HYDRAULIC DESIGN CRITERIA

SHEETS 310-1 TO 310-1/2

WAVE PRESSURES ON CREST GATES

1. A theory for the pressure resulting from a wave striking a vertical wall was developed by Sainflou (1). The particular phenomenon is known as a "clapotis." The incident wave combines with the reflected wave to produce a wave height twice that of the incident wave. The theory is valid only for wave heights which do not exceed the still-water depth. The depth of water behind spillway crest gates is normally greater than the design wave height. Therefore, the theory can be used to estimate pressure distribution for the design of crest gates and for spillway stability analysis problems.

2. Application of the Sainflou wave pressure theory to crest gates and spillways is illustrated on Hydraulic Design Chart 310-1. The first equation is a parameter of the clapotis and indicates the effective change in mean water depth resulting from transition of the wave. The second equation indicates the change in bottom pressure. The clapotis results in pressure decrease as well as a pressure increase relative to the still-water static pressure. Design problems are generally only concerned with the maximum pressure.

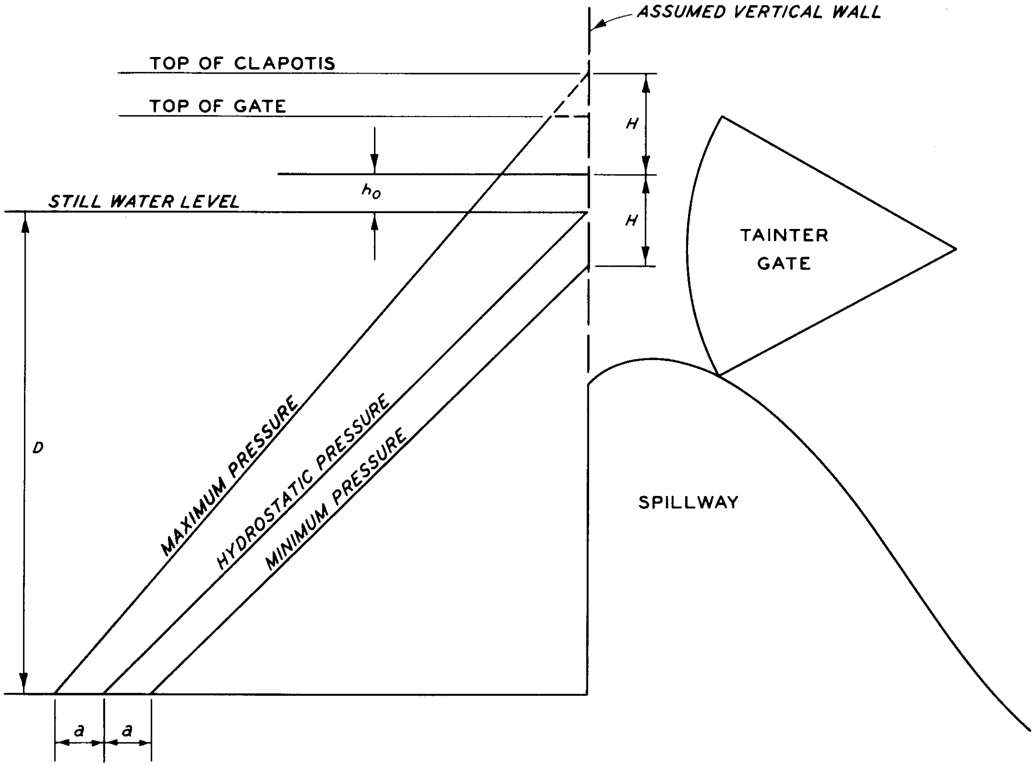
3. Overtopping of a gate by waves occurs when the clapotis rises above the gate. For this condition the maximum pressure distribution would be zero at the top of the gate and vary along a curve which would become asymptotic to the straight-line distribution at the bottom of the spillway structure. As data are not available to establish the true pressure distribution, it may be assumed for design purposes that the portion of the pressure diagram above the top of the gate is ineffective and that the pressure distribution below the top of the gate is a straight line as indicated on Chart 310-1.

4. The equations of the clapotis involve hyperbolic functions of the cosine and cotangent. Hydraulic Design Chart 310-1/1 presents graphical and tabulated values of these functions for depth-wave length ratios (D/λ) of 0.0 to 0.8.

5. Hydraulic Design Chart 310-1/2 is a sample computation illustrating use of the Sainflou theory for crest gate design and spillway stability analysis. A wave length, wave height, and approach depth of 125, 6, and 75 ft, respectively, have been assumed for the computation. The direction of approach is considered normal to the spillway.

---

(1) M. Sainflou, "Essay on vertical breakwaters," Annales des Ponts et Chaussees (July-August 1928), pp 5-48. Translated by C. R. Hatch for U. S. Army Engineer Division, Great Lakes, CE, Chicago, Ill. (No date.)



EQUATIONS

$$h_o = \frac{\pi H^2}{\lambda} \coth \frac{2\pi D}{\lambda}$$

$$a = \frac{H}{\cosh \frac{2\pi D}{\lambda}}$$

WHERE:

h_o = A PARAMETER OF THE CLAPOTIS, FT

a = A BOTTOM PRESSURE PARAMETER, FT OF WATER

D = DEPTH OF WATER (STILL WATER LEVEL TO BOTTOM), FT

H = WAVE HEIGHT, FT

λ = WAVE LENGTH, FT

NOTE: VALUES OF $\cosh \frac{2\pi D}{\lambda}$ AND $\coth \frac{2\pi D}{\lambda}$ ARE ON CHART 310-1/1.

CREST GATES
WAVE PRESSURE DESIGN ASSUMPTIONS

HYDRAULIC DESIGN CHART 310-1

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

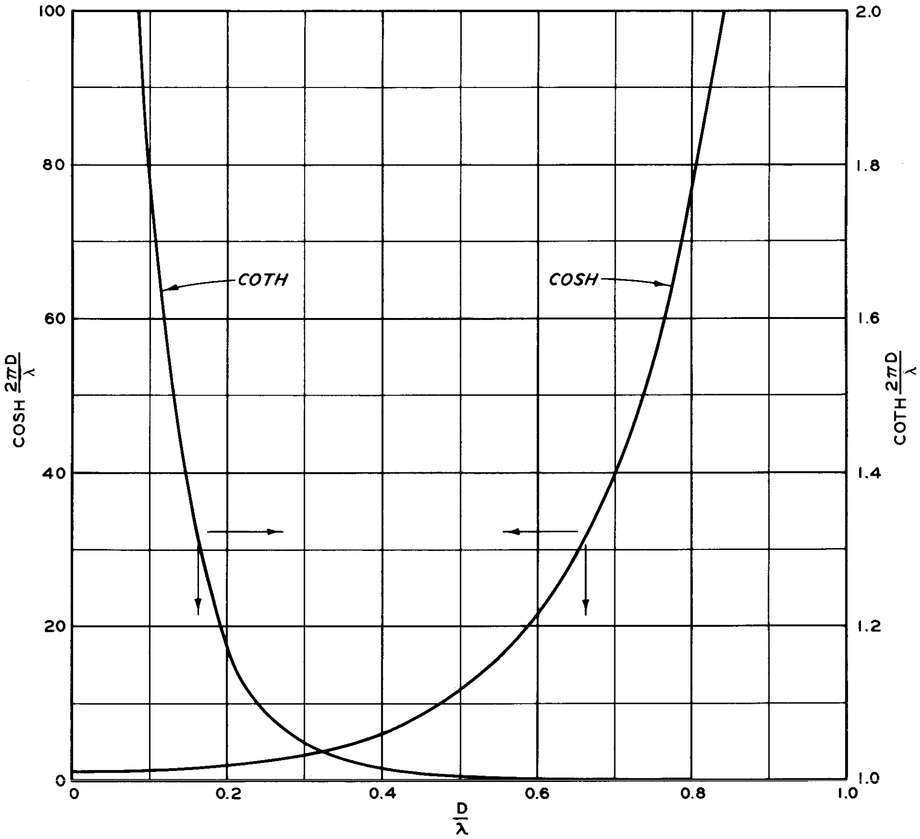


TABLE OF VALUES

$\frac{D}{\lambda}$	COSH $\frac{2\pi D}{\lambda}$	COTH $\frac{2\pi D}{\lambda}$
0	1.000	∞
0.1	1.204	1.796
0.2	1.898	1.177
0.3	3.366	1.047
0.4	6.205	1.013
0.5	11.574	1.004
0.6	21.659	1.001
0.7	40.569	1.000
0.8	76.013	1.000

NOTE: D=DEPTH OF WATER (STILL WATER LEVEL TO BOTTOM), FT
λ= WAVE LENGTH, FT

CRESTGATES

WAVE PRESSURE
HYPERBOLIC FUNCTIONS

HYDRAULIC DESIGN CHART 310-1/1

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

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

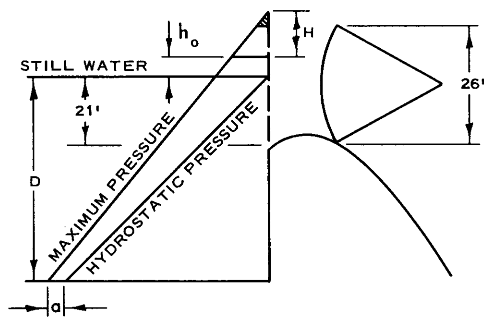
JOB CW 804 PROJECT John Doe Dam SUBJECT Crest Gates

COMPUTATION Effects of Wave Pressure

COMPUTED BY RGC DATE 6/3/60 CHECKED BY MBB DATE 6/7/60

GIVEN:

Gated spillway as shown
Design wave length (λ) = 125 ft
Design wave height (H) = 6 ft
Still-water depth (D) = 75 ft



REQUIRED:

1. Maximum pressure distribution on gate and spillway structure
2. Maximum hydraulic load per ft of width of gate
3. Maximum hydraulic load per ft of width of structure

COMPUTE:

1. Pressure distribution

(a) Maximum effective depth with wave

$$h_o = \frac{\pi H^2}{\lambda} \coth \frac{2\pi D}{\lambda} \quad \text{(Chart 310-1)}$$

$$\frac{D}{\lambda} = \frac{75}{125} = 0.6; \ \coth \frac{2\pi D}{\lambda} = 1.001 \quad \text{(Chart 310-1/1)}$$

$$h_o = \frac{3.14 \times 6^2}{125} \times 1.001 = 0.9 \text{ ft.}$$

Effective depth = $D + h_o + H = 75.0 + 0.9 + 6.0 = 81.9$ ft.

(b) Maximum effective bottom pressure with wave

$$a = \frac{H}{\cosh \frac{2\pi D}{\lambda}} \quad \text{(Chart 310-1)}$$

$$\frac{D}{\lambda} = 0.6; \ \cosh \frac{2\pi D}{\lambda} = 21.7 \quad \text{(Chart 310-1/1)}$$

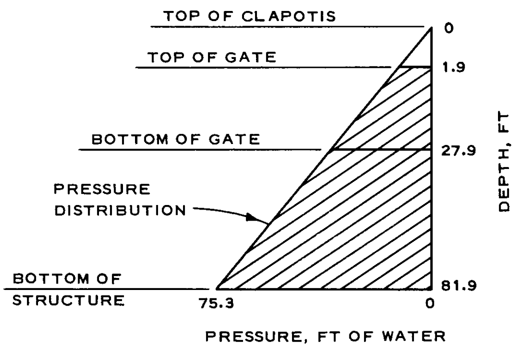
$$a = \frac{6}{21.7} = 0.3 \text{ ft.}$$

Effective pressure = $D + a = 75.0 + 0.3 = 75.3$ ft.

(c) Depth of gate overtopping

Depth = 81.9 − (75.0 − 21.0 + 26.0) = 1.9 ft.

(d) Maximum pressure distribution graph



2. Maximum hydraulic load per foot of width of gate (from 1d above)

Maximum pressure at top of gate $(P_1) = \frac{1.9}{81.9} \times 75.3 = 1.7$ ft

Maximum pressure at bottom of gate $(P_2) = \frac{27.9}{81.9} \times 75.3 = 25.7$ ft

Maximum hydraulic load on gate $(R) = \gamma \left(\frac{P_1 + P_2}{2}\right) \times$ gate height

γ = specific weight of water = 62.4 lb/ft^3

$$R = 62.4 \left(\frac{1.7 + 25.7}{2}\right) 26$$

= 22,200 lb/ft of width

Note: For still-water level maximum gate pressure is 21 ft of water and maximum hydraulic load is 13,750 lb/ft of width.

3. Maximum hydraulic load per foot of width of structure (from 1d above)

Maximum pressure at bottom of structure $(P_3) = 75.3$ ft

Maximum hydraulic load on structure $(R_h) = \gamma \left(\frac{P_1 + P_3}{2}\right) \times$ height of structure

$$= 62.4 \left(\frac{1.7 + 75.3}{2}\right) 80$$

= 192,000 lb/ft of width

Note: Equivalent for still-water level is 175,000 lb/ft of width.

CREST GATES
WAVE PRESSURE
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 310-1/2

HYDRAULIC DESIGN CRITERIA

SHEETS 311-1 TO 311-5

TAINTER GATES ON SPILLWAY CRESTS

DISCHARGE COEFFICIENTS

1. Discharge through a partially open tainter gate mounted on a spillway crest can be computed using the basic orifice equation:

$$Q = CA\sqrt{2gH}$$

where,

Q = discharge in cfs

C = discharge coefficient

A = area of orifice opening in ft^2

H = head to the center of the orifice in ft.

The coefficient (C) in the above equation is primarily dependent upon the characteristics of the flow lines approaching and leaving the orifice. In turn, these flow lines are dependent upon the shape of the crest, the radius of the gate, and the location of the trunnion.

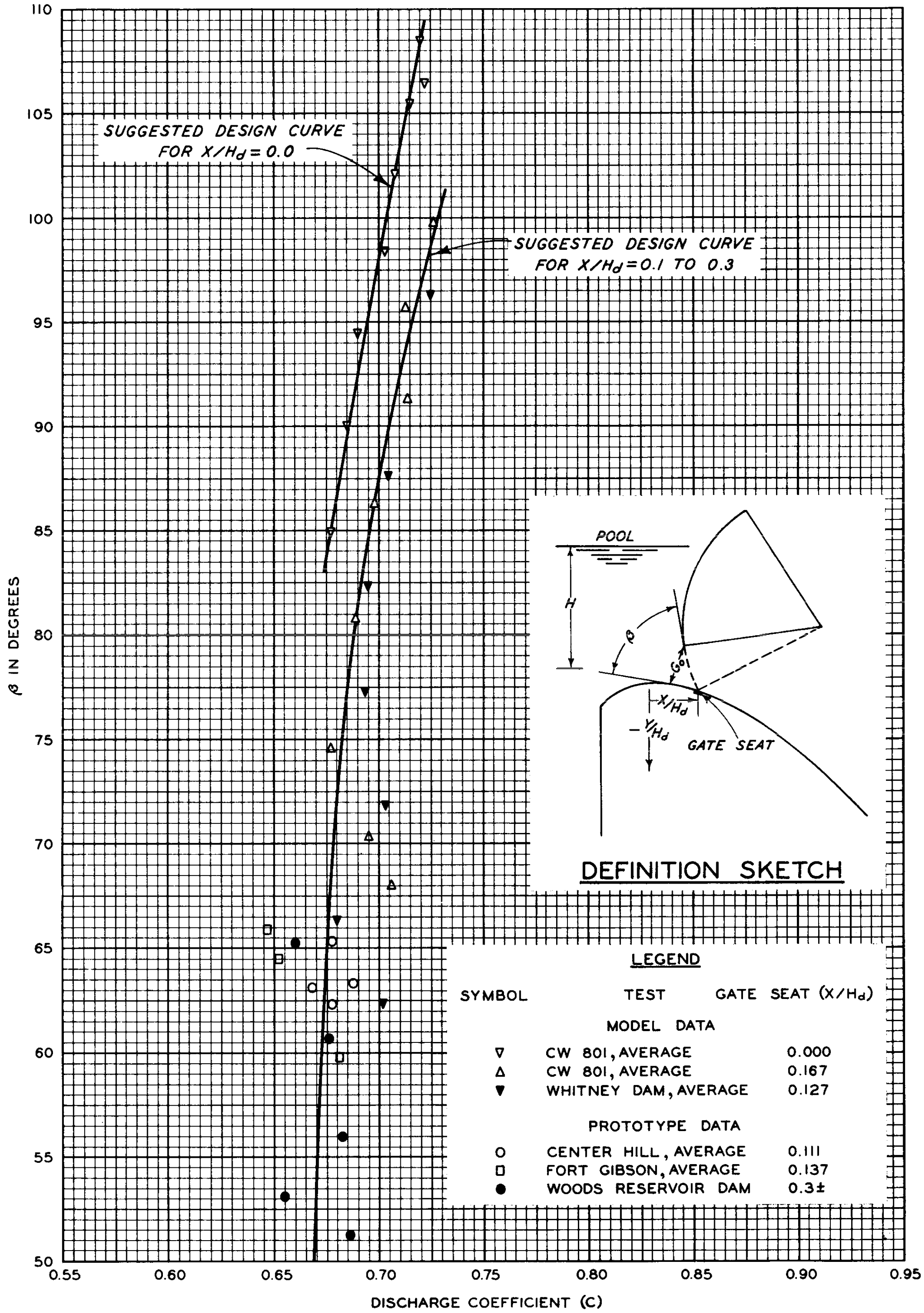
2. Discharge Coefficients. Chart 311-1 shows a plot of average discharge coefficients computed from model and prototype data for several crest shapes and tainter gate designs for nonsubmerged flow. Data shown are based principally on tests with three or more bays in operation. Discharge coefficients for a single bay would be lower because of side contractions although data are not presently available to evaluate this factor. On this chart, the discharge coefficient (C) is plotted as a function of the angle (β) formed by the tangent to the gate lip and the tangent to the crest curve at the nearest point of the crest curve. The net gate opening is considered to be the shortest distance from the gate lip to the crest curve. The angle is a function of the major geometric factors affecting the flow lines of the orifice discharge. One suggested design curve applies to tainter gates having gate seats located downstream from the crest axis. The other suggested design curve is based on tests with the gate seat located on the axis and indicates the effects of the masonry shape upstream from the crest axis.

3. Computation. Computation of discharge through a tainter gate mounted on a spillway crest is considerably complicated by the geometry involved in determining the net gate opening to be used in the orifice formula. The problem is simplified by fitting circular arcs to the crest

curve used in the design of spillways. Chart 311-2 illustrates the necessary computations to obtain the net gate opening and the angle β described in paragraph 2, for tainter gates mounted on spillway crests shaped to $X^{1.85} = -2 H_d^{0.85} Y$. All factors are expressed in terms of the design head (H_d). The method shown is applicable to other crest shapes. However, the accompanying design aids, Charts 311-3 and 311-4, apply only to standard crests.

4. To initiate the computations, Y_L/H_d values of the gate lip are assumed and corresponding values of X_L/H_d are computed (columns 1 to 6, Chart 311-2). These coordinates are then located on Chart 311-3 to determine the characteristics of a substitute arc. The substitute arc is then used to compute the net gate opening (columns 7 to 14). The point of intersection of the masonry line by the gate opening is determined by similar triangles (columns 14, 15, and 16). Design aid Chart 311-4 can be used to determine the Y_c/H_d coordinate of the gate opening and masonry line intersection (column 17), and also the slope of the masonry line (columns 18 and 19) which in turn combines with the slope of the gate lip tangent to form the angle β (column 20). If graphical methods are preferred to analytical methods, a large-scale layout will enable the head, net gate opening, and the angle β to be scaled so that the discharge can be computed with fair accuracy.

5. Chart 311-5 is a sample computation of the steps involved in the development of a rating curve for a partially open tainter gate. The final computations are dimensional and are believed accurate to within $\pm$ 2 per cent, for gate opening-head ratios (G_o/H) less than 0.6.



FORMULA

$$Q = CG_oB\sqrt{2gH}$$

WHERE:

G_o = NET GATE OPENING
B = GATE WIDTH
H = HEAD TO CENTER OF GATE OPENING

TAINTER GATES ON SPILLWAY CRESTS

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 311-1

WES 3-56

JOB CW804 PROJECT JOHN DOE DAM
SUBJECT SPILLWAY DISCHARGE
COMPUTED BY AAMG DATE 8-24-54
CHECKED BY HAB DATE 8-26-54

COMPUTATION SHEET

GATE OPENINGS AND ANGLE β

GIVEN

DESIGN HEAD (H_d) = 37.0 FT.
RADIUS OF GATE (R_G) = $0.831 H_d$.
TRUNNION COORDINATES (X_T, Y_T).
$X_T = 0.907 H_d$, $Y_T = 0.324 H_d$.

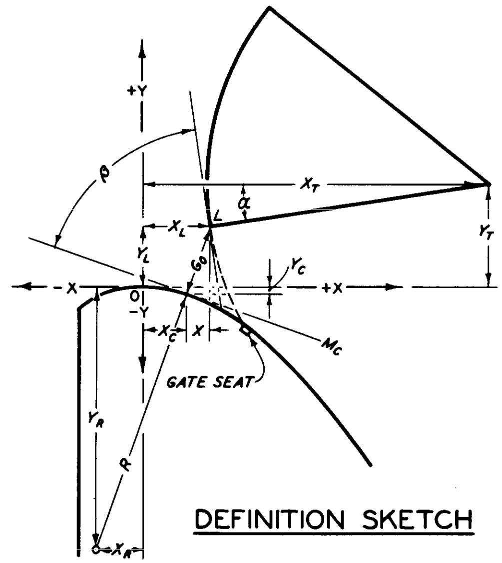
DEFINITIONS

GATE LIP COORDINATES (X_L, Y_L).
SPILLWAY CREST COORDINATES (X_C, Y_C).
SLOPE OF TANGENT TO CREST (M_C), NEGATIVE WHEN DOWNSTREAM FROM CREST.
SHORTEST DISTANCE FROM GATE LIP TO CREST (G_0).

DEFINITIONS (CONT)

α IS THE ANGLE BETWEEN A LINE CONNECTING THE GATE LIP AND THE TRUNNION CENTER, AND A HORIZONTAL LINE THROUGH THE TRUNNION, CONSIDERED POSITIVE AND NEGATIVE WHEN THE GATE LIP IS ABOVE AND BELOW THE TRUNNION, RESPECTIVELY.

NOTE: ALL DIMENSIONS USED IN COMPUTATIONS ARE IN TERMS OF DESIGN HEAD (H_d).



DEFINITION SKETCH

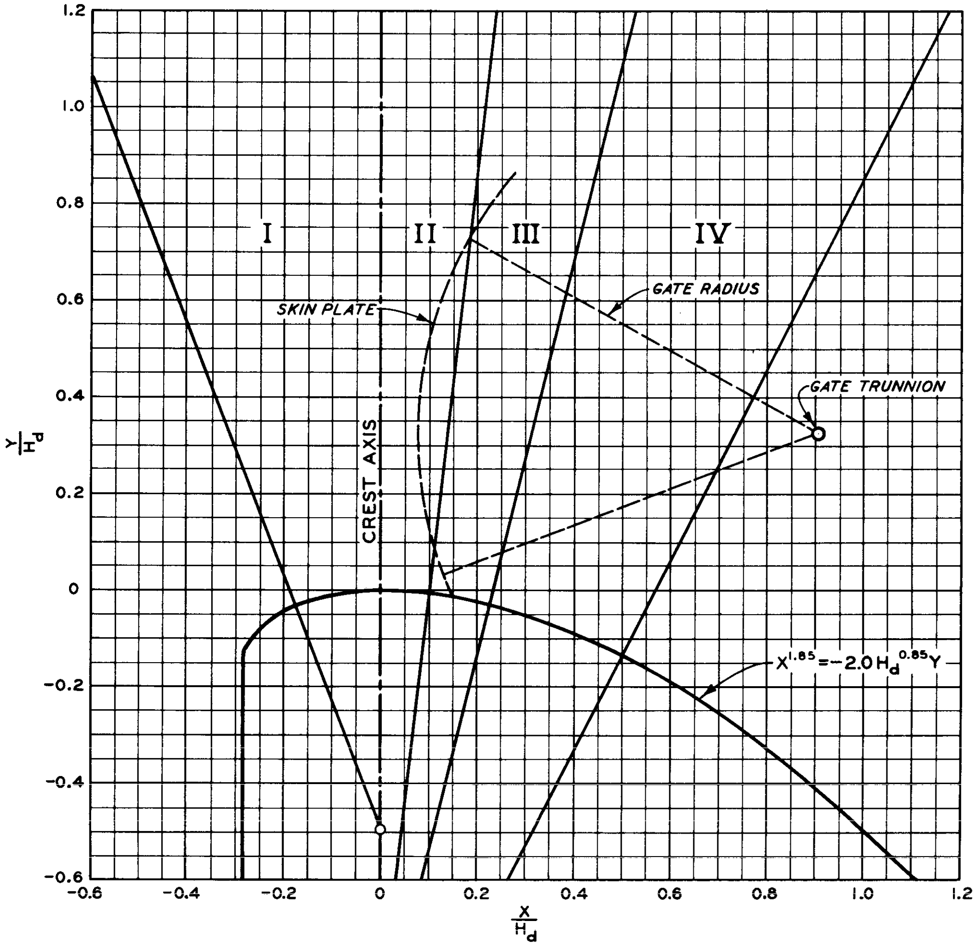
(1) Y_L	(2) Y_T-Y_L	(3) SIN α (2) ÷ R_G	(4) α DEGREES	(5) R_GCOSα 0.831COSα	(6) X_L X_T−(5)	(7) FROM CHART 311-3 CLASS	(8) X_R	(9) Y_R	(10) R	(11) X_L-X_R (6)−(8)	(12) Y_L-Y_R (1)−(9)	(13) $R+G_0$ $[(11)^2+(12)^2]^{1/2}$	(14) G_0 (13)−(10)	(15) X $\frac{(11)\times(14)}{(13)}$	(16) X_C (6)−(15)	(17) FROM CHART 311-4 Y_C	(18) M_C	(19) $TAN^{-1}M_C$ DEGREES	(20) β $90°+TAN^{-1}M_C+\alpha$ $90°+(19)+(4)$ DEGREES
0.100	0.224	0.270	−15.67	0.800	0.107	II	−0.050	−1.329	1.330	0.157	1.429	1.437	0.107	0.012	0.095	−0.0065	−0.125	−7.13	67.20
0.200	0.124	0.149	−8.57	0.821	0.086	II	−0.050	−1.329	1.330	0.136	1.529	1.535	0.205	0.018	0.068	−0.0035	−0.094	−5.35	76.06
0.300	0.024	0.029	−1.66	0.830	0.077	II	−0.050	−1.329	1.330	0.127	1.629	1.634	0.304	0.024	0.053	−0.0022	−0.076	−4.36	83.98
0.400	−0.076	−0.091	+5.22	0.829	0.078	II	−0.050	−1.329	1.330	0.128	1.729	1.733	0.403	0.030	0.048	−0.0018	−0.070	−4.02	91.20

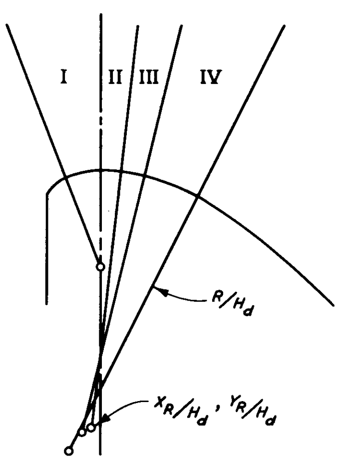
TAINTER GATES ON SPILLWAY CRESTS

SAMPLE GEOMETRIC COMPUTATION

HYDRAULIC DESIGN CHART 311-2

WES 3-56





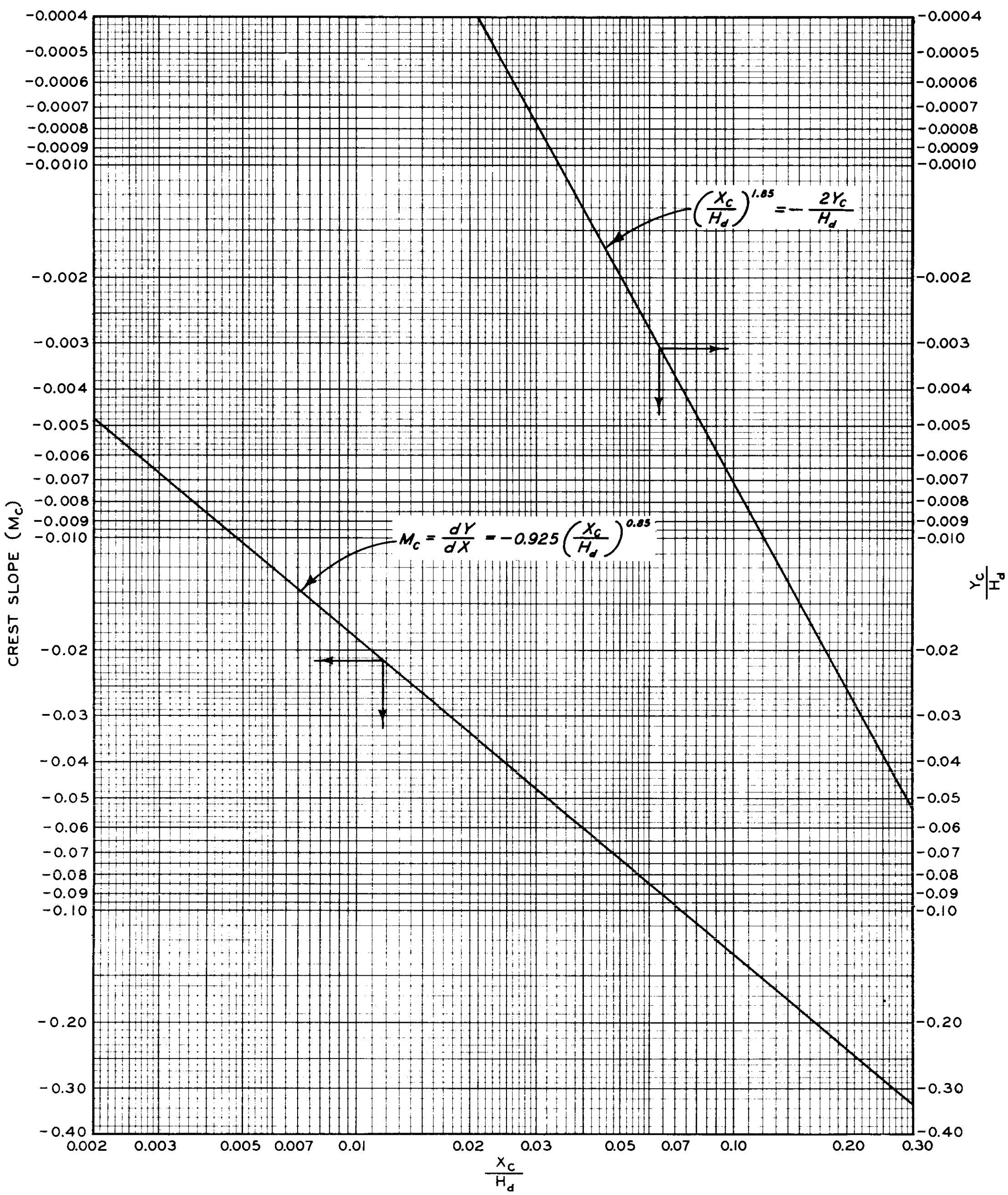
DEFINITION SKETCH

CLASS	R/H_d	X_R/H_d	Y_R/H_d
I	0.500	0.000	-0.500
II	1.330	-0.050	-1.329
III	1.359	-0.100	-1.351
IV	1.472	-0.164	-1.452

TAINTER GATES ON SPILLWAY CRESTS

GEOMETRIC FACTORS

HYDRAULIC DESIGN CHART 311-3



TAINTER GATES ON SPILLWAY CRESTS

CREST COORDINATES AND SLOPE FUNCTION

HYDRAULIC DESIGN CHART 311-4

WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB CW804 PROJECT JOHN DOE DAM SUBJECT SPILLWAY DISCHARGE

COMPUTATIONS COORDINATES FOR RATING CURVE (POOL VS DISCHARGE FOR VARIOUS GATE OPENINGS)

COMPUTED BY AAMc DATE 8-25-54 CHECKED BY RRW DATE 8-27-54

GIVEN

DESIGN HEAD (H_d) = 37.0 FT
GATE WIDTH (B) = 42.0 FT
CREST ELEV = 288.0 FT

FORMULAS

$Q = CG_0B\sqrt{2gH}$

H = POOL ELEV − 0.5[ELEV Y_L + ELEV Y_C]

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
β* DEGREES	C**	G_0/H_d*	G_0 FT	Y_L/H_d*	Y_L FT	Y_C/H_d*	Y_C FT	ELEV Y_L = 288 + Y_L FT	ELEV Y_C = 288 + Y_C FT	$\frac{(9)+(10)}{2}$	POOL FT	H (12) − (11) FT	$H^{1/2}$	Q CFS
67.20	0.676	0.107	3.96	0.100	3.70	−0.0065	−0.24	291.70	287.76	289.73	300 315 325	10.27 25.27 35.27	3.20 5.03 5.94	2,900 4,500 5,400
76.06	0.683	0.205	7.59	0.200	7.40	−0.0035	−0.13	295.40	287.87	291.64	310 315 325	18.36 23.36 33.36	4.28 4.83 5.78	7,500 8,400 10,100
83.98	0.694	0.304	11.25	0.300	11.10	−0.0022	−0.08	299.10	287.92	293.51	310 315 325	16.49 21.49 31.49	4.06 4.64 5.61	10,700 12,200 14,800
91.20	0.707	0.403	14.91	0.400	14.80	−0.0018	−0.07	302.80	287.93	295.37	315 320 325	19.63 24.63 29.63	4.43 4.96 5.44	15,800 17,600 19,300

* FROM HYDRAULIC DESIGN CHART 311-2
** FROM HYDRAULIC DESIGN CHART 311-1

TAINTER GATES ON SPILLWAY CRESTS

SAMPLE DISCHARGE COMPUTATIONS

HYDRAULIC DESIGN CHART 311-5

WES 3-56

HYDRAULIC DESIGN CRITERIA

SHEETS 311-6 AND 311-6/1

CREST PRESSURES

1. General. Pressures on standard spillways with partly open tainter gates are principally affected by the gate opening, gate geometry, and head on the gate. The effects of gate radius and trunnion elevation can be generally neglected within the limits of practical design.

2. Background. A laboratory study of the effects of gate seat location on pressures for standard shaped spillway crests (HDC 111-1 to 111-2/1) was made at WES[1] prior to 1948. A design head of 0.75 ft was used. The results of an extensive study by Lemos[2] of all geometric variables including gate seat locations upstream and downstream of the crest were published in 1965. A design head of 0.5 ft was used in this study. Comparable model[3] and prototype[4] data are also available.

3. Design Criteria. Dimensionless crest pressure profiles for small, medium, and large gate openings for the design head and 1.33 times the design head are given in HDC 311-6 and 311-6/1. The data are for gate seat locations of from $0.0H_d$ to $0.6H_d$ downstream of the crest. The study by Lemos[2] included gate seat locations from $-0.2H_d$ upstream to $0.6H_d$ downstream of the crest, gate radii of 1.0 and $1.25H_d$, trunnion elevations of from 0.2 to $1.0H_d$ above the crest, and heads of 1.0 and 1.25 H_d. Lemos' results indicate that the minor relative differences in gate radii, trunnion elevations, and gate openings of the experimental data shown on charts 311-6 and 311-6/1 should have negligible effect on crest pressures estimated from the charts. The Chief Joseph[3] and Altus[4] model curves were interpolated from observed data.

4. Application. The data given in the charts should be adequate for estimating crest pressures to be expected under normal design and operating conditions. When unusual design or operating conditions are encountered, the extensive work of Lemos can be used as a guide in estimating pressure conditions to be expected.

5. The data presented in charts 311-6 and 311-6/1 show that crest pressures resulting from normal design and operation practices are not controlling design factors. For partial gate openings the expected minimum crest pressures may range from about $-0.1H_d$ for pools at design head to about $-0.2H_d$ for heads approximating $1.3H_d$. Gated spillways are presently being built with 50-ft design heads; so for an underdesigned crest, the minimum pressure to be expected with gate control would be about -10 ft of water. This pressure would increase to -5 ft if design head was the maximum operating head. Pressures of these magnitudes should be free of cavitation. Periodic surges upstream of partially open tainter gates have been observed for certain combinations of head and gate width. Criteria for

surge prevention are given in ETL 1110-2-51.[5]

6. The pressure profiles in charts 331-6 and 311-6/1 can be used to estimate crest pressures for the design head for various gate openings and gate seat locations. The general absence of excessive negative pressures is noteworthy. Structural economy should no doubt have a strong influence on the selection of the gate seat location.

7. References.

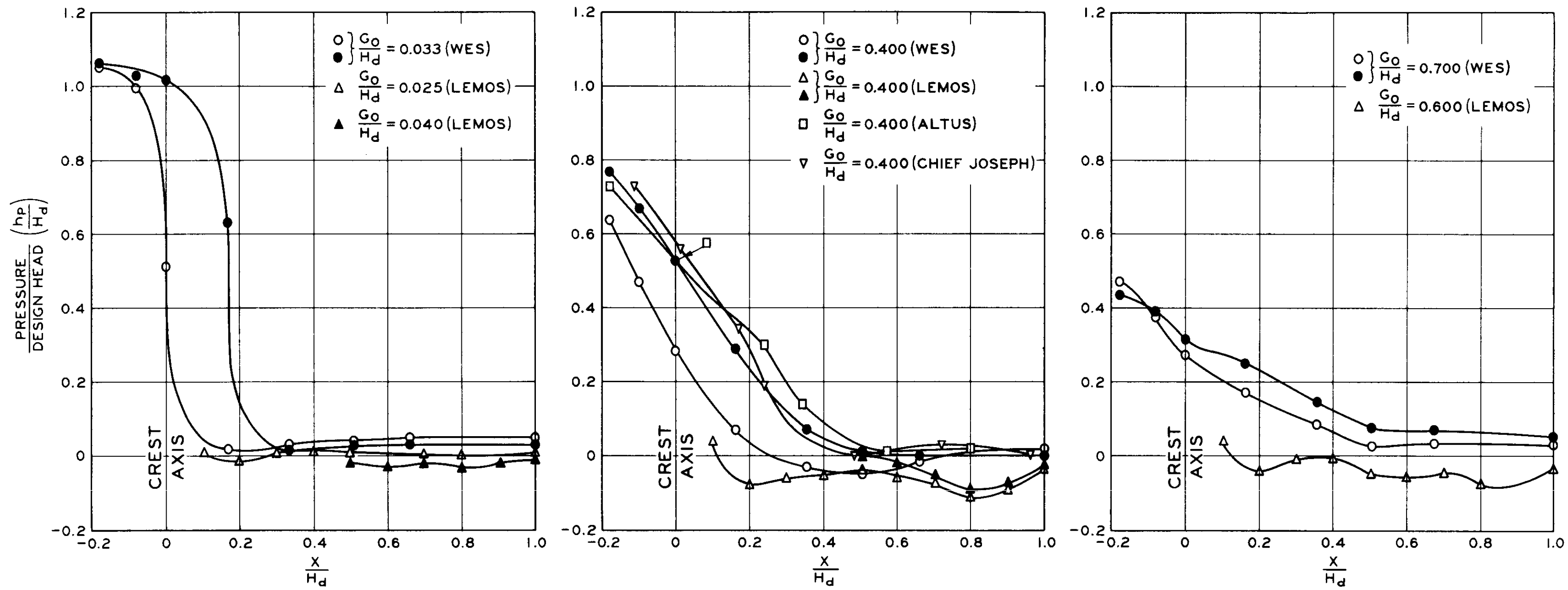
(1) U. S. Army Engineer Waterways Experiment Station, CE, General Spillway Tests (CW 801). Unpublished data.

(2) National Laboratory of Civil Engineering, Department of Hydraulics, Ministry of Public Works, Instability of the Boundary Layer - Its Effects Upon the Concept of Spillways of Dams, by F. O. Lemos. Proceedings 62/43, Lisbon, Portugal, 1965. WES Translation No. 71-3 by Jan C. Van Tienhoven, August 1971.

(3) U. S. Army Engineer Waterways Experiment Station, CE, Prototype Spillway Crest Pressures, Chief Joseph Dam, Columbia River, Washington. Miscellaneous Paper No. 2-266, Vicksburg, Miss., April 1958.

(4) Rhone, T. J., "Problems concerning use of low head radial gates." Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division, paper 1935, vol 85, No. HY2 (February 1959).

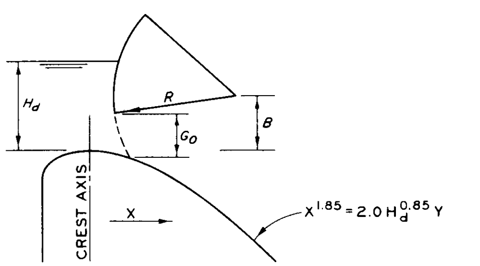
(5) U. S. Army, Office, Chief of Engineers, Engineering and Design; Design Criteria for Tainter Gate Controlled Spillways. Engineer Technical Letter No. 1110-2-51, Washington, D. C., 22 August 1968.



LEGEND

SYMBOL	TEST	GATE SEAT (X/H_d)	R/H_d	B/H_d
○———○	CW 801 (M)	0.000	1.27	0.385
●———●	CW 801 (M)	0.167	1.27	0.367
△———△	LEMOS (M)	0.000	1.25	0.560
▲———▲	LEMOS (M)	0.400	1.25	0.520
▽———▽	CHIEF JOSEPH (P) (INTERPOLATED)	0.258	1.00	0.444
□———□	ALTUS (M) (INTERPOLATED)	0.342	1.27	0.500

(M) MODEL
(P) PROTOTYPE



TAINTER GATES ON SPILLWAY CRESTS

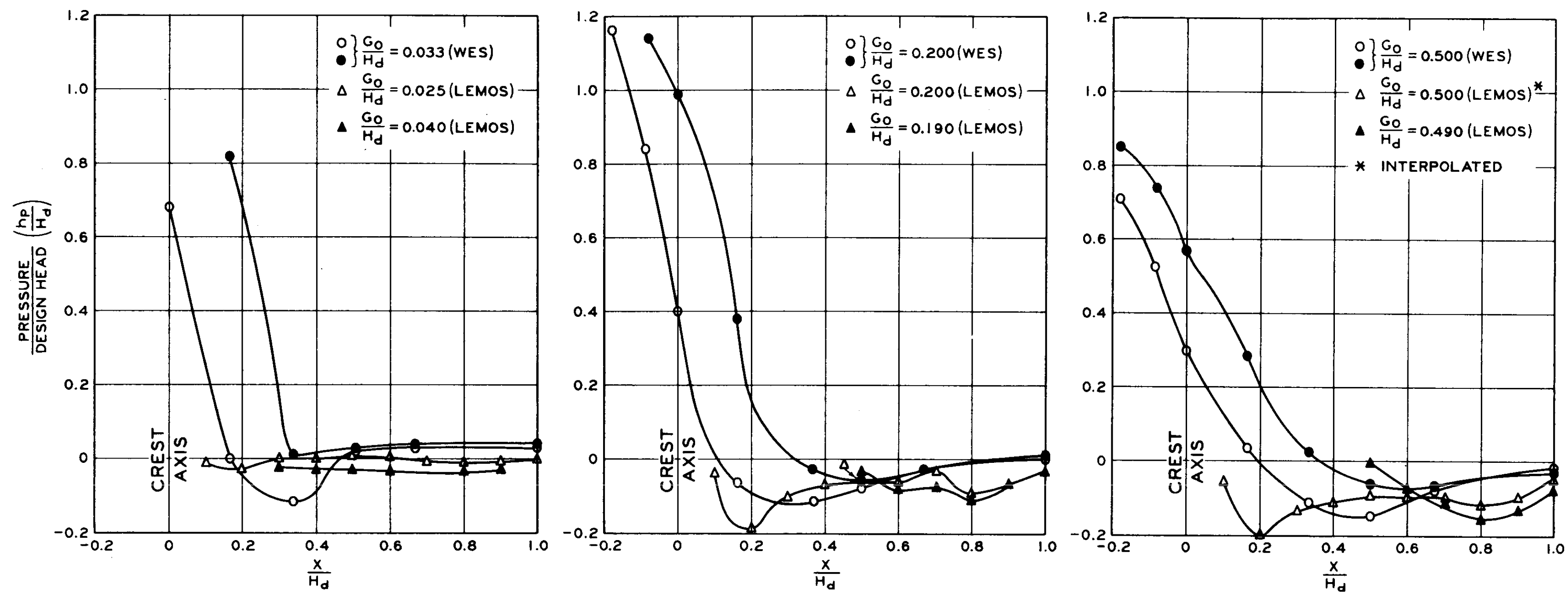
EFFECT OF GATE SEAT LOCATION ON CREST PRESSURES FOR $H = 1.00\,H_d$

HYDRAULIC DESIGN CHART 311-6

REV 7-71

WES 8-60

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



LEGEND

SYMBOL	TEST	GATE SEAT (X/H_d)	R/H_d	B/H_d	H/H_d
○—○	CW 801 (M)	0.000	1.27	0.385	1.33
●—●	CW 801 (M)	0.167	1.27	0.367	1.33
△—△	LEMOS (M)	0.000	1.25	0.560	1.25
▲—▲	LEMOS (M)	0.400	1.25	0.520	1.25

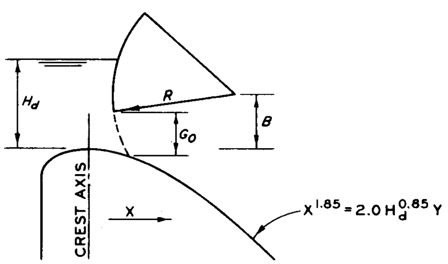


CHART 311-6/1

TAINTER GATES ON SPILLWAY CRESTS

EFFECT OF GATE SEAT LOCATION ON CREST PRESSURES FOR $H \sim 1.3 H_d$

HYDRAULIC DESIGN CHART 311-6/1

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

WES 7-71

SHEET 312

VERTICAL LIFT GATES ON SPILLWAYS

DISCHARGE COEFFICIENTS

1. Purpose. Vertical lift gates have been used on high-overflow-dam spillways. However, they are more commonly found on low-ogee-crest dams and navigation dams with low sills where reservoir pool control normally requires gate operation at partial openings. Hydraulic Design Chart 312 provides a method for computing discharge for partly opened, vertical lift gates.

2. Background. Discharge under high head, vertical lift gates can be computed using the standard orifice equation given in Sheets 311-1 to 311-5. The equation recommended by King[1] for discharge through low head orifices involves the head to the three-halves power. For flow under a low head gate, this equation can be expressed as

$$Q_G = C_{d1}\sqrt{2g}\, L\left(H_2^{3/2} - H_1^{3/2}\right) \qquad (1)$$

where Q_G is the gate controlled discharge, C_{d1} the discharge coefficient, g the acceleration of gravity, L the gate width, and H_1 and H_2 are the heads on the gate lip and gate seat, respectively.

3. A recent U. S. Army Engineers Waterways Experiment Station[2] study of discharge data from four laboratory investigations[3-6] failed to indicate correlation of discharge coefficients computed using equation 1 above or the equation given in Sheets 311-1 to 311-5. However, the concept of relating gate-controlled discharge to free discharge was developed in that study. The free discharge equation is

$$Q = C_d\sqrt{2g}\, LH^{3/2} \qquad (2)$$

where H is the head on the crest. The relation of controlled to free discharge was obtained by dividing equation 1 by equation 2.

$$\frac{Q_G}{Q} = \frac{C_{d1}}{C_d}\left(\frac{H_2^{3/2} - H_1^{3/2}}{H^{3/2}}\right) \qquad (3)$$

4. Analysis. The analysis of data taken from references 3 through 7 indicated reasonable correlation between free and controlled discharge. The results are shown in Chart 312. This study indicated that the relation C_{d1}/C_d varied slightly with the discharge ratio but could be assumed

as unity. Data from studies[6,7] with the gate seat located appreciably downstream from the crest showed good correlation with data for on-crest gate seat locations.

5. Application. Application of Chart 312 to the gate-discharge problem requires information on the head-discharge relation for free overflow for the crest under consideration. These data are usually available from spillway rating curves. Chart 312 should be a useful tool for the development of rating curves for vertical lift gates.

6. References.

(1) King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, revised by E. F. Brater, 4th ed. McGraw-Hill Book Co., Inc., New York, N. Y., 1954, pp 3-9.

(2) U. S. Army Engineer Waterways Experiment Station, CE, Discharge Rating Curves for Vertical Lift Gates on Spillway Crests, by R. H. Multer. Miscellaneous Paper No. 2-606, Vicksburg, Miss., October 1963.

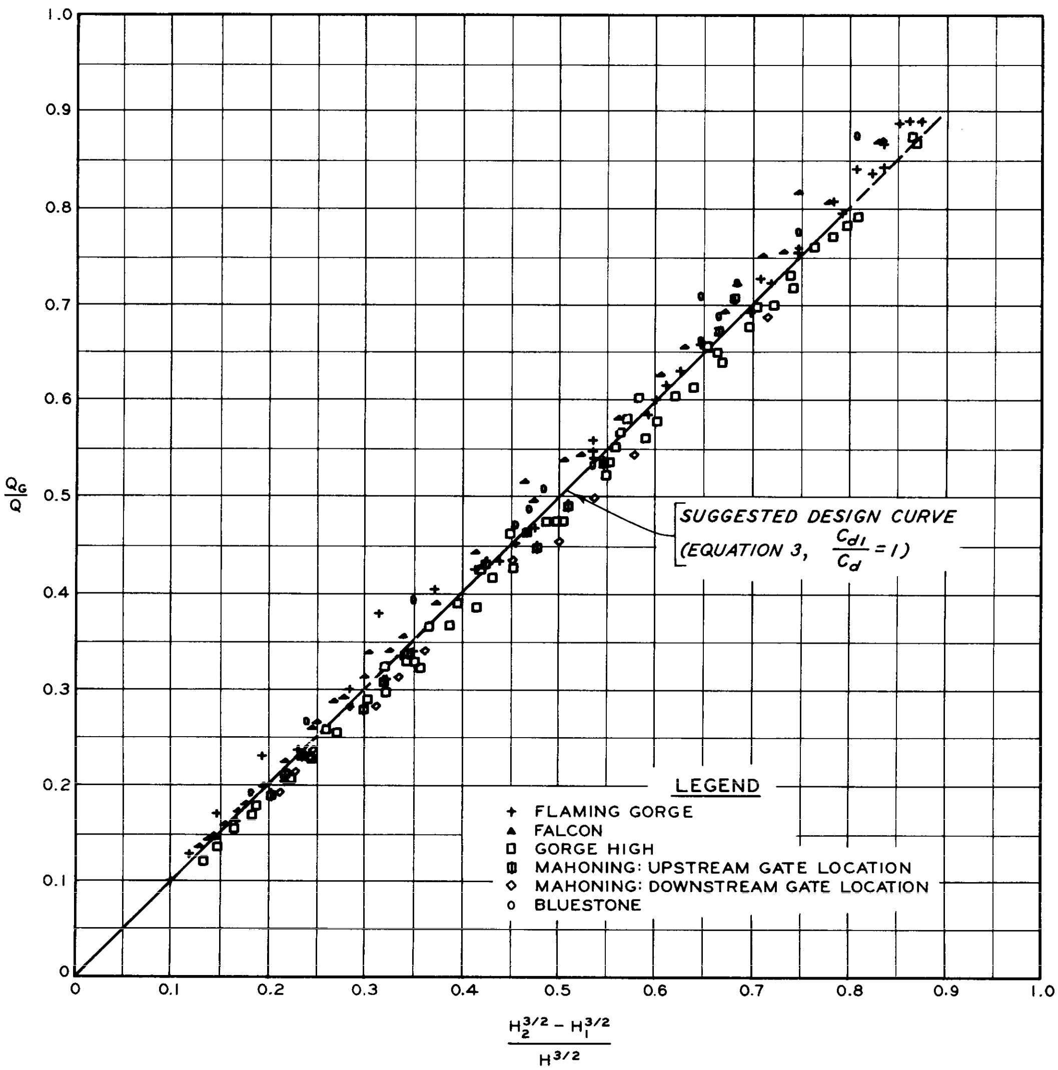
(3) U. S. Bureau of Reclamation, Hydraulic Model Studies of Falcon Dam, by A. S. Reinhart. Hydraulic Laboratory Report No. HYD-276, July 1950.

(4) __________, Hydraulic Model Studies of Gorge High Dam Spillway and Outlet Works, by W. E. Wagner. Hydraulic Laboratory Report No. HYD-403, September 1955.

(5) Carnegie Institute of Technology, Laboratory Tests on Hydraulic Models of Bluestone Dam, New River, Hinton, W. Va. Final report, prepared for the U. S. Army Engineer District, Huntington, W. Va., February 1937.

(6) Case School of Applied Science, A Report on Hydraulic Model Studies for the Spillway and Outlet Works of Mahoning Dam on Mahoning Creek, Near Punxsutawney, Pa., by G. E. Barnes. Prepared for the U. S. Army Engineer District, Pittsburgh, Pa., May 1938.

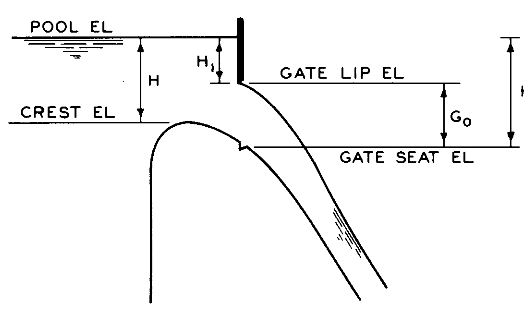
(7) U. S. Bureau of Reclamation, Hydraulic Model Studies of Flaming Gorge Dam Spillway and Outlet Works, by T. J. Rhone. Hydraulic Laboratory Report No. HYD-531, May 1964.



NOTE: Q = FREE-FLOW DISCHARGE AT HEAD H

Q_G = DISCHARGE AT HEAD H AND GATE OPENING G_O

$H_1 = H_2 - G_O$



DEFINITION SKETCH

VERTICAL LIFT GATES ON SPILLWAYS

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 312

REV 1-68 WES 1-66

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

HYDRAULIC DESIGN CRITERIA

SHEET 320-1

CONTROL GATES

DISCHARGE COEFFICIENTS

1. General. The accompanying Hydraulic Design Chart 320-1 represents test data on the discharge coefficients applicable to partial openings of both slide and tractor gates. The basic orifice equation is expressed as follows:

$$Q = C\ G_o\ B \sqrt{2gH'}$$

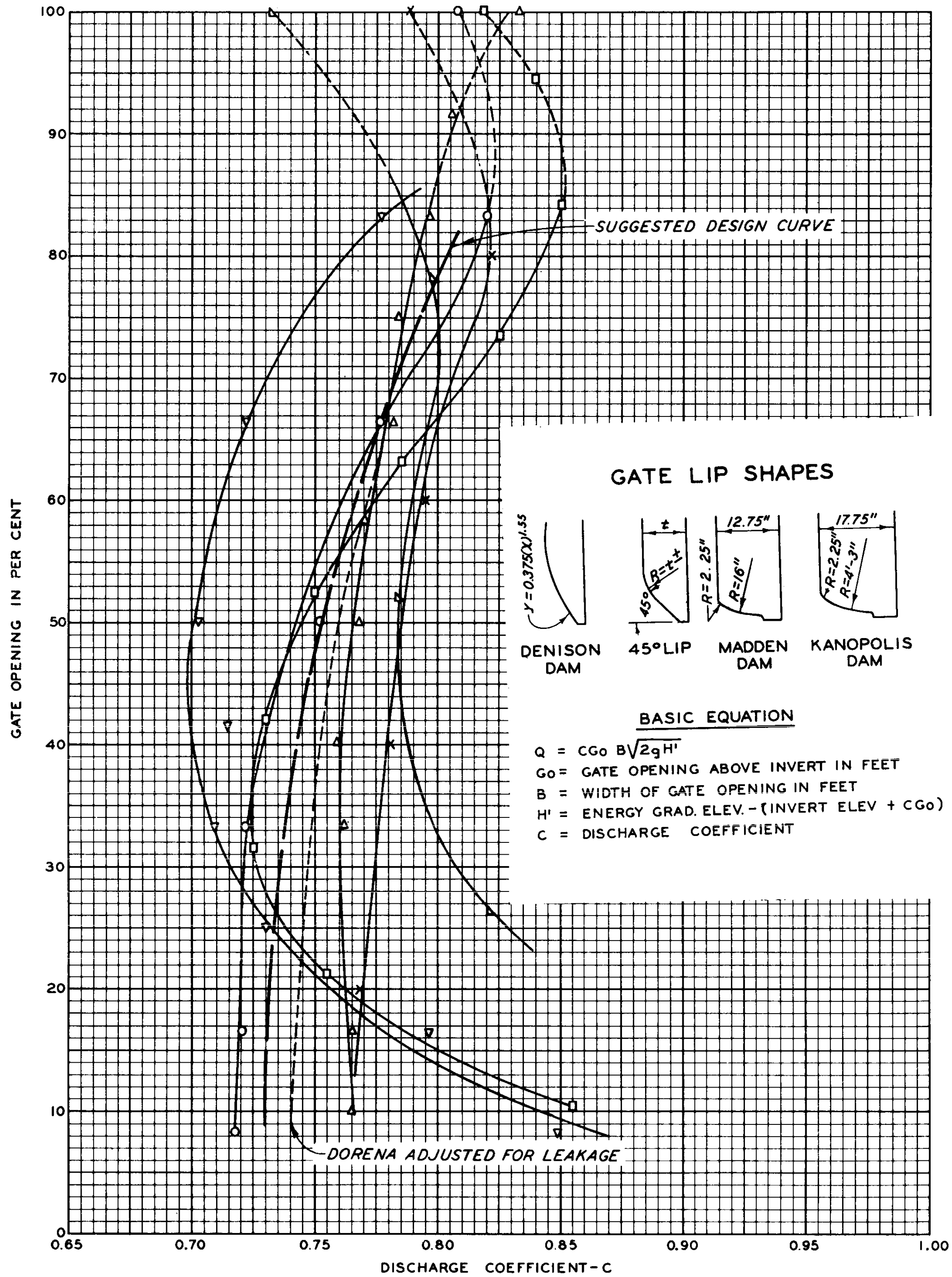
The coefficient C is actually a contraction coefficient if the gate is located near the tunnel entrance and the entrance energy loss is neglected. When the gate is located near the conduit entrance the head (H') is measured from the reservoir water surface to the top of the vena contracta. However, when the gate is located a considerable distance downstream of the conduit entrance, H' should be measured from the energy gradient just upstream of the gate to the top of the vena contracta because of appreciable losses upstream of the gate. The evaluation of H' requires successive approximation in the analysis of test data. However, the determination of H' in preparation of a rating curve can be easily accomplished by referring to the chart for C.

2. Discharge Coefficients. Discharge coefficients for tractor and slide gates are sensitive to the shape of the gate lip. Also, coefficients for small gate openings are materially affected by leakage over and around the gate. Chart 320-1 presents discharge coefficients determined from tests on model and prototype structures having various gate clearances and lip shapes. The points plotted on the 100 per cent opening are not affected by the gate but rather by friction and other loss factors in the conduit. For this reason the curves are shown by dashed lines above 85 per cent gate opening.

3. Suggested Criteria. Model and prototype tests prove that the 45° gate lip is hydraulically superior to other gate lip shapes. Therefore, the 45° gate lip has been recommended for high head structures. In the 1949 model tests leakage over the gate was reduced to a minimum. Correction of the Dorena Dam data for leakage results in a discharge coefficient curve that is in close agreement with the 1949 curve. The average of these two curves shown on Chart 320-1 is the suggested design curve. For small gate openings special allowances should be made by the designer for any expected excessive intake friction losses and gate leakage.

4. Values from the suggested design curve are tabulated below for the convenience of the designer.

Gate Opening, Per Cent	Discharge Coefficient
10	0.73
20	0.73
30	0.74
40	0.74
50	0.75
60	0.77
70	0.78
80	0.80



LEGEND

- ◭ FORT RANDALL MODEL } 45° LIP
- O WES MODEL TESTS CW 803 } 45° LIP
- Δ DORENA PROTOTYPE } 45° LIP
- □ DENISON PROTOTYPE
- X MADDEN PROTOTYPE
- ∇ KANOPOLIS PROTOTYPE

CONTROL GATES

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 320-1

WES 4-1-53

HYDRAULIC DESIGN CRITERIA

SHEETS 320-2 TO 320-2/3

VERTICAL LIFT GATES

HYDRAULIC AND GRAVITY FORCES

1. Purpose. The purpose of HDC's 320-2 to 320-2/2, which apply to the hydraulic forces on vertical lift gates, is to make the results of investigations of such forces available in a convenient nondimensional form. These charts are equally applicable to tractor gates and slide gates.

2. Definition. HDC 320-2 is included to simplify the definition of the hydraulic forces involved. For purposes of discussing buoyancy, a gate may be assumed to be a rectangular parallelepiped with the vertical axis coincident with the direction of gravity. If the body is completely inclosed, the buoyant force in still water is equal to the difference between the total pressure on top (downthrust) and the total pressure on the bottom (upthrust). For such an inclosed vertical body, water pressure on the upstream face has no vertical component of pressure.

3. Some engineers use the expression, the "wet weight" of a gate. This is simply the dry weight in air minus the buoyant force. If the body is cellular or lacks an upstream skin plate, the wet weight differs from that of a completely inclosed body. The gate shown in HDC 320-2 is an inclosed body and is further considered to have no horizontal projections such as gate seals.

4. The unit pressure on top of the gate, or downthrust, is dependent on the head of water in the gate well or the pressure head in the bonnet. This head in turn depends on the relation of the pressure difference across the gap and the area of the upstream gap coupled to the pressure differences and area of the downstream gap. Actually, the flow across the top of the gate has a hydrodynamic effect; but, for the purpose of these charts, this effect is not considered important.

5. The hydrodynamic effect of water flowing past the bottom of the gate is substantial. A reduction of pressure on the bottom from the theoretical static head is generally called "downpull," which may be viewed either as a reduction in upthrust or a reduction in buoyancy. Downpull is dependent upon the geometry of the gate bottom. HDC's 320-2 to 320-2/3 are concerned principally with the 45-degree gate bottom, for which experimental data are presented.

6. Vertical Stability. The gate well can be sucked completely dry of water with certain combinations of upstream and downstream gap areas between the gate and the roof of the conduit. If the upthrust then exceeds the weight of the gate, the entire body of the gate will be thrust

vertically upward. The experimental data on upthrust are of value in checking the design for such a possibility. However, discharge coefficients for the upstream and downstream gaps must be assumed to determine whether a gate opening exists that could cause a practically dry well.

7. Upthrust. Dimensionless plots of unit upthrust on the sloping bottom of four 45-degree gate-bottom designs are shown in HDC 320-2/1. The data sources are listed in paragraph 11. The data include both model and prototype pressure measurements. The Fort Randall gate has a downstream skin plate and downstream seals, and the 45-degree sloping gate bottom has an upstream skin plate. The Pine Flat and Norfork gates have upstream skin plates and downstream seals.

8. The upthrust force was computed from observed pressure data on the sloping gate bottom. These data were plotted on the horizontal plane of projection of the gate bottom. Pressure contours in feet of water were drawn, integrated, and divided by the area of projection between the conduit walls to determine the upthrust per unit area of cross section. The plots of data indicate that the conduit width-average gate thickness ratio is a factor in the magnitude of upthrust per unit area. The average gate thickness includes the gate bottom seal.

9. Pressure per unit area on top of the gate can be determined from HDC 320-2/2. The Fort Randall Dam data shown in the chart are based on field and model measurements of gate-well water-surface elevations. The Pine Flat and Norfork Dam data result from field measurements of bonnet pressures at these structures. Details of clearances between the gates and the gate recesses are also shown. The area of the top of the gate tc be used in computation of the downthrust should include the area of the gate within the gate slots, the area between the conduit walls and the area of the gate top seal.

10. Application. HDC 320-2/3 is a sample computation illustrating the use of HDC's 320-2/1 and 320-2/2 in the solution of a hydraulic and gravity force problem. In this computation the hydraulic force is based on the cross-sectional area of the gate between the conduit walls. In actual design, the effects of the top and bottom gate seals and the area of the gate within the gate slots should also be considered.

11. Data Sources.

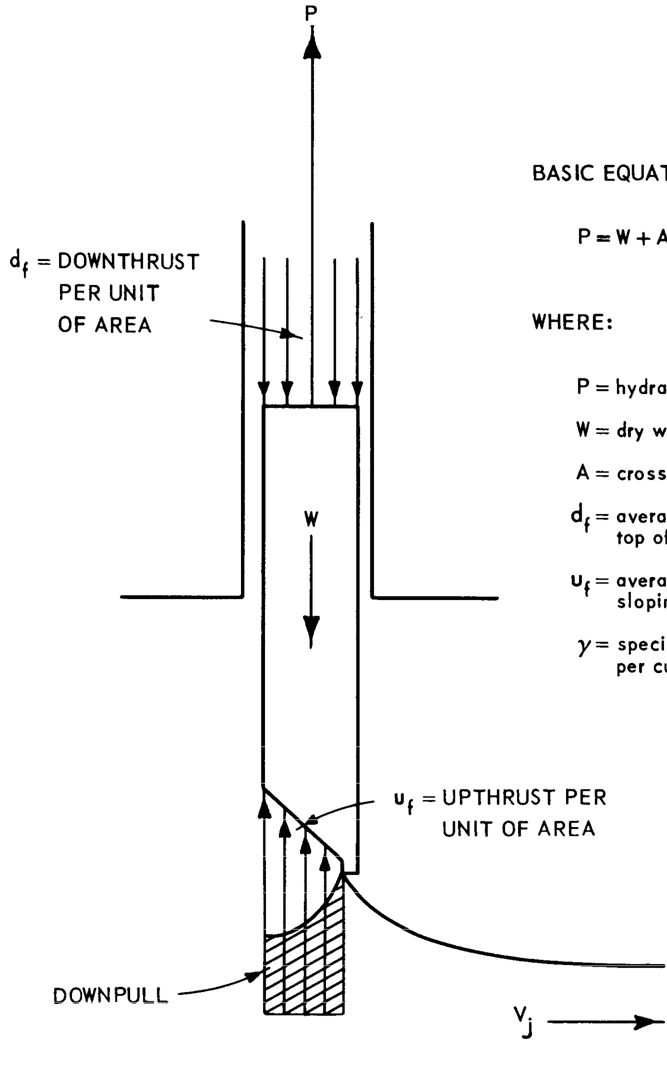
(1) U. S. Army Engineer Waterways Experiment Station, CE, Vibration, Pressure and Air-Demand Tests in Flood-Control Sluice, Pine Flat Dam, Kings River, California. Miscellaneous Paper No. 2-75, Vicksburg, Miss., February 1954, and subsequent unpublished test data.

(2) __________, Slide Gate Tests, Norfork Dam, North Fork River, Arkansas. Technical Memorandum No. 2-389, Vicksburg, Miss., July 1954.

(3) __________, Vibration and Pressure-Cell Tests, Flood-Control Intake

Gates, Fort Randall Dam, Missouri River, South Dakota. Technical Report No. 2-435, Vicksburg, Miss., June 1956.

(4) U. S. Army Engineer Waterways Experiment Station, CE, Spillway and Outlet Works, Fort Randall Dam, Missouri River, South Dakota. Technical Report No. 2-528, Vicksburg, Miss., October 1959.



BASIC EQUATION

$$P = W + A\,(d_f - u_f)\cdot\gamma$$

WHERE:

P = hydraulic and gravity forces in tons

W = dry weight of gate in tons

A = cross-sectional area of gate in sq ft

d_f = average downthrust per unit of area on top of gate in feet of water

u_f = average upthrust per unit of area on sloping bottom of gate in feet of water

γ = specific weight of water, 0.0312 ton per cu ft

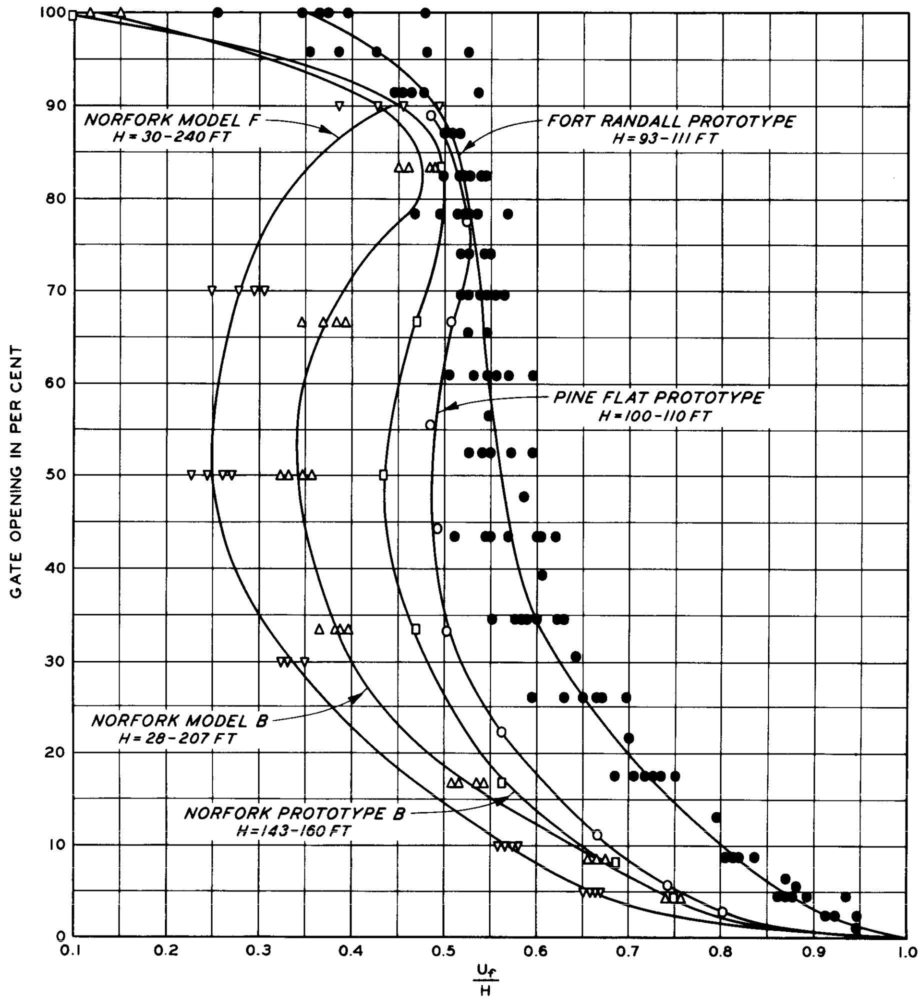
Note: Does not include factor for frictional and other mechanical forces.

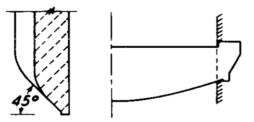
d_f = gate well water surface above conduit invert (H_w) minus sum of gate height (D) and gate opening (G_o).

VERTICAL LIFT GATES

HYDRAULIC AND GRAVITY FORCES DEFINITION AND APPLICATION

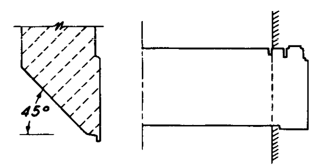
HYDRAULIC DESIGN CHART 320-2





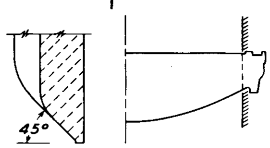
NORFORK TYPE B

$\frac{W}{T} = 5.00$



FORT RANDALL

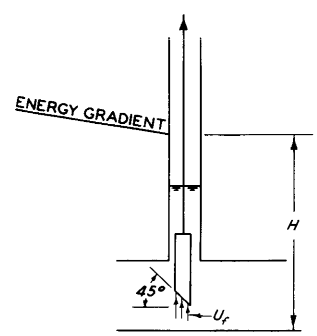
$\frac{W}{T} = 3.24$



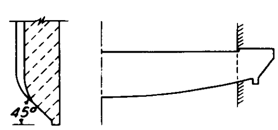
PINE FLAT

$\frac{W}{T} = 4.16$

NOTE: T = AVERAGE THICKNESS OF GATE - FT
W = WIDTH OF CONDUIT - FT



DEFINITION SKETCH



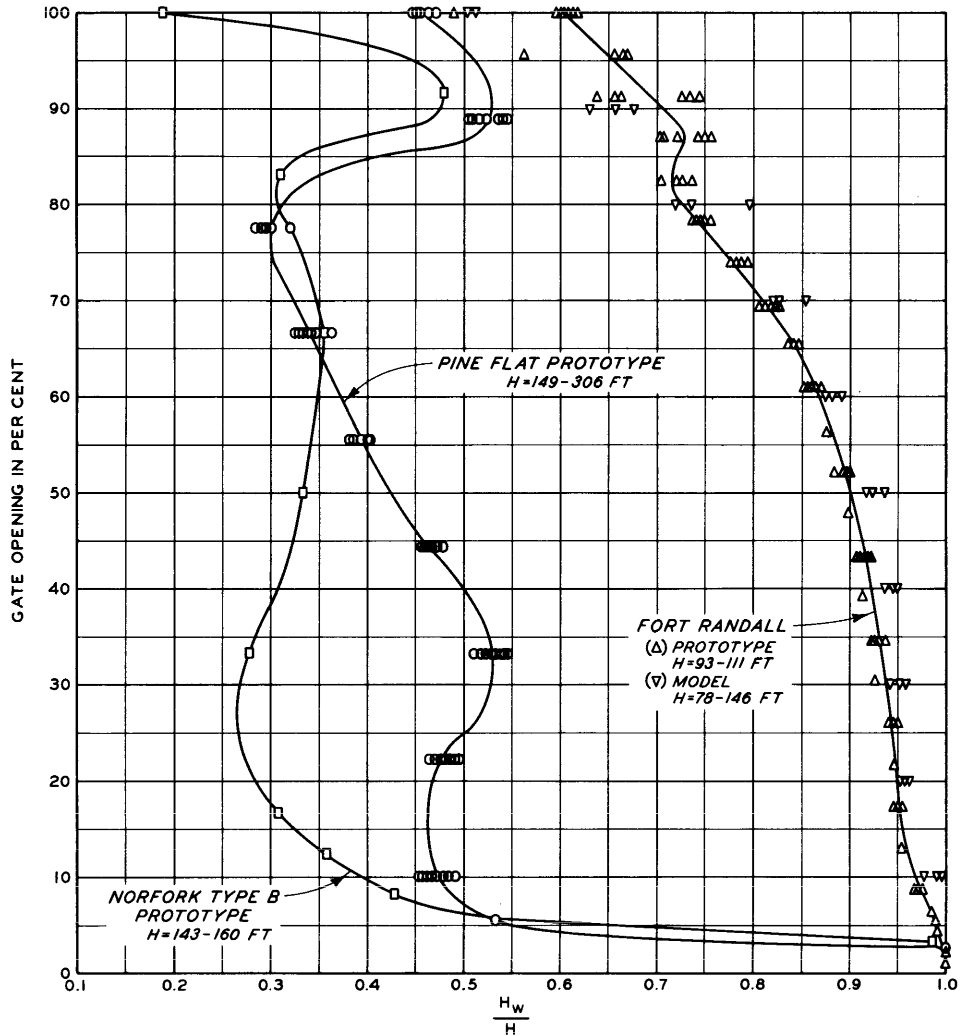
NORFORK TYPE F

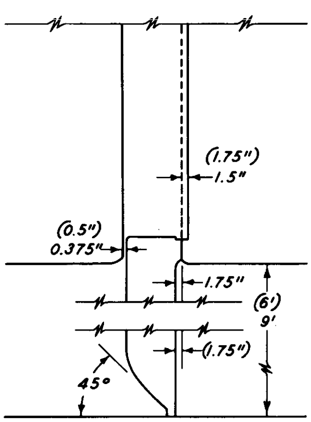
$\frac{W}{T} = 7.34$

VERTICAL LIFT GATES

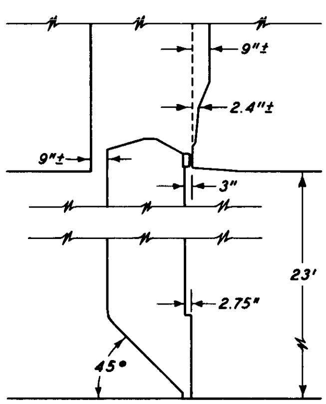
UPTHRUST ON GATE BOTTOM

HYDRAULIC DESIGN CHART 320-2/1

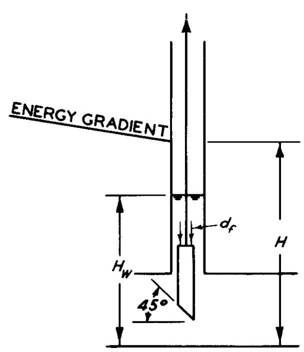




PINE FLAT
(NORFORK)



FORT RANDALL



DEFINITION SKETCH

VERTICAL LIFT GATES

GATE WELL WATER SURFACE

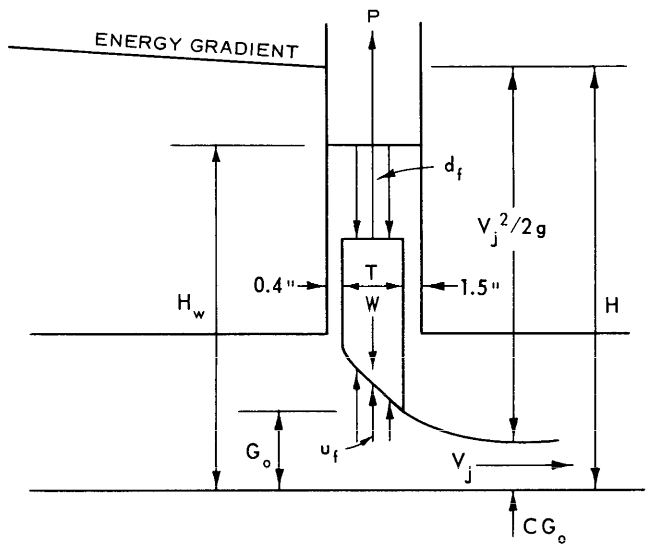
HYDRAULIC DESIGN CHART 320-2/2

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

JOB CW 804 PROJECT John Doe Dam SUBJECT Vertical Lift Gates
COMPUTATION Hydraulic and Gravity Forces
COMPUTED BY MBB DATE 4/10/61 CHECKED BY CWD DATE 4/20/61

GIVEN:

Gate - Pine Flat type (HDC 320-2/1)
Height (D) = 9.0
Width (B) = 5.0
Average thickness (T) = 1.2 ft
Upstream gate clearance = 0.4 in.
Downstream gate clearance = 1.5 in.
Dry weight (W) = 8 tons
Gate opening (G_o) = 3.0 ft
Discharge (Q) = 1200 cfs



DETERMINE:

1. Energy head above conduit invert (H)

   Gate opening (G_o) percent

   $$\frac{G_o}{D} \times 100 = \frac{3}{9} \times 100 = 33.3$$

   Gate coefficient (C) = 0.737 (HDC 320-1)

   Velocity of jet (V_j)

   $$\frac{Q}{CG_oB} = \frac{1200}{0.737 \times 3 \times 5} = 108.5 \text{ ft/sec}$$

   Velocity head of jet ($V_j^2/2g$)

   $$\frac{V_j^2}{2g} = \frac{(108.5)^2}{64.4} = 182.8 \text{ ft}$$

   Energy head above conduit invert

   $$H = CG_o + V_j^2/2g$$
   $$= (0.737 \times 3) + (182.8) = 185.0 \text{ ft}$$

2. Unit upthrust (u_f)

   For Pine Flat from HDC 320-2/1

   $$\frac{u_f}{H} = 0.51 \text{ for } G_o = 33.3 \text{ percent}$$
   $$u_f = 0.51\ (185.0) = 94.4 \text{ ft}$$

3. Unit downthrust (d_f)

   For Pine Flat from HDC 320-2/2

   Gate well water surface above conduit invert (H_w)

   $$\frac{H_w}{H} = 0.53 \text{ for } G_o = 33.3 \text{ percent}$$
   $$H_w = 0.53\ (185.0) = 98.0 \text{ ft}$$

   Unit downthrust

   $$d_f = H_w - (D + G_o) = 98.0 - (9 + 3)$$
   $$= 86.0 \text{ ft}$$

4. Hoist load (P) (HDC 320-2)

   $$P = W + A\ (d_f - u_f)\ \gamma$$
   $$= 8 + (5 \times 1.2)\ (86.0 - 94.4)\ 0.0312$$
   $$= 8 - 1.6 = 6.4 \text{ tons}$$

5. Repeat computations for other gate openings to develop gate hoist load curve.

Note:
1. The vertical load resulting from the friction between the gate and the gate guides has not been included in this computation.
2. In actual problems the difference between the projected areas of the top and bottom of the gate including seals and areas within the gate slots should be considered.

VERTICAL LIFT GATES
HYDRAULIC AND GRAVITY FORCES
SAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 320-2/3

HYDRAULIC DESIGN CRITERIA

SHEET 320-3

TAINTER GATES IN CONDUITS

DISCHARGE COEFFICIENTS

1. HDC 320-3 presents coefficient curves for tainter gates in conduits for use in the discharge equation:

$$Q = C\ G_o\ B \sqrt{2gH}$$

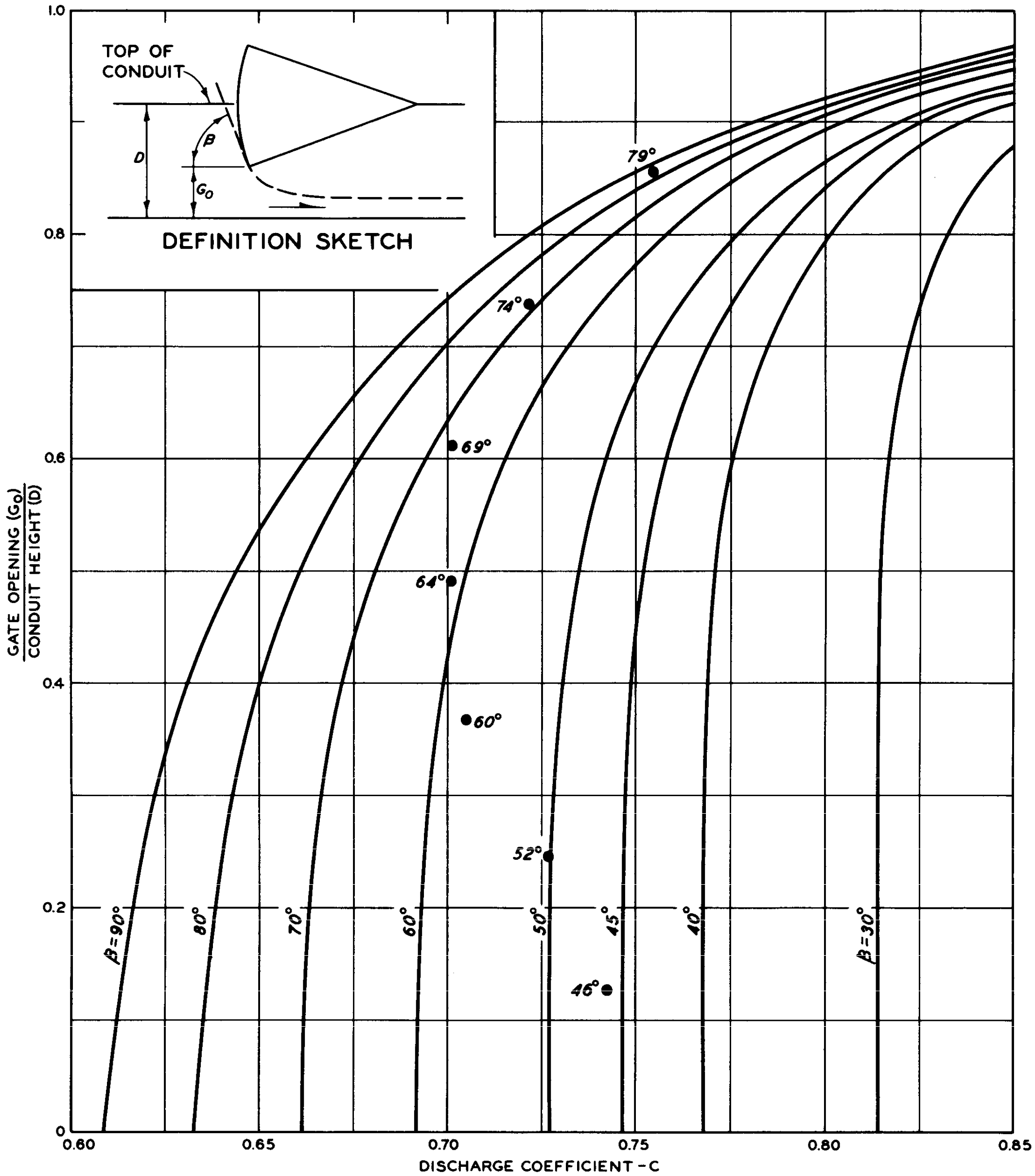
The coefficient C is actually a contraction coefficient when the head H is measured from the energy gradient just upstream from the gate to the top of the vena contracta downstream.

2. The curves shown in HDC 320-3 are based on an equation by R. von Mises* for the contraction coefficient for two-dimensional flow through slots. The solution of this equation requires successive approximation of the contraction coefficient. The computations were made on an electronic digital computer. The sketch shown in the chart is considered to be a half-section of the symmetrical slot condition investigated by Von Mises. The conduit invert represents the center line of his geometry and the roof one of the parallel approach boundaries. The tangent to the gate lip is assumed to be the sloping boundary from which the jet issues. The plotted data result from controlled tests on the Garrison tunnel model** in which leakage around or over the gate was negligible and discharge under the gate was carefully measured. The agreement between the curves and Garrison data indicates the applicability of the curves to tainter gates in conduits with straight inverts.

---

* Mises, R. von, "Berechnung von Ausfluss - und ueberfallzahlen (Computation of coefficients of out-flow and overfall)," Zeitschrift des Vereines deutscher Ingenieure, Band 61, Nr. 22 (2 June 1917), p 473.

** U. S. Army Engineer Waterways Experiment Station, CE, Outlet Works and Spillway for Garrison Dam, Missouri River, North Dakota, Technical Memorandum No. 2-431 (Vicksburg, Miss., March 1956).



BASIC EQUATION

$$Q = C G_0 B \sqrt{2gH}$$

WHERE :

Q =DISCHARGE–CFS
C =DISCHARGE COEFFICIENT
G_0=GATE OPENING–FT.
B =WIDTH OF GATE OPENING–FT.
H =ENERGY GRAD. ELEV. -(INVERT ELEV. + CG_0)

LEGEND

——— VON MISES
● GARRISON MODEL

TAINTER GATES IN CONDUITS

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 320-3

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

HYDRAULIC DESIGN CRITERIA

SHEETS 320-4 TO 320-7

TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENTS

1. Free discharge through a partially open tainter gate in an open channel can be computed using the equation:

$$Q = C_1 C_2 G_o B \sqrt{2gh}$$

The coefficient (C_1) depends on the vena contracta, the shape of which is a function of the gate opening (G_o), gate radius (R), trunnion height (a), and upstream depth (h) for gate sills at streambed elevations. When the gate sill is above streambed elevation, the coefficient also depends upon sill height (P) and sill length (L).

2. Hydraulic Design Charts 320-4 to 320-6 present discharge coefficients (C_1) for tainter gates with sills at streambed elevation. The insert graphs on the charts indicate adjustment factors (C_2) for raised sill conditions. Charts are included for a/R ratios of 0.1, 0.5, and 0.9. Coefficients for other a/R values can be obtained by interpolation between the charts. The coefficient is plotted in terms of the h/R ratio for G_o/R values of 0.05 to 0.5. The effect of G_o/h is inherent in the solution and is indicated by the limit-use curve $G_o/h = 0.8$.

3. The basic curves on Charts 320-4 to 320-6 were prepared from tests reported by Toch (3), Metzler (2), and Gentilini (1). The method of plotting was developed by Toch. Cross plots of the Toch, Metzler, and Gentilini data resulted in the interpolated curves. Good correlation of test results was obtained for the larger gate openings. Similar correlation was not obtained in all cases for the smaller gate openings. The Gentilini data for the smaller G_o/R ratios and their general correlation with Metzler's data resulted in the interpolated curves for G_o/R values of 0.05 and 0.1. The 0.2 curve is in close agreement with results reported by Toch. Interpolated coefficients from the C_1 curve indicate general agreement with experimental results to within ± 3 per cent.

4. Charts 320-4 to 320-6 also apply to raised sill design problems when the adjustment factor curve shown on the auxiliary graph is considered. The C_2 curve was developed from U. S. Army Corps of Engineers (4-7) studies and indicates the effects of the L/P ratio on the discharge coefficient. This adjustment results in reasonable agreement with experimental data. Sufficient information is not available to determine the effects, if any, of the parameter P/R.

5. Hydraulic Design Chart 320-7 is a sample computation sheet illustrating application of Charts 320-4 to 320-6.

6. References.

(1) Gentilini, B., "Flow under inclined or radial sluice gates - technical and experimental results." La Houille Blanche, vol 2 (1947), p 145. WES Translation No. 51-9 by Jan C. Van Tienhoven, November 1951.

(2) Metzler, D. E., A Model Study of Tainter Gate Operation. State University of Iowa Master's Thesis, August 1948.

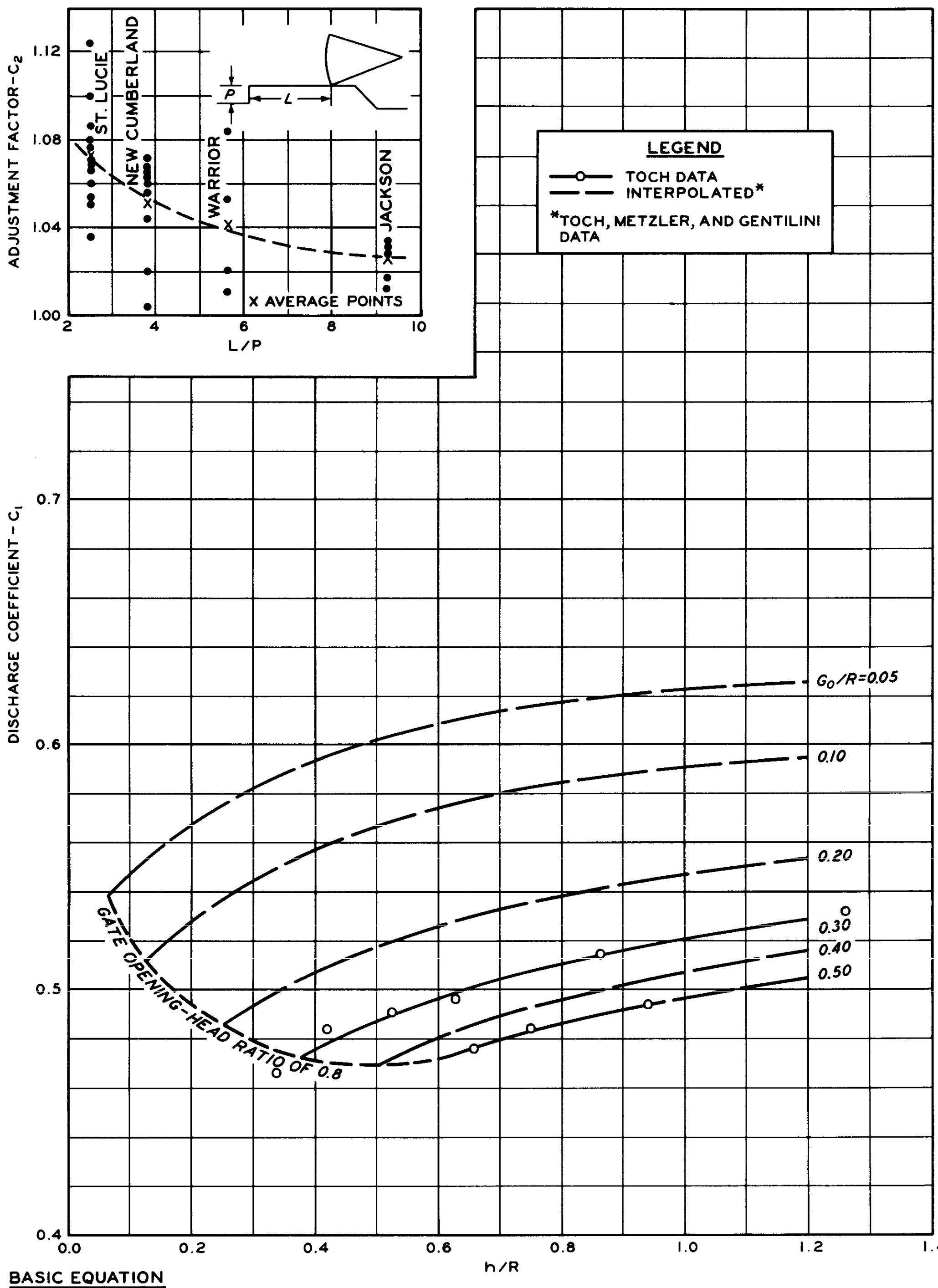
(3) Toch, A., The Effect of a Lip Angle Upon Flow Under a Tainter Gate. State University of Iowa Master's Thesis, February 1952.

(4) U. S. Army Engineer Waterways Experiment Station, CE, Model Study of the Spillway for New Lock and Dam No. 1, St. Lucie Canal, Florida. Technical Memorandum No. 153-1, Vicksburg, Miss., June 1939.

(5) __________, Spillway for New Cumberland Dam, Ohio River, West Virginia. Technical Memorandum No. 2-386, Vicksburg, Miss., July 1954.

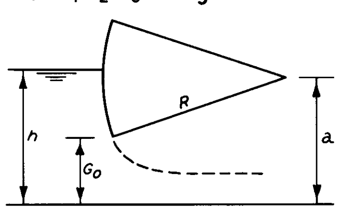
(6) __________, Stilling Basin for Warrior Dam, Warrior River, Alabama. Technical Report No. 2-485, Vicksburg, Miss., July 1958.

(7) __________, Spillways and Stilling Basins, Jackson Dam, Tombigbee River, Alabama. Technical Report No. 2-531, Vicksburg, Miss., January 1960.



BASIC EQUATION

$$Q = C_1 C_2 G_0 B \sqrt{2gh}$$



DEFINITION SKETCH

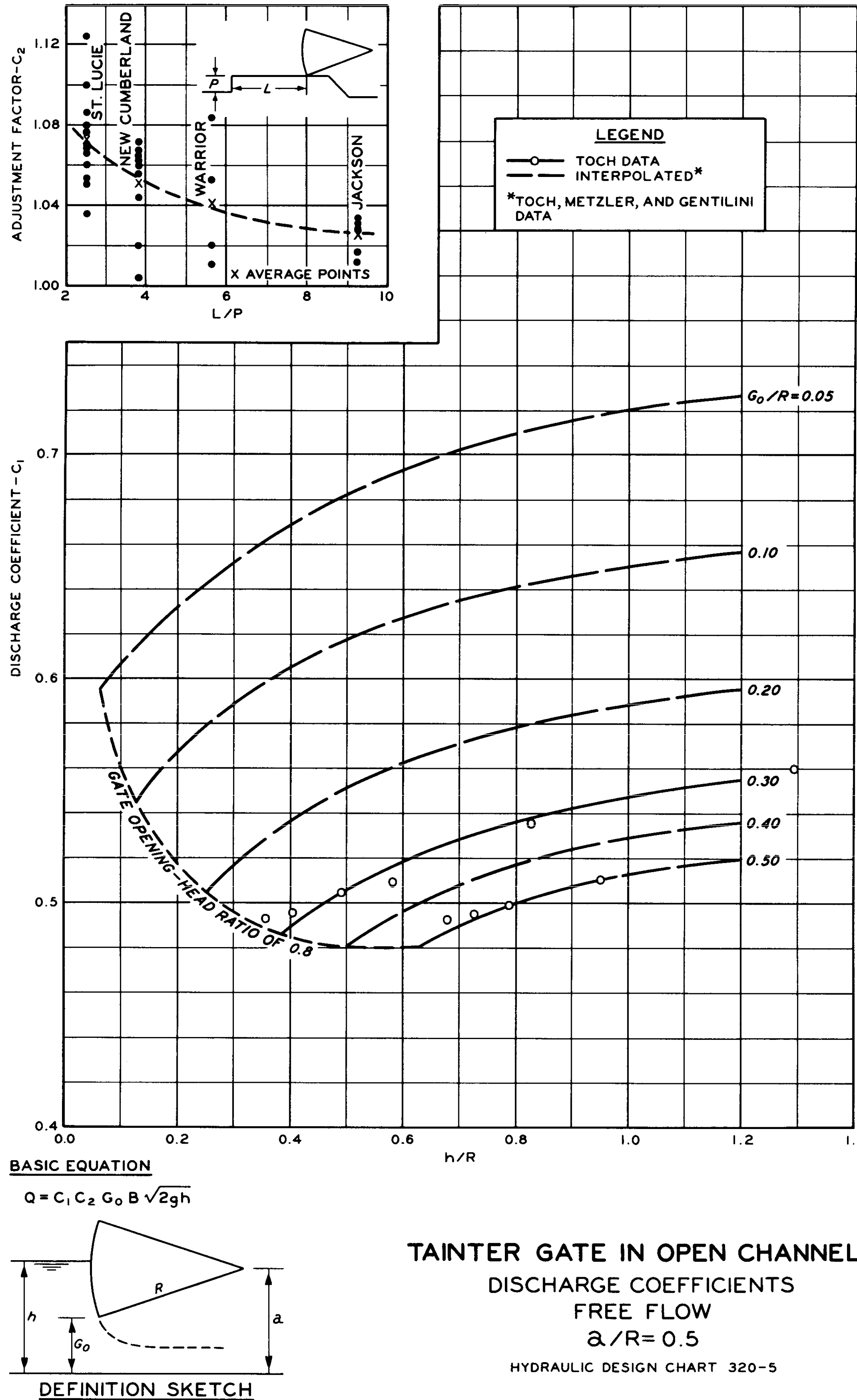
TAINTER GATE IN OPEN CHANNELS

DISCHARGE COEFFICIENTS
FREE FLOW
a/R = 0.1

HYDRAULIC DESIGN CHART 320-4

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

WES 8-60



BASIC EQUATION

$$Q = C_1 C_2 G_O B \sqrt{2gh}$$

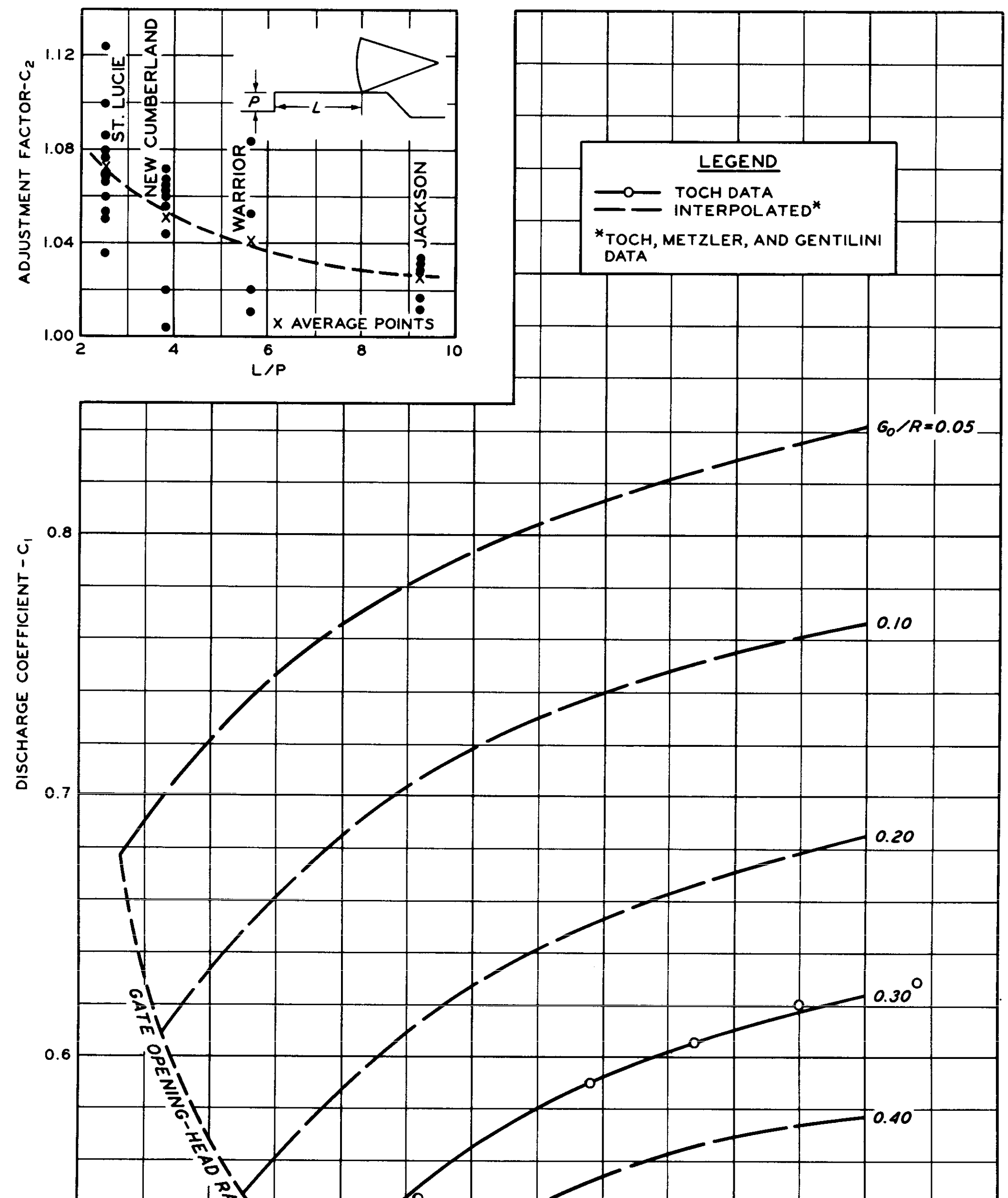
TAINTER GATE IN OPEN CHANNELS

DISCHARGE COEFFICIENTS
FREE FLOW
$a/R = 0.5$

HYDRAULIC DESIGN CHART 320-5

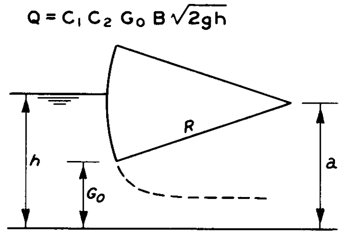
PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 8-60



BASIC EQUATION

$$Q = C_1 C_2 G_0 B \sqrt{2gh}$$



DEFINITION SKETCH

TAINTER GATE IN OPEN CHANNELS

DISCHARGE COEFFICIENTS
FREE FLOW
$a/R = 0.9$

HYDRAULIC DESIGN CHART 320-6

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

WES 8-60

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

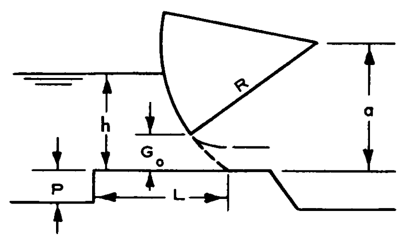
JOB CW 804 PROJECT John Doe River SUBJECT Tainter Gate in Open Channels

COMPUTATION Free Discharge for Gate Rating

COMPUTED BY MBB DATE 5/9/60 CHECKED BY RGC DATE 5/17/60

GIVEN:

Tainter gate installation as shown
Upstream depth $(h) = 15$ ft
Gate opening $(G_o) = 4$ ft
Gate radius $(R) = 25$ ft
Trunnion height $(a) = 20$ ft
Bay width $(B) = 60$ ft
Length – step to gate seat $(L) = 20$ ft
Height of step $(P) = 5$ ft



$Q = C_1 C_2 G_o B\sqrt{2gh}$

REQUIRED:

Free discharge for gate rating

COMPUTE:

1. Parameters

   $a/R = 0.8,\ h/R = 0.6,\ G_o/R = 0.16,\ L/P = 4$

2. Discharge coefficient (C_1) for unstepped condition for $a/R = 0.8$

   Chart 320-5 $(a/R = 0.5,\ h/R = 0.6,\ G_o/R = 0.16)$, $C_1 = 0.587$

   Chart 320-6 $(a/R = 0.9,\ h/R = 0.6,\ G_o/R = 0.16)$, $C_1 = 0.664$

   By interpolation for $a/R = 0.8$

   $$C_1 = 0.587 + \frac{0.8 - 0.5}{0.9 - 0.5}(0.664 - 0.587)$$
   $$= 0.645$$

3. Adjustment for stepped sill

   For $L/P = 4$

   Adjustment factor $(C_2) = 1.05$ (see chart insert)

   $C_1 C_2 = 0.645\ (1.05) = 0.678$

4. Discharge

   $$Q = C_1 C_2 G_o B\sqrt{2gh}$$
   $$= 0.678\ (4)\ \ (60)\sqrt{64.4 \times 15}$$
   $$= 5050 \text{ cfs}$$

TAINTER GATE IN OPEN CHANNELS
DISCHARGE COEFFICIENTS
FREE FLOW
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 320-7

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

HYDRAULIC DESIGN CRITERIA

SHEETS 320-8 AND 320-8/1

TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENTS

SUBMERGED FLOW

1. Tainter gates on low sills at navigation dams frequently operate at tailwater elevations resulting in submerged flow conditions. The discharge under the gate is controlled by the difference in the upper and lower pool elevations, the degree of sill submergence by the tailwater, the gate opening, and, to a lesser extent, the stilling basin apron elevation. Hydraulic Design Charts 320-8 and 320-8/1 present discharge coefficient data for computing flows under tainter gates on low sills operating under submerged conditions.

2. Basic Data. The U. S. Army Engineer Waterways Experiment Station (WES)[1] has developed the following equation for computing flows under gates on low sills with tailwater elevations greater than gate sill elevation.

$$Q = C_s L h_s \sqrt{2gh} \qquad (1)$$

where

Q = discharge, cfs

C_s = submerged flow discharge coefficient, a function of the sill submergence-gate opening ratio

L = bay width, ft

h_s = tailwater depth over sill, ft

g = acceleration, gravitational, ft per sec^2

h = total head differential pool to tailwater, ft (including approach velocity head)

Equation 1 results in good correlation of experimental data when C_s is plotted as a function of the submergence-gate opening ratio (h_s/G_o). The equation was developed by modifying the standard orifice equation as follows

$$Q = CLG_o \sqrt{2gh} \qquad (2)$$

or

$$Q\left(\frac{G_o}{h_s}\right) = CLG_o\left(\frac{G_o}{h_s}\right)\sqrt{2gh}$$

$$Q = C_sLG_o\left(\frac{h_s}{G_o}\right)\sqrt{2gh}$$

$$Q = C_sLh_s\sqrt{2gh} \tag{3}$$

where

$C_s = C(G_o/h_s)$

G_o = gate opening

3. Chart 320-8 presents the results of extensive model tests[2,3,4,5] and limited prototype data.[6] The plotted curves are based on careful measurements and are believed to be representative of the best available data. The model data and most of the prototype data were obtained with the gates adjacent to the test gate open the same amount as the test gate. The plotted curves indicate the effects of the relation of the elevation of the stilling basin apron to that of the gate sill. The portions of the curves having C_s values less than 0.1 are based on prototype gate openings of 1 ft or less and on model gate openings of about 0.05 ft. The experimental data are omitted from this chart in the interest of clarity. Chart 320-8/1 is included to illustrate the degree of data correlation resulting in the curves presented in Chart 320-8.

4. Application. The suggested design curve in Chart 320-8 should be useful for developing pool regulation curves for navigation dam spillways consisting of tainter gates on low sills. The curves presented generally represent sill elevations about 5 ft above streambed and stilling basin apron elevations 3.5 to 31 ft below sill elevation. The Hannibal and Cannelton spillway sills are located about 15 and 19 ft above streambed, respectively. The height of the sill above the approach bed does not seem to be an important factor in submerged flow controlled by gates. However, the coefficient data presented include all the geometric effects of each structure as well as the effects of adjacent gate operation. The curve most applicable to spillway design conditions should be used for developing discharge regulation curves.

5. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Typical Spillway Structure for Central and Southern Florida Water-Control Project; Hydraulic Model Investigation, by J. L. Grace, Jr. Technical Report No. 2-633, Vicksburg, Miss., September 1963.

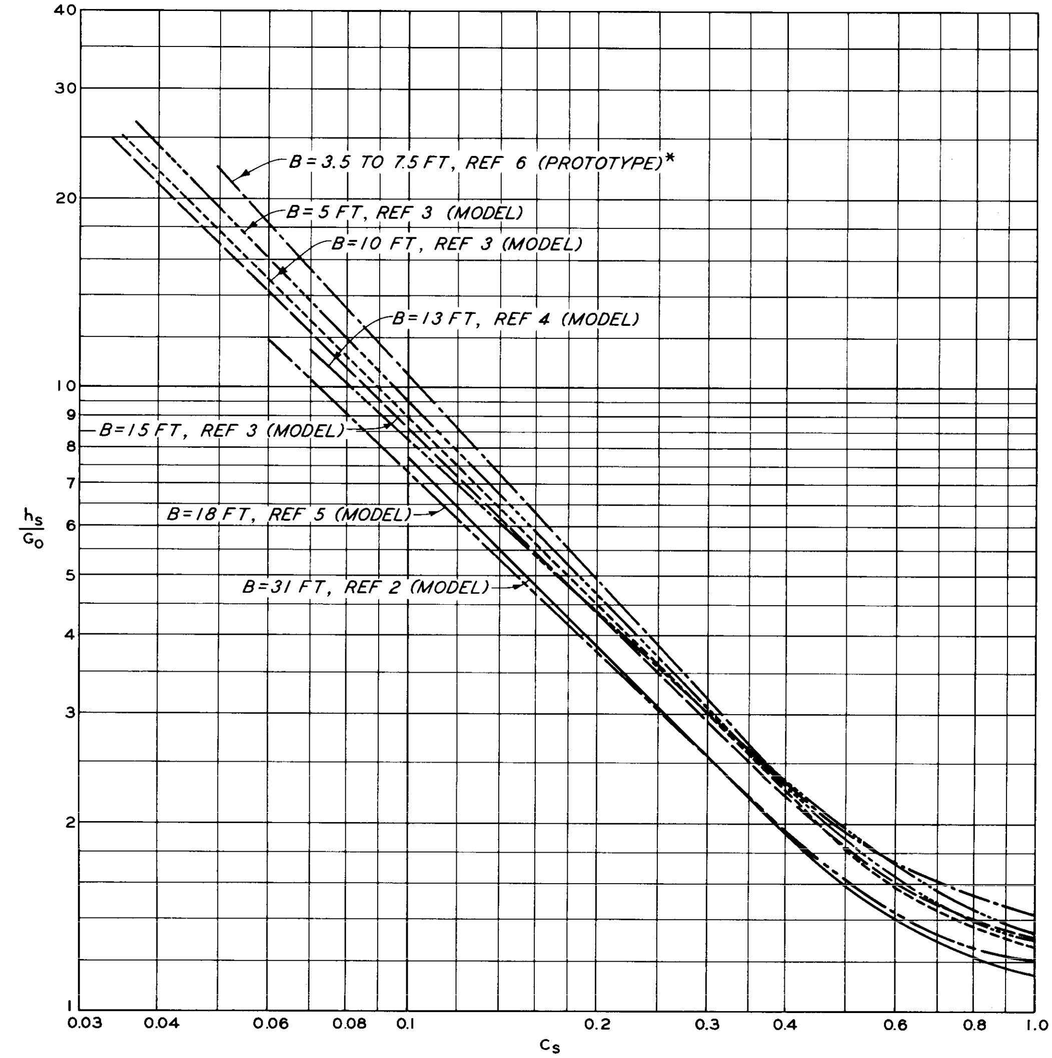
(2) __________, Spillway, Millers Ferry Lock and Dam, Alabama River, Alabama; Hydraulic Model Investigation, by G. A. Pickering. Technical Report No. 2-643, Vicksburg, Miss., February 1964.

(3) __________, Spillway for Typical Low-Head Navigation Dam, Arkansas River, Arkansas; Hydraulic Model Investigation, by J. L. Grace, Jr. Technical Report No. 2-655, Vicksburg, Miss., September 1964.

(4) __________, Spillway for Cannelton Locks and Dam, Ohio River, Kentucky and Indiana; Hydraulic Model Investigation, by G. A. Pickering and J. L. Grace, Jr. Technical Report No. 2-710, Vicksburg, Miss., December 1965.

(5) __________, Spillway, Hannibal Locks and Dam, Ohio River, Ohio and West Virginia; Hydraulic Model Investigation. Technical Report No. 2-731, Vicksburg, Miss., June 1966.

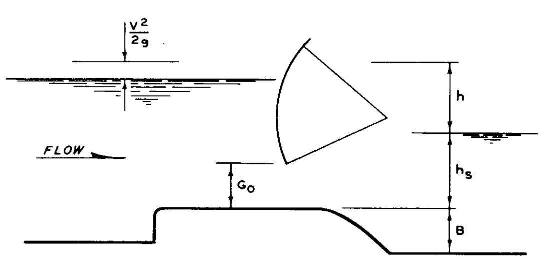
(6) Denzel, C. W., Submerged Tainter Gate Flow Calibration. 1965, U. S. Army Engineer District, St. Louis, Mo. (unpublished memorandum).



BASIC EQUATION

$$Q = C_s L h_s \sqrt{2gh}$$

*MISSISSIPPI RIVER DAMS 2, 5A, AND 26

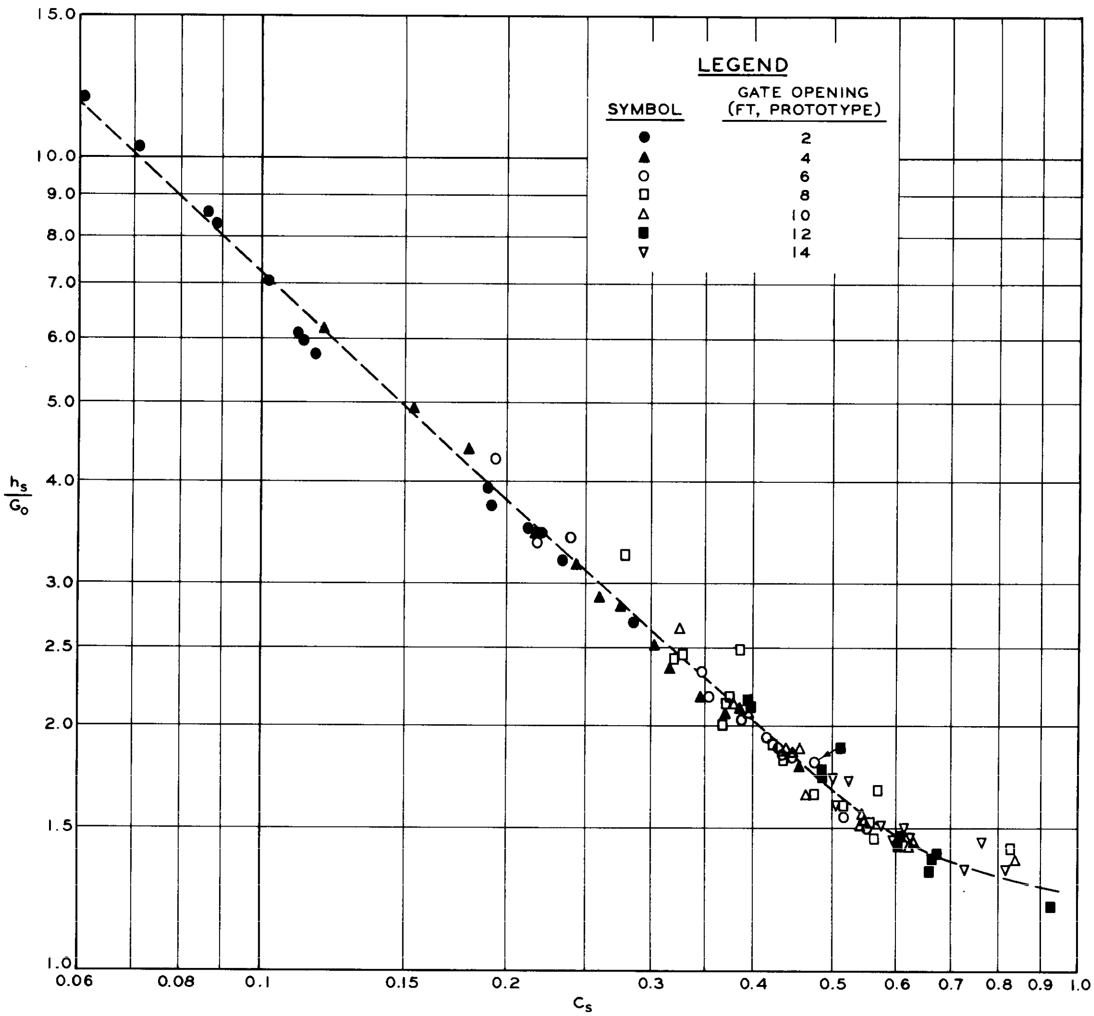


DEFINITION SKETCH

TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENT SUBMERGED FLOW

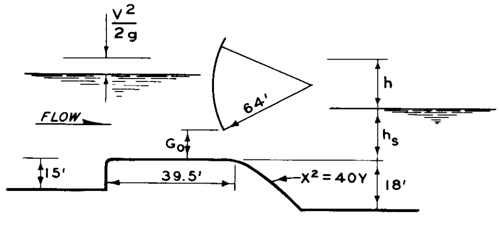
HYDRAULIC DESIGN CHART 320-8



BASIC EQUATION

$$Q = C_s L h_s \sqrt{2gh}$$

NOTE: DATA FROM HANNIBAL MODEL, REF 5



DEFINITION SKETCH

TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENT SUBMERGED FLOW TYPICAL CORRELATION

HYDRAULIC DESIGN CHART 320-8/1

HYDRAULIC DESIGN CRITERIA

SHEETS 330-1 AND 330-1/1

GATE VALVES

DISCHARGE CHARACTERISTICS

1. The discharge characteristics of a flow control valve may be expressed in terms of a loss coefficient for valves along a full-flowing pipeline, or in terms of a discharge coefficient for free flow from a valve located at the downstream end of a pipeline. Loss and discharge coefficients for gate valves are given on Hydraulic Design Charts 330-1 and 330-1/1, respectively.

2. Loss Coefficient. The loss of head caused by a valve occurs not only in the valve itself but also in the pipe as far downstream as the velocity distribution is distorted. Tests to determine this total loss, exclusive of friction, have been conducted on several makes and sizes of gate valves at the University of Wisconsin(1) and the Alden Hydraulic Laboratory.(2) The results of these tests on the larger sizes of valves are given on Chart 330-1 as loss coefficients in terms of the velocity head immediately upstream from the valve. Data are given for both a simple disk gate valve having a crescent-shaped water passage at partial openings and a ring-follower type of gate valve having a lens-shaped water passage at partial openings. The scatter in the Wisconsin data is attributed to minor variations in the geometry of the different makes of valves tested.

3. Discharge Coefficients. Discharge coefficients for free flow from a gate valve at the downstream end of a pipeline have been determined by the Bureau of Reclamation(3) for several makes and sizes of simple disk gate valves. The results of these tests are given on Chart 330-1/1 as discharge coefficients in terms of the total energy head immediately upstream from the valve. The scatter in these data is attributed to minor variations in geometry of the valves tested.

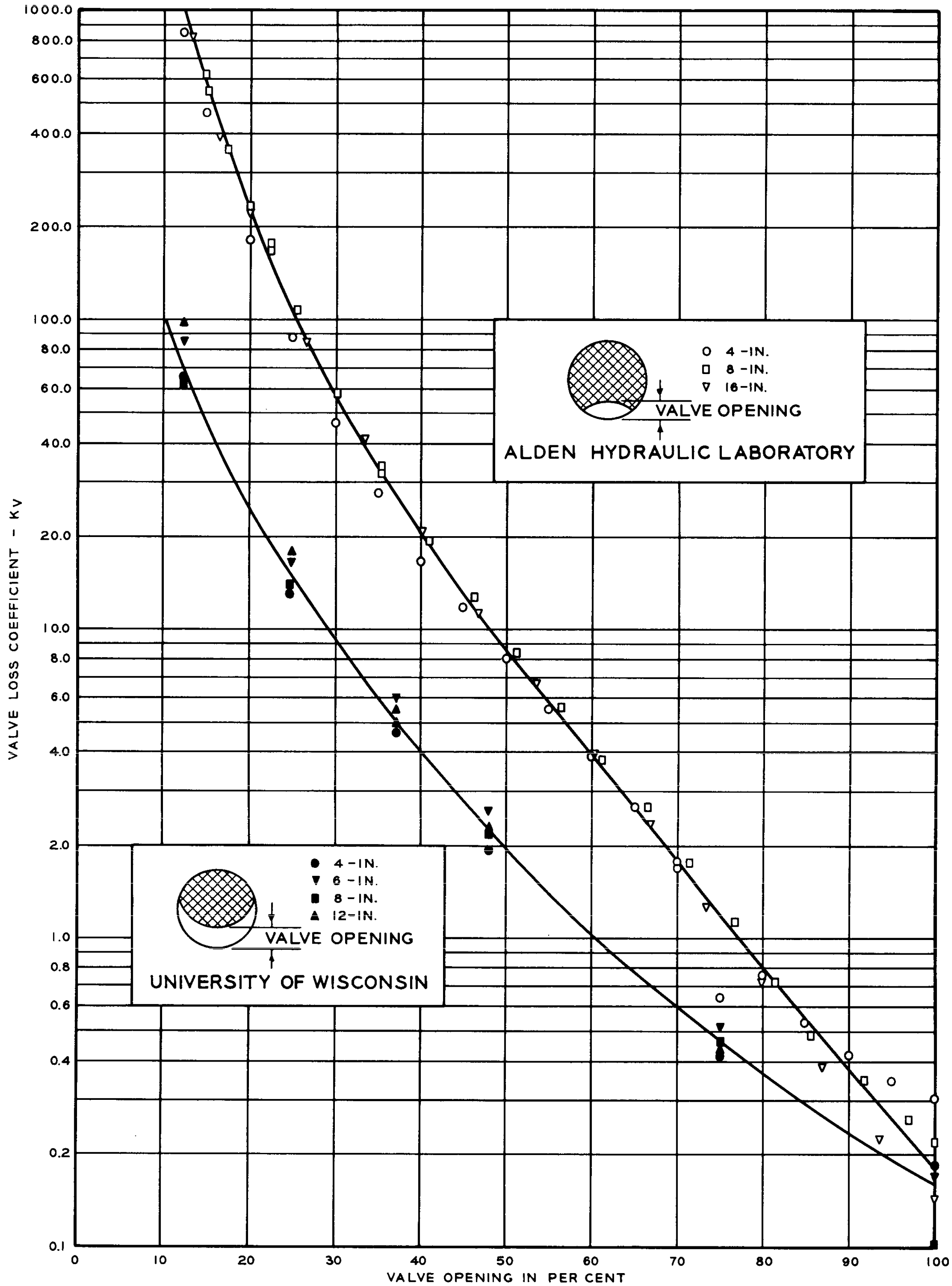
4. Application. The loss data given on Chart 330-1 are applicable to valves installed in full-flowing pipelines having no bends or other disturbances within several diameters upstream and downstream from the valve. The discharge coefficients on Chart 330-1/1 are for valves installed at the downstream end of several diameters of straight pipe and discharging into the atmosphere.

5. List of References.

(1) Corps, C. I., and Ruble, R. O., Experiments on Loss of Head in Valves and Pipes of One-half to Twelve Inches Diameter. University of Wisconsin Engineering Experiment Station Bulletin, vol. IX, No. 1, Madison, Wis., 1922.

(2) Hooper, L. J., Tests of 4-, 8-, and 16-Inch Series 600 Rising Stem Valves for the W-K-M Division of ACE Industries, Houston, Texas. Alden Hydraulic Laboratory, Worcester Polytechnic Institute, Worcester, Mass., Sept. 1949.

(3) U. S. Bureau of Reclamation, Study of Gate Valves and Globe Valves as Flow Regulators for Irrigation Distribution Systems Under Heads Up to About 125 Feet of Water. Hydraulic Laboratory Report No. Hyd-337, Denver, Colo., 13 Jan. 1956.



BASIC EQUATION $K_V = \frac{H_L}{V^2/2g}$

WHERE:

K_V = VALVE LOSS COEFFICIENT

H_L = HEAD LOSS THROUGH VALVE

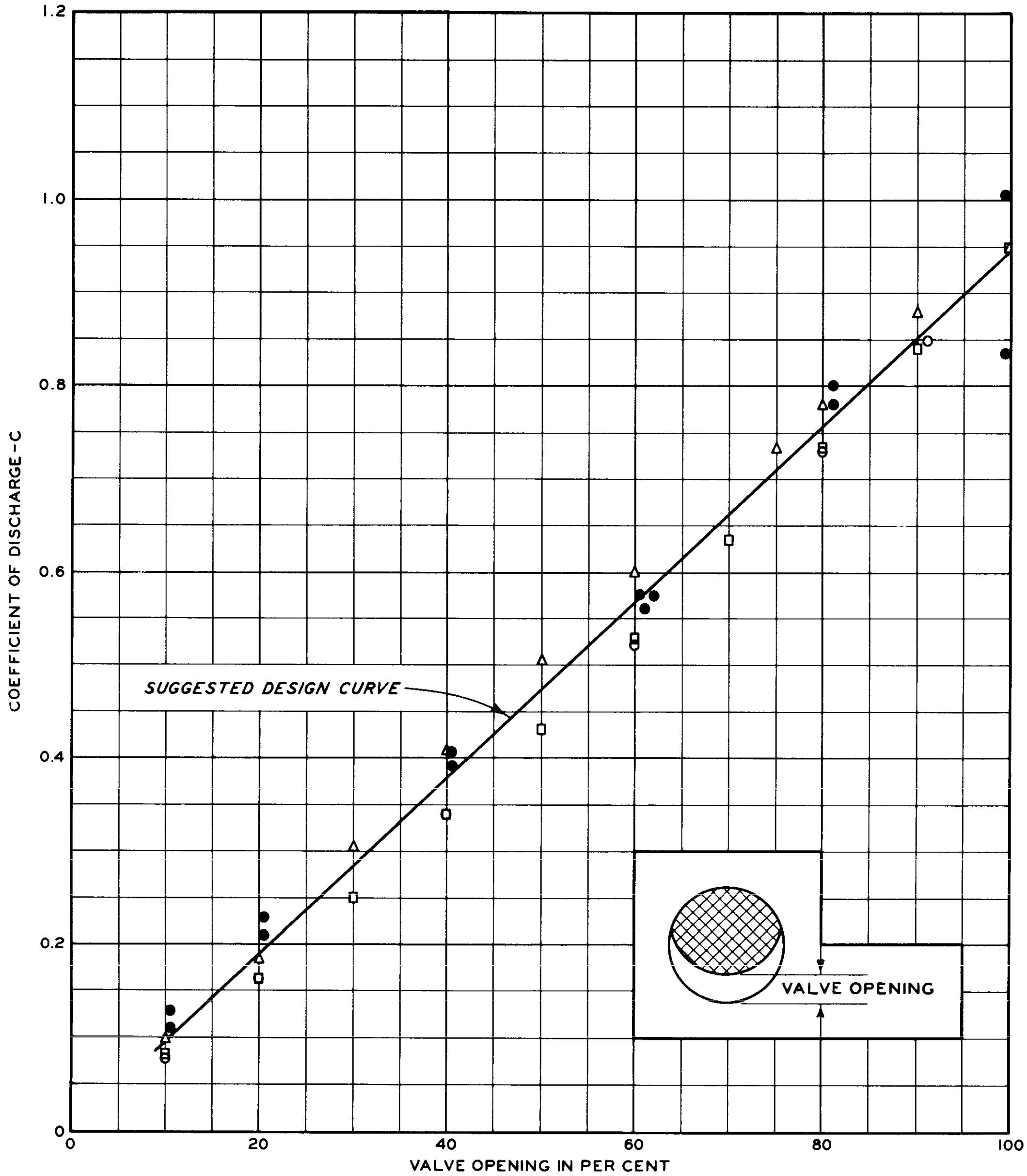
V = AVERAGE VELOCITY IN PIPE

NOTE:

DATA ARE FOR VALVES HAVING SAME DIAMETER AS PIPE AND FOR DOWNSTREAM PIPE FLOWING FULL.

GATE VALVES
LOSS COEFFICIENTS
HYDRAULIC DESIGN CHART 330-1

WES 6-57



BASIC EQUATION $Q = CA\sqrt{2gH_e}$

WHERE:

C = VALVE DISCHARGE COEFFICIENT

A = AREA BASED ON NOMINAL VALVE DIAMETER

H_e = ENERGY HEAD MEASURED TO CENTER LINE OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE

NOTE:

DATA ARE FROM USBR TESTS FOR FREE FLOW FROM 8- TO 12-INCH-DIAMETER GATE VALVES AT DOWNSTREAM END OF CONDUIT OF SAME NOMINAL DIAMETER AS VALVE.

GATE VALVES

FREE FLOW

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 330-1/1

HYDRAULIC DESIGN CRITERIA

SHEETS 331-1 to 331-3

BUTTERFLY VALVES

DISCHARGE AND HYDRAULIC TORQUE CHARACTERISTICS

1. The discharge and torque characteristics of butterfly valves can be expressed in terms of discharge and torque coefficients as functions of the angle of rotation of the valve vane from opened position. The discharge coefficient is primarily a function of the orifice opening whereas the hydraulic torque coefficient depends upon the geometry of the valve vane. Thus, differences in torque coefficients are to be expected for various shaped vanes at the same opening. Although considerable data have been published(2), only data indicated as the original computations or curves of the investigators have been included in Design Charts 331-1 to 331-2/1.

2. Discharge Coefficients. A modified form of the standard orifice equation has been used for computation of valve discharge. The area used in the equation is based on the nominal diameter of the valve because of difficulty in determining the actual areas of the orifice openings for partially opened valves. The discharge coefficient varies inversely with the angle of rotation of the valve from opened position. Two valve locations have been tested; one in which the valve is near the outflow end of the pipe, and the other in which the valve is well within a straight reach of pipe. Hydraulic Design Chart 331-1 presents discharge coefficients for valves located within the pipe. Chart 331-1/1 presents similar data for valves located near the end of the pipe. The material used in these charts is taken from the following investigators: McPherson(7), Dickey-Coplen(4), Gaden(5), Colleville(8), DeWitt(3), and Armanet(1). The Dickey-Coplen data are from air tests on a thin circular damper. The Armanet tests reflect the effects of convergence in the valve housing downstream from the vane pivot.

3. Torque Coefficients. Torque coefficient data are presented in Charts 331-2 and 2/1. The available information is limited. Chart 331-2 pertains to valves located within the pipe and Chart 331-2/1 applies to valves located near the end of the pipe. The Keller and Salzmann(6) data in Chart 331-2 were obtained from air tests. The DeWitt curve in Chart 331-2/1 was computed from published prototype torque curves. The Gaden curves are based on carefully controlled laboratory tests which included measurement of and correction for pressure distribution on the downstream face of the valve vane. The Armanet curves reflect the effects of convergence in the valve body. The scarcity of torque coefficient data is indicative of the need for torque tests on butterfly valves of American manufacture.

4. Application. A sample computation for torque is given in Chart 331-3. Final computations should be based on the recommendations of the valve manufacturer at which time friction torque and seating torque data should be considered.

5. List of References.

(1) Armanet, L., "Vannes-Papillon Des Turbines." *Génissiat,* Numéro Hors Série De La Houille Blanche, pp 199-219.

(2) Cohn, S. D., "Performance analysis of butterfly valves." *Instruments,* vol 24, No. 8 (August 1951), p 880-884.

(3) DeWitt, C., "Operating a 24-in. butterfly valve under a head of 223 ft." *Engineering News-Record* (18 September 1930), pp 460-462.

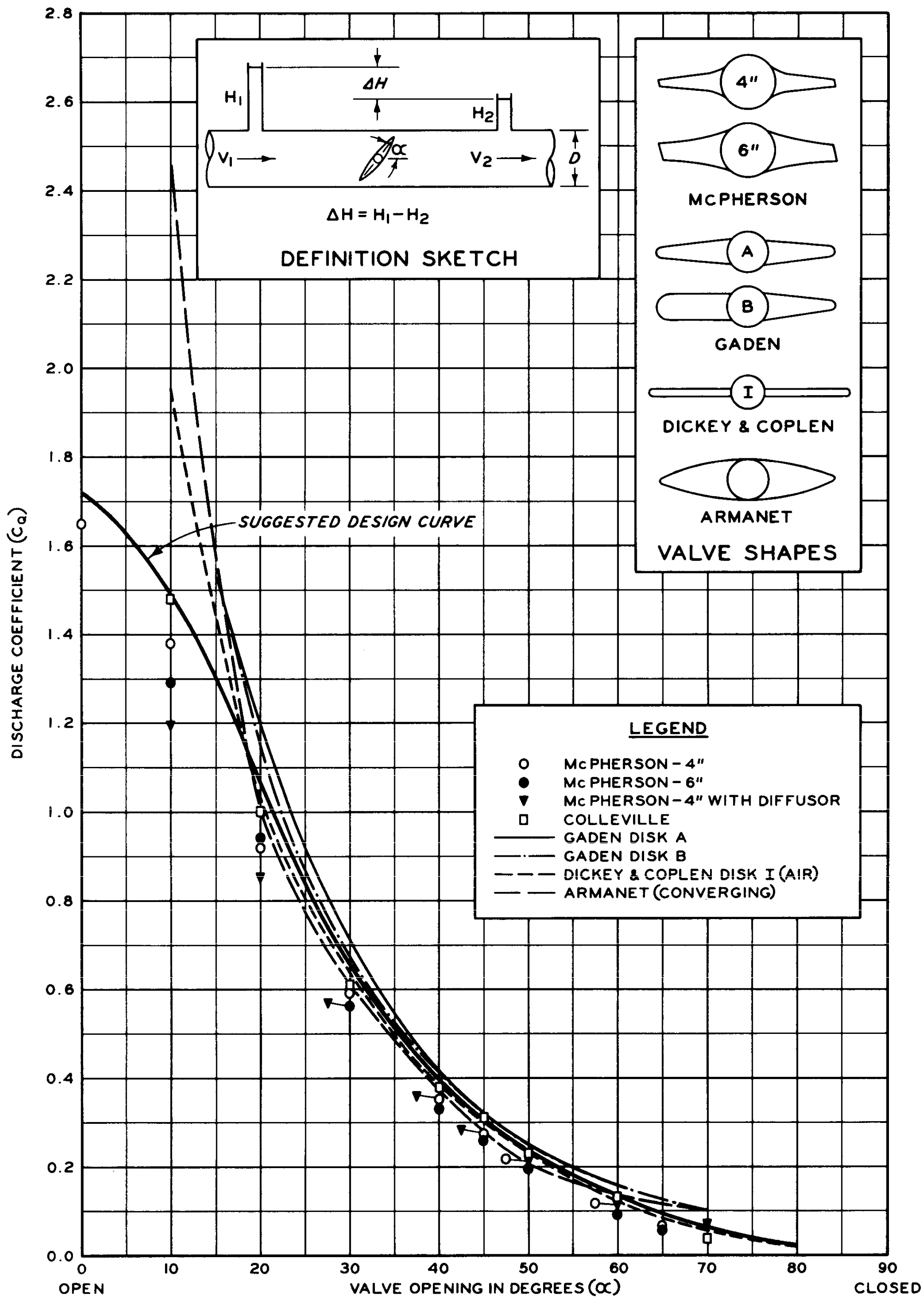
(4) Dickey, P. S., and Coplen, H. L., "A study of damper characteristics." *Transactions,* ASME, vol 64, No. 2 (February 1942).

(5) Gaden, D., "Contribution to study of butterfly valves." *Schweizerische Bauzeitung,* vol III, Nos. 21, 22, and 23 (May 21 and 28 and June 4, 1938). Similar material by D. Gaden was also published in England in *Water Power* (December 1951 and January 1952).

(6) Keller, C., and Salzmann, F., "Aerodynamic model tests on butterfly valves." *Escher-Wyss News,* vol IX, No. 1 (January-March 1936).

(7) McPherson, M. B., Strausser, H. S., and Williams, J. C., Jr., "Butterfly valve flow characteristics." *Proceedings,* ASCE, paper 1167, vol 83, No. HY1 (February 1957).

(8) Voltmann, Henry, discussion of reference 7. *Proceedings,* ASCE, vol 83, No. HY4 (August 1957), pp 1348-48 and 49.



BASIC EQUATION

$$Q = C_Q D^2 \sqrt{g} \sqrt{\Delta H}$$

WHERE:

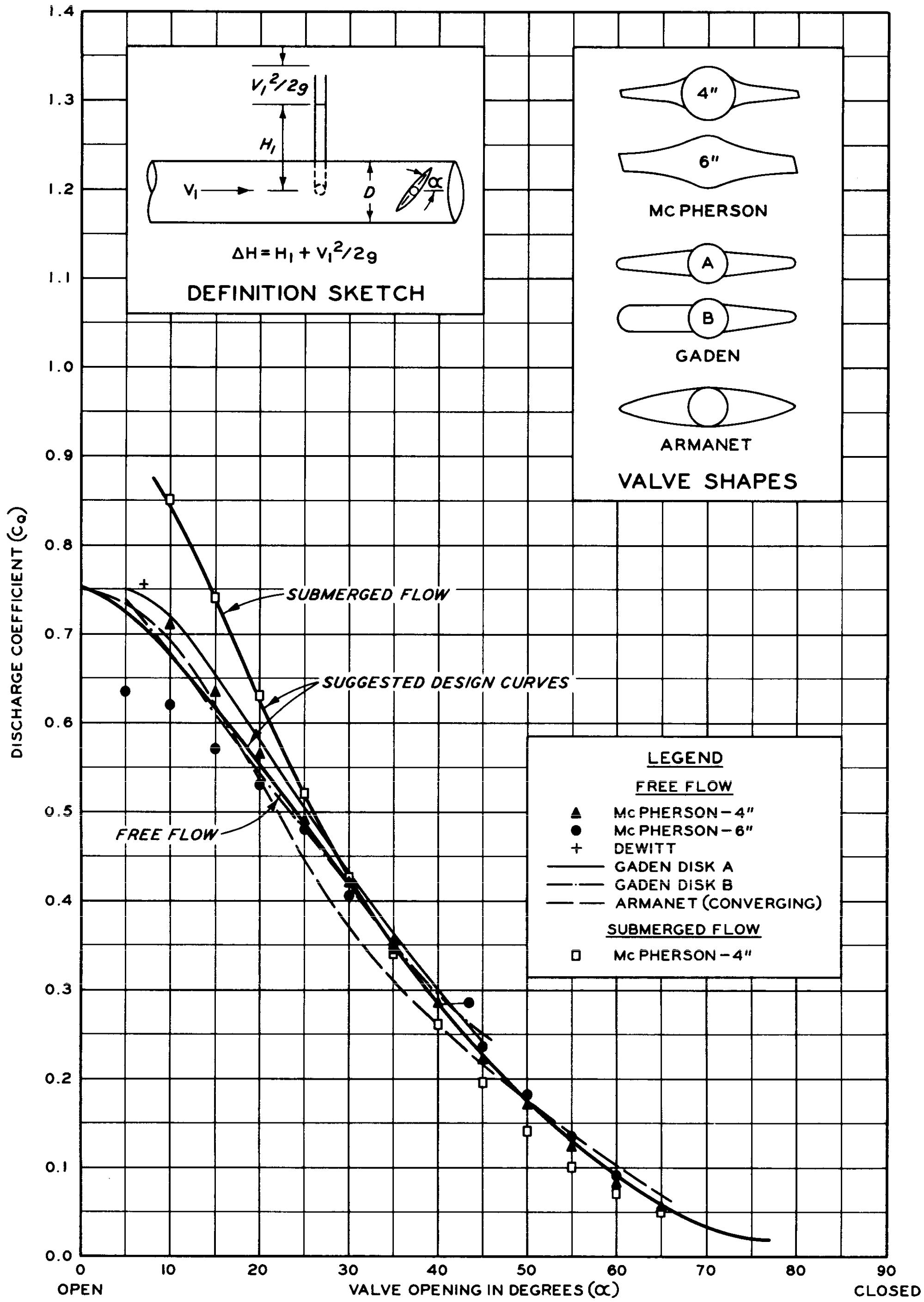
Q = DISCHARGE IN CFS
C_Q = DISCHARGE COEFFICIENT
D = VALVE DIAMETER IN FT
g = GRAVITY CONSTANT = 32.2 FT/SEC^2
ΔH = PRESSURE DROP ACROSS THE VALVE IN FT OF WATER

BUTTERFLY VALVES

DISCHARGE COEFFICIENTS VALVE IN PIPE

HYDRAULIC DESIGN CHART 331-1

WES 8-58



BASIC EQUATION

$$Q = C_Q D^2 \sqrt{g}\,\sqrt{\Delta H}$$

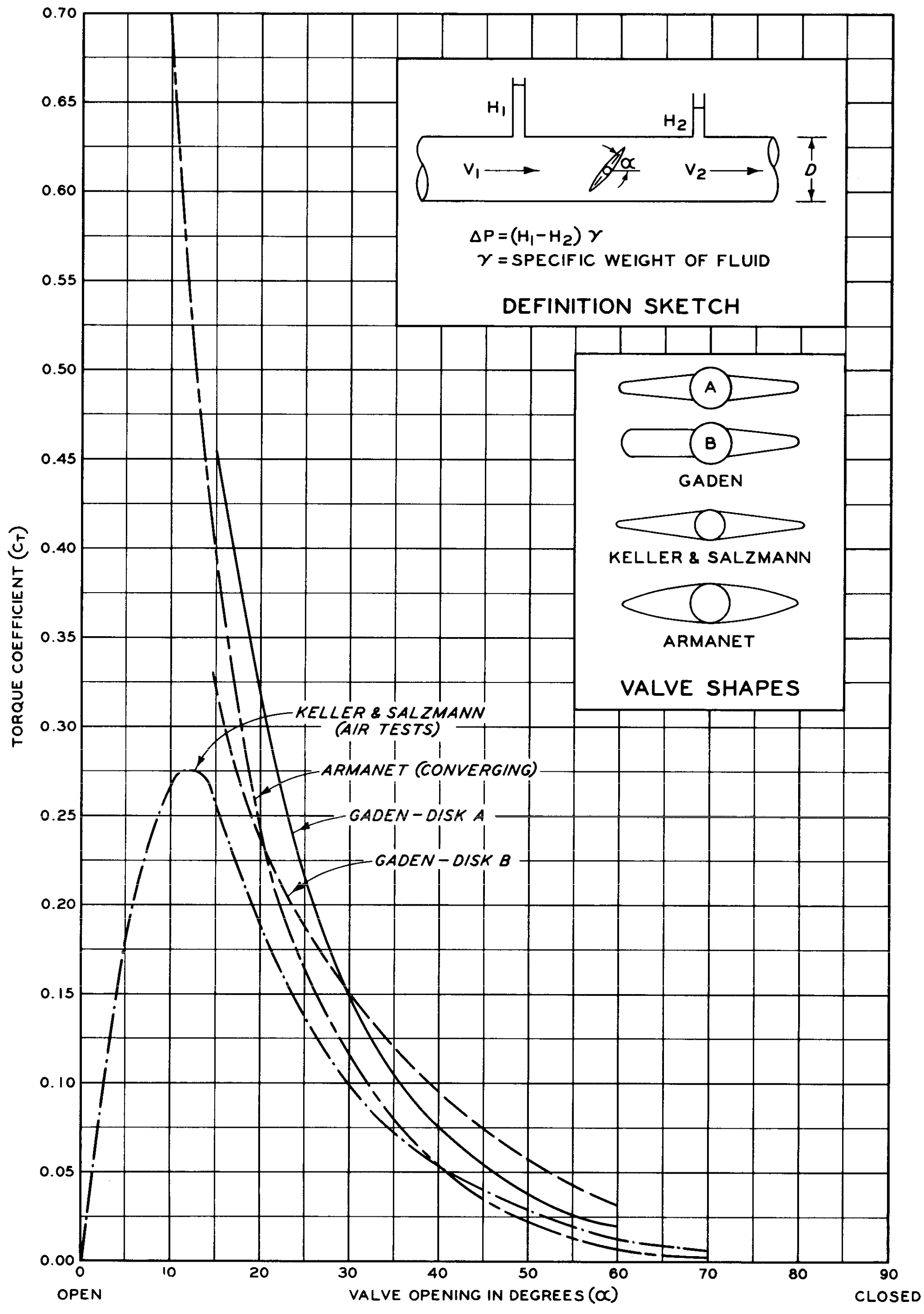
WHERE:

Q = DISCHARGE IN CFS
C_Q = DISCHARGE COEFFICIENT
D = VALVE DIAMETER IN FT
g = GRAVITY CONSTANT = 32.2 FT/SEC^2
ΔH = TOTAL ENERGY HEAD IN FT OF WATER UPSTREAM OF VALVE

BUTTERFLY VALVES
DISCHARGE COEFFICIENTS
VALVE IN END OF PIPE

HYDRAULIC DESIGN CHART 331-1/1

WES 8-58



<u>BASIC EQUATION</u>

$$T = C_T D^3 \Delta P$$

WHERE:

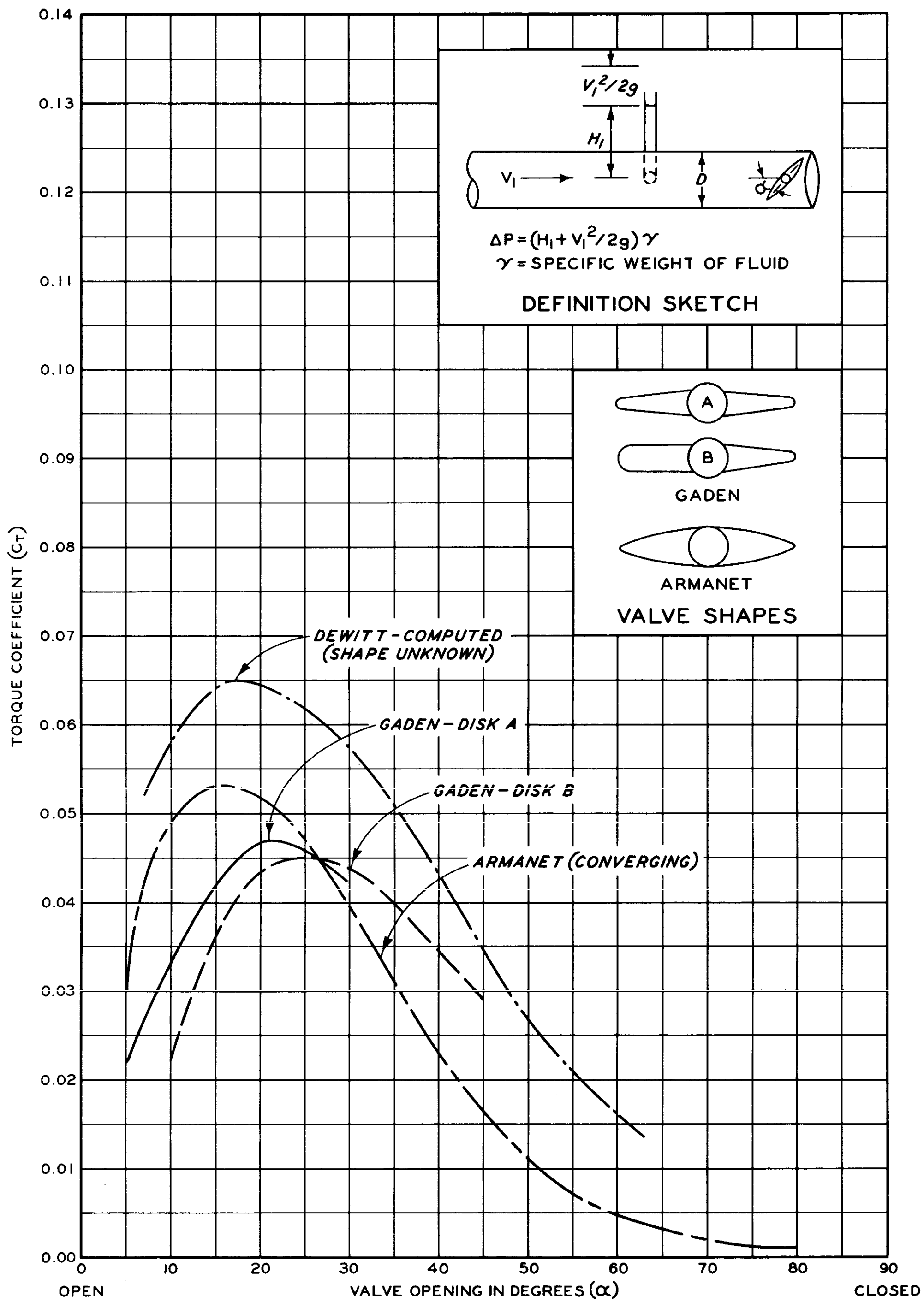
- T = TORQUE IN FT-LB
- C_T = TORQUE COEFFICIENT
- D = VALVE DIAMETER IN FT
- ΔP = PRESSURE DIFFERENTIAL IN LB/SQ FT

BUTTERFLY VALVES

TORQUE COEFFICIENTS
VALVE IN PIPE

HYDRAULIC DESIGN CHART 331-2

WES 8-58



BASIC EQUATION

$$T = C_T D^3 \Delta P$$

WHERE:

- T = TORQUE IN FT-LB
- C_T = TORQUE COEFFICIENT
- D = VALVE DIAMETER IN FT
- ΔP = TOTAL ENERGY HEAD AT UPSTREAM SIDE OF VALVE IN LB/SQ FT

BUTTERFLY VALVES

TORQUE COEFFICIENTS VALVE IN END OF PIPE

HYDRAULIC DESIGN CHART 331-2/1

WES 8-58

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB CW 804 PROJECT John Doe Dam SUBJECT Butterfly Valves

COMPUTATION Valve Opening and Hydraulic Torque

COMPUTED BY WCB DATE 2/26/58 CHECKED BY RGC DATE 2/27/58

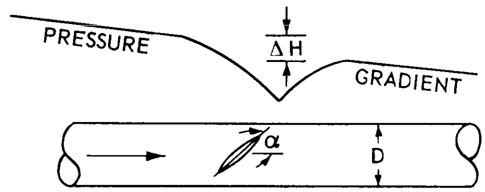
GIVEN:

Total available head (H_T) = 225 ft

Valve diameter (D) = 4 ft

Valve shape - Gaden-Disk A on Chart 331-1

Energy loss in system without valve (H_L) = $0.3\ V^2/2g$



ASSUME:

Discharge (Q) = 600 cfs

COMPUTE:

1. Head loss (H_L) in system without valve

$$V = \frac{Q}{A} = 48 \text{ ft per sec}$$

$$H_v = V^2/2g = 35 \text{ ft}$$

$$H_L = 0.3\ H_v = 10 \text{ ft}$$

2. Required valve loss (ΔH) for Q = 600 cfs

$$\Delta H = H_T - H_L - H_v = 225 - 10 - 35 = 180 \text{ ft}$$

Discharge coefficient (C_Q)

$$Q = C_Q D^2 \sqrt{g}\sqrt{\Delta H} \quad \text{(Chart 331-1)}$$

$$C_Q = \frac{600}{16 \times \sqrt{32.2} \times \sqrt{180}} = 0.49$$

From suggested design curve on Chart 331-1, valve opening (α) = 36° for C_Q of 0.49.

3. Hydraulic torque (T) for Q = 600 cfs and α = 36°. From Chart 331-2, torque coefficient (C_T) for Gaden-Disk A valve open 36° = 0.10.

$$T = C_T D^3 \Delta P \quad \text{(Chart 331-2)}$$

Where $\Delta P = (H_1 - H_2)\gamma = \Delta H\gamma$

$$T = 0.10 \times 64 \times 180 \times 62.5 = 72{,}000 \text{ ft-lb}$$

Repeat computations for other assumed discharges to determine discharge and hydraulic torque curves.

BUTTERFLY VALVES

SAMPLE COMPUTATION
DISCHARGE AND TORQUE

HYDRAULIC DESIGN CHART 331-3

WES 8-58

HYDRAULIC DESIGN CRITERIA

SHEETS 332-1 AND 1/1

HOWELL-BUNGER VALVES

DISCHARGE COEFFICIENTS

1. General. The Howell-Bunger valve is essentially a cylinder gate mounted with the axis horizontal. A conical end piece with its apex upstream is connected to the valve body by vanes. A movable external horizontal sleeve controls the discharge by varying the opening between the sleeve and the cone. The discharge is in the form of a diverging hollow conical jet. Diameters of valves range from 1.5 to 9 ft. Some valves have four vanes while others have six vanes. Separate discharge coefficient charts are presented for four- and six-vane valves.

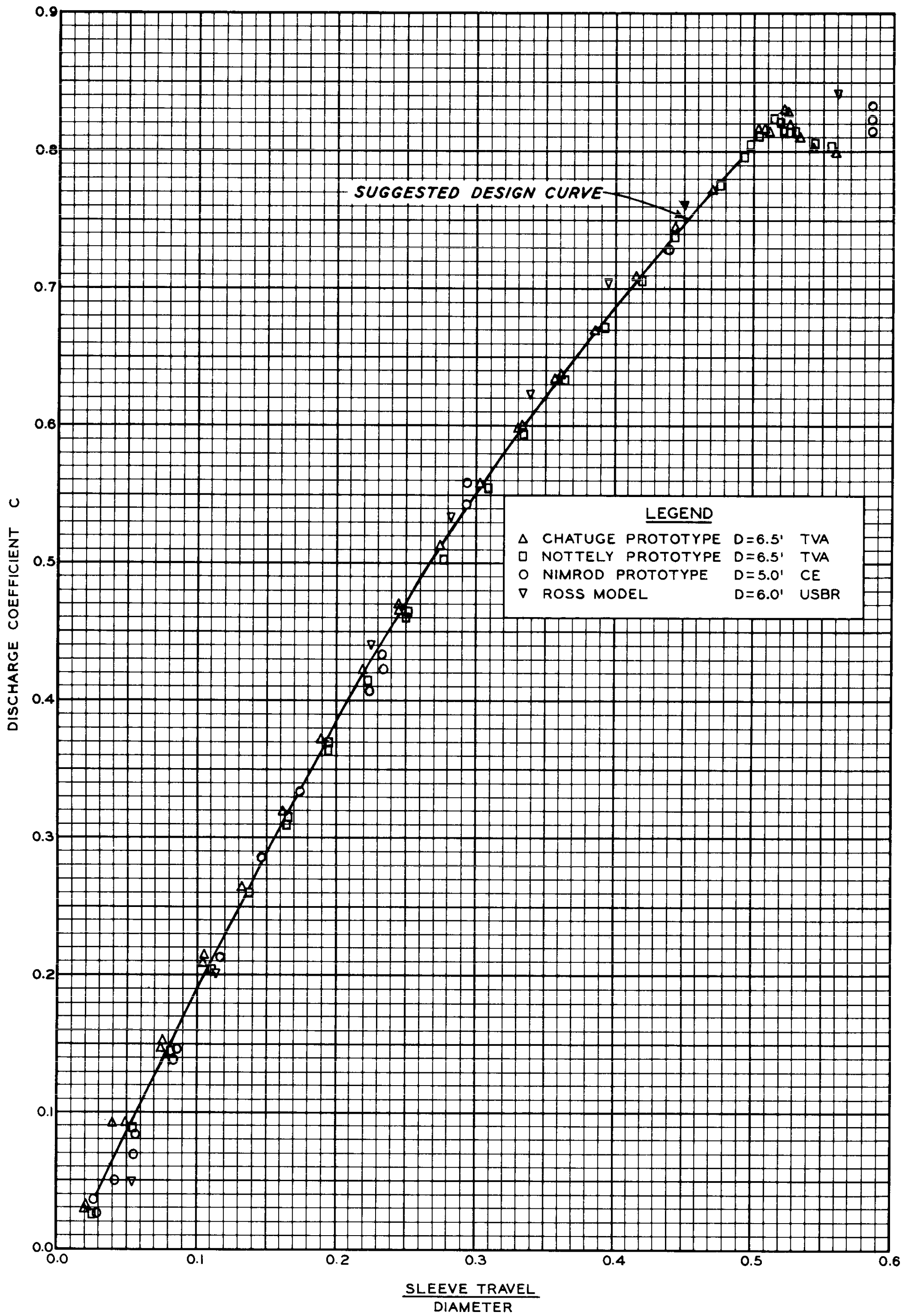
2. Discharge Coefficients. Discharge coefficients for Howell-Bunger valves have been computed for various dimensional features of the valves. However, the discharge coefficients shown on Charts 332-1 and 1/1 are based on the area of the conduit immediately upstream from the valve. The basic equation used is shown on each chart. The computed coefficients are plotted against the dimensionless factor, sleeve travel divided by conduit diameter.

3. Experimental Data. Discharge coefficients for Chatuge, Nottely, Watauga, and Fontana Dams were computed from prototype data published by the Tennessee Valley Authority(1). Coefficients for Ross Dam are based on model data published by the Bureau of Reclamation(2). Coefficients for Nimrod Dam result from discharge measurements made by the Little Rock District, CE. Coefficients for Narrows Dam result from model data obtained by the Waterways Experiment Station. The data presented on Charts 332-1 and 332-1/1 indicate discharge coefficients of 0.82 and 0.87 for full openings of the four- and six-vane valves, respectively.

---

(1) R. A. Elder and G. B. Dougherty, "Hydraulic Characteristics of Howell-Bunger Valves and Their Associated Structures," TVA Report dated 1 Nov. 1950.

(2) "Investigation of Hydraulic Properties of the Revised Howell-Bunger Valve, City of Seattle, Washington," Hydraulic Laboratory Report No. 168, Bureau of Reclamation, April 1945.



BASIC EQUATION

$$Q = CA\sqrt{2gH_e}$$

WHERE:

C = DISCHARGE COEFFICIENT

A = AREA OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN SQ FT

H_e = ENERGY HEAD MEASURED TO CENTERLINE OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN FT

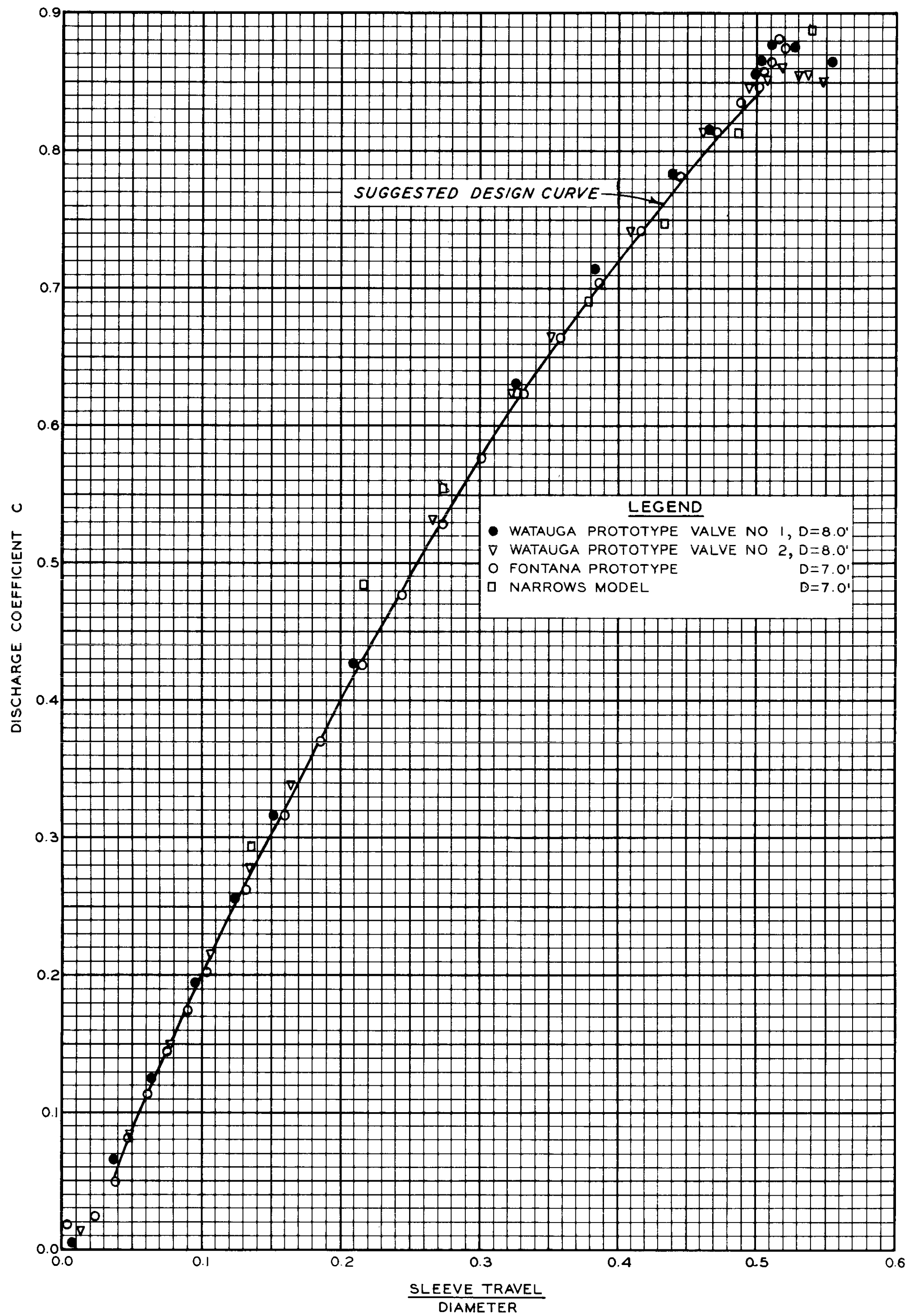
HOWELL-BUNGER VALVES

DISCHARGE COEFFICIENTS

FOUR VANES

HYDRAULIC DESIGN CHART 332-1

WES 2-54



BASIC EQUATION

$$Q = CA\sqrt{2gH_e}$$

WHERE:

C = DISCHARGE COEFFICIENT

A = AREA OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN SQ FT

H_e = ENERGY HEAD MEASURED TO CENTERLINE OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN FT

HOWELL - BUNGER VALVES

DISCHARGE COEFFICIENTS

SIX VANES

HYDRAULIC DESIGN CHART 332-1/1

WES 2-54

HYDRAULIC DESIGN CRITERIA

SHEET 340-1

FLAP GATES

HEAD LOSS COEFFICIENTS

1. Flap gate head losses can be determined by the equation:

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

where

H_L = head loss in ft of water
K = head loss coefficient
V = conduit velocity in ft per sec

2. Hydraulic Design Chart 340-1 presents head loss coefficients for submerged flap gates. The data result from tests by Nagler (1) on 18-in.-, 24-in.-, and 30-in.-diameter gates.

3. Modern flap gates are heavier but similar in design to those tested by Nagler. It is suggested that Chart 340-1 be used for design purposes for submerged flow conditions until additional data become available. Head loss coefficient data are not available for free discharge.

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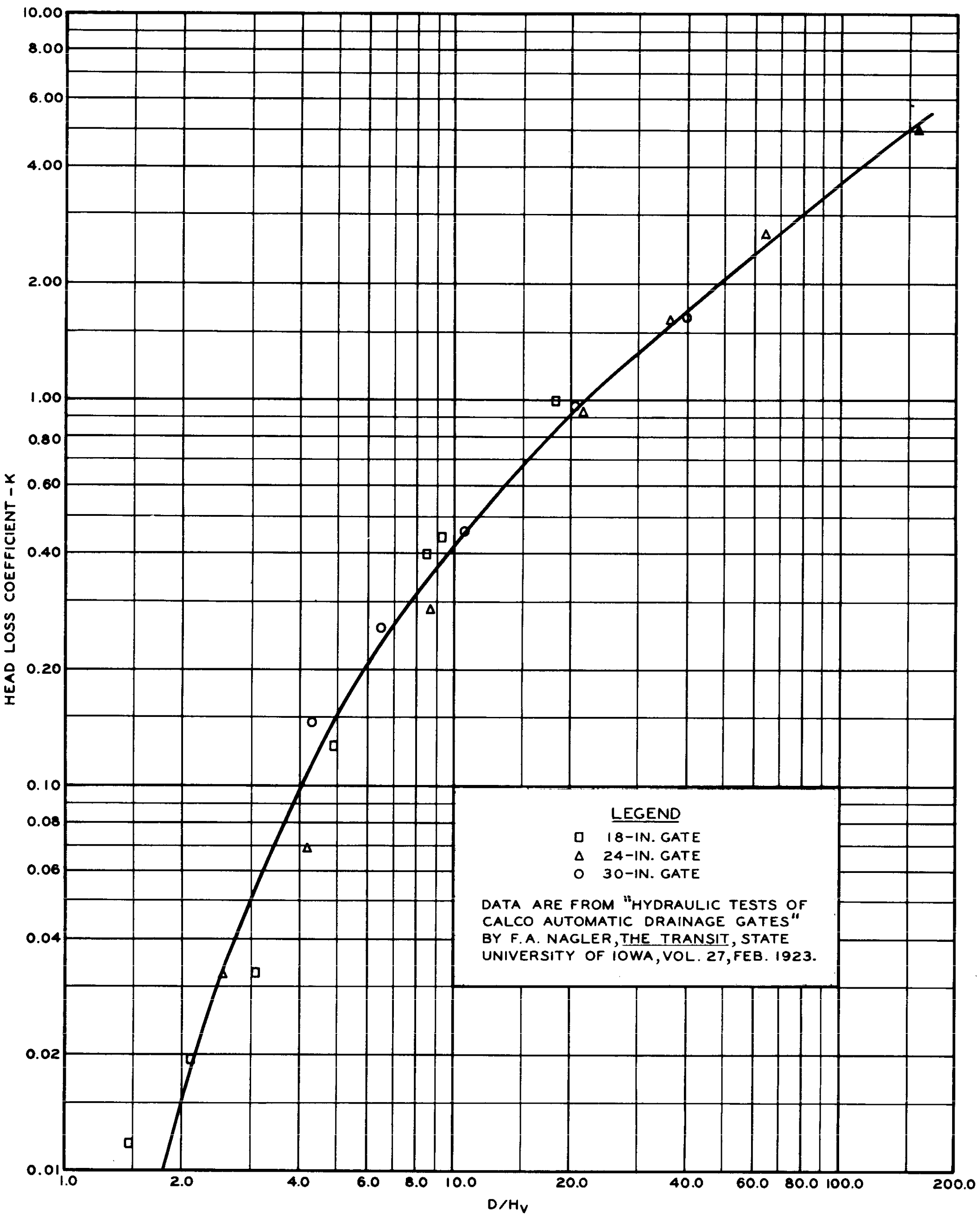
(1) F. A. Nagler, "Hydraulic tests of Calco automatic drainage gates," The Transit, State University of Iowa, vol 27 (February 1923).



EQUATIONS

$$K = \frac{H_L}{H_V} \; ; \; H_V = \frac{V^2}{2g}$$

NOTE: K = HEAD LOSS COEFFICIENT
H_L = HEAD LOSS, FT
D = CONDUIT DIAMETER, FT
V = CONDUIT VELOCITY, FT/SEC
g = ACCELERATION OF GRAVITY, FT/SEC^2

FLAP GATES
HEAD LOSS COEFFICIENTS
SUBMERGED FLOW

HYDRAULIC DESIGN CHART 340-1

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