HYDRAULIC DESIGN CRITERIA

SHEET 122-1/2

OVERFLOW SPILLWAYS

DISCHARGE COEFFICIENTS

DESIGN HEAD

1. Purpose. Hydraulic Design Chart 122-1/2 can be used for selecting the spillway design head for upstream spillway face slope and approach depth conditions not covered by other Hydraulic Design Criteria charts. The chart, used in conjunction with the USBR[1] extensive tables of lower and upper nappe surface coordinate data for weirs sloping downstream and for vertical weirs, should permit optimization of spillway crest shape design for free overfall spillways having upstream face slope ratios from 3V on 12H to vertical and approach depths from 0.15 to 10 times the design head (H_d) .

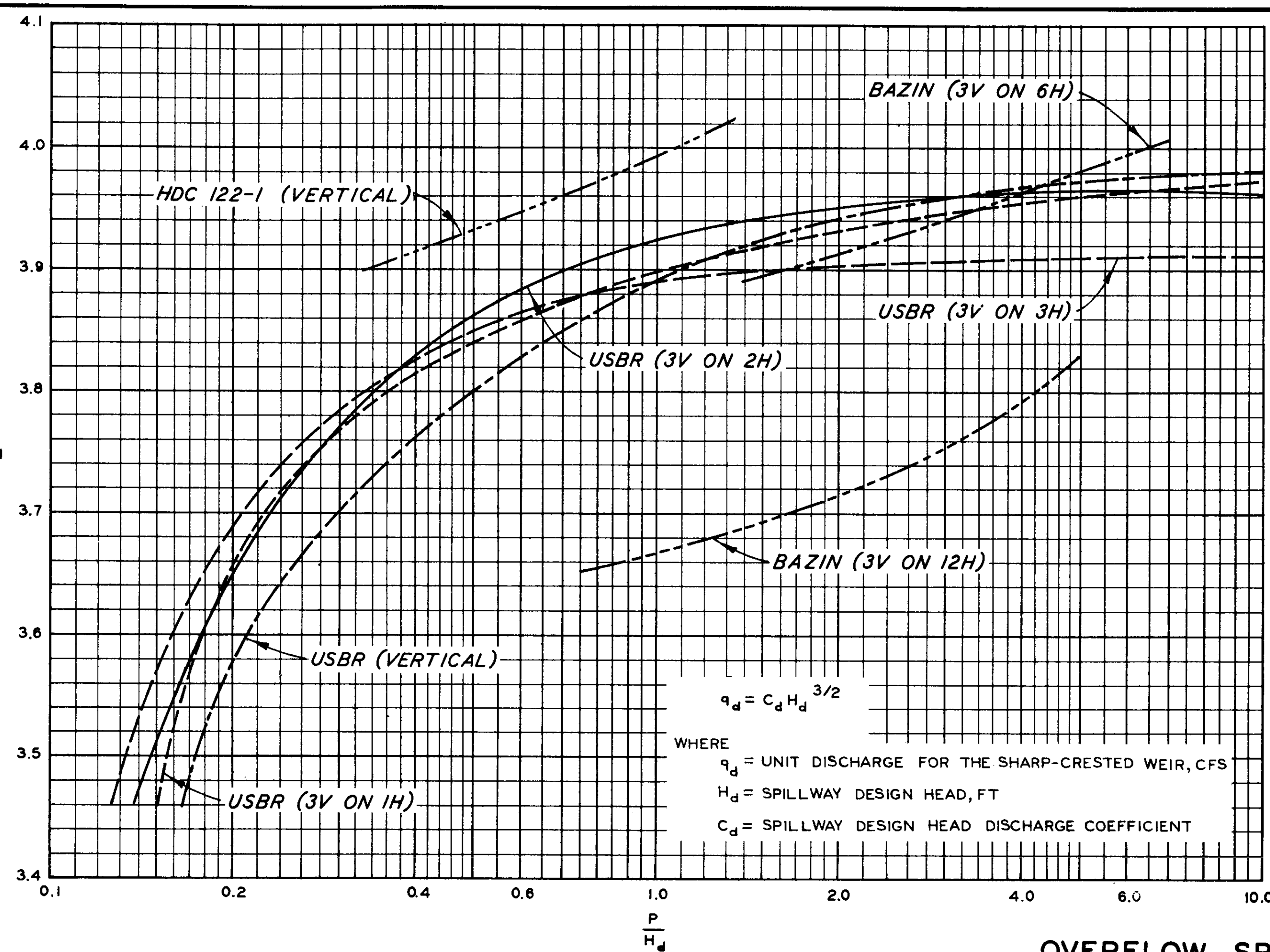
2. Background. The USBR tests were made in a 2-ft-wide, 9-ft-deep rectangular flume. Approach depths varied from less than 0.1 ft to 5 ft. Heads on the sharp-crested weirs ranged from about 0.1 to 1.0 ft. Discharge coefficients have been converted into values applicable to heads on the rounded crest. USBR coordinates of the upper and lower nappes of the weir overflow are in terms of the head on the sharp-crested weir. Comparable presentations of Bazin's[2] data for weirs sloping downstream (3V on 6H and 3V on 12H) are also included on the chart and in the USBR tabulation. A plot of the WES experimental data (Chart 122-1) for $H/H_d = 1$ and $P/H_d = 0.3$ to 1 is included for comparison. The WES discharge coefficients for rounded crests have consistently been about 3 percent higher than comparable USBR coefficients for sharp-crested weirs (HDC 122-4). The use of the USBR coefficients for the design of unmodeled spillways should result in conservative design.

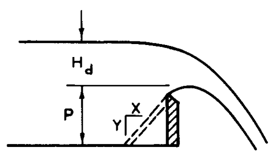
3. Application. The spillway design flow Q_d is computed using an appropriate coefficient from Chart 122-1/2 and pier and abutment contraction coefficients from other applicable charts (see Charts 111-3/1, 111-3/2, 111-5, 116-6, and 122-2). The spillway design head H_d and the computed design discharge Q_d are then used with Chart 111-3/3 to develop a spillway rating curve for uncontrolled flow as described in paragraph 5a of Sheet 111-3/3.

4. 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) Bazin, M., "Recent experiments on the flow of water over weirs." Annales des Ponts et Chaussées, October 1888.





NOTE: CURVES BASED ON FIGURES 15 AND 21, BULLETIN 3, PART VI, BOULDER CANYON PROJECT, FINAL REPORT, USBR 1948.

OVERFLOW SPILLWAYS
DISCHARGE COEFFICIENTS
DESIGN HEAD

HYDRAULIC DESIGN CHART 122-1/2

WES 2-72

HYDRAULIC DESIGN CRITERIA

SHEETS 122-3 to 122-3/5

LOW OGEE CRESTS

CREST SHAPE

45-DEGREE UPSTREAM SLOPE

1. General. An alternate method to that presented in Chart 111-20 for ogee spillway crests with a 45-deg upstream face slope is presented here. Coordinates of the lower nappe profile for sharp-crested weirs sloping 45 deg downstream and various approach depths have been determined and published by the U. S. Bureau of Reclamation (USBR) (reference 1). The published coordinates are in terms of the head on the sharp-crested weir.

2. For low ogee crests, the head is defined as the total energy head upstream from the spillway crest, and the approach depth as the height of the spillway crest above the approach channel bottom. For convenience of design, the shape downstream from the apex of the crest is considered separately from the shape upstream. The USBR (referenced 2) has published curves for determining the location of the apex of the crest, and for selecting coefficients and powers applicable to the general downstream quadrant crest shape equation. These curves are in a somewhat different form from that used by the U. S. Army Corps of Engineers.

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

$$X^n = \frac{1}{K}\left(H_d\right)^{(n-1)}Y \qquad \text{Corps of Engineers form}$$

where

Y = vertical coordinate positive downward

H_d = design head

K = variable dependent on approach depth

X = horizontal coordinate positive to the right

n = variable, however usually set equal to 1.85

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.

3. <u>Preferred Shapes.</u> Chart 122-3 shows the relation between the approach depth P and the approach velocity head h_a in terms of the design head H_d based on the USBR data (reference 1). The increments of h_a/H_d used in the original study were fairly close together, although the actual shape change is not great until the velocity of the approach flow becomes substantial. Three preferred shapes which would be considered reasonable for a range of h_a/H_d and their corresponding values of P/H_d are indicated in Chart 122-3. These three selected crest shapes are suggested for projects subject to model testing to provide information systematically and economically on pressure characteristics and discharge coefficients for spillways designed using the USBR method. For cases in which model studies are not contemplated, use of model study results presented in Sheets 111-20 to 111-20/1 is recommended.

4. <u>Downstream Quadrant Shape.</u> Chart 122-3/1 presents the published USBR curves for the coefficient K and exponent n of the downstream quadrant shape equation. The curves are applicable to approach velocity head-design ratio h_a/H_d of 0.00 to 0.20. The USBR curves of X_e/H_d and Y_e/H_d for locating the apex of the crest are also given in the chart. Downstream quadrant equations for spillways having approach velocity head-design ratios of 0.08 and 0.12 are given in Charts 122-3/2 and 122-3/3, respectively. The condition of $h_a/H_d = 0.00$ is presented in Chart 111-9. The coefficients in the equations given in these charts as noted in paragraph 2 are reciprocals of those given in Chart 122-3/1. Tables of the functions required in the evaluation of the equations are not available elsewhere; consequently, they are included in the charts to assist the designer in computing the required coordinates. Tabulations of the slope of the downstream quadrant shape are also included in the charts to facilitate location of the beginning of the toe curve or the tangent section.

5. <u>Upstream Quadrant Shape.</u> Upstream quadrant shapes have frequently been defined by circular arcs fitted to the experimental data. This procedure usually results in a surface of discontinuity where the curved crest meets the sloping upstream face. The possible effects of this surface of discontinuity are discussed in paragraph 4 of Sheets 111-1 to 111-2/1. WES has derived upstream quadrant equations based on the USBR basic data (reference 1) for 45-deg downstream sloping weirs. Chart 122-3/4 gives the general form of the equation in terms of the sharp-crested weir. Curves of the coefficients and the exponents required for evaluation of the equation are given in this chart and in Chart 122-3/1. The curves are applicable to approach velocity head-design head ratios h_a/H_d of 0.00 to 0.20. Upstream quadrant shapes based on these curves should satisfy the crest location criteria given in Chart 122-3/1, result in zero slope at the spillway crest, minimize

the discontinuity at the 45-deg upstream face, and result in good agreement with the experimental data.

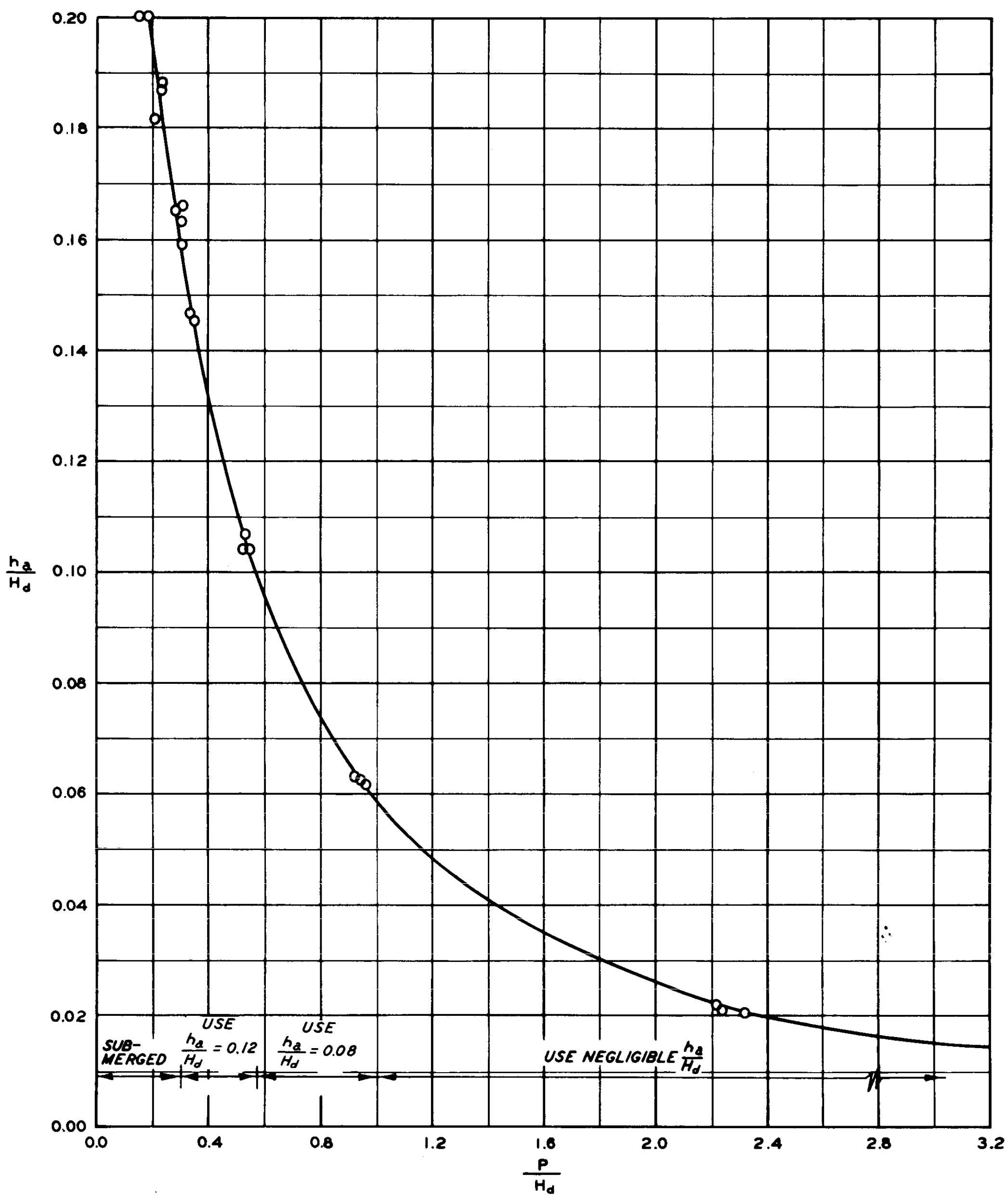
6. The solution of the equation in Chart 122-3/4 gives coordinates in terms of the sharp-crested weir. Transfer of the coordinate origin to the spillway crest and the reference head to the design head results in the cumbersome general equation shown in Chart 122-3/5. Upstream quadrant coordinates referenced to the crest apex and design head are tabulated in this chart for h_a/H_d values of 0.08 and 0.12 to simplify the design procedure.

7. The crest shapes defined in Charts 122-3 to 122-3/5 have not been tested in the laboratory for determination of pressures and discharge coefficients.

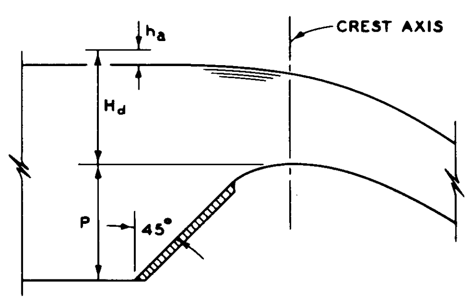
8. References.

(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) __________, Design of Small Dams, Washington D. C., 1960.



NOTE: DATA FROM TABLE 8, BOULDER CANYON REPORT, BULLETIN 3, USBR, 1948.



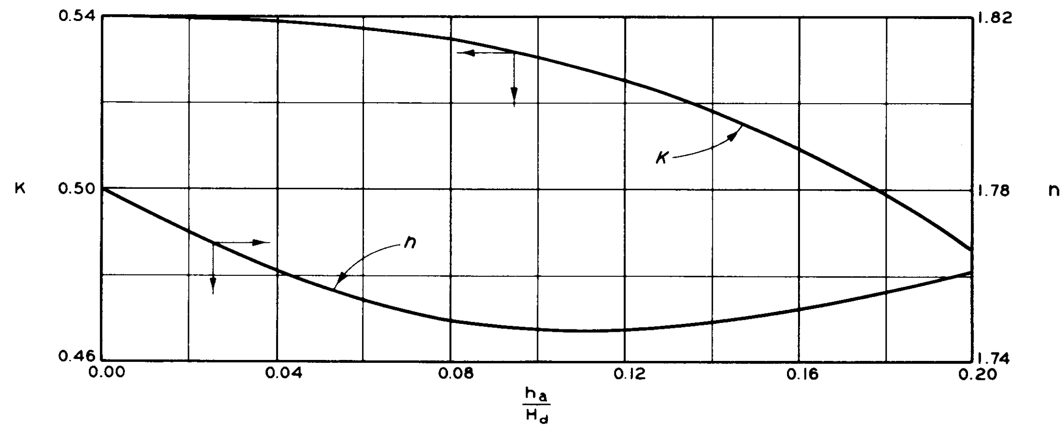
DEFINITION SKETCH

LOW OGEE CRESTS
45-DEGREE UPSTREAM SLOPE
APPROACH HYDRAULICS

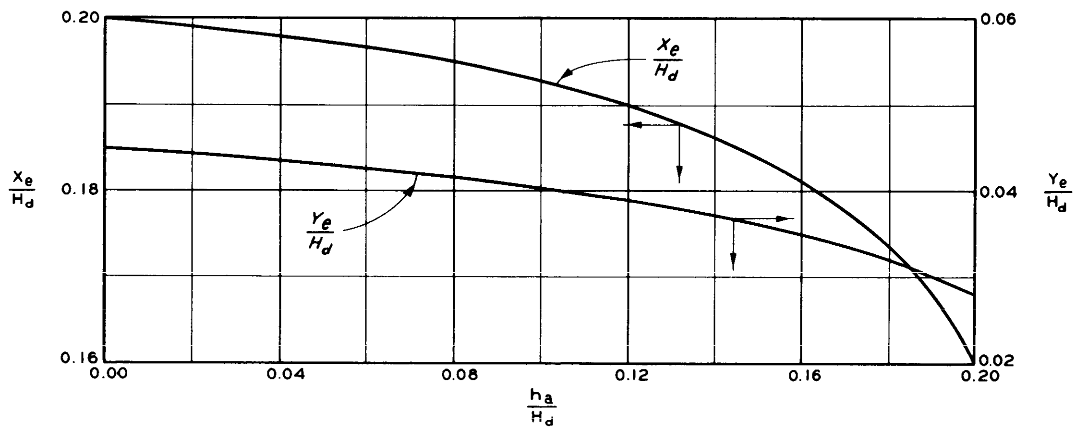
HYDRAULIC DESIGN CHART 122-3

REV 1-64 WES 10-61

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

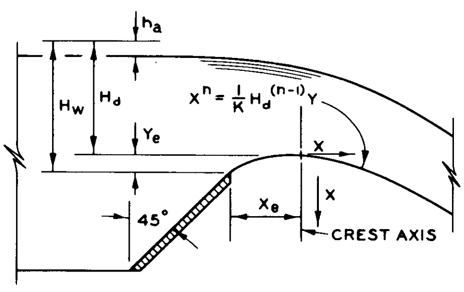


A. CONSTANTS OF DOWNSTREAM QUADRANT EQUATIONS



B. WEIR NAPPE GEOMETRY

NOTE: CURVES REPRODUCED FROM FIGURE 187, <u>DESIGN OF SMALL DAMS</u>, USBR, 1960.



DEFINITION SKETCH

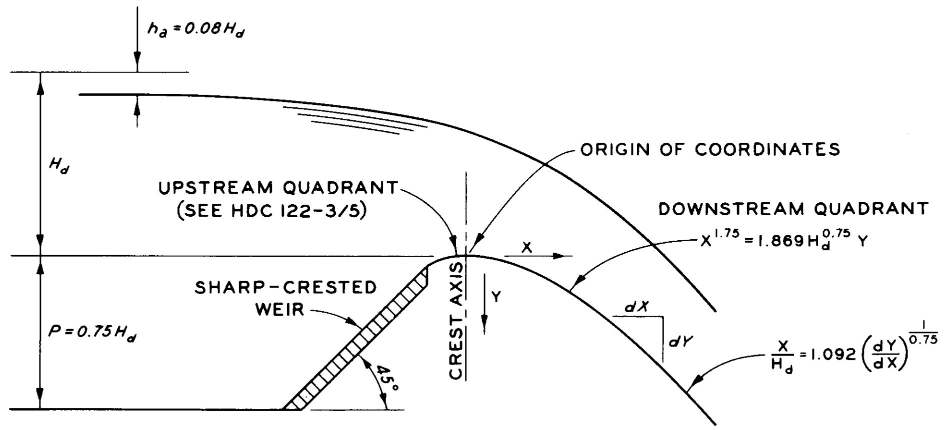
LOW OGEE CRESTS
45-DEGREE UPSTREAM SLOPE
CREST SHAPE FACTORS

HYDRAULIC DESIGN CHART 122-3/1

REV 1-64

WES 10-61

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



NOTE: EQUATION BASED ON USBR CURVES, HDC 122-3/1.

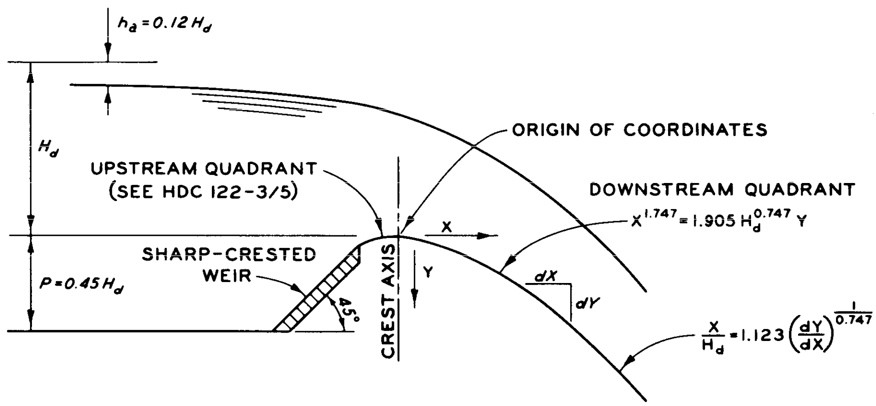
DOWNSTREAM QUADRANT DATA								SLOPE DATA	
X	$X^{1.75}$	X	$X^{1.75}$	H_d	$1.869 \times H_d^{0.75}$	H_d	$1.869 \times H_d^{0.75}$	$\frac{dY}{dX}$	$\frac{X}{H_d}$
0.10	0.0177	6	23.002	1	1.869	26	21.522	0.50	0.433
0.15	0.0361	7	30.125	2	3.144	27	22.139	0.60	0.553
0.20	0.0598	8	38.055	3	4.261	28	22.752	0.70	0.679
0.25	0.0883	9	46.765	4	5.287	29	23.358	0.80	0.811
0.30	0.1216	10	56.234	5	6.250	30	23.960	0.90	0.949
0.35	0.1592	12	77.369	6	7.166	31	24.557	1.00	1.092
0.40	0.2011	14	101.327	7	8.044	32	25.148	1.05	1.165
0.45	0.2472	16	128.000	8	8.891	33	25.735	1.10	1.240
0.50	0.2973	18	157.299	9	9.712	34	26.318	1.15	1.315
0.60	0.4090	20	189.148	10	10.511	35	26.897	1.20	1.392
0.70	0.5356	25	279.508	11	11.290	36	27.471	1.25	1.470
0.80	0.6767	30	384.558	12	12.051	37	28.041	1.30	1.549
0.90	0.8316	35	503.639	13	12.797	38	28.608	1.35	1.629
1.00	1.000	40	636.217	14	13.528	39	29.171	1.40	1.710
1.20	1.376	45	781.847	15	14.247	40	29.730	1.45	1.792
1.40	1.802	50	940.151	16	14.953	41	30.285	1.50	1.875
1.60	2.276	55	1110.797	17	15.649	42	30.838	1.60	2.043
1.80	2.797	60	1293.495	18	16.334	43	31.387	1.70	2.215
2.00	3.364	65	1487.984	19	17.010	44	31.933	1.80	2.391
2.50	4.970	70	1694.032	20	17.677	45	32.475		
3.00	6.839	75	1911.425	21	18.336	46	33.015		
3.50	8.956	80	2139.969	22	18.987	47	33.552		
4.00	11.314	90	2629.810	23	19.631	48	34.086		
4.50	13.903	100	3162.278	24	20.268	49	34.617		
5.00	16.719			25	20.898	50	35.146		

LOW OGEE CRESTS

45-DEGREE UPSTREAM SLOPE
DOWNSTREAM QUADRANT – $h_a = 0.08 H_d$

HYDRAULIC DESIGN CHART 122-3/2

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



NOTE: EQUATION BASED ON USBR CURVES, HDC 122-3/1.

DOWNSTREAM QUADRANT DATA / SLOPE DATA

X	$X^{1.747}$	X	$X^{1.747}$	H_d	$1.905 \times H_d^{0.747}$	H_d	$1.905 \times H_d^{0.747}$	$\frac{dY}{dX}$	$\frac{X}{H_d}$
0.10	0.0179	6	22.879	1	1.905	26	21.718	0.50	0.444
0.15	0.0363	7	29.949	2	3.197	27	22.339	0.60	0.567
0.20	0.0601	8	37.818	3	4.328	28	22.954	0.70	0.696
0.25	0.0887	9	46.458	4	5.365	29	23.564	0.80	0.833
0.30	0.1220	10	55.847	5	6.338	30	24.169	0.90	0.975
0.35	0.1597	12	76.794	6	7.263	31	24.768	1.00	1.123
0.40	0.2017	14	100.528	7	8.149	32	25.362	1.05	1.198
0.45	0.2478	16	126.940	8	9.004	33	25.952	1.10	1.275
0.50	0.2979	18	155.941	9	9.832	34	26.537	1.15	1.354
0.60	0.4096	20	187.456	10	10.638	35	27.118	1.20	1.433
0.70	0.5362	25	276.822	11	11.422	36	27.695	1.25	1.514
0.80	0.6771	30	380.654	12	12.190	37	28.267	1.30	1.595
0.90	0.8318	35	498.295	13	12.941	38	28.836	1.35	1.678
1.00	1.000	40	629.215	14	13.677	39	29.401	1.40	1.762
1.20	1.375	45	772.969	15	14.401	40	29.963	1.45	1.846
1.40	1.800	50	929.182	16	15.112	41	30.520	1.50	1.932
1.60	2.273	55	1097.523	17	15.812	42	31.075	1.60	2.106
1.80	2.792	60	1277.704	18	16.502	43	31.626	1.70	2.284
2.00	3.357	65	1469.466	19	17.182	44	32.174	1.80	2.466
2.50	4.957	70	1672.578	20	17.853	45	32.718		
3.00	6.816	75	1886.828	21	18.516	46	33.260		
3.50	8.923	80	2112.021	22	19.170	47	33.799		
4.00	11.267	90	2594.548	23	19.818	48	34.334		
4.50	13.841	100	3118.889	24	20.458	49	34.867		
5.00	16.638			25	21.091	50	35.397		

LOW OGEE CRESTS

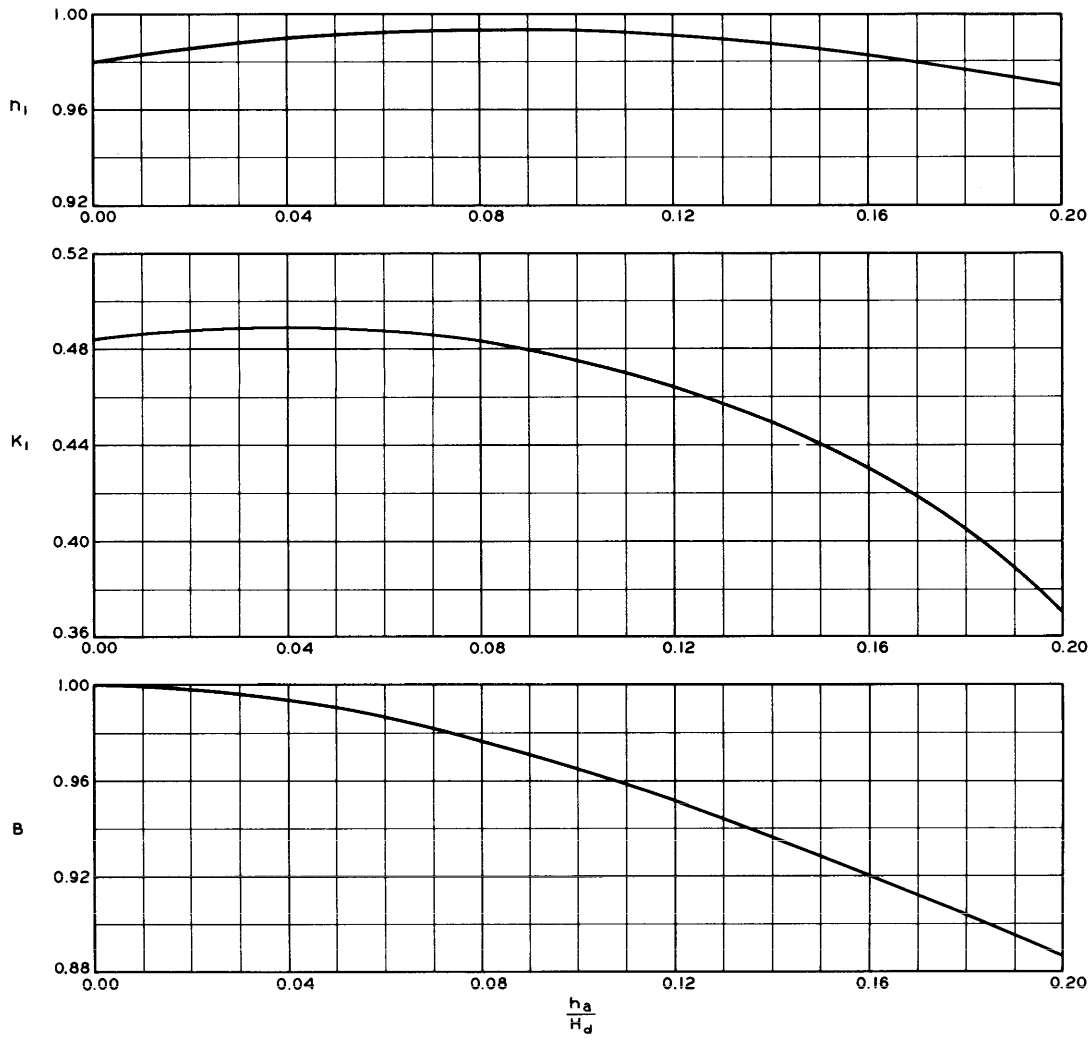
45-DEGREE UPSTREAM SLOPE

DOWNSTREAM QUADRANT – $h_a = 0.12 H_d$

HYDRAULIC DESIGN CHART 122-3/3

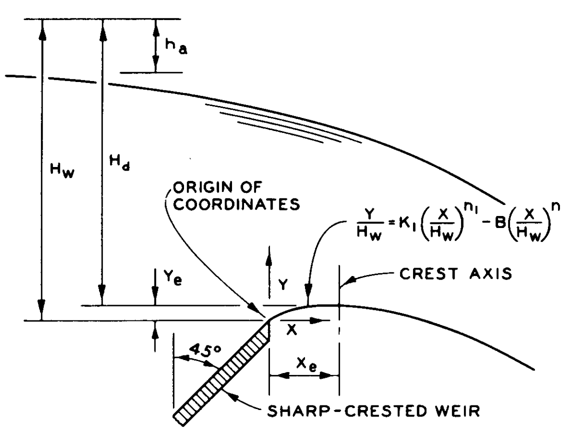
PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 1-64



NOTES: 1. CURVES BASED ON USBR DATA IN BOULDER CANYON REPORT, BULLETIN 3, PART VI, 1948, AND ON USBR CURVES ON HYDRAULIC DESIGN CHART 122-3/1.

2. n IS THE SAME IN BOTH UPSTREAM AND DOWNSTREAM QUADRANT EQUATIONS.



DEFINITION SKETCH

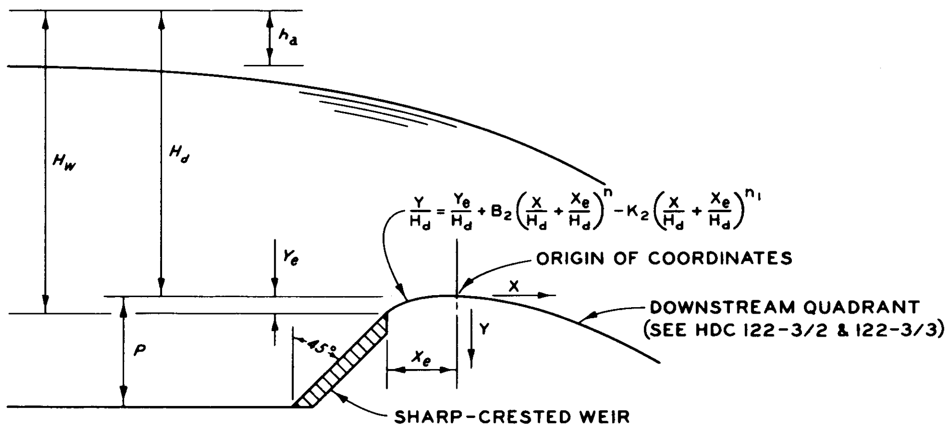
LOW OGEE CRESTS

45-DEGREE UPSTREAM SLOPE UPSTREAM QUADRANT FACTORS

HYDRAULIC DESIGN CHART 122-3/4

WES 1-64

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



	Y/H_d			Y/H_d	
X/H_d \ h_a/H_d	0.08	0.12	X/H_d \ h_a/H_d	0.08	0.12
-0.000	0.0000	0.0000	-0.150	0.0235	0.0231
-0.020	0.0004	0.0004	-0.155	0.0252	0.0248
-0.040	0.0016	0.0015	-0.160	0.0270	0.0265
-0.060	0.0035	0.0035	-0.165	0.0288	0.0284
-0.080	0.0064	0.0062	-0.170	0.0308	0.0303
-0.100	0.0101	0.0099	-0.175	0.0328	0.0323
-0.110	0.0122	0.0120	-0.180	0.0349	0.0344
-0.120	0.0147	0.0144	-0.185	0.0372	0.0366
-0.130	0.0174	0.0170	-0.190	0.0395	0.0390
-0.140	0.0203	0.0199	-0.195	0.0420	
-0.145	0.0219	0.0215			

NOTE: COORDINATES BASED ON
HDC 122-3/1 AND 122-3/4.

LOW OGEE CRESTS

45-DEGREE UPSTREAM SLOPE
UPSTREAM QUADRANT COORDINATES

HYDRAULIC DESIGN CHART 122-3/5

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

WES 1-64

SHEETS 122-3/9 TO 122-3/10

LOW OGEE CRESTS

WATER-SURFACE PROFILES

45-DEGREE UPSTREAM SLOPE

1. The shapes of the upper nappe profiles for low, ogee crests are required for the design of spillway abutment and pier heights and for the selection of trunnion elevations for tainter gates. Coordinates for the upper nappe profile for sharp-crested weirs sloping 45 degrees downstream have been determined and published by the USBR.* The published coordinates apply, in a strict sense, only to the unsupported free-falling jet. When the jet is supported, the development of the turbulent boundary layer will influence the thickness of the jet. The jet thickness will also be affected if the beginning of the toe curve or the tangent chute is close to the crest. However, the published data can be used for estimating the water-surface profile in the vicinity of the spillway crest for the design.

2. For low, ogee crests, the head is defined as the total energy head upstream from the spillway crest, and the approach depth as the height of the spillway crest above the approach channel invert. HDC 122-3/9 shows the relation between the design head-approach channel depth ratio $\left(\frac{H_d}{P_w + Y_e}\right)$ and the approach velocity parameter $\left(\frac{h_a}{H_w}\right)$ used by the USBR. This chart can be used in conjunction with HDC 122-3 and 122-3/1 for selection of the most appropriate upper nappe profile coordinates from the published data.

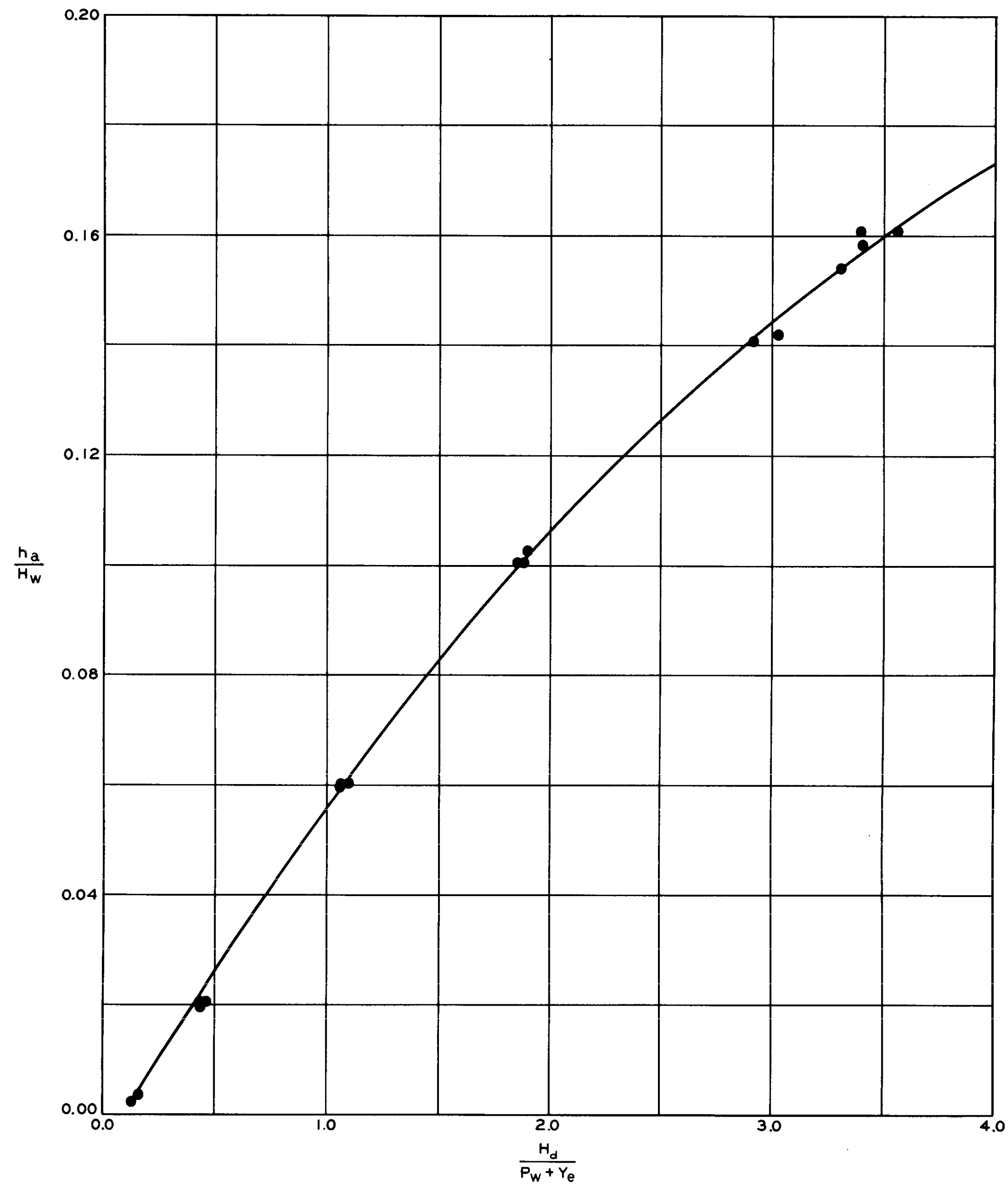
3. The published data are in the terms of the head on the sharp-crested weir with the origin of the data at the weir crest. For design purposes, coordinates are expressed in terms of the head on the crest with their origin at the crest apex. The necessary conversion can be accomplished using the relation between the weir head and the design head

$$H_w = H_d + Y_e$$

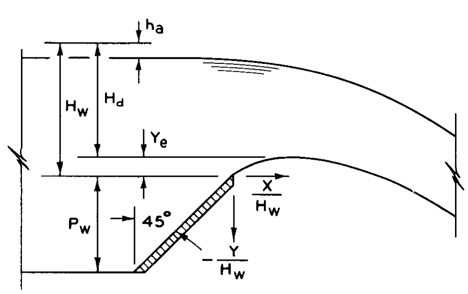
and the values of X_e/H_d and Y_e/H_d given in HDC 122-3/1. HDC 122-3/10 illustrates the computations required for this conversion.

---

* 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.



NOTE: DATA FROM TABLE 8, BOULDER CANYON REPORT, BULLETIN 3, USBR, 1948.



DEFINITION SKETCH

LOW OGEE CRESTS
45-DEGREE UPSTREAM SLOPE
APPROACH VELOCITY

HYDRAULIC DESIGN CHART 122-3/9

WES 1-64

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

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB: ES 804 PROJECT: John Doe Dam SUBJECT: Spillway

COMPUTATION: Upper Water Surface Profile

COMPUTED BY: RGC DATE: 1/6/64 CHECKED BY: MBB DATE: 1/10/64

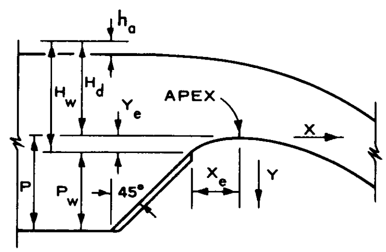
GIVEN:

Spillway Crest

$H_d = 20$ ft

$P/H_d = 0.74$ } HDC 122-3

$h_a/H_d = 0.08$ }



REQUIRED:

Coordinates for upper nappe profile for design head (H_d), origin at crest apex.

COMPUTE:

1. Required relationships

$Y_e/H_d = 0.042$ (HDC 122-3/1)

$X_e/H_d = 0.195$ (HDC 122-3/1)

$h_a/H_w = 0.076$ (HDC 122-3/9)

$H_w = H_d + Y_e$ (Definition sketch)

$$\frac{H_w}{H_d} = 1 + \frac{Y_e}{H_d} = 1 + 0.042 = 1.042$$

$$\frac{X}{H_d} = 1.042\frac{X}{H_w};\quad \frac{Y}{H_d} = 1.042\frac{Y}{H_w} \quad \text{(Origin at sharp crest)}$$

$$\frac{X}{H_d} = 1.042\frac{X}{H_w} - \frac{X_e}{H_d};\quad \frac{Y}{H_w} = 1.042\frac{Y}{H_w} + \frac{Y_e}{H_d} \quad \text{(Origin at crest apex)}$$

2. Coordinates - Upper water-surface profile

Origin at Weir Crest				Origin at Crest Apex			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
X/H_w*	Y/H_w**	X/H_d (1.042 × (1))	Y/H_d (1.042 × (2))	X/H_d ((3) − 0.195))	Y/H_d ((4) + 0.042))	X (ft)	Y (ft)
-4.00	-0.910	-4.168	-0.948	-4.363	-0.906	-87.26	-18.12
-3.00	-0.906	-3.126	-0.944	-3.321	-0.902	-66.42	-18.04
-2.00	-0.899	-2.084	-0.937	-2.279	-0.895	-45.58	-17.90
-1.00	-0.881	-1.042	-0.918	-1.237	-0.876	-24.74	-17.52
0.00	-0.760	0.000	-0.792	-0.195	-0.750	-3.90	-15.00
0.50	-0.580	0.521	-0.604	0.326	-0.562	6.52	-11.24
1.00	-0.257	1.042	-0.268	0.847	-0.226	16.94	-4.52
1.50	0.247	1.563	0.257	1.368	0.299	27.36	5.98
2.00	0.931	2.084	0.970	1.889	1.012	37.78	20.24

*From Table 21, p. 80, Boulder Canyon report, Bulletin 3, USBR, 1948

**From Table 21, cited above, interpolated for $h_a/H_w = 0.076$

LOW OGEE CRESTS

45-DEGREE UPSTREAM SLOPE UPPER WATER-SURFACE PROFILE SAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 122-3/10

HYDRAULIC DESIGN CRITERIA

SHEET 122-5

LOW OGEE CRESTS

TOE CURVE PRESSURES

1. Purpose. Experimental laboratory data indicate that toe curve pressures for low ogee crests are approximately a maximum from the third point to the end of the toe curve. Analytical and flow net studies further indicate that the toe curve affects the boundary pressure immediately upstream and downstream from the curve. The relatively high pressure at the end of the curve may be transmitted to the underside of the chute slabs immediately downstream. Also, for low ogee crests the toe curve may have a submergence effect upon the flow over the crest. Therefore, estimates of boundary pressure relating to toe curves may be useful in studies of the structural design of the curve, the stability of the slabs immediately downstream, and the capacity of the spillway. HDC 122-5 can be used as a guide in estimating the pressure distribution on toe curves for low ogee spillways.

2. Design Criteria. The results of a Waterways Experiment Station (WES)[1] semiempirical study on flip bucket and toe curve pressures for high overflow spillways have been published and are summarized in HDC 112-7. The study included analysis of data from five hydraulic model investigations of low ogee spillways (Gavins Point,[2] Dickinson,[3] Fresno,[4] Bonny,[5] and Keyhole[6] Dams). The parameters defined in HDC 112-7 were used in the analysis. Reasonable correlation of the data for ratios of toe curve radius-total head $R/H_T \leq 1.0$ was obtained as shown in HDC 122-5. The curves in the chart are based on HDC 112-7. For R/H_T ratios > 1.0, the dimensionless pressure term can be expressed as $h_p/H_T + \Delta(h_p/H_T)$ where h_p/H_T is the value of the pressure parameter from HDC 122-5 and $\Delta(h_p/H_T)$ is an additive value from the insert graph in the chart. Revision of the curves in this chart may be desirable as additional data become available.

3. Application. The values of the parameters $q/R\sqrt{2gH_T}$, α/α_T , and R/H_T are computed for the design or maximum flow as the case may be. The notation is defined in HDC 112-7 and in the definition sketch in HDC 122-5. The values of h_p/H_T for α/α_T of 0.0 and 0.25 to 1.00 are read directly from the chart if $R/H_T < 1.0$. Values of $\Delta(h_p/H_T)$ from the chart insert should be added to the values read from the chart proper if $R/H_T > 1.0$. The pressure upstream and downstream from the toe curve reduces rapidly to the normal hydrostatic pressure and can be evaluated by a flow net or model study. However, an estimate of these pressures can be obtained by graphical extrapolation of the pressure pattern of the toe curve.

4. References.

(1) 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.

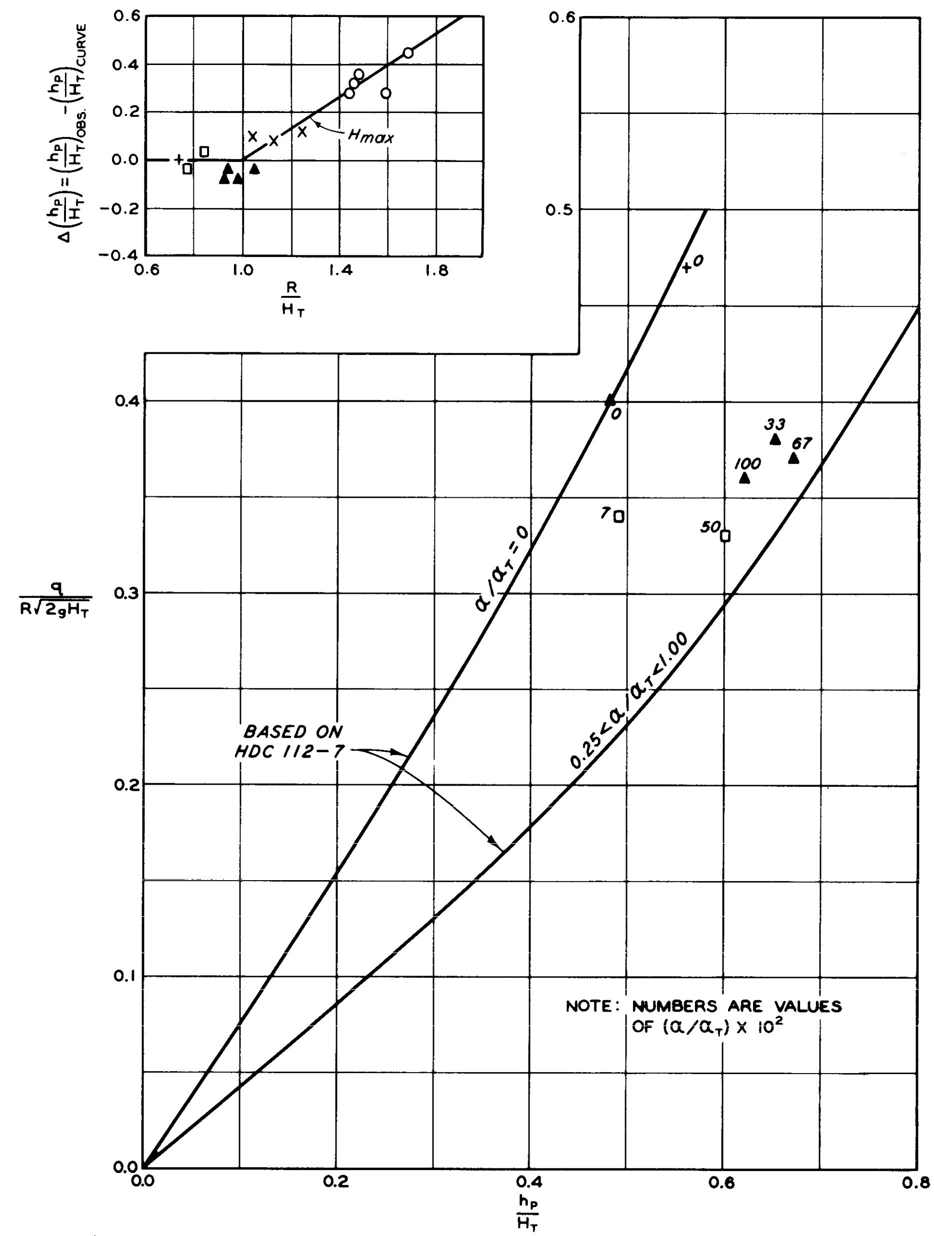
(2) U. S. Army Engineer Waterways Experiment Station, CE, Spillway for Gavins Point Dam, Missouri River, Nebraska; Hydraulic Model Investigation. Technical Memorandum No. 2-404, Vicksburg, Miss., May 1955.

(3) Beichley, G. L., Hydraulic Model Studies of Dickinson Dam Spillway. Hydraulic Laboratory Report No. HYD-267, U. S. Bureau of Reclamation, 30 December 1949.

(4) Ball, J. W., and Besel, R. C., Hydraulic Model Studies of the Spillway and Outlets for the Fresno Dam, Milk River Project. Hydraulic Laboratory Report No. HYD-177, U. S. Bureau of Reclamation, 6 July 1945.

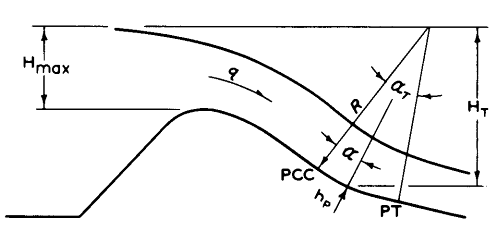
(5) Rusho, E. J., Hydraulic Model Studies of the Overflow Spillway and the Hale Ditch Irrigation Outlet, Bonny Dam, Missouri River Basin Project. Hydraulic Laboratory Report No. HYD-331, U. S. Bureau of Reclamation, 31 January 1952.

(6) Beichley, G. L., Hydraulic Model Studies of Keyhole Dam Spillway, Missouri River Project. Hydraulic Laboratory Report No. HYD-271, U. S. Bureau of Reclamation, 8 January 1952.



LEGEND

- X GAVINS POINT DAM
- ▲ DICKINSON DAM
- O FRESNO DAM
- ◻ BONNY DAM
- \+ KEY HOLE DAM



DEFINITION SKETCH

LOW OGEE CRESTS

TOE CURVE PRESSURES

HYDRAULIC DESIGN CHART 122-5

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

WES 1-66

SPILLWAY CHUTES

ENERGY - DEPTH CURVES

1. General. The design procedure to determine the water-surface profile in a spillway chute often employs the step method of computation. The curve can be solved by use of the varied flow function. The method devised by Bakhmeteff[1] is preferred by some engineers and the type of profile is classified as the S_1 curve. However, when the step method is employed, a trial-and-error procedure is normally used to resolve the total energy into depth and velocity head from the equation:

$$E = d + \frac{V^2}{2g}$$

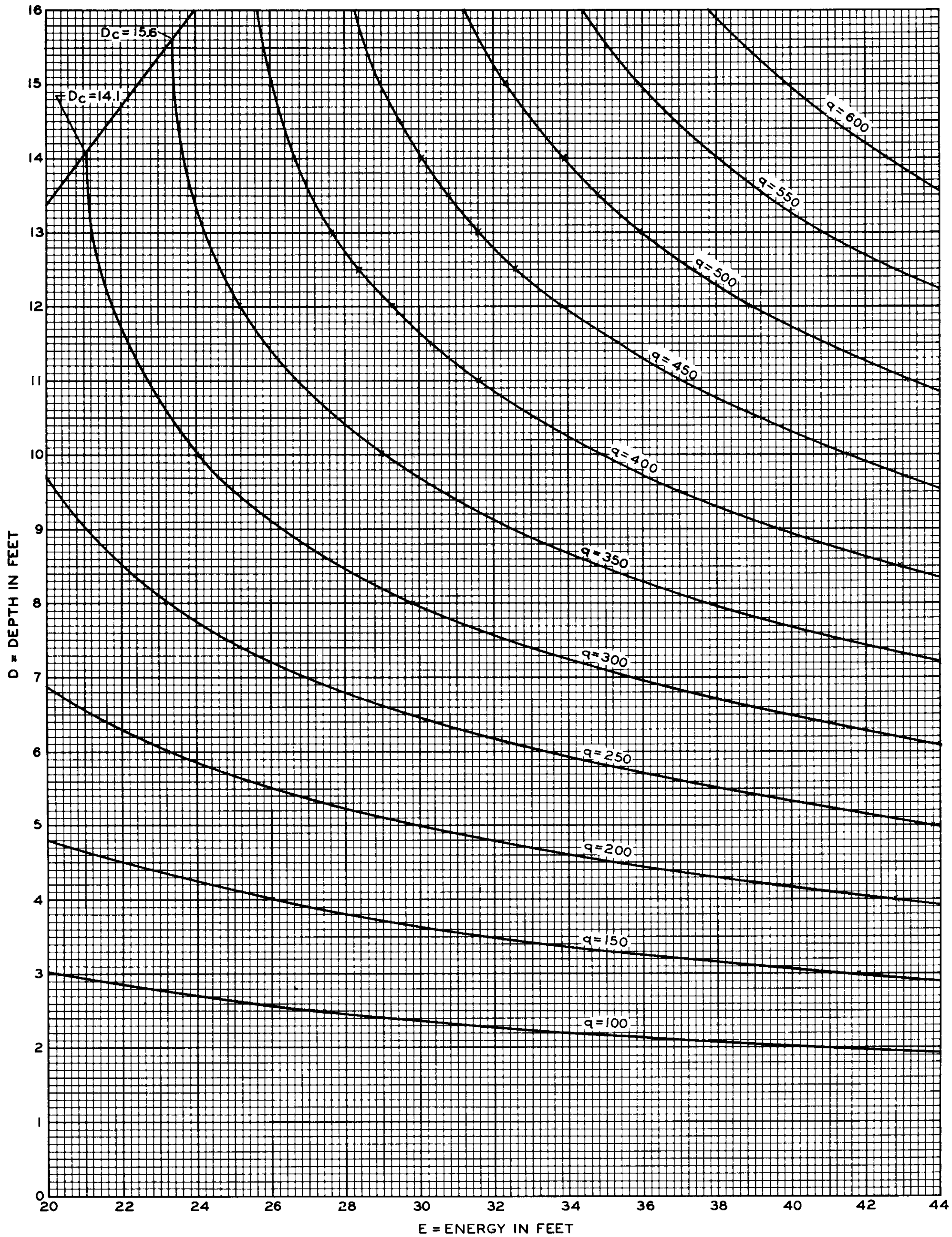
where E is the energy in feet and is the difference between the energy gradient and the bottom of the chute, d is the depth of flow and $V^2/2g$ is the velocity head.

2. Energy-depth Curves. A graph with energy and depth as ordinates and a family of curves representing various rates of flow per unit width (q) greatly facilitates the step method. This type of graph is not new.

3. Sample Computation. The sample computation chart (123-6) is included only to demonstrate the application of the energy-depth curves. Many different arrangements of the computation form have been used. The sample computation is carried to one-hundredths of a foot to indicate that the last significant figure can be estimated from the energy-depth curve. The starting station of the water-surface profile is commonly taken to be the tangent point of the chute to the ogee. If one assumes no energy loss between the reservoir pool and the starting station, the initial energy is the difference in elevation between the reservoir and the chute floor at the starting station. Entering the energy-depth graph with the initial energy and the proper discharge per unit width, the depth can be determined. Subtracting the depth from the energy, the velocity head is found. The energy-depth curves are then used at each successive phase of the computation after a new energy value is determined. Reproductions of the energy-depth curves on double size sheets are available upon request.

---

(1) Boris A. Bakhmeteff, Hydraulics of Open Channels (McGraw-Hill, 1932).

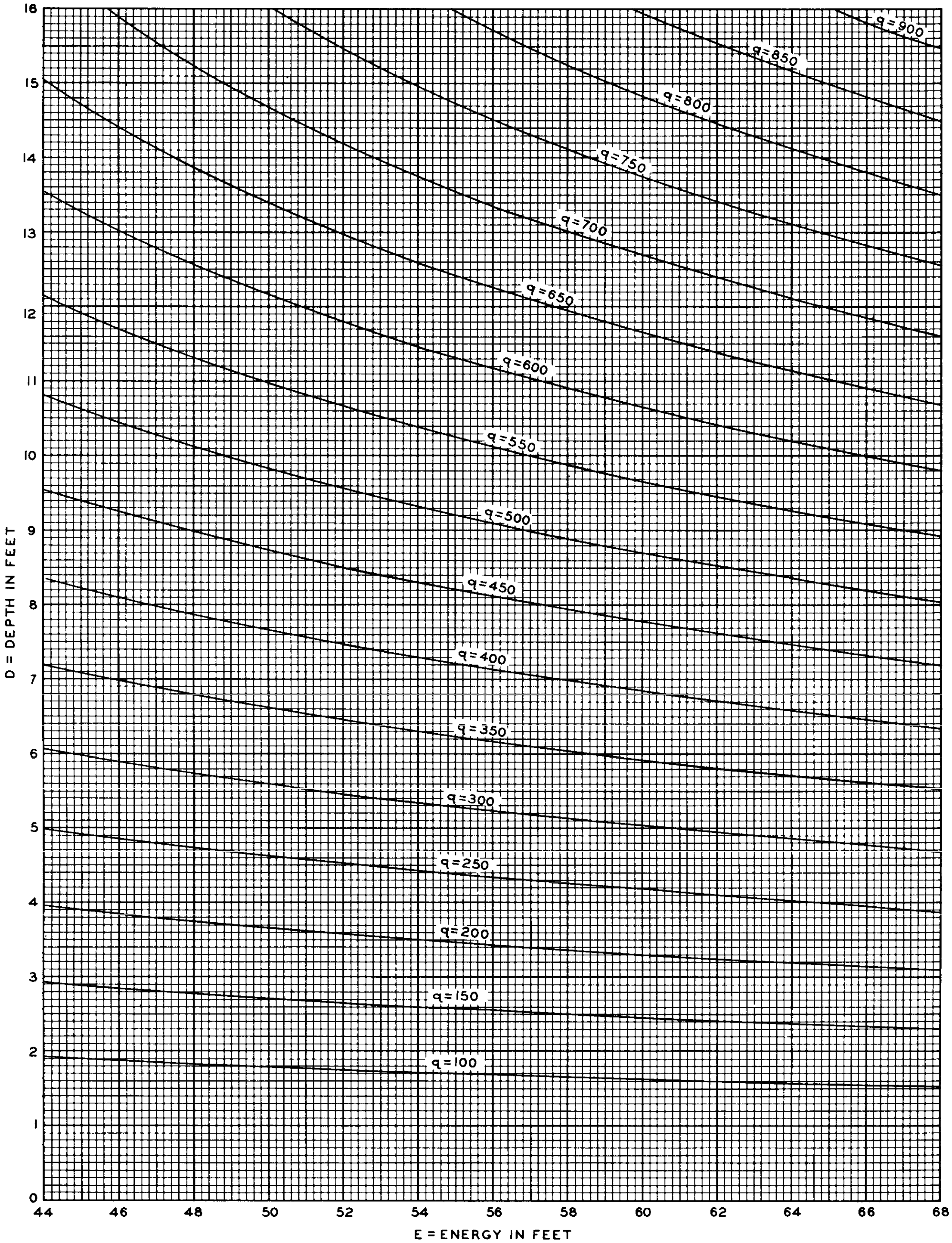


D = DEPTH IN FEET.
D_c = CRITICAL DEPTH IN FEET.
q = DISCHARGE PER FOOT OF WIDTH IN CFS.
E = ENERGY IN FEET $\left(E = D + V^2/2g\right)$.
V = VELOCITY IN FT PER SEC.

ENERGY-DEPTH CURVES
SUPERCRITICAL FLOW
ENERGY - 20 TO 44 FEET

HYDRAULIC DESIGN CHART 123-2

WES 4-1-52



D = DEPTH IN FEET.
D_c = CRITICAL DEPTH IN FEET.
q = DISCHARGE PER FOOT OF WIDTH IN CFS.
E = ENERGY IN FEET $\left(E = D + \frac{V^2}{2g}\right)$.
V = VELOCITY IN FT PER SEC.

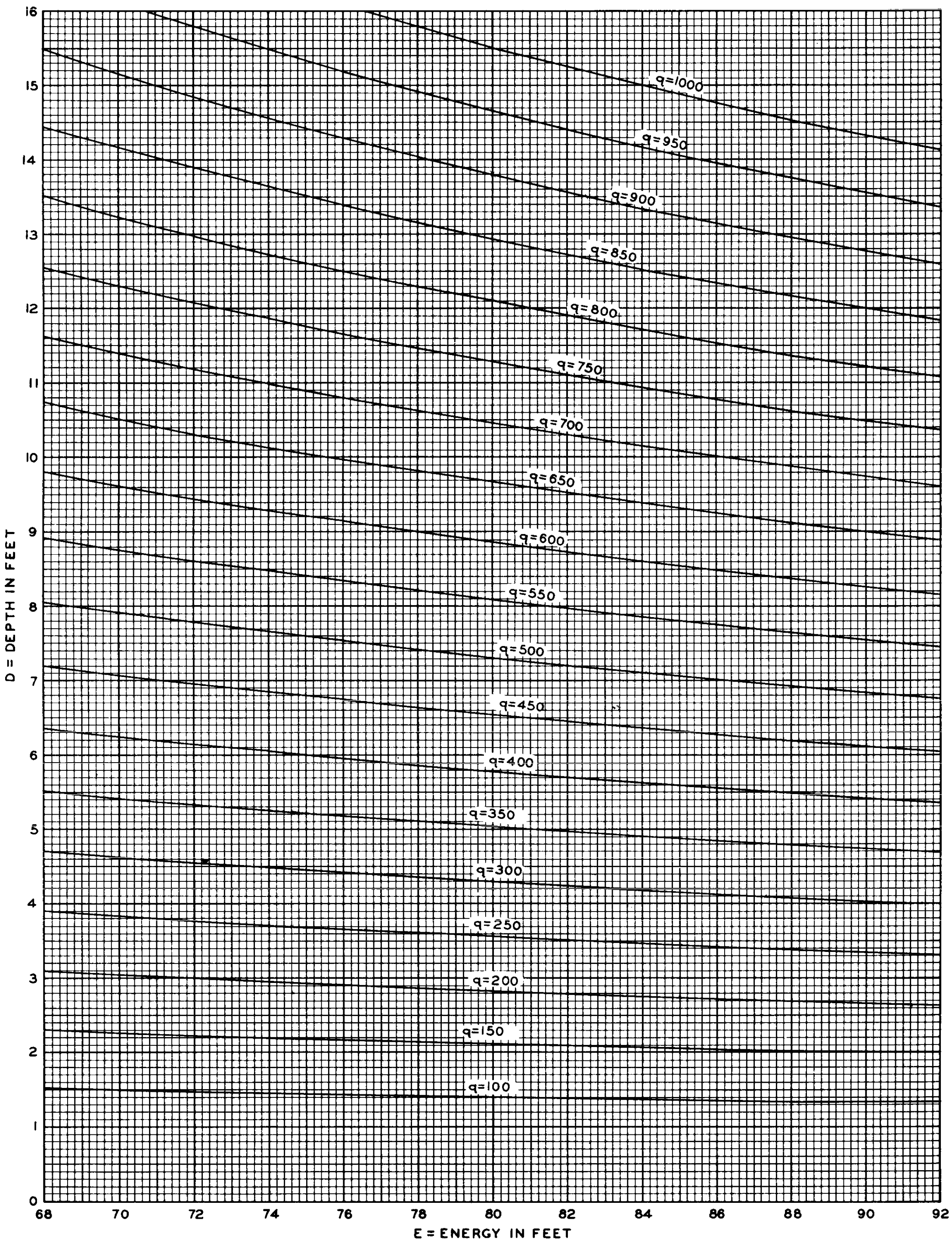
ENERGY-DEPTH CURVES

SUPERCRITICAL FLOW

ENERGY-44 TO 68 FEET

HYDRAULIC DESIGN CHART 123-3

WES 4-1-52



D = DEPTH IN FEET.
D_c = CRITICAL DEPTH IN FEET.
q = DISCHARGE PER FOOT OF WIDTH IN CFS.
E = ENERGY IN FEET $\left(E = D + \frac{V^2}{2g}\right)$.
V = VELOCITY IN FT PER SEC.

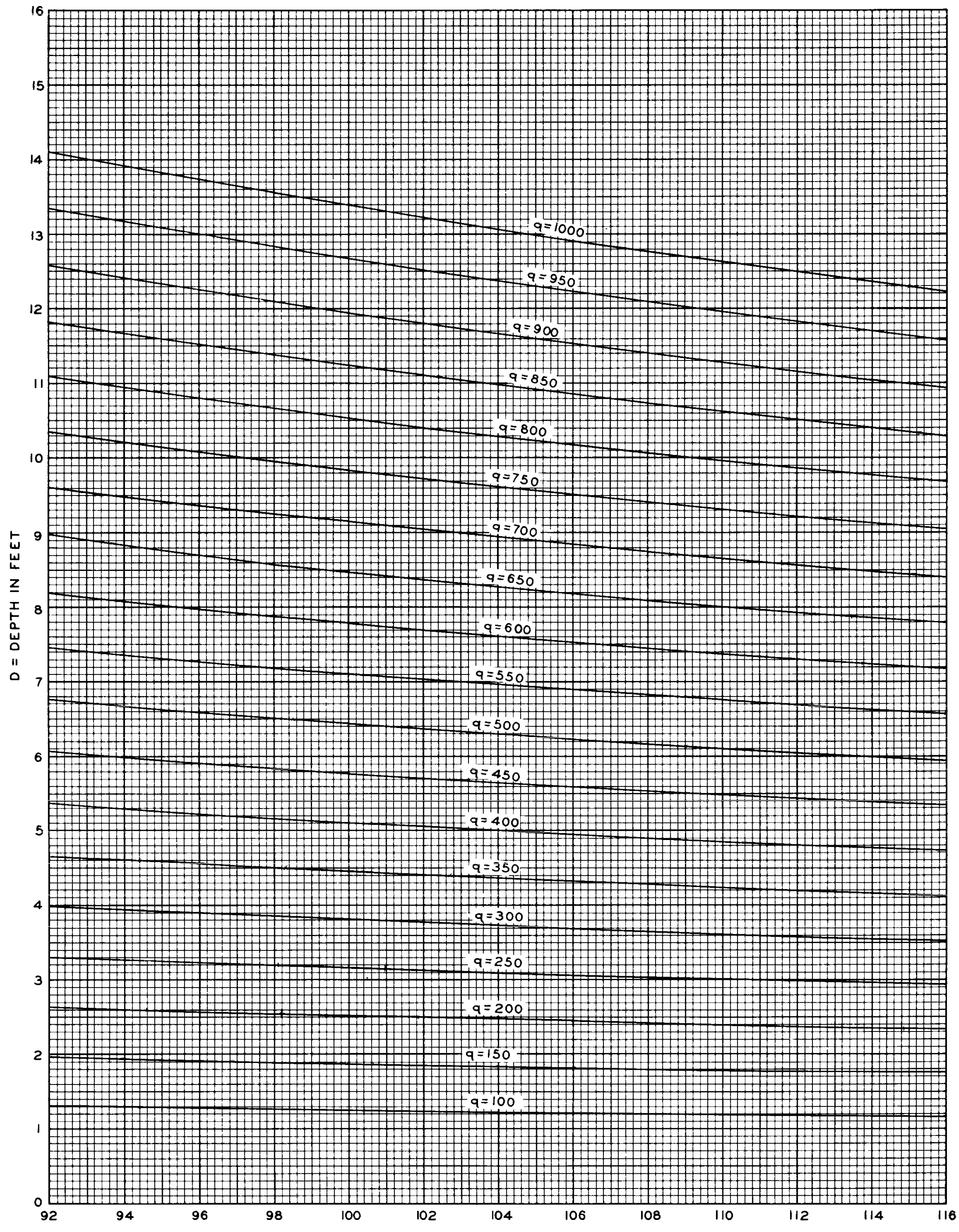
ENERGY-DEPTH CURVES

SUPERCRITICAL FLOW

ENERGY-68 TO 92 FEET

HYDRAULIC DESIGN CHART 123-4

WES 4-1-52



D = DEPTH IN FEET.
D_c = CRITICAL DEPTH IN FEET.
q = DISCHARGE PER FOOT OF WIDTH IN CFS.
E = ENERGY IN FEET $\left(E = D + V^2/2g\right)$.
V = VELOCITY IN FT PER SEC.

ENERGY-DEPTH CURVES
SUPERCRITICAL FLOW
ENERGY-92 TO 116 FEET
HYDRAULIC DESIGN CHART 123-5

WES 4-1-52

WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

Job: CW-804 Project: JOHN DOE DAM Subject: CHUTE SPILLWAY
Computations: WATER SURFACE PROFILE
Computed By: A.A.Mc. Date: 4-1-52 Checked By: B.G. Date: 4-1-52

GIVEN:

$Q = 120{,}000$ cfs
$W = 400$ ft
$n = 0.013$
$\Delta L = 100$ ft
Chute Slope = 0.10
Initial Energy = 28.0 ft

FORMULAS:

$$\Delta h_f = \Delta L n^2 \left(\frac{V^2}{2.21 R^{4/3}} \right)$$

$\Delta L n^2 = 0.0169$
$q = Q/W = 300$ cfs

STATION	FLOOR ELEV	ENERGY GRAD ELEV	E	d	R	$\frac{V^2}{2g}$	V	$\frac{V^2}{2.21R^{4/3}}$	Δh_f
10+50	599.25	627.25	28.0	8.45	8.11	19.55	35.5	34.92	0.590(2)
		626.66(1)							0.908(4)
11+50	589.25	626.34(3)	37.41	6.78	6.56	30.63	44.5	72.57	1.226
		625.11							1.568
12+50	579.25	624.77	45.86	5.92	5.75	39.94	50.7	113.01	1.910
		622.86							2.273
13+50	569.25	622.50	53.61	5.35	5.21	48.26	55.81	155.95	2.636
		619.86							2.972
14+50	559.25	619.53	60.61	5.00	4.88	55.61	59.82	195.71	3.308
		616.22							3.647
15+50	549.25	615.88	66.97	4.72	4.61	62.25	63.44	235.85	3.986
								$\Sigma \Delta h_f$ = 11.368	

Check Energy Gradient Elev 627.25
615.88 + 11.368 = 627.248 Check

(1) First estimate of energy gradient based on Δh_f from station 10 + 50(2)
(3) Adjusted energy gradient based on average of Δh_f between 10 + 50 & 11 + 50(4)

NOTATION:

- d = Depth of Flow in Chute - ft
- E = Energy = $d + V^2/2g$ - ft
- Δhf = Increment of Friction Loss - ft
- ΔL = Increment of Chute Length - ft
- n = Manning's Friction Coefficient
- q = Discharge per foot of width - cfs
- Q = Discharge - cfs
- R = Hydraulic Radius - ft
- V = Velocity - ft/sec
- W = Width of Chute - ft

CHUTE SPILLWAYS
ENERGY - DEPTH CURVES
SAMPLE COMPUTATION

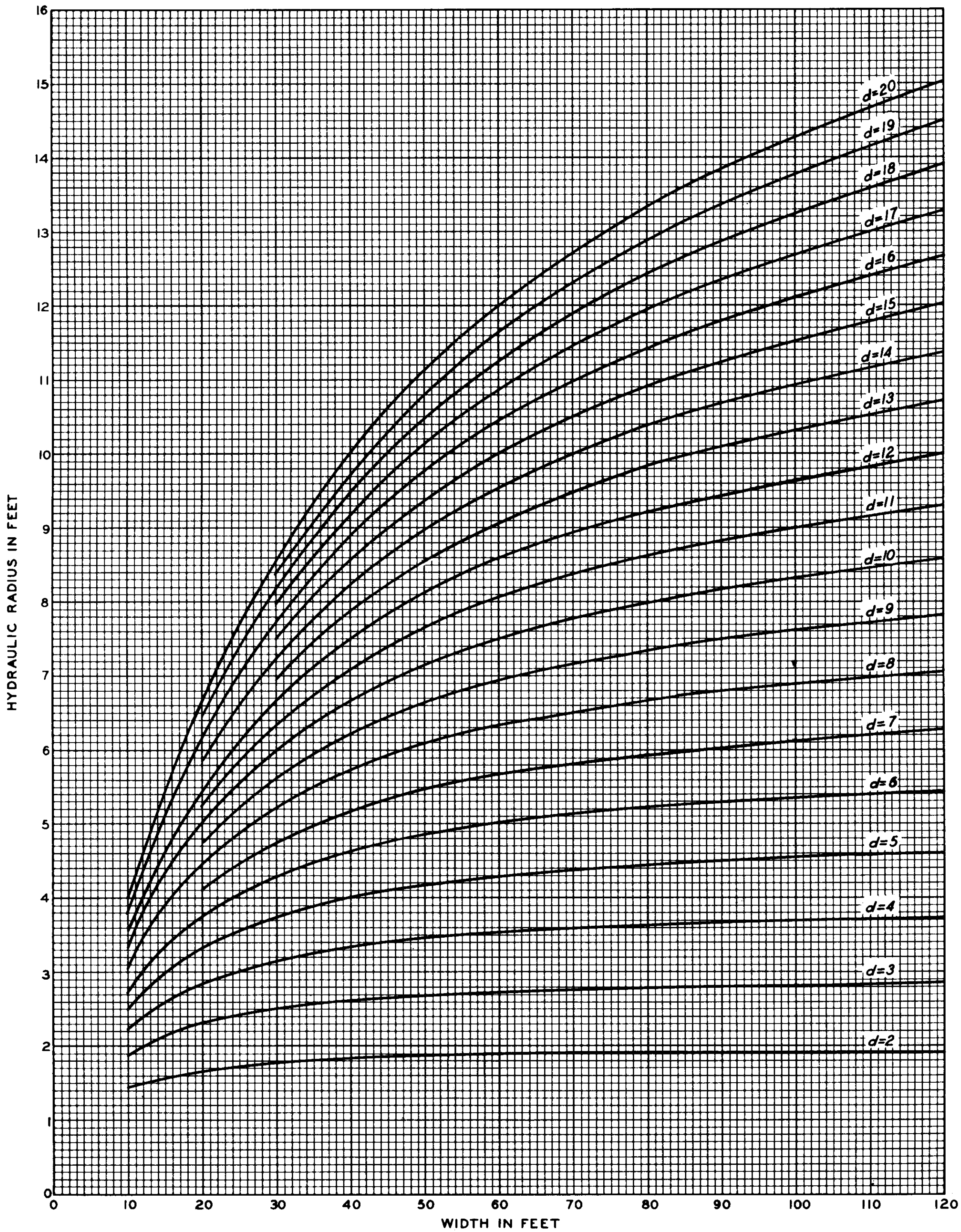
HYDRAULIC DESIGN CRITERIA

SHEETS 123-7 TO 123-9

CHUTE SPILLWAY

COMPUTATION AIDS

1. The curves on Hydraulic Design Chart 123-7 to 123-9 were prepared as design aids for problems similar to the one given in the sample computation on Chart 123-6 previously issued. The first two charts facilitate the determination of the hydraulic radius for a given rectangular chute width when the depth becomes known. Chart 123-7 is for widths from 10 to 120 ft and Chart 123-8 is for widths from 100 to 1200 ft. The last chart enables the designer to find the velocity and the function $V^2/2.21\ R^{4/3}$ after the velocity head has been computed.



BASIC EQUATION

$$R = \frac{Wd}{W+2d}$$

WHERE:

W = WIDTH OF CHUTE
d = DEPTH OF WATER
R = HYDRAULIC RADIUS

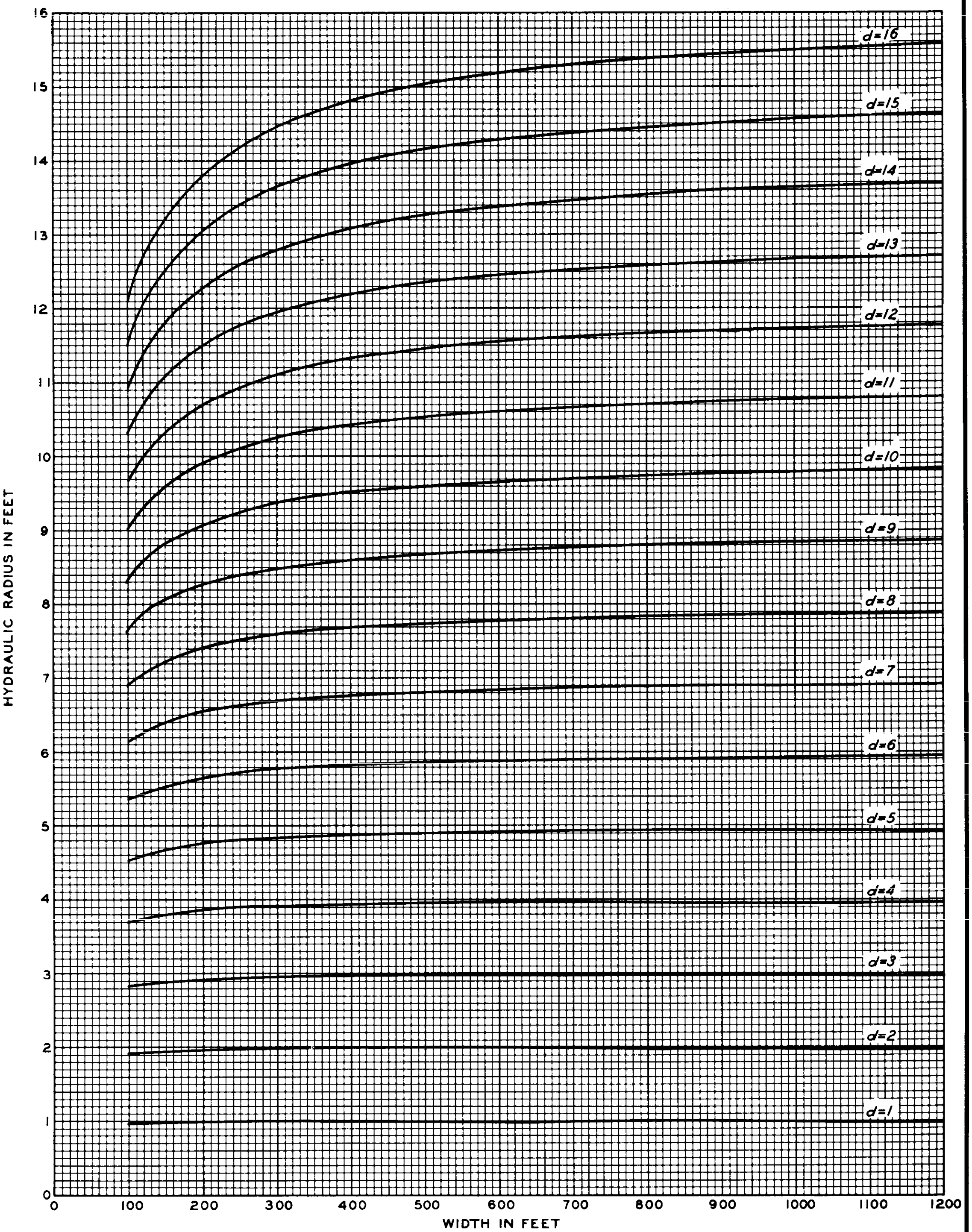
CHUTE SPILLWAYS

HYDRAULIC RADIUS-WIDTH-DEPTH CURVES

WIDTH 10 TO 120 FT

HYDRAULIC DESIGN CHART 123-7

WES 4-1-53



BASIC EQUATION

$$R = \frac{Wd}{W+2d}$$

WHERE:

W = WIDTH OF CHUTE
d = DEPTH OF WATER
R = HYDRAULIC RADIUS

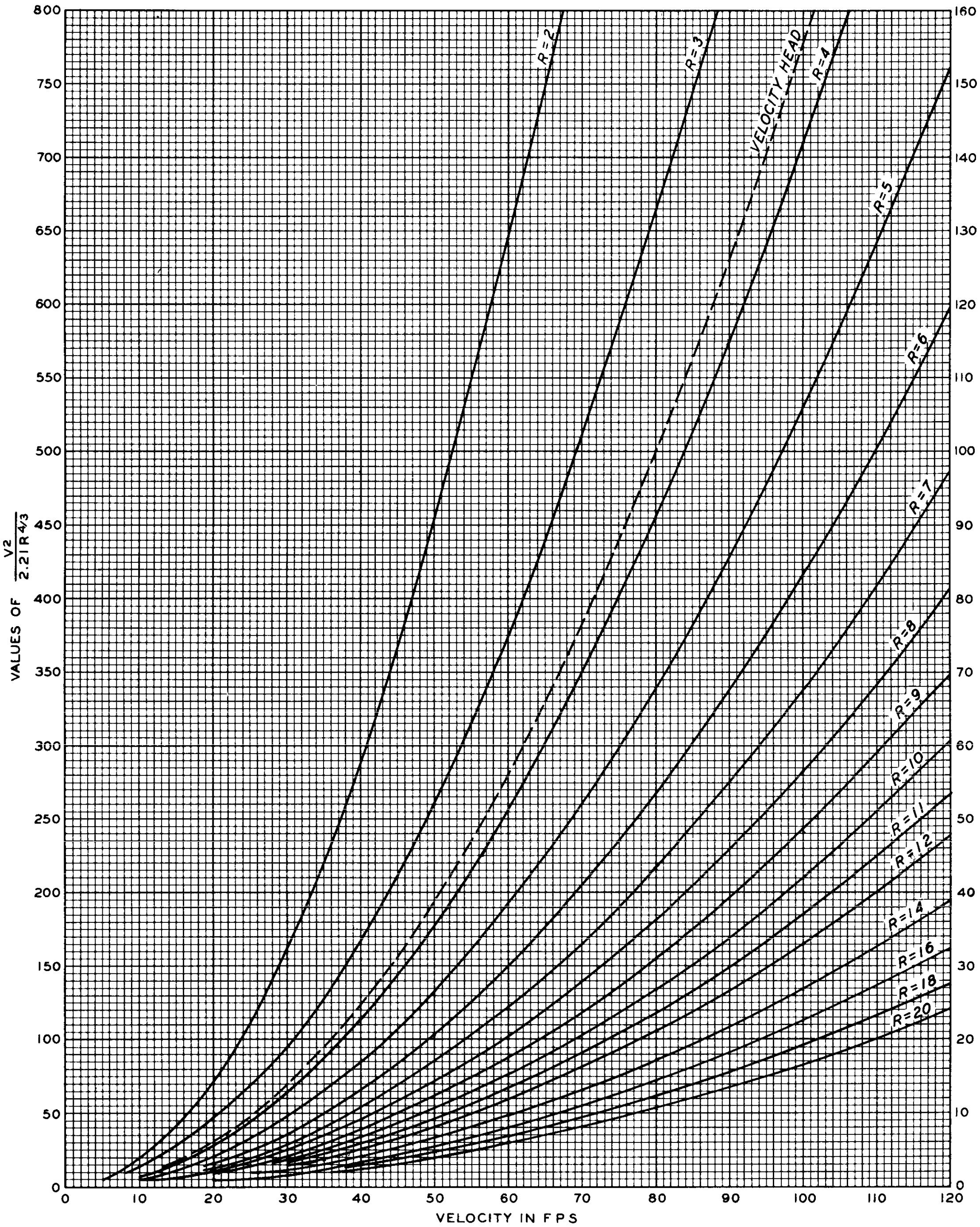
CHUTE SPILLWAYS

HYDRAULIC RADIUS-
WIDTH-DEPTH CURVES

WIDTH 100 TO 1200 FT

HYDRAULIC DESIGN CHART 123-8

WES 4-1-53



CHUTE SPILLWAYS

VELOCITY-HEAD AND $\frac{V^2}{2.21 R^{4/3}}$ CURVES

HYDRAULIC DESIGN CHART 123-9

WES 4-1-53

HYDRAULIC DESIGN CRITERIA

SHEETS 124-1 TO 124-1/1

CHUTE SPILLWAYS

STILLING BASINS

LENGTH OF HYDRAULIC JUMP

1. Purpose. The hydraulic jump is commonly used for energy dissipation at the end of spillway chutes. The jump may occur on the sloping chute, on both the sloping chute and the horizontal apron, or on the horizontal apron, depending upon tailwater conditions. In each case it is necessary to determine the length of the stilling basin walls required to confine the jump. HDC 124-1 can be used to estimate jump lengths when excess tailwater forces the jump to occur entirely on the chute slope. HDC 124-1/1 is applicable when the length of the jump spans the intersection of the sloping chute and the horizontal apron.

2. Laboratory Investigation. Laboratory experiments on the hydraulic jump on sloping aprons have been conducted by Bakhmeteff and Matzke,[1] Kindsvater,[2] Lin and Priest,[3] and Bradley and Peterka.[4,5] Difficulty has been found in correlating the results of various investigations of length of a jump on a sloping floor. The apparent reason for this is differences in the definition of the jump length used by the investigators. Bradley and Peterka[4] define the end of the jump as "The point where the high velocity jet begins to lift from the floor, or a point on the tailwater surface immediately downstream of the surface roller, whichever occurs farthest downstream." Bradley and Peterka's jump-length curves have been reproduced as HDC 124-1 and 124-1/1 and are recommended for design purposes because of the extensiveness of the tests compared with those of other investigators. The experimental data points have been omitted from the charts to simplify their use. Data points for existing basins with sloping aprons are plotted in HDC 124-1/1 and show good agreement with the curves. These points, selected from project tabulations published by Bradley and Peterka,[4] are limited to those cases where the locations of the jumps are dimensionally defined.

3. The length L of the jump in terms of the theoretical depth d_2 for zero slope is plotted as a function of the entering Froude number F_1 in HDC 124-1. Jump-length curves for continuous slopes of 0.0 to 0.33 are given. The tailwater depth d_2' required for the jump to occur completely on the sloping apron is shown by the insert graph.

4. For noncontinuous slopes, the length L_t of that portion of the jump occurring on the slope is given as a function of the tailwater depth TW in HDC 124-1/1. The theoretical depths d_2 for the horizontal apron jump have been used to develop the dimensionless curves for apron slopes of 0.05 to 0.33.

5. Application. HDC 124-1 and 124-1/1 can be used in the following manner:

a. Continuous Slope. Compute the Froude number of the entering flow and the theoretical depth d_2 for the jump on a horizontal floor. The latter can be estimated from HDC 112-3 or 112-4 and 112-5. From the insert graph in HDC 124-1, determine the tailwater depth d_2' for the slope of interest. This depth locates the end of the jump. From the chart proper determine the jump length for the entering Froude number and the chute slope. Locate the toe of the jump using the computed tailwater depth d_2' and jump length L. Check the Froude number at the toe of the jump against the Froude number computed for the entering flow and repeat the computations if necessary.

b. Noncontinuous Slope. For a noncontinuous slope, the procedure is similar to that for continuous slopes. If the existing tailwater depth TW is less than the d_2' obtained from the insert graph in HDC 124-1 but greater than the theoretical depth d_2, the hydraulic jump will occur partly on the slope and partly on the horizontal apron. The length of the portion of the jump on the slope is readily determined from HDC 124-1/1. The computed length L_t locates the toe of the jump. If the Froude number used in the computation does not approximate that existing at the toe of the jump, the computations should be repeated using the new Froude number at the toe of the jump. Bradley and Peterka suggest that the jump length curves given in HDC 124-1 for continuous slopes are also applicable, with negligible error, to noncontinuous slopes.

6. References.

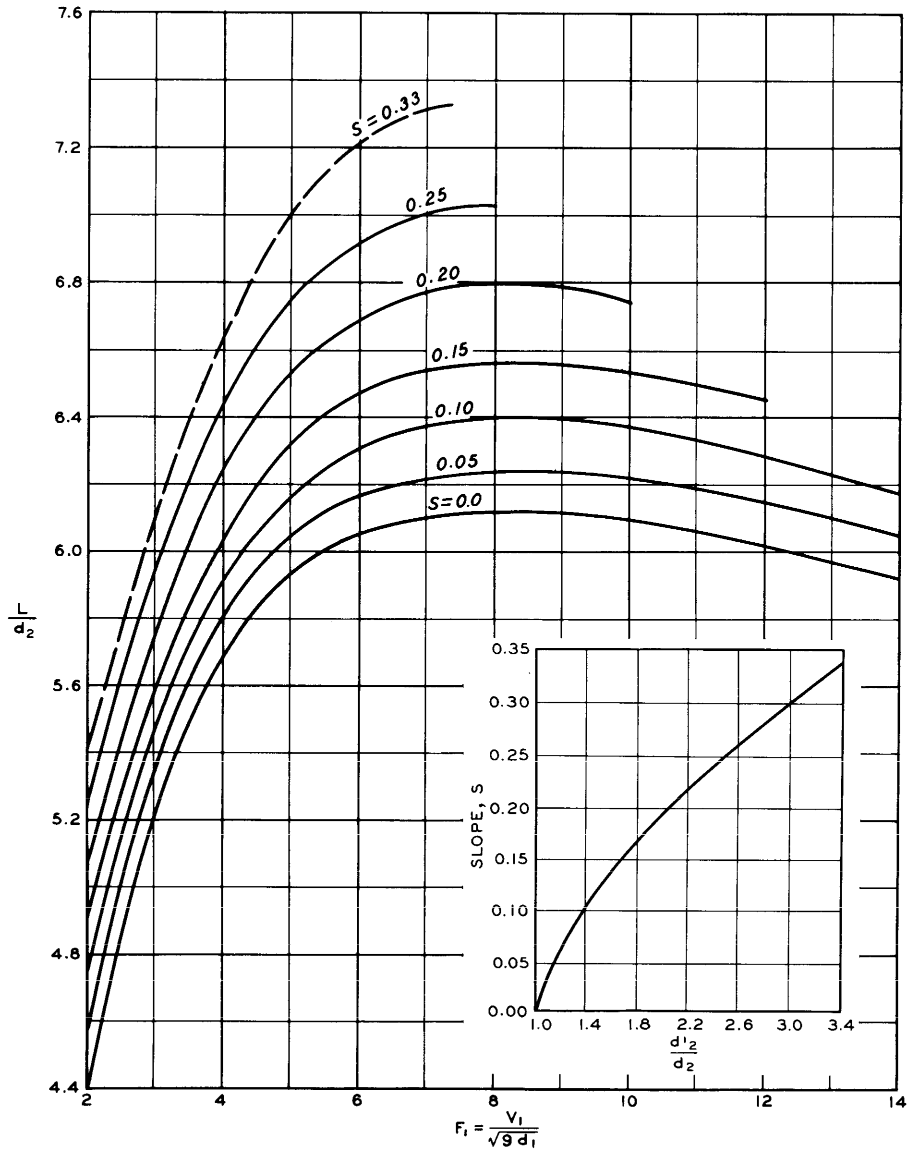
(1) Bakhmeteff, B. A., and Matzke, A. E., "The hydraulic jump in terms of dynamic similarity." Transactions, American Society of Civil Engineers, vol 101 (1936), pp 630-680.

(2) Kindsvater, C. E., "The hydraulic jump in sloping channels." Transactions, American Society of Civil Engineers, vol 109 (1944), pp 1107-1120.

(3) Lin, Kuang-ming, and Priest, M. S., The Hydraulic Jump Over a Plane Inclined Bottom. Alabama Polytechnic Institute, Engineering Experiment Station Bulletin 30, April 1958.

(4) Bradley, J. N., and Peterka, A. J., "Hydraulic design of stilling basins: stilling basin with sloping apron (basin V)." ASCE Hydraulics Division Journal, vol 83, HY 5 (October 1957), pp 1-32.

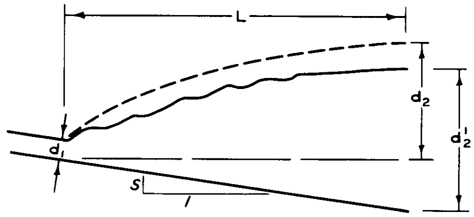
(5) U. S. Bureau of Reclamation, Hydraulic Design of Stilling Basins and Bucket Energy Dissipators, revised July 1963, by A. J. Peterka. Engineering Monograph No. 25.



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

$$\frac{d_2}{d_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right)$$

NOTE: CURVES DEVELOPED BY BRADLEY AND PETERKA FROM EXPERIMENTAL DATA. DATA POINTS OMITTED TO SIMPLIFY CHART.
CURVE FOR S=0.33 EXTRAPOLATED



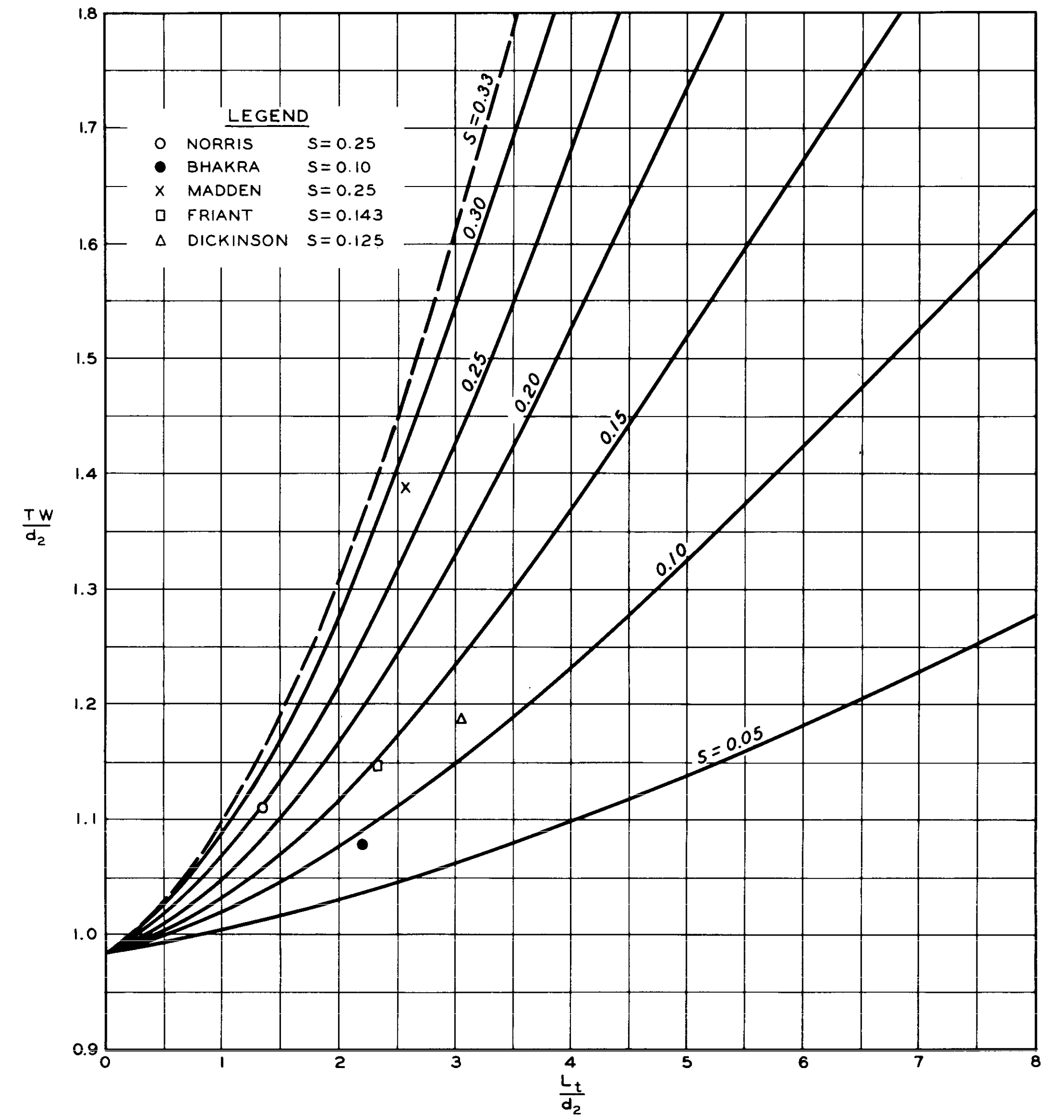
DEFINITION SKETCH

CHUTE SPILLWAYS STILLING BASINS

CONTINUOUS SLOPE LENGTH OF HYDRAULIC JUMP

HYDRAULIC DESIGN CHART 124-1

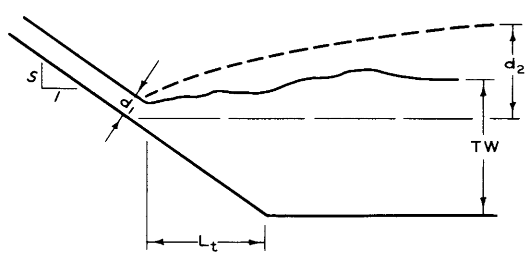
PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI



$$\frac{d_2}{d_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right)$$

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

NOTE: CURVES DEVELOPED BY BRADLEY AND PETERKA FROM EXPERIMENTAL DATA. DATA POINTS OMITTED TO SIMPLIFY CHART.
CURVE FOR S = 0.33 EXTRAPOLATED



DEFINITION SKETCH

CHUTE SPILLWAYS STILLING BASINS

NONCONTINUOUS SLOPE JUMP LENGTH ON SLOPE

HYDRAULIC DESIGN CHART 124-1/1

HYDRAULIC DESIGN CRITERIA

SHEETS 140-1 TO 140-1/8

MORNING GLORY SPILLWAYS

1. Background. Morning glory or shaft spillways utilize a crest circular in plan. The outflow is carried by a vertical or sloping shaft to a horizontal tunnel at approximately streambed level. The capacity of the morning glory spillway is limited by the size of the circular crest that can be fitted to the topography and by the head on the crest. Under various hydraulic conditions, the flow may be controlled by the crest, the throat, or the friction of the entire system flowing under pressure. A recent design of the USBR includes an inclined shaft with a vertical bend at the bottom that has a radius five times the diameter.(2) The USBR recommends that the horizontal tunnel of morning glory spillways be designed to flow only 75 percent full to eliminate instability.(8)

2. Laboratory Investigation. Laboratory investigations by Camp,(3,4) Wagner,(9) Lazzari,(5) and others on flow over circular sharp-crested weirs have been used as the basis for design of morning glory spillways. The most complete study was that made by Wagner on a 20-in.-diameter weir. The results of this study have been used for the development of HDC's 140-1 to 140-1/8.

3. Design Discharge. Morning glory spillways are generally designed for crest control or free-flow conditions. Laboratory tests indicate that submergence begins to affect the discharge when the ratio of the head to weir radius is greater than 0.45. The discharge may be determined by a modified weir equation:

$$Q = C\ (2\pi R)\ H_d^{3/2}$$

where

Q = discharge, cfs
C = discharge coefficient
R = radius of sharp crest, ft
H_d = design head on spillway crest, ft

HDC 140-1 permits a preliminary estimate of the discharge-head-radius relation for deep approach and free-flow crest conditions. The discharge curves on this chart are for head-radius ratios of 0.20, 0.30, and 0.40.

4. The experimental discharge coefficients are for the head on the circular, sharp-crested weir. Discharge coefficient curves for design head on the spillway crest have been published by the USBR(8) and are reproduced in HDC 140-1/1 for use with the equation given in paragraph 3. Curves for

three approach depth conditions are shown.

5. Crest Shape. HDC's 140-1/2 to 140-1/5 present dimensionless crest profiles and coordinates in terms of the head on the sharp crest. Tabulations are included for ratios of head to weir radius of 0.2 to 2.0 and ratios of approach depth to radius of 2, 0.30, and 0.15. The ratio of head (H_s) to weir radius (R) is required for use of these charts. This relation can be determined from a USBR design aid(8) reproduced as HDC 140-1/6.

6. Crest Shape Equations. Equations of the lower surface of the nappe have been determined for a limited number of conditions. The converging flow over the crest results in complex equations for morning glory spillway crest shapes. Upstream and downstream quadrant shape equations for three approach depth conditions and for three ratios of head to radius for each depth are given in HDC 140-1/7. The equations are considered adequate for defining crest shapes within the limits indicated on the chart.

7. Transition Shape. The crest shape is generally connected to the vertical shaft by a transition section. A procedure which can be used for transition shape design has been published by the USBR.(8)

8. Application Procedure. HDC 140-1/8 is a sample computation illustrating the use of HDC's 140-1/1 to 140-1/7 in morning glory spillway design. In this computation the horizontal tunnel was considered as flowing 75 percent full.

9. Design Factors. A number of problems encountered in the design and operation of morning glory spillways have been reported.(1,2,6,7) In the design of these structures the engineer is sometimes concerned with flow regulation by crest gates. Information may also be required on discharge coefficients and crest pressures for less than design flow and possible effects of adjacent topography on radial flow to the crest. Therefore, in some cases a model study may be required before selection of the final design.

10. References.

(1) Abecasis, F. N., "The behavior of morning glory shaft spillways." IAHR, Proceedings of the Sixth General Meeting, The Hague, 1955, vol 3, pp C8-1-C8-10.

(2) Bradley, J. N., "Prototype behavior," in "Morning glory shaft spillways: A symposium." Transactions, American Society of Civil Engineers, vol 121 (1956), pp 312-344.

(3) Camp, C. S., Determination of Shape of Nappe and Coefficient of Discharge of a Vertical Sharp-crested Weir, Circular in Plan with Radially Inward Flow. State University of Iowa thesis, 1937.

(4) Camp, C. S., and Howe, J. W., "Tests of circular weirs." Civil

Engineering, vol 9, No. 4 (April 1939), pp 247-248.

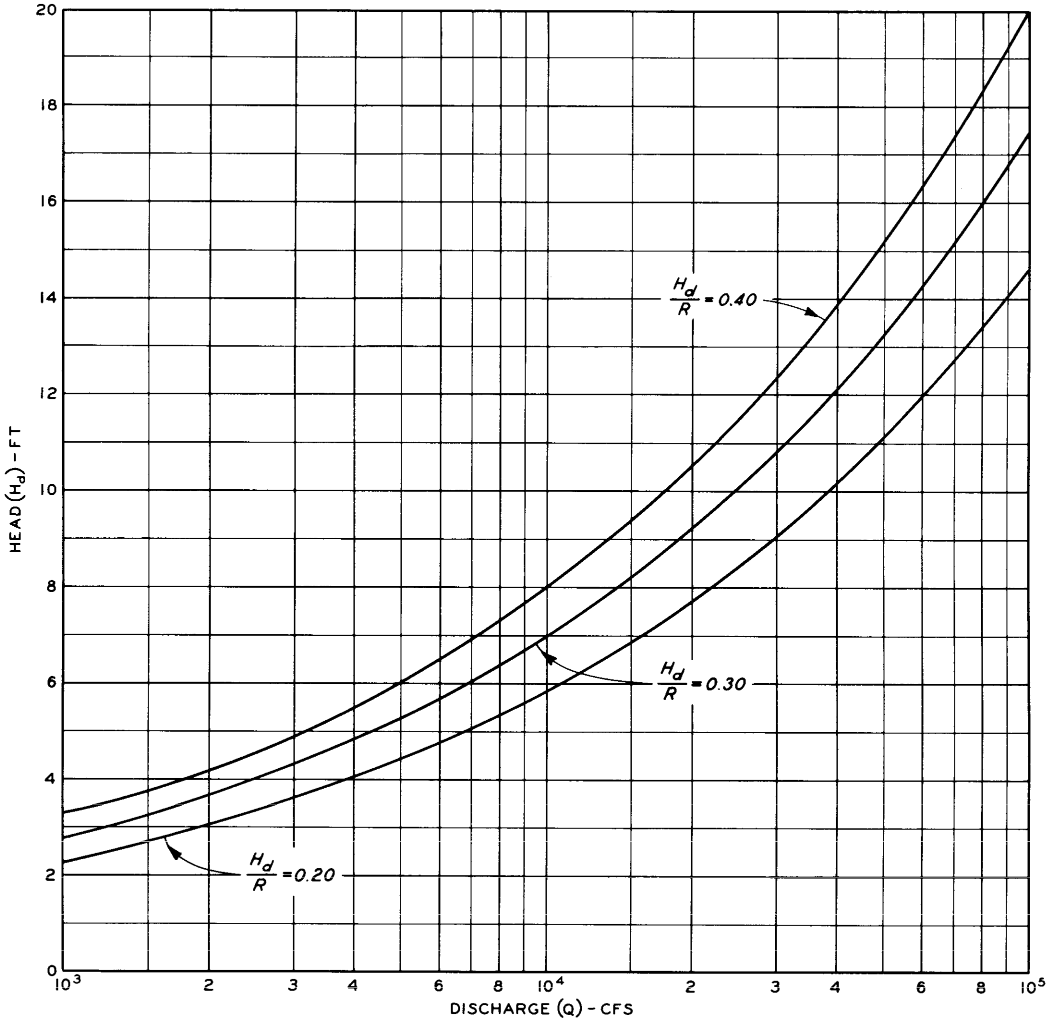
(5) Lazzari, E., "Ricerca sperimentale sullo sfioratore a pianta circolare." L'Energia Elettrica (November 1954), pp 838-840.

(6) Peterka, A. J., "Performance tests on prototype and model," in "Morning glory shaft spillways: A symposium." Transactions, American Society of Civil Engineers, vol 121 (1956), pp 385-409.

(7) U. S. Bureau of Reclamation, "Tests on preliminary designs of spillways," in Model Studies of Spillways. Boulder Canyon Project Final Reports, Part VI, Hydraulic Investigations, Bulletin 1 (Denver, Colo., 1938), Chapter II.

(8) __________, Design of Small Dams, 1st ed. 1960.

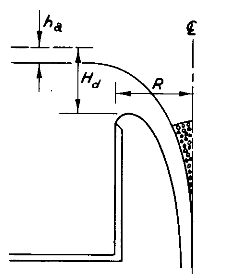
(9) Wagner, W. E., "Shaft spillways: determination of pressure-controlled profiles." Transactions, American Society of Civil Engineers, vol 121 (1956).



EQUATION

$$Q = C\,(2\pi R)\,H_d^{3/2}$$

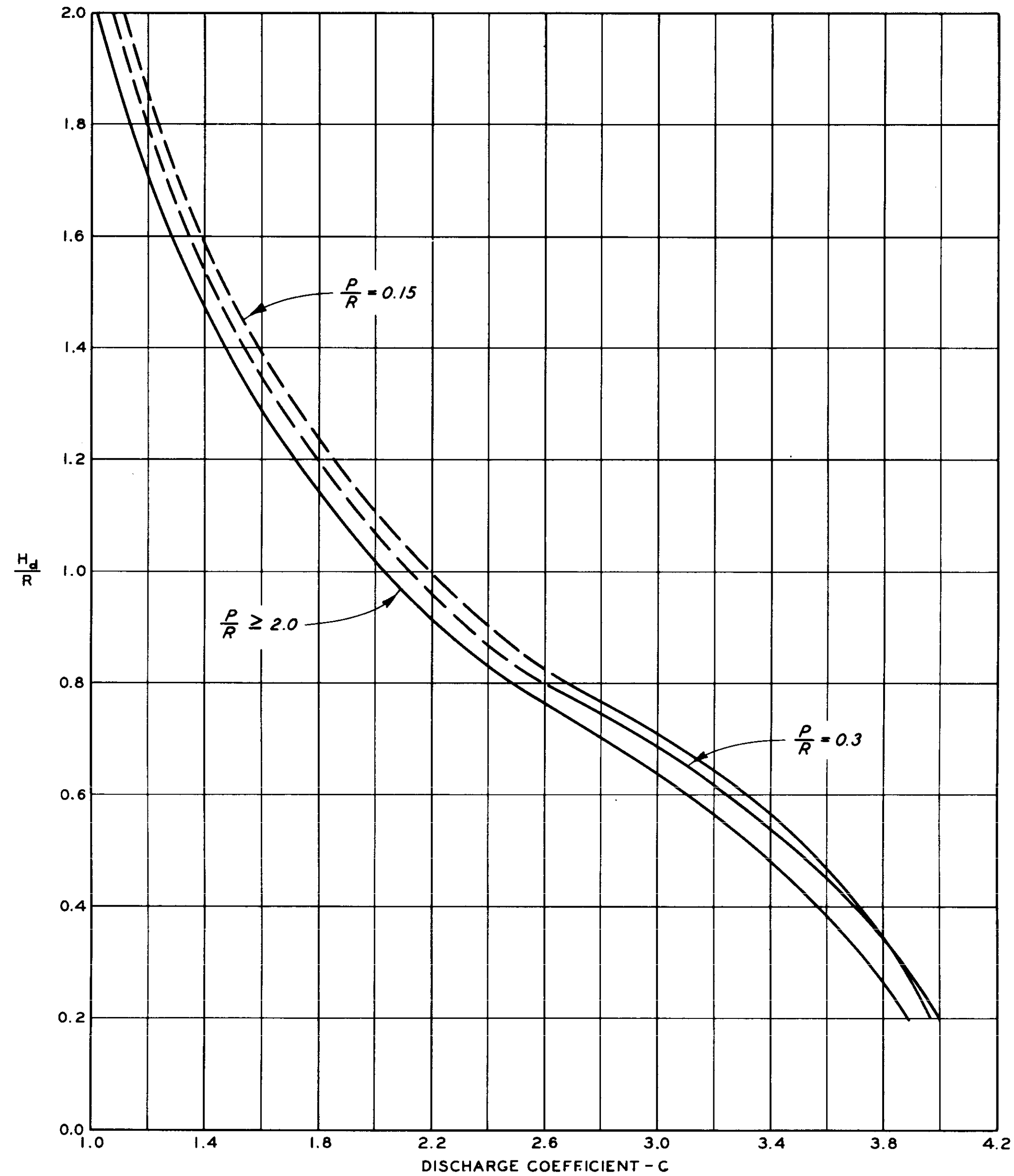
WHERE:
- Q = DISCHARGE, CFS
- C = DISCHARGE COEFFICIENT
- R = RADIUS OF SHARP CREST, FT
- H_d = DESIGN HEAD ON SPILLWAY CREST, FT



DEFINITION SKETCH

MORNING GLORY SPILLWAYS
DEEP APPROACH - CREST CONTROL
DESIGN DISCHARGE

HYDRAULIC DESIGN CHART 140-1



NOTE: CURVES ARE TAKEN FROM USBR DESIGN OF SMALL DAMS AND ARE BASED ON WAGNER'S DATA FOR FULLY AERATED FLOW OVER A SHARP-CRESTED WEIR.

DASHED CURVES ARE BASED ON EXTRAPOLATED VALUES OF H_d/R (CHART 140-1/6).

P = APPROACH DEPTH TO SHARP CREST, FT.

EQUATION

$$Q = C(2\pi R) H_d^{3/2}$$

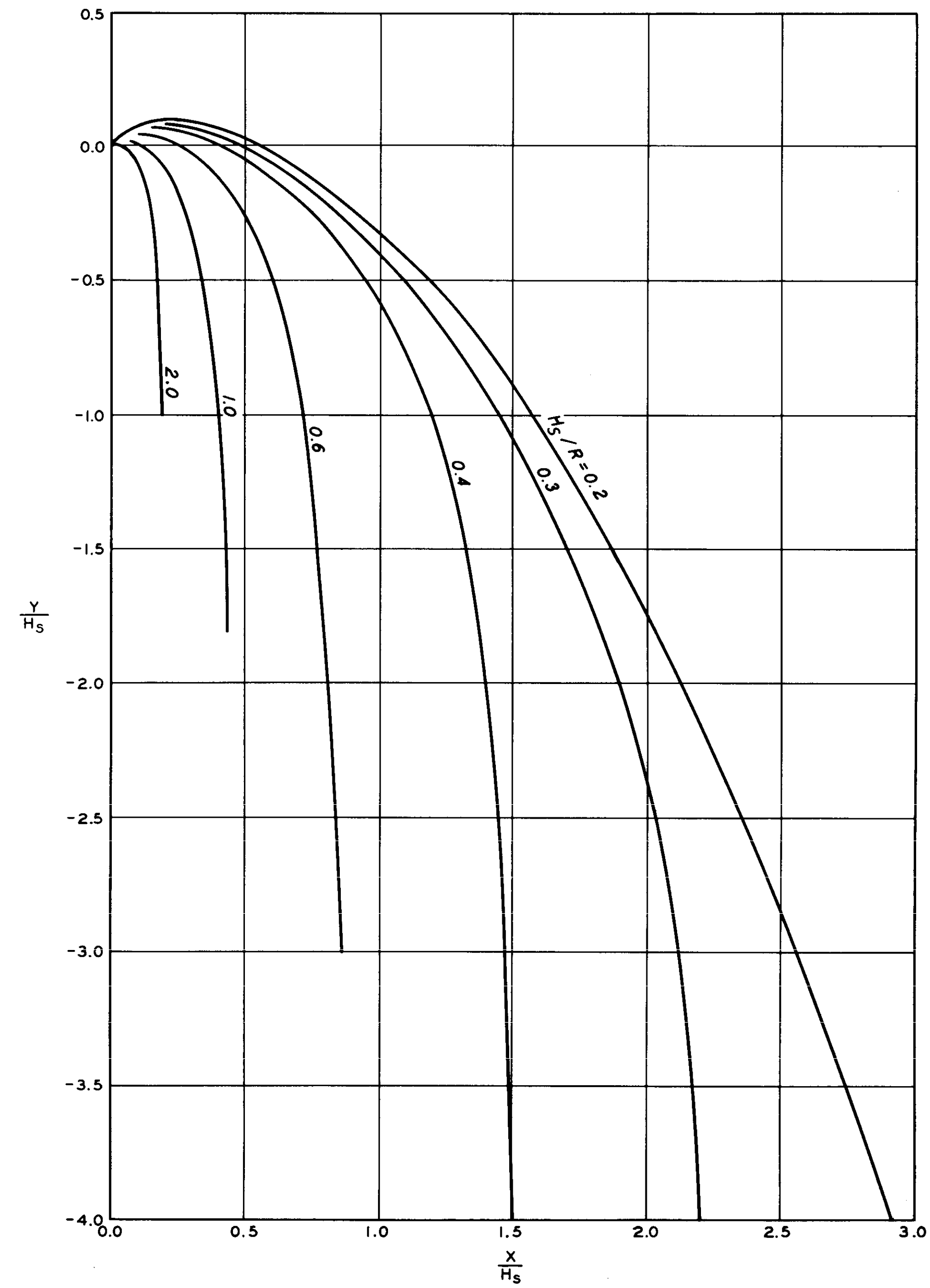
WHERE:

- Q = DISCHARGE, CFS.
- C = DISCHARGE COEFFICIENT.
- R = RADIUS OF SHARP CREST, FT.
- H_d = DESIGN HEAD ON SPILLWAY CREST, FT.

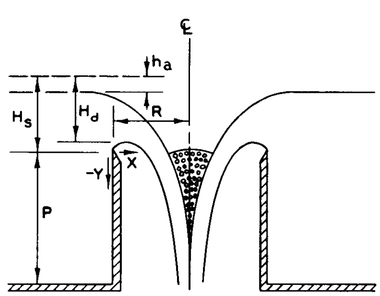
MORNING GLORY SPILLWAYS

DISCHARGE COEFFICIENT DESIGN HEAD

HYDRAULIC DESIGN CHART 140-1/1



NOTE: $\frac{P}{R} \geq 2$, NEGLIGIBLE VELOCITY OF APPROACH. NAPPE AERATED.



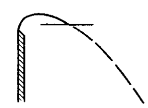
DEFINITION SKETCH

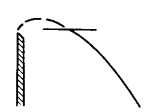
MORNING GLORY SPILLWAYS
LOWER NAPPE PROFILES

HYDRAULIC DESIGN CHART 140-1/2

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

$\frac{H_s}{R}$	0.20	0.30	0.40	0.50	0.60	1.00	1.50	2.00
$\frac{X}{H_s}$	$\frac{Y}{H_s}$ FOR PORTION OF PROFILE ABOVE WEIR CREST							
0.000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
.010	.0133	.0128	.0122	.0116	.0112	.0095	.0077	.0070
.020	.0250	.0236	.0225	.0213	.0202	.0159	.0115	.0090
.030	.0350	.0327	.0308	.0289	.0270	.0198	.0126	.0085
.040	.0435	.0403	.0377	.0351	.0324	.0220	.0117	.0050
.050	.0506	.0471	.0436	.0402	.0368	.0226	.0092	
.060	.0570	.0531	.0489	.0448	.0404	.0220	.0053	
.070	.0627	.0584	.0537	.0487	.0432	.0201	.0001	
.080	.0677	.0630	.0578	.0521	.0455	.0172		
.090	.0722	.0670	.0613	.0549	.0471	.0135		
.100	.0762	.0705	.0642	.0570	.0482	.0089		
.120	.0826	.0758	.0683	.0596	.0483			
.140	.0872	.0792	.0705	.0599	.0460			
.160	.0905	.0812	.0710	.0585	.0418			
.180	.0927	.0820	.0705	.0559	.0361			
.200	.0938	.0819	.0688	.0521	.0292			
.250	.0926	.0773	.0596	.0380	.0068			
.300	.0850	.0668	.0446	.0174				
.350	.0750	.0540	.0280					
.400	.0620	.0365	.0060					
.450	.0450	.0170						
.500	.0250							
.550	.0020							
.600								
.650								
$\frac{Y}{H_s}$	$\frac{X}{H_s}$ FOR PORTION OF PROFILE BELOW WEIR CREST							
0.000	0.554	0.487	0.413	0.334	0.262	0.116	0.070	0.048
−.020	.592	.526	.452	.369	.293	.145	.096	.074
−.040	.627	.563	.487	.400	.320	.165	.115	.088
−.060	.660	.596	.519	.428	.342	.182	.129	.100
−.080	.692	.628	.549	.454	.363	.197	.140	.110
−.100	.722	.657	.577	.478	.381	.210	.150	.118
−.150	.793	.725	.641	.531	.423	.238	.170	.132
−.200	.860	.790	.698	.575	.459	.260	.184	.144
−.250	.919	.847	.750	.613	.490	.280	.196	.153
−.300	.976	.900	.797	.648	.518	.296	.206	.160
−.400	1.079	1.000	.880	.706	.562	.322	.220	.168
−.500	1.172	1.087	.951	.753	.598	.342	.232	.173
−.600	1.260	1.167	1.012	.793	.627	.359	.240	.179
−.800	1.422	1.312	1.112	.854	.673	.384	.253	.184
−1.000	1.564	1.440	1.189	.899	.710	.402	.260	.188
−1.200	1.691	1.553	1.248	.933	.739	.417	.266	
−1.400	1.808	1.653	1.293	.963	.760	.423		
−1.600	1.918	1.742	1.330	.988	.780	.430		
−1.800	2.024	1.821	1.358	1.008	.797	.433		
−2.000	2.126	1.891	1.381	1.025	.810			
−2.500	2.354	2.027	1.430	1.059	.838			
−3.000	2.559	2.119	1.468	1.086	.853			
−3.500	2.749	2.171	1.489	1.102				
−4.000	2.914	2.201	1.500					
−4.500	3.053	2.220	1.509					
−5.000	3.178	2.227						
−5.500	3.294	2.229						
−6.000	3.405	2.232						





NOTE: NEGLIGIBLE VELOCITY OF APPROACH. NAPPE AERATED. TABLE BY WAGNER, TRANSACTIONS, ASCE, 1956.

MORNING GLORY SPILLWAYS

LOWER NAPPE SURFACE COORDINATES

P/R ≥ 2

HYDRAULIC DESIGN CHART 140-1/3

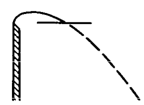
WES 10-61

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

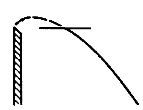
MORNING GLORY SPILLWAYS

LOWER NAPPE SURFACE COORDINATES P/R = 0.30

$\frac{H_s}{R}$	0.20	0.30	0.40	0.50	0.60	0.80
$\frac{X}{H_s}$	$\frac{Y}{H_s}$ FOR PORTION OF PROFILE ABOVE WEIR CREST					
0.000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
.010	.0130	.0130	.0120	.0115	.0110	.0100
.020	.0245	.0240	.0225	.0195	.0180	.0170
.030	.0340	.0330	.0300	.0270	.0240	.0210
.040	.0415	.0390	.0365	.0320	.0285	.0240
.050	.0495	.0455	.0420	.0370	.0325	.0245
.060	.0560	.0505	.0460	.0405	.0350	.0250
.070	.0610	.0550	.0500	.0440	.0370	.0245
.080	.0660	.0590	.0530	.0460	.0385	.0235
.090	.0705	.0625	.0550	.0480	.0390	.0215
.100	.0740	.0660	.0575	.0500	.0395	.0190
.120	.0800	.0705	.0600	.0510	.0380	.0120
.140	.0840	.0735	.0615	.0515	.0355	.0020
.160	.0870	.0750	.0610	.0500	.0310	
.180	.0885	.0755	.0600	.0475	.0250	
.200	.0885	.0745	.0575	.0435	.0180	
.250	.0855	.0685	.0480	.0270		
.300	.0780	.0580	.0340	.0050		
.350	.0660	.0425	.0150			
.400	.0495	.0240				
.450	.0300	.0025				
.500	.0090					
.550						



$\frac{Y}{H_s}$	$\frac{X}{H_s}$ FOR PORTION OF PROFILE BELOW WEIR CREST					
0.000	0.519	0.455	0.384	0.310	0.238	0.144
−.020	.560	.495	.423	.345	.272	.174
−.040	.598	.532	.458	.376	.300	.198
−.060	.632	.567	.491	.406	.324	.220
−.080	.664	.600	.522	.432	.348	.238
−.100	.693	.631	.552	.456	.368	.254
−.150	.760	.701	.618	.510	.412	.290
−.200	.831	.763	.677	.558	.451	.317
−.250	.893	.826	.729	.599	.483	.341
−.300	.953	.880	.779	.634	.510	.362
−.400	1.060	.981	.867	.692	.556	.396
−.500	1.156	1.072	.938	.745	.595	.424
−.600	1.242	1.153	1.000	.780	.627	.446
−.800	1.403	1.301	1.101	.845	.672	.478
−1.000	1.549	1.430	1.180	.892	.707	.504
−1.200	1.680	1.543	1.240	.930	.733	.524
−1.400	1.800	1.647	1.287	.959	.757	.540
−1.600	1.912	1.740	1.323	.983	.778	.551
−1.800	2.018	1.821	1.353	1.005	.797	.560
−2.000	2.120	1.892	1.380	1.022	.810	.569
−2.500	2.351	2.027	1.428	1.059	.837	
−3.000	2.557	2.113	1.464	1.081	.852	
−3.500	2.748	2.167	1.489	1.099		
−4.000	2.911	2.200	1.499			
−4.500	3.052	2.217	1.507			
−5.000	3.173	2.223				
−5.500	3.290	2.228				
−6.000	3.400					

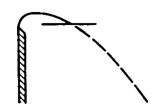


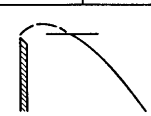
NOTE: APPRECIABLE VELOCITY OF APPROACH. NAPPE AERATED. TABLE BY WAGNER, TRANSACTIONS, ASCE, 1956.

HYDRAULIC DESIGN CHART 140-1/4

WES 10-61

$\frac{H_s}{R}$	0.20	0.30	0.40	0.50	0.60	0.80
$\frac{X}{H_s}$	$\frac{Y}{H_s}$ FOR PORTION OF PROFILE ABOVE WEIR CREST					
0.000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
.010	.0120	.0115	.0110	.0105	.0100	.0090
.020	.0210	.0195	.0185	.0170	.0160	.0140
.030	.0285	.0265	.0250	.0225	.0200	.0165
.040	.0345	.0325	.0300	.0265	.0230	.0170
.050	.0405	.0375	.0345	.0300	.0250	.0170
.060	.0450	.0420	.0380	.0330	.0265	.0165
.070	.0495	.0455	.0410	.0350	.0270	.0150
.080	.0525	.0485	.0435	.0365	.0270	.0130
.090	.0560	.0510	.0455	.0370	.0265	.0100
.100	.0590	.0535	.0465	.0375	.0255	.0065
.120	.0630	.0570	.0480	.0365	.0220	
.140	.0660	.0585	.0475	.0345	.0175	
.160	.0670	.0590	.0460	.0305	.0110	
.180	.0675	.0580	.0435	.0260	.0040	
.200	.0670	.0560	.0395	.0200		
.250	.0615	.0470	.0265	.0015		
.300	.0520	.0330	.0100			
.350	.0380	.0165				
.400	.0210					
.450	.0015					
.500						
.550						
$\frac{Y}{H_s}$	$\frac{X}{H_s}$ FOR PORTION OF PROFILE BELOW WEIR CREST					
0.000	0.454	0.392	0.325	0.253	0.189	0.116
-.020	.499	.437	.369	.292	.228	.149
-.040	.540	.478	.407	.328	.259	.174
-.060	.579	.516	.443	.358	.286	.195
-.080	.615	.550	.476	.386	.310	.213
-.100	.650	.584	.506	.412	.331	.228
-.150	.726	.660	.577	.468	.376	.263
-.200	.795	.729	.639	.516	.413	.293
-.250	.862	.790	.692	.557	.445	.319
-.300	.922	.843	.741	.594	.474	.342
-.400	1.029	.947	.828	.656	.523	.381
-.500	1.128	1.040	.902	.710	.567	.413
-.600	1.220	1.129	.967	.753	.601	.439
-.800	1.380	1.285	1.080	.827	.655	.473
-1.000	1.525	1.420	1.164	.878	.696	.498
-1.200	1.659	1.537	1.228	.917	.725	.517
-1.400	1.780	1.639	1.276	.949	.750	.531
-1.600	1.897	1.729	1.316	.973	.770	.544
-1.800	2.003	1.809	1.347	.997	.787	.553
-2.000	2.104	1.879	1.372	1.013	.801	.560
-2.500	2.340	2.017	1.423	1.049	.827	
-3.000	2.550	2.105	1.457	1.073	.840	
-3.500	2.740	2.153	1.475	1.088		
-4.000	2.904	2.180	1.487			
-4.500	3.048	2.198	1.491			
-5.000	3.169	2.207				
-5.500	3.286	2.210				
-6.000	3.396					





NOTE: APPRECIABLE VELOCITY OF APPROACH. NAPPE AERATED. TABLE BY WAGNER, TRANSACTIONS, ASCE, 1956.

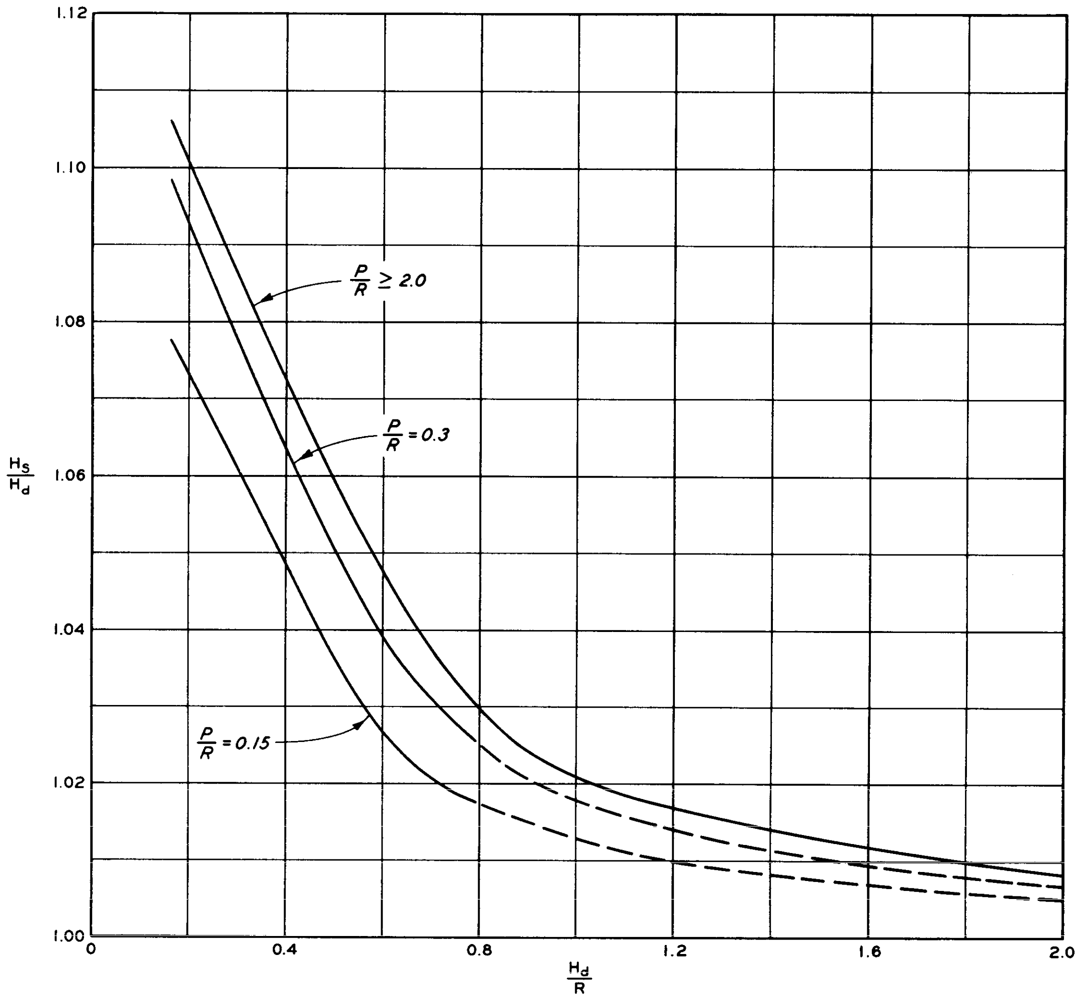
MORNING GLORY SPILLWAYS

LOWER NAPPE SURFACE COORDINATES

P/R = 0.15

HYDRAULIC DESIGN CHART 140-1/5

WES 10-61



NOTE: P = DEPTH OF APPROACH TO SHARP CREST.
R = RADIUS OF SHARP CREST.
H_S = DESIGN HEAD ON SHARP CREST.
H_d = DESIGN HEAD ON SPILLWAY CREST.

CURVES ARE TAKEN FROM USBR <u>DESIGN OF SMALL DAMS.</u>

DASHED LINES ARE BASED ON EXTRAPOLATION OF DATA.

MORNING GLORY SPILLWAYS

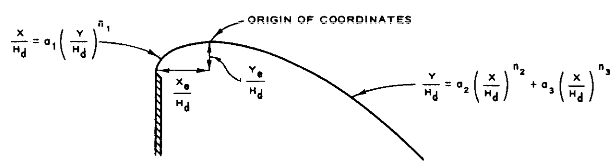
$\frac{H_S}{H_d}$ VS $\frac{H_d}{R}$

HYDRAULIC DESIGN CHART 140-1/6

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

WES 10-61

$\frac{P}{R}$	$\frac{H_s}{R}$	$\frac{X_e}{H_d}$	$\frac{Y_e}{H_d}$	UPSTREAM QUADRANT EQUATIONS	LIMITING $\frac{X}{H_d}$	DOWNSTREAM QUADRANT EQUATIONS	LIMITING $\frac{X}{H_d}$
≥2	0.2	−0.237	0.1035	$\frac{X}{H_d} = -0.635\left(\frac{Y}{H_d}\right)^{0.410}$	−0.190	$\frac{Y}{H_d} = 0.610\left(\frac{X}{H_d}\right)^{1.85}$	3.20
	0.3	−0.209	0.0893	$\frac{X}{H_d} = -0.568\left(\frac{Y}{H_d}\right)^{0.397}$	−0.166	$\frac{Y}{H_d} = 0.685\left(\frac{X}{H_d}\right)^{1.85} + 0.000009\left(\frac{X}{H_d}\right)^{15.6}$	2.25
	0.4	−0.174	0.0764	$\frac{X}{H_d} = -0.538\left(\frac{Y}{H_d}\right)^{0.424}$	−0.145	$\frac{Y}{H_d} = 0.830\left(\frac{X}{H_d}\right)^{1.85} + 0.035\left(\frac{X}{H_d}\right)^{12.2}$	1.45
0.30	0.2	−0.219	0.0972	$\frac{X}{H_d} = -0.622\left(\frac{Y}{H_d}\right)^{0.419}$	−0.155	$\frac{Y}{H_d} = 0.590\left(\frac{X}{H_d}\right)^{1.73} + 0.00375\left(\frac{X}{H_d}\right)^{4.73}$	3.50
	0.3	−0.189	0.0817	$\frac{X}{H_d} = -0.637\left(\frac{Y}{H_d}\right)^{0.451}$	−0.140	$\frac{Y}{H_d} = 0.650\left(\frac{X}{H_d}\right)^{1.73} + 0.00174\left(\frac{X}{H_d}\right)^{5.68}$	2.15
	0.4	−0.156	0.0655	$\frac{X}{H_d} = -0.556\left(\frac{Y}{H_d}\right)^{0.440}$	−0.120	$\frac{Y}{H_d} = 0.725\left(\frac{X}{H_d}\right)^{1.73} + 0.140\left(\frac{X}{H_d}\right)^{6.93}$	1.35
0.15	0.2	−0.192	0.0724	$\frac{X}{H_d} = -0.625\left(\frac{Y}{H_d}\right)^{0.430}$	−0.160	$\frac{Y}{H_d} = 0.600\left(\frac{X}{H_d}\right)^{1.73} + 0.00375\left(\frac{X}{H_d}\right)^{4.77}$	3.45
	0.3	−0.164	0.0627	$\frac{X}{H_d} = -0.665\left(\frac{Y}{H_d}\right)^{0.476}$	−0.125	$\frac{Y}{H_d} = 0.660\left(\frac{X}{H_d}\right)^{1.73} + 0.0008\left(\frac{X}{H_d}\right)^{10.0}$	2.15
	0.4	−0.132	0.0504	$\frac{X}{H_d} = -0.540\left(\frac{Y}{H_d}\right)^{0.451}$	−0.105	$\frac{Y}{H_d} = 0.760\left(\frac{X}{H_d}\right)^{1.73} + 0.155\left(\frac{X}{H_d}\right)^{6.67}$	1.35



MORNING GLORY SPILLWAYS

CREST SHAPE EQUATIONS

HYDRAULIC DESIGN CHART 140-1/7

WES 10-61

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

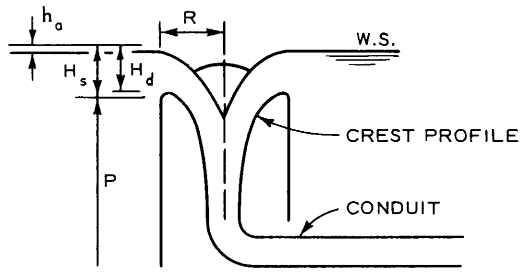
U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB CW 804 PROJECT John Doe Dam SUBJECT Morning Glory Spillway

COMPUTATION Spillway Design

COMPUTED BY CWD DATE 6/14/61 CHECKED BY MBB DATE 6/30/61



GIVEN:

Design discharge (Q) = 10,000 cfs
Design head (H_d) = 10 ft
Approach depth (P) $\geq$ 2 R
Conduit to be designed for free flow

REQUIRED:

Spillway shape with minimum crest radius

COMPUTE:

1. Crest radius required to pass design discharge.

Assume R = 16.4 ft

$$\frac{H_d}{R} = \frac{10}{16.4} = 0.61$$

For $\frac{H_d}{R} = 0.61$ and $\frac{P}{R} \geq 2$

$C = 3.08$ (HDC 140-1/1)

$$Q = C\,(2\,\pi\,R)\,H_d^{3/2}$$
$$= 3.08 \times 6.28 \times 16.4 \times 10^{3/2}$$
$$= 10{,}030 \text{ cfs*}$$

2. Ratio of head on weir to crest radius (H_s/R)

For $\frac{H_d}{R} = 0.61$ and $\frac{P}{R} \geq 2$

$\frac{H_s}{H_d} = 1.047$ (HDC 140-1/6)

$H_s = 1.047 \times 10 = 10.47$ ft

$\frac{H_s}{R} = \frac{10.47}{16.4} = 0.64$ (Partial submergence, $0.45 < H_s/R < 1.00$)

3. Crest profile for $H_s/R = 0.64$ by interpolation from table on HDC 140-1/3.

*If computed discharge does not closely approximate design discharge, assume new radius and repeat computation.

MORNING GLORY SPILLWAYS

SPILLWAY DESIGN

SAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 140-1/8