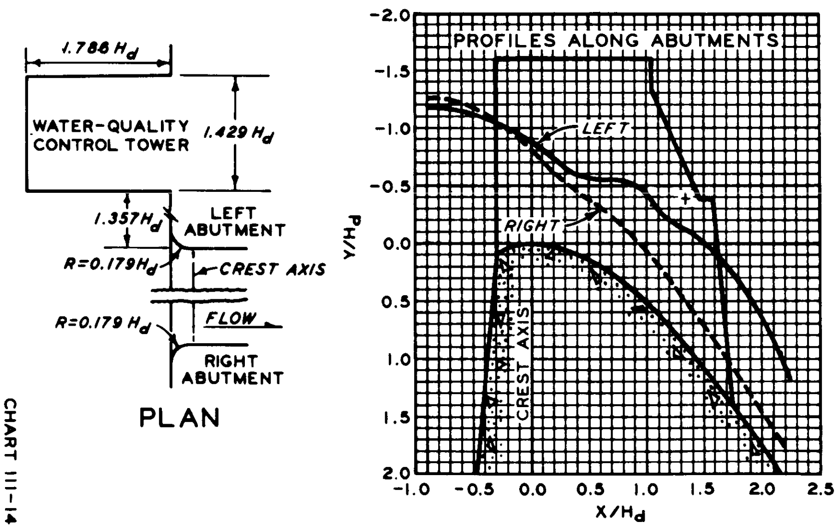
ABUTMENT EFFECTS

HYDRAULIC DESIGN CHART 111-13/1

WES 9-54, REV 9-75







NOTE: PROFILES BASED ON ROWLESBURG MODEL TESTS FOR SPILLWAYS HAVING SEVEN 1.607 H_d-WIDE BAYS SEPARATED BY 0.357 H_d-WIDE PIERS SIMILAR TO TYPE 3 (HYDRAULIC DESIGN CHART 111-5).

$H/H_d = 1.34$

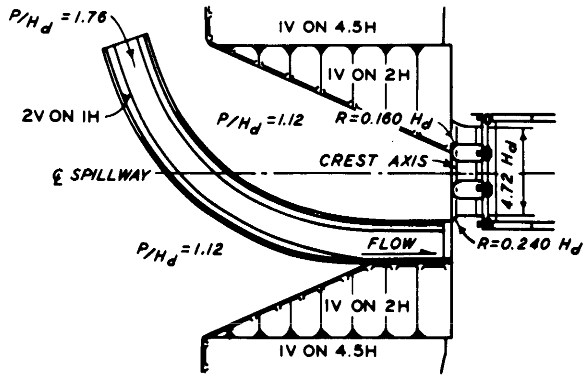
OVERFLOW SPILLWAY CREST

UPPER NAPPE PROFILES ALONG ABUTMENTS

APPROACH CHANNEL AND ABUTMENT CURVATURE EFFECTS

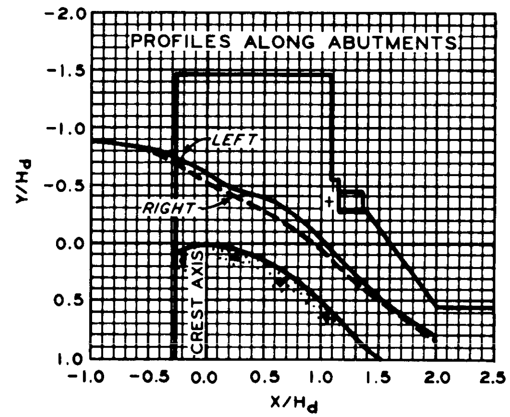
HYDRAULIC DESIGN CHART 111-14

WES 7-73

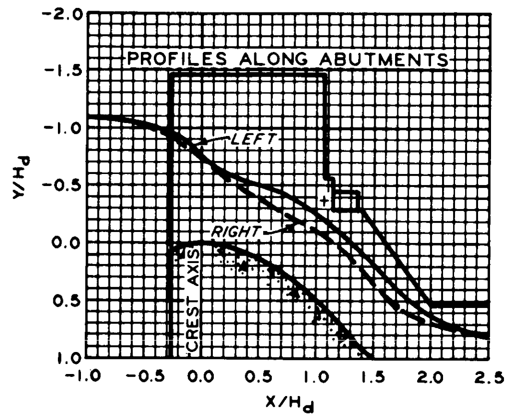


NOTE: PROFILES BASED ON ALUM CREEK MODEL TESTS FOR SPILLWAY HAVING THREE 1.360 H_d - WIDE BAYS SEPARATED BY 0.320 H_d - WIDE PIERS SIMILAR TO TYPE 2 (HYDRAULIC DESIGN CHART III-5).

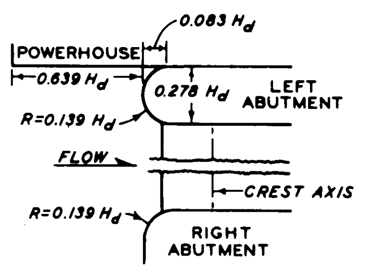
PLAN



$H/H_d = 0.92$

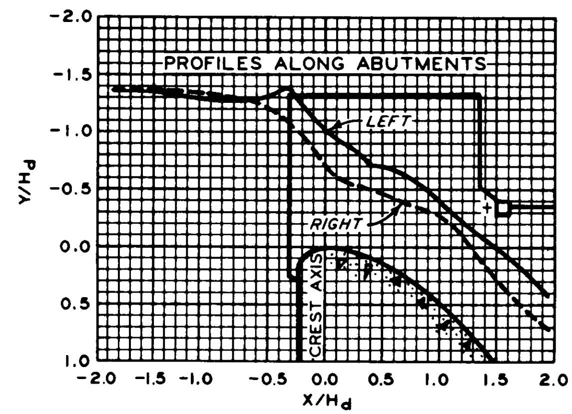


$H/H_d = 1.14$



NOTE: PROFILE BASED ON CLARENCE CANNON MODEL TESTS FOR SPILLWAY HAVING FOUR 1.389 H_d - WIDE BAYS SEPARATED BY 0.278 H_d - WIDE PIERS SIMILAR TO TYPE 2 (HYDRAULIC DESIGN CHART III-5).

PLAN



$H/H_d = 1.35$

OVERFLOW SPILLWAY CREST

UPPER NAPPE PROFILES ALONG ABUTMENTS

APPROACH CHANNEL AND ABUTMENT CURVATURE EFFECTS

HYDRAULIC DESIGN CHART III-14/1

WES 8-75