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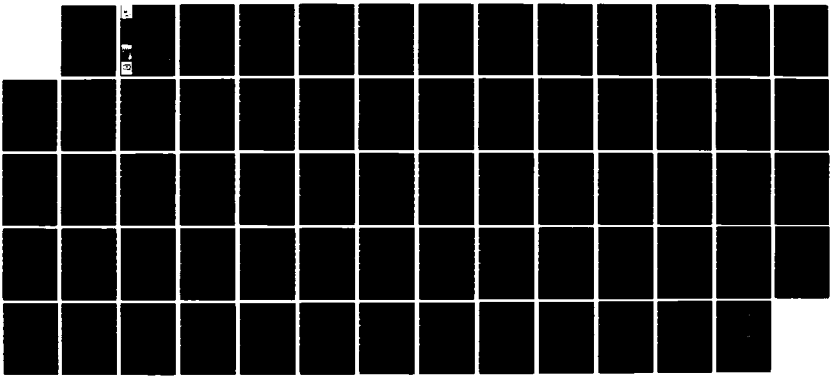

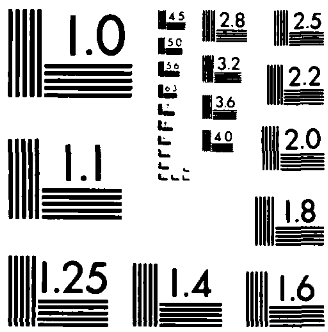

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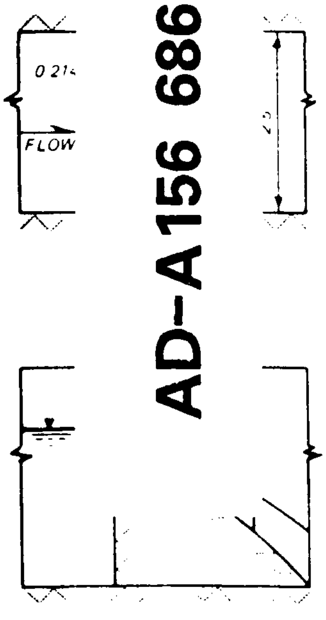

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(2)

TECHNICAL REPORT HL-85-1

GENERAL SPILLWAY INVESTIGATION

Hydraulic Model Investigation

by

Stephen T. Maynord

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631
Vicksburg, Mississippi 39180-0631

March 1985

Final Report

Approved For Public Release: Distribution Unlimited

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Washington, DC 20314-1000

Under Civil Works Investigation
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A design procedure for spillway crests has been developed that is applicable to a wide range of approach depths, upstream slopes, and heads on the crest. This report presents the results of systematic model tests of spillway crests designed using the procedure and covering a wide range of design conditions. Discharge coefficients, crest pressures, and water-surface profiles were determined both with and without piers on the crest.

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PREFACE

The study described herein was performed at the U. S. Army Engineer Waterways Experiment Station (WES), during the period 1977 to 1982, for the Office, Chief of Engineers, U. S. Army, as part of the Civil Works Research and Development program. Funds utilized were allotted under Civil Works Investigation, Work Unit 030200/31177, "General Spillway Tests." The study was accomplished under the direction of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, J. L. Grace, Jr., Chief of the Hydraulic Structures Division, and N. R. Oswalt, Chief of the Spillways and Channels Branch. Mr. T. E. Murphy (retired) provided guidance throughout the course of the study. This report was prepared by Mr. S. T. Maynord.

Commanders and Directors of WES during the period of this study and the preparation and publication of this report were COL G. H. Hilt, CE, COL John L. Cannon, CE, COL Nelson P. Conover, CE, COL Tilford C. Creel, CE, and COL Robert C. Lee, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, US CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

US customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
inches	25.4	millimetres

GENERAL SPILLWAY INVESTIGATION

Hydraulic Model Investigation

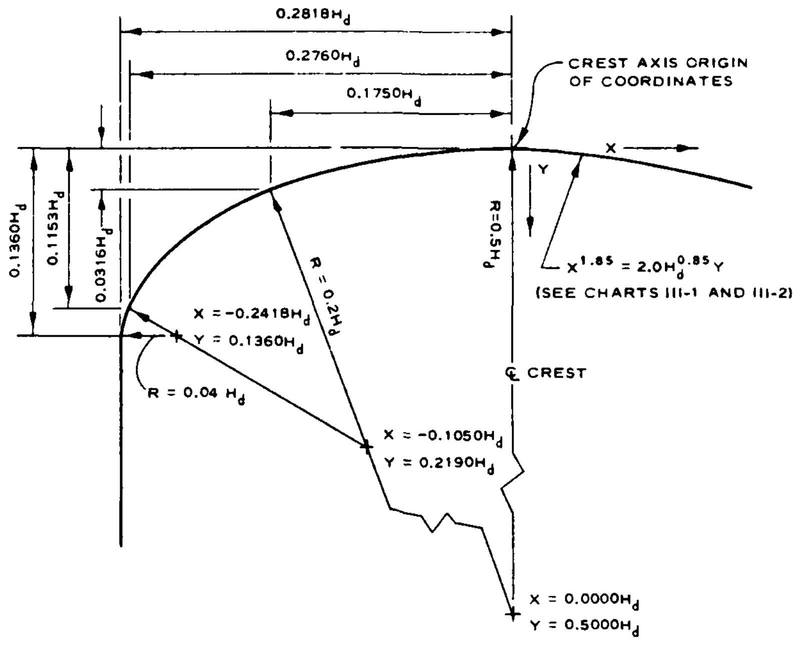
PART I: INTRODUCTION

1. Considerable work has been done to determine the shape of the crest of an overflow spillway and different methods are available that depend on the relative height and upstream face slope of the spillway. With few exceptions, these methods are derived from the extensive data taken by the U. S. Bureau of Reclamation (USBR) (USBR 1948) defining the profile of the lower nappe of flow over a sharp-crested weir for a wide range of relative heights and upstream face slopes. The design methods presented by the U. S. Army Engineer Waterways Experiment Station (WES) (USACE 1952) and the USBR (1974) are probably the most widely used methods in this country.

2. The Corps of Engineers design guidance for shaping the crest of spillways having a vertical upstream face and negligible velocity of approach is shown in Figures 1a and 1b. In Figure 1a, the small radius $R = 0.04H_d$ that connects the curved portion of the crest to the upstream face was added to eliminate the surface discontinuity. This resulted in improved pressure conditions and discharge coefficients at heads exceeding the design head. In Figure 1b, an alternate method for shaping the upstream quadrant of crests having vertical upstream faces and negligible velocity of approach is found in Engineer Manual 1110-2-1603 (OCE 1965). Corps design guidance for shaping the crests of spillways having sloping upstream faces and negligible velocity of approach is shown in Figures 2a, 2b, and 2c for sloping faces of 3V on 1H, 3V on 2H, and 3V on 3H, respectively. Note that the crest designs shown in Figure 2 have a discontinuity at the upstream face. Corps of Engineers guidance for shaping the crest of spillways having appreciable velocity of approach and a 3V-on-3H upstream face is given in Figure 3. No guidance is presented for other upstream face slopes where the velocity of approach is appreciable.

3. The USBR criteria (1974) for shaping the crests of spillways is shown in Figure 4. These crests have a surface discontinuity at the intersection of the curved portion of the crest and the upstream face of the crest.

4. A design procedure for shaping spillway crests was needed that eliminated the surface discontinuity at the upstream face and was applicable to a



COORDINATES FOR UPSTREAM QUADRANT

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

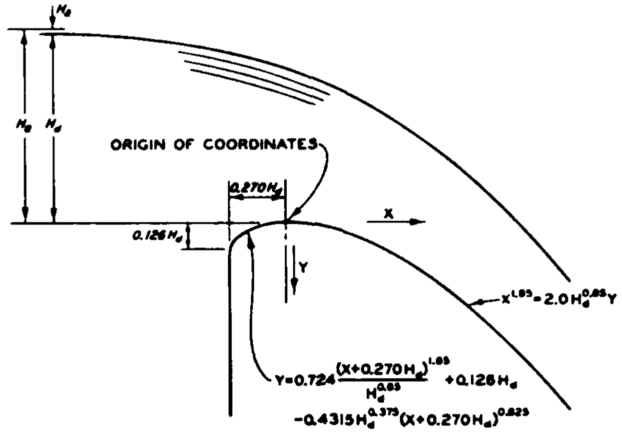
NOTE: H_d = DESIGN HEAD BASED ON LOWER SURFACE OF NAPPE FROM SHARP-CRESTED WEIR WITH NEGLIGIBLE VELOCITY OF APPROACH AND CREST AT $X = -0.2818H_d$, $Y = 0.1259H_d$.

OVERFLOW SPILLWAY CREST
UPSTREAM QUADRANT

HYDRAULIC DESIGN CHART 111-2/1

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

Figure 1. Crest shape, vertical face, negligible velocity of approach (Continued)



COORDINATES FOR UPSTREAM QUADRANT

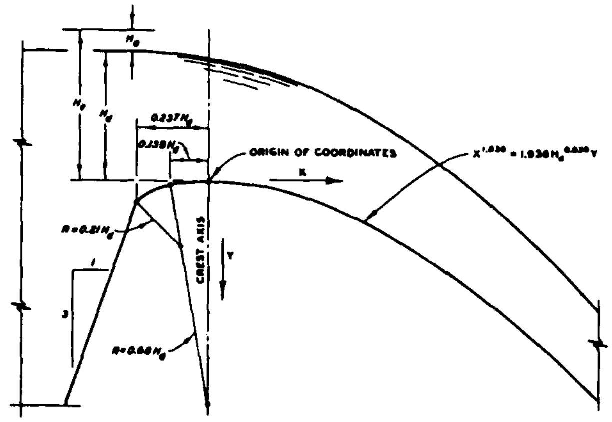
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	$\frac{X}{H_d}$	$\frac{Y}{H_d}$
0.0000	0.0000	-0.2000	0.0494
-0.0200	0.0004	-0.2100	0.0556
-0.0400	0.0016	-0.2200	0.0624
-0.0600	0.0036	-0.2300	0.0701
-0.0800	0.0068	-0.2400	0.0787
-0.1000	0.0108	-0.2450	0.0836
-0.1200	0.0158	-0.2500	0.0889
-0.1400	0.0221	-0.2550	0.0948
-0.1600	0.0296	-0.2600	0.1016
-0.1700	0.0339	-0.2650	0.1099
-0.1800	0.0386	-0.2689	0.1165
-0.1900	0.0437	-0.2703	0.1261

NOTE: H_d = DESIGN HEAD BASED ON LOWER SURFACE OF NAPPE FROM SHARP-CRESTED WEIR WITH NEGLIGIBLE VELOCITY OF APPROACH.

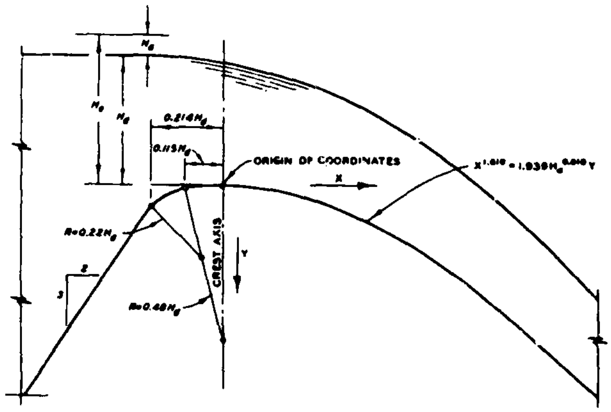
HIGH OVERFLOW SPILLWAYS
CREST SHAPE
UPSTREAM QUADRANT
FROM EM 1110-2-1603

b.

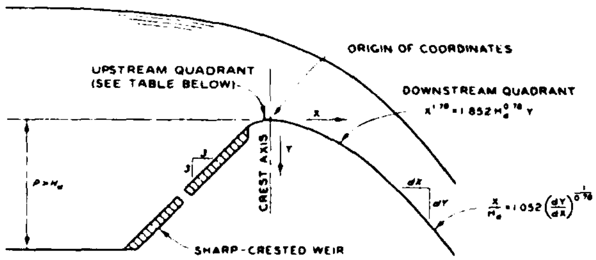
Figure 1. (Concluded)



a. 3V ON 1H



b. 3V ON 2H



Upstream Quadrant Coordinates

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

c 3V ON 3H

Figure 2. Crest shape, sloping face, negligible velocity of approach from Hydraulic Design Criteria (USACE 1952)

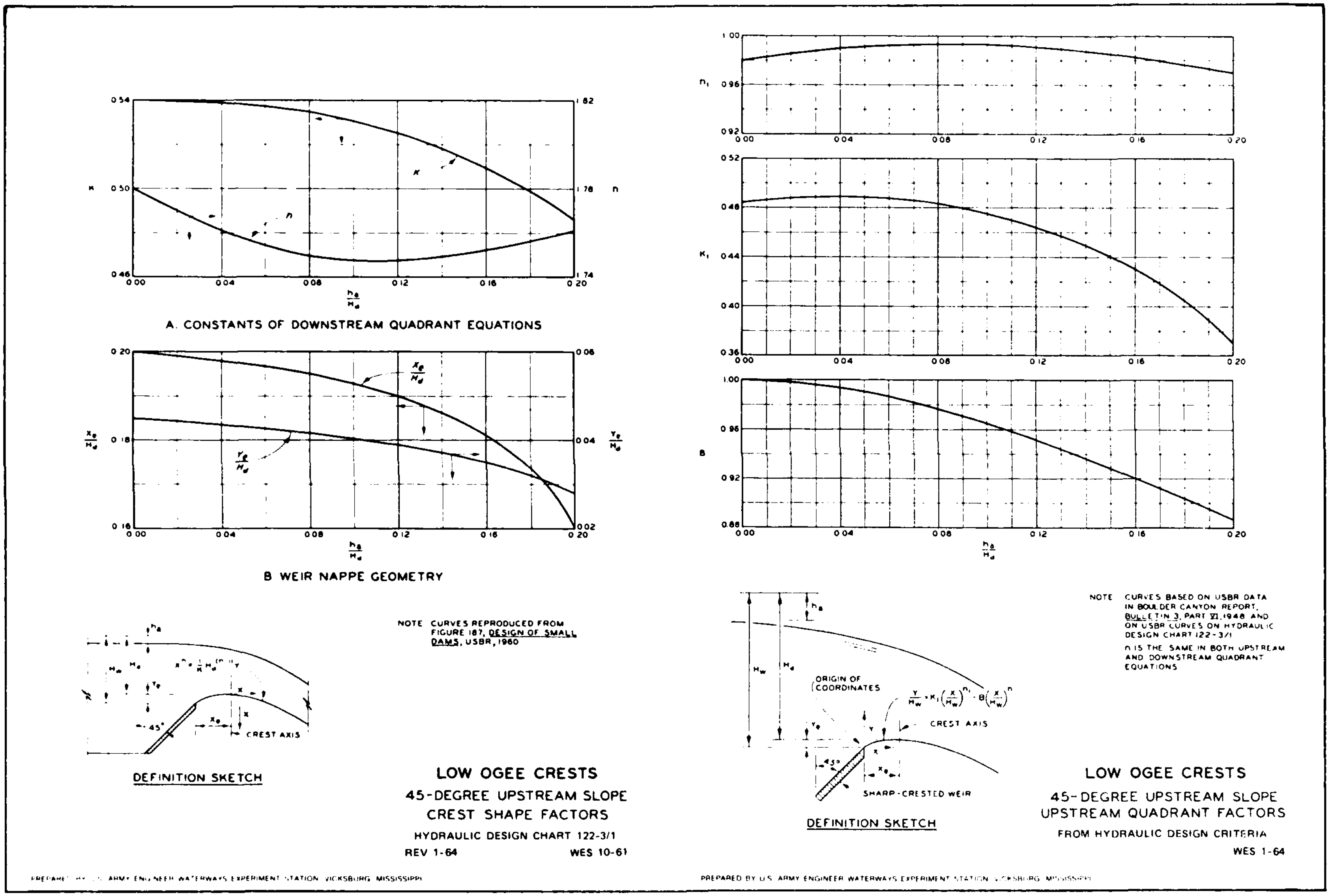


Figure 3. Crest shape, sloping face, low ogee crest from Hydraulic Design Criteria (USACE 1952)

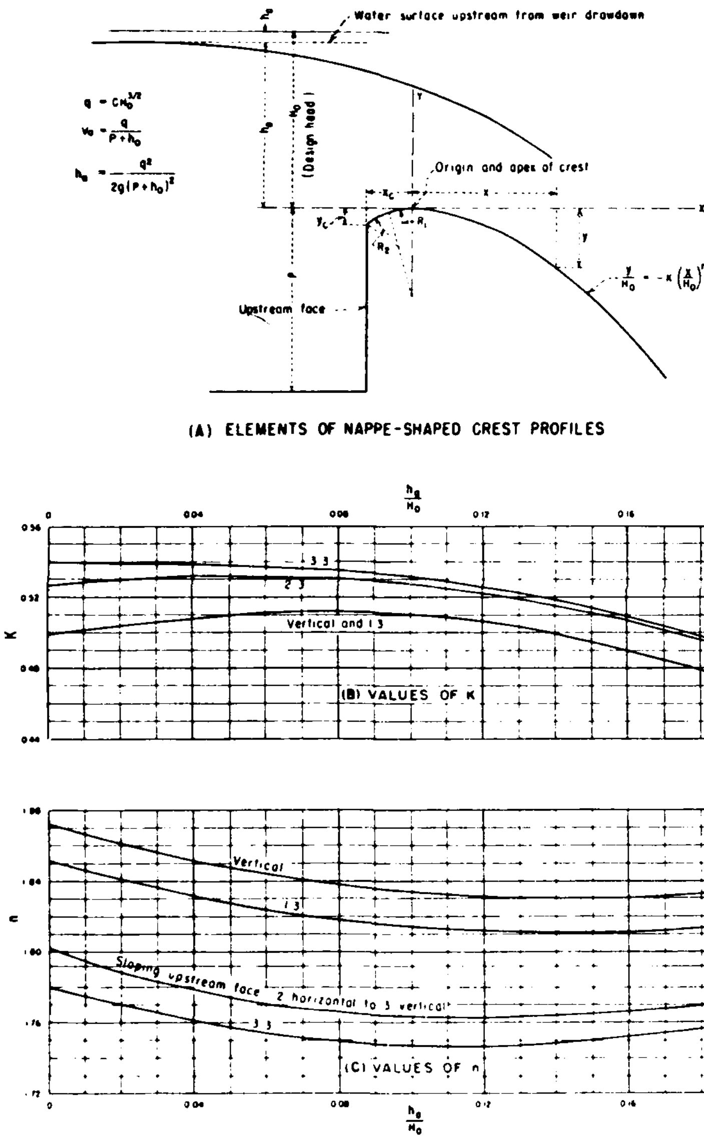


Figure 247 Factors for definition of nappe-shaped crest profiles 288-D-2406 (Sheet 1 of 2.)

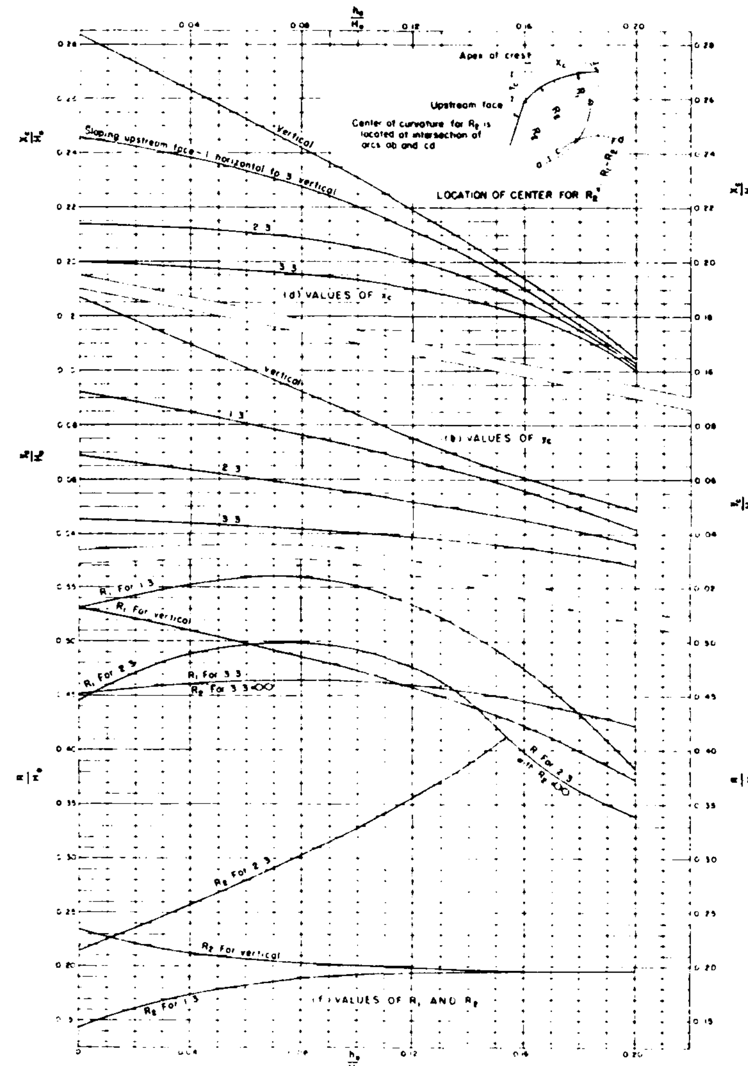


Figure 247 Factors for definition of nappe shaped crest profiles 288-D-2407 (Sheet 2 of 2.)

Figure 4. Crest shape by USBR (1974)

wide range of approach velocities and upstream face slopes. Crests with this surface discontinuity experience low pressures just downstream of the discontinuity (Abecasis 1961). Murphy (1973) proposed a design procedure using an elliptical upstream quadrant that is tangent to the upstream face and applicable to any approach velocity or upstream face slope. The axes of the ellipse were varied with approach velocity to obtain the best fit of the USBR data defining the lower nappe of flow over a sharp-crested weir. Plots of ellipse axes versus approach depths, all as ratios of design head, are shown in Figure 5. If a sloping upstream face of the spillway is desirable then an ellipse the same as that for a vertical face is used, and the sloping face is attached tangent to the ellipse. For the downstream quadrant, the values n and k in the Corps of Engineers equation

$$x^n = kH_d^{n-1}y$$

were determined using the USBR data. It was found that the USBR data for a vertical plate could be matched closely by maintaining n at a value of 1.85 for all depths of approach and varying k with depth of approach as shown in Figure 5. The downstream quadrants for spillways with sloped upstream face are designed using Figure 5 for vertical upstream face. A full development of this design procedure is found in Murphy (1973).

5. This design procedure has been verified for spillways having a negligible velocity of approach by Melsheimer and Murphy (1970). The objective of this investigation was to conduct systematic physical model tests to verify Murphy's procedure for spillways covering a wide range of approach velocities and upstream face slopes. Future references in his report to approach velocity or approach depth will be discussed in terms of the ratio P/H_d, the approach depth/design head on the crest. This report covers P/H_d values of 0.25 to 2.0. For each value of P/H_d, a range of upstream face slopes was tested. Discharge coefficients, pressures, and water-surface profiles were determined both with and without piers. Height of crest above the downstream apron floor was always greater than $2H_d$ so that free flow conditions were maintained. Either design head (H_d) or design energy head is used to denote the head on the crest which determines the shape of the crest and always includes the velocity head in the approach flow.

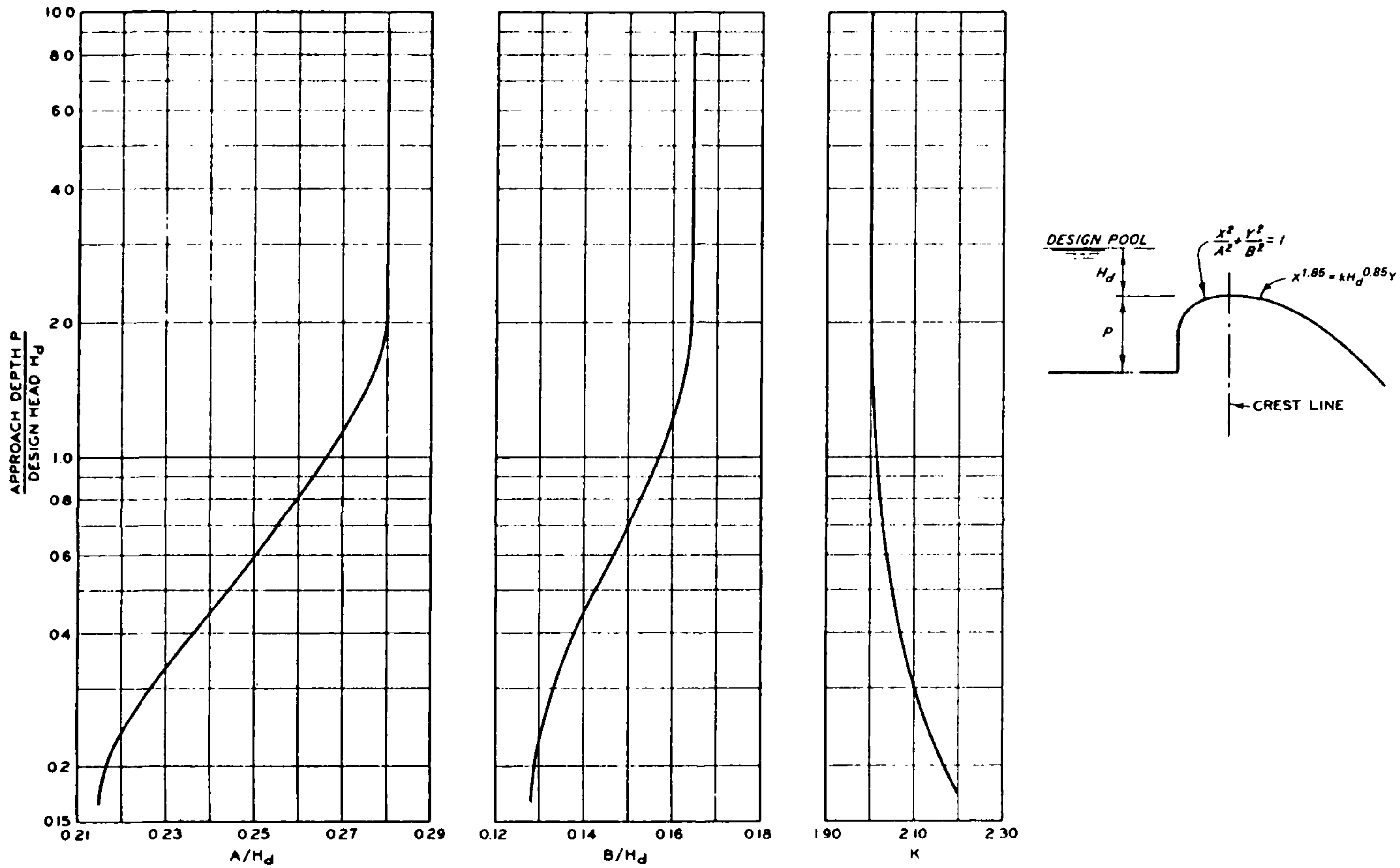


Figure 5. Spillway crest design curves (from Murphy 1973)

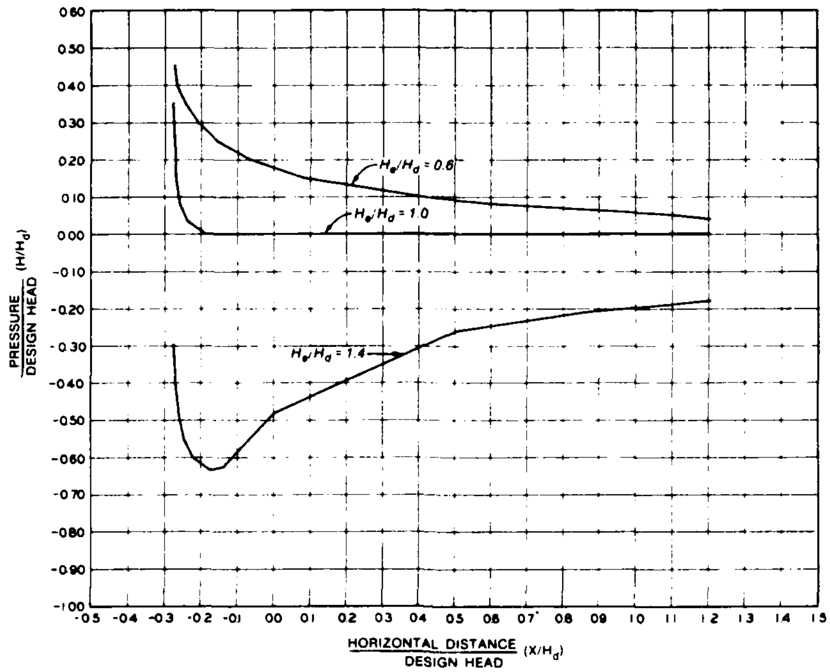
PART II: PERTINENT LITERATURE

6. Excellent history of the development of spillway crest design techniques can be found in Borland (1946), USBR (1948), Thorson (1953), and Cassidy (1964). A bibliography on the hydraulic design of spillways can be found in American Society of Civil Engineers (ASCE 1963). A literature search as part of the present investigation was conducted to identify the following:

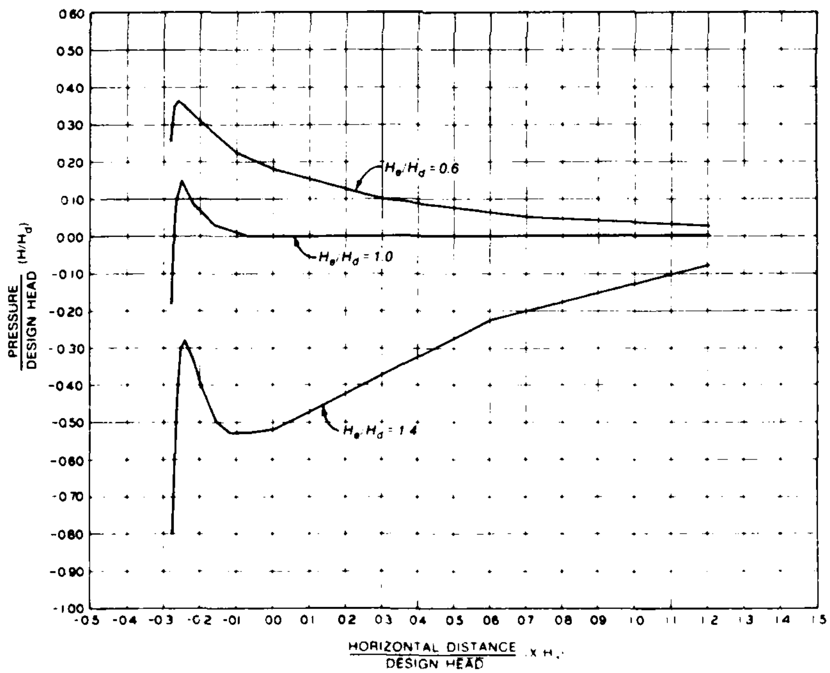
a. Past work to compare with the design procedure by Murphy (1973) (paragraphs 7 to 14).

b. Past work on scale effects and model-prototype comparisons of spillway performance to aid in selecting the proper model size to be used in this investigation (paragraphs 15 to 25).

7. The most important difference of the crest shape proposed by Murphy (1973) is the shape of the crest at the intersection of the curved portion of the crest and the upstream face. Lower nappe profiles over sharp-crested weirs (Bazin 1890, USBR 1948) show a sharp break in slope at this point. Most crest shape design procedures reflect this sharp break in slope (Figures 2 and 4). McCornmach (1968) reported that this sharp break in slope might result in rapid development of the boundary layer, an increase in the amount of air entrainment, and possibly a need for higher sidewalls in some designs. Murphy's procedure results in the curved portion of the crest being tangent with the upstream face for any P/H_d or upstream face slope. Rouse and Reid (1935) reported that the profile of the lower nappe over a sharp-crested weir is affected by surface tension at the weir crest that is not present with flow over a solid crest. Rouse and Reid (1935) slightly altered the upstream quadrant profile to obtain atmospheric pressure along the face of the spillway at the design head. This alteration also resulted in an increase in discharge coefficient. Abecasis (1961) conducted tests of various crest shapes to determine the effect of rounding the intersection of the upstream quadrant and the upstream face of the spillway. Abecasis' results showed that a $0.04H_d$ radius (Figure 1a) should be used at the intersection of the upstream face and upstream quadrants for both the USBR and WES spillways. This radius eliminated the area of high negative pressure located just downstream of the intersection of the upstream face. Pressures for the USBR profile with and without the $0.04H_d$ radius are shown in Figure 6. Abecasis' work was limited to vertical face spillways having negligible velocity of approach. Abecasis also conducted pressure measurements of various crest shapes including tests in a



WITH 0.04 H_d RADIUS



WITHOUT 0.04 H_d RADIUS

Figure 6. Crest pressures for USBR profile

vacuum channel to predict incipient cavitation along the face of a spillway crest (Figure 7).

8. Dillman (1933) used an ellipse to form the upstream quadrant of a high (large P/H_d) crest according to the equation

$$\frac{x^2}{(0.24K)^2} + \frac{y^2}{(0.11K)^2} = 1$$

where K is the index head measured above the sharp-crested weir used to shape the crest. In terms of H_d (measured above top of crest), Dillman's equation becomes

$$\frac{x^2}{\left(0.271H_d\right)^2} + \frac{y^2}{\left(0.130H_d\right)^2} = 1$$

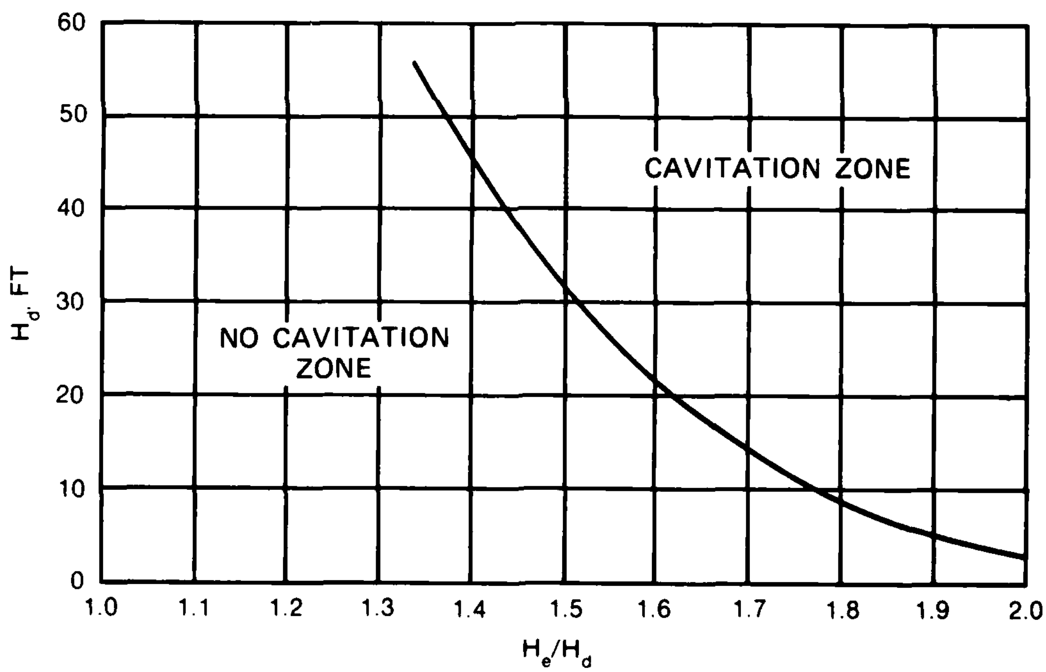


Figure 7. Proposed limit for cavitation based on tests in vacuum channel. Replotted from Abecasis (1961) USBR profile with $0.04H_d$ radius at upstream face

Murphy's (1973) equation for high spillways ($P/H_d \geq 2.0$) is

$$\frac{x^2}{\left(0.280H_d\right)^2} + \frac{y^2}{\left(0.167H_d\right)^2} = 1$$

which is similar to Dillman's equation. Dillman concluded that for the high spillway, higher discharge coefficients resulted from the vertical upstream face than with the inclined upstream face.

9. Bradley (1952) gives the effect of irregular spillway crest shapes for 34 free overfall spillways. Discharge coefficients from the original model study are compared with the discharge coefficient for the correct spillway shape. Included are data on Wilson Dam which used an ellipse on the upstream quadrant of the crest.

10. Thorson (1953) conducted tests of flow over sharp-crested weirs and found that the flow springs free from the weir face along a curve tangent to the weir face at the upstream edge of the crest. The radius of curvature is at first small but steadily increases with distance from the weir.

11. McNown, Hsu, and Yih (1955) used relaxation techniques to determine the profile of the lower nappe of flow over a very high sharp-crested weir. This method results in the lower nappe leaving the weir tangent to the upstream face of the crest and is the basis for the shape presented in Figure 1b.

12. Underdesigning the spillway crest is the procedure of selecting a design head which is less than the maximum head anticipated. This results in obtaining the higher discharge coefficients that occur at heads higher than the design head. The amount to which a crest may be underdesigned depends on the magnitude of the negative pressures that occur at heads higher than the design head. Webster (1959) reported on the use of underdesigned crests in the Pacific Northwest. Crests were designed for a head equal to 75 percent of the maximum head and significant savings in the cost of the structure were realized by reducing the number of gate bays required to pass the maximum discharge.

13. Cassidy (1964) compared results by means of potential flow theory with experimental data and found good agreement of water-surface profiles and pressure. Cassidy also concludes that the close agreement of the pressure distributions for the potential flow and physical models indicates that the effects of viscosity are negligible, and thus the pressure characteristics of

the spillway are functions of geometry alone for the size model spillway (H_d = 0.267 ft*) used in his investigation. Discharge coefficients for the potential flow theory were approximately 1 to 2 percent greater than those measured experimentally.

14. Cassidy (1970) reported on the benefits of designing a crest for a given negative pressure rather than some percentage of the maximum head as reported by Webster (1959).

15. Model scale effects and model-prototype conformity concerning flow over weirs and spillway crests have received considerable study over the years. The extensive research on flow over sharp-crested weirs by Schoder and Turner (1929) shows that heads classified as extremely low (<0.2 ft) deviate significantly from the predictive equations.

16. Nagler and Davis (1930) reported on a model-prototype comparison of a 1:11-scale model and Keokuk Dam. Model heads were greater than 0.4 ft for all discharges. The effect of surface roughness of the model crest was investigated and found to be negligible. Model and prototype discharges were within 1 percent. Groat (1930) states "hydraulic friction is of little or no account in its effects on weir discharge, unless the head is abnormally small or the kinematic viscosity abnormally large."

17. Eisner (1933) conducted overfall (weir) tests with sharp-crested weirs at various scale ratios and found that discharge coefficients determined at overfall heads of 0.16 ft differed from prototype discharges by up to 7 percent. Randolph (1938) found that prototype measurements of discharge resulted in values up to 13 percent higher than those measured in the model.

18. Hickox (1944) reported on a model-prototype comparison of Norris Dam and a 1:72-scale model by the USBR. Results indicated an average model discharge coefficient of 3.9 percent greater than that observed in the prototype. Stevens and Cochrane (1944) reported on a model-prototype comparison of Bonneville Dam and 1:5-scale model of a 10-ft-wide section of the prototype crest. Pressures along the face of the spillway were measured at the same locations in the model and in the prototype. A vertical control gate is used at Bonneville Dam and can be positioned either 9 ft upstream or 8 ft downstream of the center line of the crest. Tests were conducted with the gate at

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* A table of factors for converting U. S. customary units of measurements to metric (SI) units is presented on page 3.

each location in both the model and the prototype and the results were compared. When the gate was in the downstream position, the pressures "checked fairly well." When the gate was in the upstream position, there was considerable disagreement between model and prototype pressures.

19. Testing by Soucek (1944) compared a 1:12-scale model with the prototype which has heads up to 6.3 ft and concluded that the model discharges averaged approximately 5 percent less than comparable discharges obtained for the prototype. Soucek stated that it was by no means certain that the discrepancy is chargeable to the model and went on to say that "the discharge indicated by a well-designed spillway model, when the model head exceeds 3 or 4 in., is fully as accurate as that obtainable by any field method in current use." Johnson (1944) reported on a study where models having scales of 1:25, 1:40, and 1:100 were constructed of the Upper Narrows Dam. To investigate the effects of model roughness, fine sand was attached to the surface of the 1:25- and 1:40-scale models. Both the 1:25- and the 1:40-scale models predicted the same discharge relation with or without the roughened surface. The 1:100-scale model predicted discharge coefficients approximately 4 percent less than the larger models.

20. The USBR (1948) conducted extensive tests to measure the profile of the lower nappe of flow over a sharp-crested weir and used heads of greater than 0.4 ft in all but a few tests to eliminate scale effects that were attributed to surface tension effects.

21. Peterka (1954) reported a comparison of 1:21.5-scale model and the Heart Butte Dam morning glory spillway. Three measurements in the prototype resulted in differences of 4.6, 1.1, and 1.7 percent between model and prototype. A fourth reading at a lower discharge was 27.4 percent in error. However, several problems in obtaining an accurate reading of both stage and discharge in the prototype were noted for this fourth reading. Ownbey (1949) reported on three models, constructed at scale ratios of 1:50, 1:100, and 1:200, of the Pickwick Landing Spillway. Discharge coefficients for the three models were compared and no significant variation with model size was found. Manning (1951) conducted tests to determine viscous effects on flow over model weirs and reported that model heads below 0.30 ft are affected by viscous effects.

22. Guyton (1958) reported excellent agreement of model and prototype crest pressures at Chief Joseph Dam. These tests were conducted at 1.1 times

the design head for both gated and free flows. Lomax (1965) reported that the 1:50-scale model discharges averaged 1.1 percent less than the measured prototype discharges for Rock Island Dam. Maxwell and Weggel (1969) reported that surface tension has little effect on the limiting size of a Froude model. Other effects such as accuracy of reference data, accuracy of data obtained, and accuracy of model construction (including roughness) limit the size of model required before surface tension effects limit model size. The influence of viscosity may outweigh all of the above but viscosity was not studied in their investigation.

23. Sarginson (1972) reported that surface tension effects are negligible with heads above 0.10 ft for circular-crested weirs. Varshney (1977) reported on a method for estimating the discharge coefficient of a prototype spillway from a scale model depending on model Reynolds number and the formation of the boundary layer along the crest. Results by Varshney (1977) showed the following model discharge coefficients for a prototype having a design head of 1.63 ft and a design discharge coefficient $C_d = 3.97$:

Model Scale	Model Head	Model C_d	Percent Error
1:40	0.041	3.09	22
1:30	0.054	3.25	18
1:20	0.082	3.43	14
1:14	0.109	3.58	10
1:10	0.163	3.73	6

24. Raju and Asawa (1977) reported that viscous and surface tension effects are responsible for errors in sharp-crested weir measurements at low heads. At heads greater than 0.36 ft the effects are negligible. At heads of 0.2 ft predicted discharges are 5 percent low.

25. The following information is shown in Figure 8 relative to the effect of model head on discharge coefficient:

a. Results of testing different scale models by Varshney (1977) and Eisner (1933), and a comparison of discharge coefficients for model crest having design heads of 3.0 ft and 0.8 ft that were conducted for this investigation.

b. Model-prototype comparisons conducted by Randolph (1938), Soucek (1944), and Johnson (1944).

c. Unpublished results of Civil Works Investigation 801 (Figure 9). Available prototype data show that the discharge coefficient

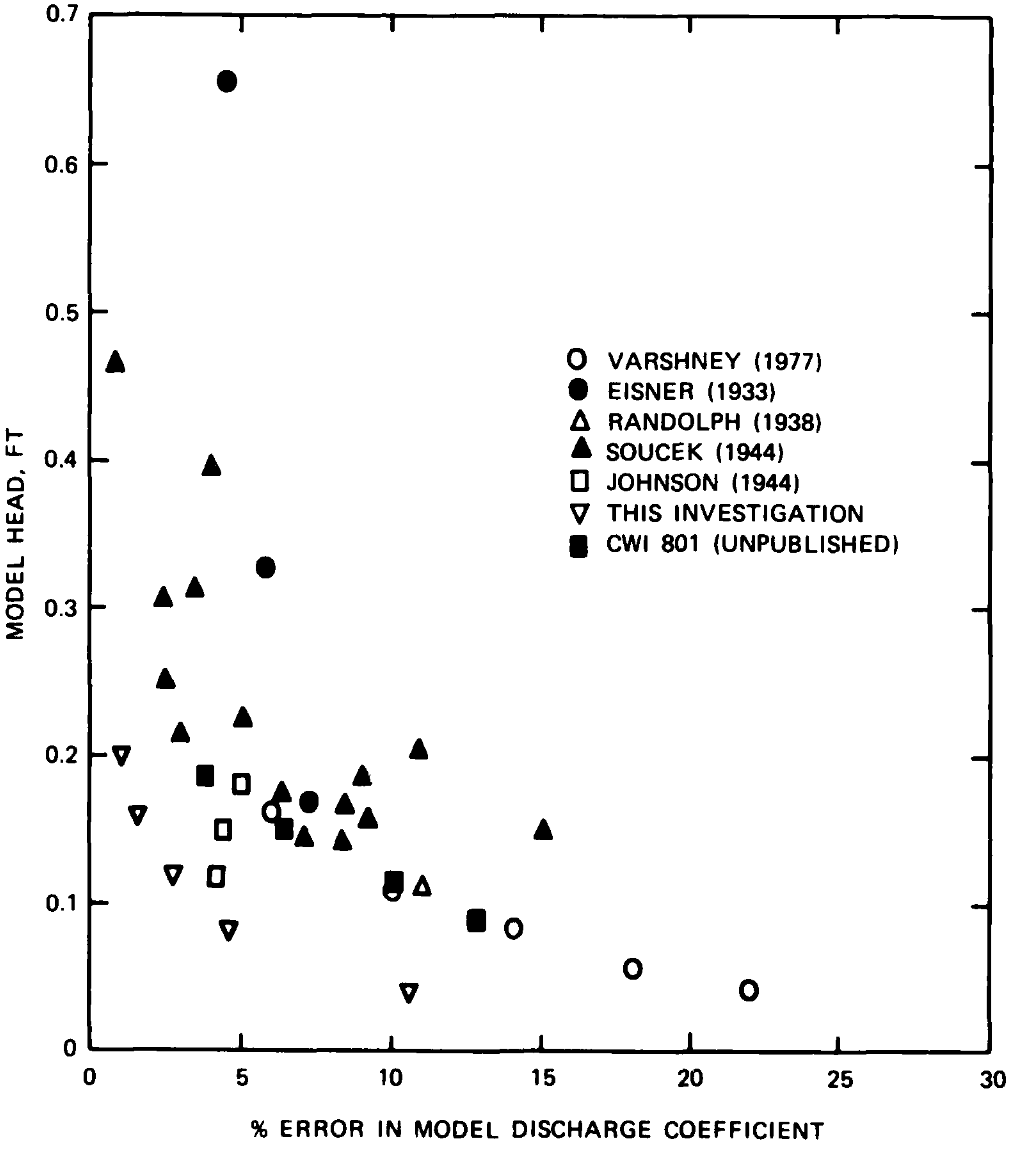

Figure 8. Percent error in model discharge coefficient

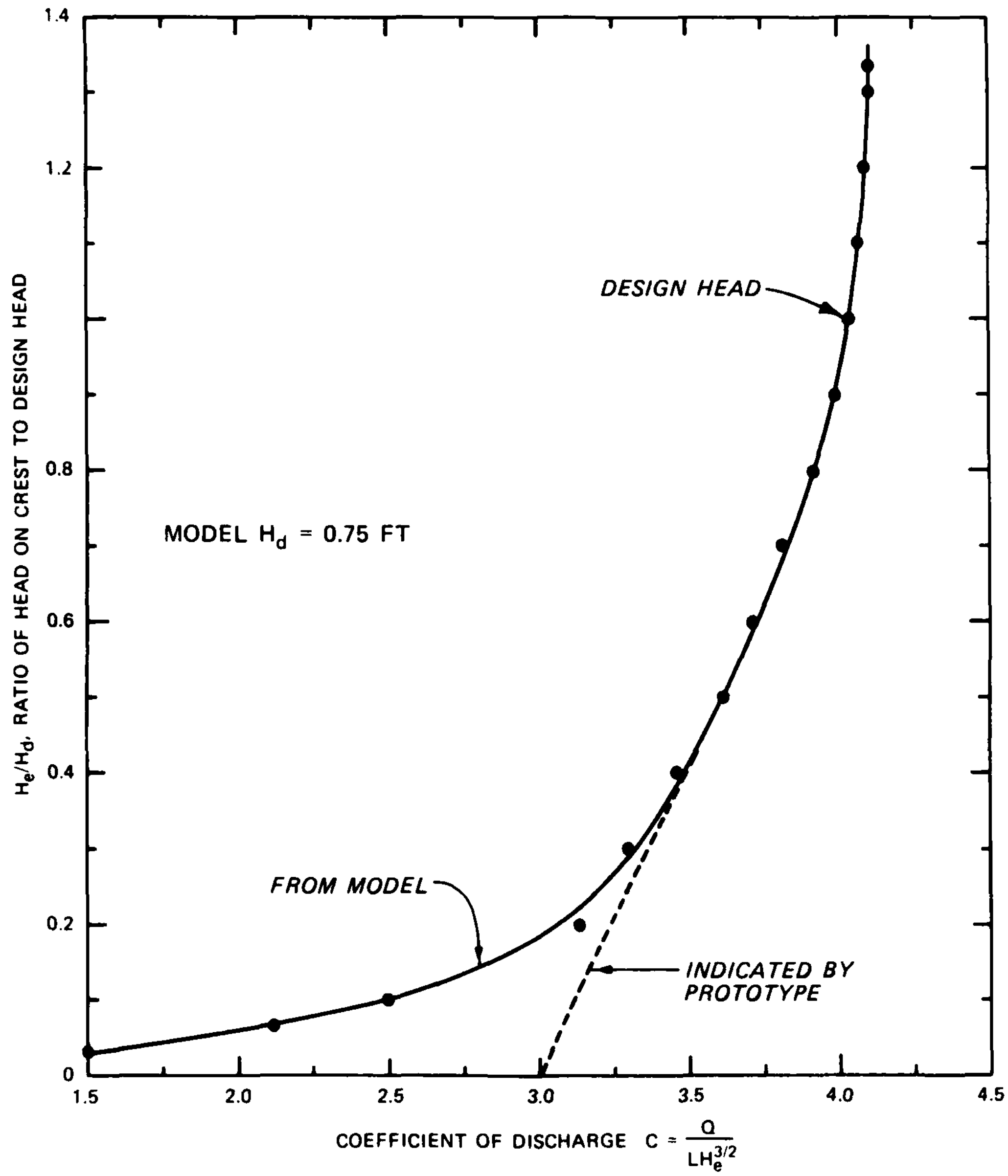

Figure 9. Results of unpublished Civil Works Investigation

for a round-crested weir approaches that for a broad-crested weir (C = 3.087) at heads on the crest approaching zero. This is shown by the dashed line in Figure 9 which is compared with the measured model coefficient by the solid line.

Figure 8 shows considerable scatter in the data but significant errors in model discharge coefficient can be expected at heads that are too small. For this investigation, a minimum head of 0.30 ft will be used to minimize model scale effects.

PART III: MODEL APPURTENANCES AND TEST PROCEDURES

26. A 2.5-ft-wide flume (Plate 1) with a horizontal floor and vertical sides lined with sheet metal was used in the investigation. Baffles were used at the inlet to distribute flow approaching the crest as uniformly as reasonably possible.

27 Discharges were measured in 8- by 4- and 20- by 10-in. venturi meters. These meters were calibrated against a 90-deg V-notch weir, and the empirical curve for the venturi meters and the measured curve based on the V-notch agreed to within less than 1 percent for both meters. Water-surface elevations were measured with a point gage mounted on rails set to grade along the sides of the flume.

28. The crests used in the investigation were constructed of sheet metal to conform to the shapes required for the design head of 0.80 ft. Simple piezometers were installed along the face of the crest to measure the average pressures acting on the crest. A typical crest with piezometers is shown in Plate 2.

29. Tests were conducted at discharges which resulted in values of H_e/H_d from about 0.4 to 1.5. Each test began with determination of the discharge coefficient C . Next, pressures along the face of the crest and water-surface profiles were measured. Then piers were attached to the crests and the pier contraction coefficients were determined. Finally, pressures and water-surface profiles were measured at the center line of the bay and along the piers.

PART IV: TEST RESULTS

Discharge Characteristics - Uncontrolled Crest

30. The spillway crests tested to determine discharge coefficients are shown in Table 1. An uncontrolled crest refers to a crest that does not have piers or gates that will affect or control flow. The following two alternatives were considered for analyzing the spillway data taken in this investigation:

a. Determine discharge coefficient C to fit the standard weir equation

$$Q = CLH_e^{1.5}$$

where

Q = discharge
C = discharge coefficient
L = crest length
H_e = energy head on crest

The discharge coefficient C will vary with approach depth, upstream face slope, and relative head on the crest, H_e/H_d. This approach with C usually given as a function of H_e/H_d is taken in the Hydraulic Design Criteria (USACE 1952) and is used in this investigation.

b. Determine C and λ to fit the following weir equation:

$$Q = CLH_e^{\lambda}$$

This alternative would be an improvement over the first if λ and/or C were constant for either approach depth, upstream slope, or relative head on the crest. This alternative was investigated at length and both λ and C vary with approach depth and relative head on the crest. Using this method as a general design procedure would be more difficult than the first method since there are two variables (λ, C) instead of one (C). Either technique, a or b, will result in the same accuracy in predicting discharge.

31. The Hydraulic Design Charts (HDC) utilize the power function

$$\frac{Q}{Q_d} = \left(\frac{H_e}{H_d}\right)^{1.6}$$

where

Q = discharge at H_e

Q_d = discharge at design energy head

H_e = energy head on crest

H_d = design energy head on crest

to determine discharge over the crest at heads other than the design head on the crest. Use of this method was considered for the present investigation but the power (1.6 in HDC) was found to vary with P/H_d. This method was compared with data given in the *HDC* for both high and low spillway crests (Plate 3), and the power function does not reproduce the C versus H_e/H_d curves and will not be used in this investigation. Borland (1946) showed that a power function relating C/C_d and H_e/H_d resulted in $C = 0$ at $H_e/H_d = 0$. At $H_e/H_d = 0$, C should be 3.08 or slightly less due to friction effects.

32. Model discharge, static head, and computed energy head are shown in Tables 2-5 for P/H_d of 0.25, 0.50, 1.0, and 2.0, respectively. Static head measurements for the $P/H_d = 0.25$ crests were difficult at heads greater than 0.8 ft because of significant turbulence in the approach flow. The energy correction factor α was assumed equal to 1.0 for all computations. The relationship of discharge coefficients as a function of the relative head on the crest, H_e/H_d, is shown in Plates 4-7 for P/H_d of 0.25, 0.50, 1.0, and 2.0, respectively. Murphy's procedure (Figure 5) was used for the crest shapes (see example in Plate 2).

33. The curves of Plates 4-7 were used to develop the suggested design curves shown in Plates 8 and 9 for upstream face slopes of vertical and 1V on 1H, respectively. Because of the possible scale effects mentioned earlier, discharge coefficients were not available at $H_e/H_d < 0.4$. Prototype experience has shown that spillway crests at very low heads exhibit the same discharge characteristics as a broad-crested weir. This indicates that the discharge coefficient at $H_e/H_d = 0$ should be 3.08 and this value was used in Plates 8 and 9. The curve for $H_e/H_d = 0.2$ was developed by interpolating between model discharge coefficient data at $H_e/H_d = 0.4$ and the prototype $C = 3.08$ at $H_e/H_d = 0$.

Crest Pressures - Uncontrolled Crest

34. Pressures along the face of the uncontrolled crests were measured for the following crests and are shown in the plates indicated:

P/H_d	Upstream Face Slope	Plate No.
0.25	Vertical	10
0.25	1V on 1H	10
0.50	Vertical	11
0.50	1V on 1H	11
1.00	Vertical	12
1.00	1V on 1H	12

In Plate 13, crest pressures from Melsheimer and Murphy (1970) are shown for a crest with P/H_d of 3.4.

35. These pressures indicate variations, some of which are caused by the method of constructing the model crests. As pointed out in PART III, the crests were constructed of sheet metal. Plastic crests were used by Melsheimer and Murphy (1970) in their tests and were considered the best method of reproducing the true shape of the crest. The chance for an imperfect shape existed even with the machined plastic crests because a machine buffing of the plastic crests formed by a planer was necessary to smooth out the individual cuts and steps used to approximate the true crest shape. Considering the number of crests needed and used in this investigation, the use of machined plastic crests would have resulted in costs above available funds. Sheet-metal crests are much less difficult to construct and give a good representation of the crest shape. The only problem is that the sheet metal may not bend in a true arc but in a series of small cords which can lead to local variation of the pressures measured on the crest. This problem may have a positive aspect in that prototype construction methods may not result in relative tolerances any better than the sheet-metal crests used in the model.

36. At $P/H_d = 0.25$, pressures were only measured for $H_e/H_d = 0.50$ and 1.0 because using an underdesigned crest for this low P/H_d crest does not result in increased discharge coefficients above $H_e/H_d = 1.0$.

37. From pressures for P/H_d of 0.50 to 3.4 the maximum negative pressure for each value of H_e/H_d does not vary with P/H_d. The maximum

negative pressure as a function of H_e/H_d is shown in Plate 14 for values of P/H_d greater than 0.50. The spillway design head H_d is selected so that the minimum crest pressure for the maximum expected head H_e is no lower than -15 to -20 ft of water to ensure cavitation-free operation. Comparison of Plate 14 and the work of Abecasis (1961) (Figure 7) shows that his recommended curve corresponds to a minimum crest pressure of -25 ft of water.

Water-Surface Profiles - Uncontrolled Crest

38. Water-surface profile data for uncontrolled crests were taken for crests with P/H_d of 0.25, 0.50, and 1.0 and are shown in Plate 15. Water-surface profile data for high crests can be found in Hydraulic Design Criteria (USACE 1952). Different upstream face slopes had little effect on water-surface elevations for use in designing walls, determining clearances, etc.

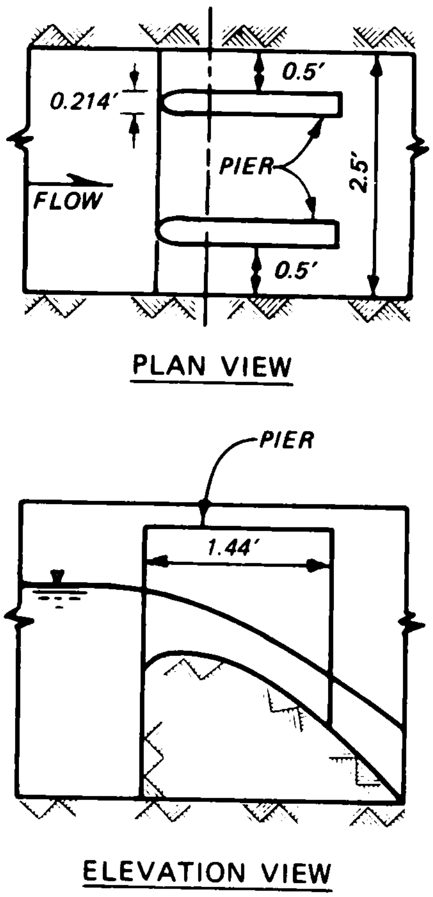

Figure 10. Crest pier details

Pier Contraction Coefficients

39. Pier contraction coefficients K_p were determined for P/H_d of 0.25, 0.50, and 1.0. Two piers were placed on the 2.5-ft-wide crests and located as shown in Figure 10. The pier nose used for all crests was the type 3 shown in HDC 111-5. The pier nose was located in the same plane as the upstream face for the vertical spillway. For the 1V-on-1H upstream slope, the pier nose location was determined by maintaining the same distance from pier nose to crest axis as used in the vertical upstream-faced crest. Basic data for upstream face slopes of vertical and 1V on 1H and P/H_d of 0.25, 0.50, and 1.0 are shown in Tables 6-8. A plot of K_p as a function of H_e/H_d is shown in Plate 16.

Crest Pressures - Controlled Crest

40. A controlled crest is one having piers that affect flow over the crest. The effects of spillway gates were not included in this investigation. Pressures were measured along the face of the controlled crest for the following crests and are shown in the plates indicated:

P/H_d	Upstream Face Slope	Location	Plate
0.25	Vertical	Center line of bay	17
0.25	Vertical	Along piers	17
0.25	1V on 1H	Center line of bay	17
0.25	1V on 1H	Along piers	17
0.50	Vertical	Center line of bay	18
0.50	Vertical	Along piers	18
0.50	1V on 1H	Center line of bay	18
0.50	1V on 1H	Along piers	18
1.0	Vertical	Center line of bay	19
1.0	Vertical	Along piers	19
1.0	1V on 1H	Center line of bay	19
1.0	1V on 1H	Along piers	19

Piezometers along the piers were located 0.02 ft from the edge of the pier to the center line of the hole. Plates 20 and 21 are the plots of crest pressures for a P/H_d of 3.4 for center line of gate bay and along piers, respectively, taken from Melsheimer and Murphy (1970).

Water-Surface Profiles - Controlled Crest

41. Water-surface profile data for controlled crests were taken from the following crests:

P/H_d	Plate
0.25	22
0.50	23
1.0	24

Water-surface profile data for high controlled crests can also be found in Hydraulic Design Criteria (USACE 1952).

PART V: DISCUSSION OF RESULTS

42. A search of the literature confirms the accuracy of model tests in predicting prototype performance of spillway crests if the model heads are not too low. It is likely that the accuracy of the model crest is within the accuracy of prototype measuring techniques particularly when comparing discharge over the crest.

43. Testing of a relatively low crest having a $P/H_d = 0.25$ resulted in no significant increase in discharge coefficients above $H_e/H_d = 1$.

44. Discharge coefficients increase with P/H_d for the same crest shape and upstream face slope. For $P/H_d > 0.25$, the flatter upstream slopes result in lower discharge coefficients. The concept of an underdesigned crest should be utilized for $P/H_d \geq 0.5$ since discharge coefficients increase above $H_e/H_d = 1.0$. Use of the power function presented in Hydraulic Design Criteria (USACE 1952) for determining discharges at heads other than the design head resulted in poor correlation with measured values. The relationship of C, H_e/H_d, P/H_d, and upstream face slope are presented in Plates 8 and 9 and result in greater accuracy than the power function.

45. Pressures on the uncontrolled crest are close to atmospheric at $H_e/H_d = 1.0$ which confirms the shape proposed by Murphy (1973). Pressures become negative at heads greater than the design head at a point on the crest located just upstream of the crest axis.

46. Pier contraction coefficients should be used with caution because the angle of flow approaching the spillway has a significant bearing on the pier coefficients.

47. Negative pressures at the center line of the bay do not become adversely negative when underdesigning the crest. Negative pressures along the piers are similar in magnitude to the uncontrolled crest and must be considered when underdesigning a controlled crest. One caution is to extend the piers on the crest far enough downstream beyond the location of low pressure, or aeration of the nappe can occur leading to possible separation and flutter. Gate slots should also be away from the core of low pressure. Aeration of the nappe occurred in the model when the gate slots were centered on the crest.

48. These test results provide a range of discharge coefficients, pressures, and water-surface profiles that allow Murphy's (1973) design procedure to be used for a wide range of spillway crest designs.

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Table 1

Crests Tested to Determine Discharge Coefficients

Crest Number	P/H_d	Upstream Face Slope
1	0.25	Vertical
2	0.25	3V on 2H
3	0.25	1V on 1H
4	0.25	2V on 3H
5	0.25	1V on 2H
6	0.50	Vertical
7	0.50	4V on 1H
8	0.50	3V on 2H
9	0.50	1V on 1H
10	0.50	2V on 3H
11	1.00	Vertical
12	1.00	4V on 1H
13	1.00	2V on 1H
14	1.00	1V on 1H
15	1.00	2V on 3H
16	2.00	Vertical
17	2.00	1V on 1H

Table 2

Basic Data, $P/H_d = 0.25$, No Piers

Vertical			3V on 2H			1V on 1H			2V on 3H			1V on 2H		
Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft
1.51	0.292*	0.315	1.49	0.285*	0.308	1.51	0.288*	0.312	1.49	0.286*	0.309	1.60	0.303*	0.328
1.97	0.341*	0.374	2.01	0.337*	0.372	2.01	0.338*	0.373	2.01	0.339*	0.374	2.02	0.341*	0.376
2.51	0.385*	0.431	2.51	0.381*	0.427	2.50	0.381*	0.427	2.55	0.386*	0.433	2.48	0.384*	0.429
2.99	0.419*	0.477	2.97	0.415*	0.473	3.01	0.416*	0.475	2.98	0.417*	0.475	2.98	0.421*	0.478
3.52	0.452*	0.525	3.52	0.452*	0.525	3.54	0.454*	0.527	3.52	0.458*	0.529	3.52	0.461*	0.532
4.01	0.484*	0.569	3.99	0.479*	0.565	3.98	0.479*	0.564	4.00	0.483*	0.568	3.98	0.485*	0.569
4.51	0.517*	0.615	4.54	0.513*	0.614	4.51	0.514*	0.613	4.54	0.519*	0.618	4.52	0.517*	0.616
5.04	0.545*	0.659	5.06	0.541*	0.657	5.05	0.540*	0.656	5.05	0.546*	0.660	5.07	0.547*	0.662
5.53	0.574*	0.701	5.51	0.573*	0.699	5.54	0.564*	0.695	5.52	0.585*	0.708	5.51	0.585*	0.708
6.01	0.600*	0.740	6.01	0.593*	0.736	5.97	0.595*	0.735	6.04	0.595*	0.739	6.05	0.602*	0.744
6.54	0.623*	0.780	6.50	0.616**	0.774	6.38	0.618**	0.769	6.46	0.630**	0.781	6.54	0.622**	0.779
7.06	0.645**	0.819	7.02	0.631**	0.809	6.97	0.628**	0.804	6.99	0.650**	0.818	7.06	0.651**	0.822
7.59	0.683**	0.867	7.61	0.674**	0.863	7.55	0.671**	0.858	7.61	0.684**	0.868	7.68	0.688**	0.874
8.13	0.671**	0.888	8.11	0.662**	0.882	7.94	0.656**	0.870	8.14	0.669**	0.887	8.07	0.671**	0.885
8.65	0.691**	0.925	8.68	0.697**	0.930	8.68	0.699**	0.931	8.62	0.705**	0.931	8.68	0.704**	0.933
9.02	0.715**	0.957	9.06	0.717**	0.960	9.02	0.714**	0.956	9.11	0.722**	0.965	9.13	0.723**	0.966
9.48	0.736**	0.991	9.51	0.738**	0.994	9.57	0.739**	0.997	9.57	0.743**	0.999	9.46	0.743**	0.993
9.98	0.755**	1.027	10.03	0.763**	1.033	10.03	0.756**	1.03	9.98	0.762**	1.030	10.08	0.766**	1.037
10.55	0.771**	1.065	10.58	0.775**	1.068	10.53	0.768**	1.062	10.62	0.778**	1.071	10.38	0.777**	1.058
10.95	0.764**	1.085	11.05	0.772**	1.093	10.91	0.764**	1.083	11.00	0.770**	1.090	11.01	0.776**	1.093
11.52	0.800**	1.13	11.50	0.798**	1.128	11.50	0.806**	1.131	11.45	0.800**	1.126	11.54	0.810**	1.135
12.14	0.849**	1.182	12.02	0.842**	1.173	11.97	0.846**	1.172	11.97	0.844**	1.171	11.97	0.849**	1.173
12.54	0.881**	1.216	12.64	0.887**	1.223	12.43	0.887**	1.212	12.51	0.881**	1.214	12.47	0.888**	1.215
12.99	0.916**	1.253	12.99	0.914**	1.252	12.91	0.917**	1.249	13.15	0.924**	1.264	12.98	0.916**	1.253
13.60	0.944**	1.296	13.60	0.948**	1.297	13.57	0.942**	1.293	13.64	0.951**	1.300	13.42	0.947**	1.288

(All five slope groups fall under the heading: Upstream Face Slope)

Note: $H_d = 0.8$ ft.

* Measured head on crest excluding velocity head measured $4.0 \times H_d$ upstream of crest.

** Represents average of six readings of stage.

Table 3

Basic Data, $P/H_d = 0.5$, No Piers

Upstream Face Slope														
Vertical			4V on 1H			3V on 2H			1V on 1H			2V on 3H		
Q	H	H_e	Q	H	H_e	Q	H	H_e	Q	H	H_e	Q	H	H_e
cfs	ft	ft	cfs	ft	ft	cfs	ft	ft	cfs	ft	ft	cfs	ft	ft
1.51	0.300*	0.312	1.51	0.297*	0.309	1.51	0.300*	0.312	1.51	0.301*	0.313	1.51	0.301*	0.313
2.01	0.354*	0.372	2.01	0.350*	0.368	2.01	0.353*	0.371	2.01	0.355*	0.373	2.01	0.356*	0.374
2.50	0.401*	0.425	2.50	0.397*	0.421	2.50	0.400*	0.424	2.50	0.403*	0.427	2.50	0.403*	0.427
2.98	0.438*	0.469	2.98	0.447*	0.478	2.98	0.437*	0.469	2.98	0.434*	0.466	2.98	0.441*	0.472
3.52	0.484*	0.523	3.52	0.474*	0.514	3.52	0.482*	0.522	3.52	0.474*	0.514	3.52	0.479*	0.519
3.98	0.518*	0.565	4.02	0.512*	0.560	3.98	0.521*	0.567	3.98	0.521*	0.567	3.98	0.522*	0.568
4.54	0.560*	0.616	4.50	0.547*	0.603	4.50	0.556*	0.611	4.50	0.559*	0.614	4.50	0.556*	0.611
5.03	0.589*	0.653	5.03	0.586*	0.651	5.03	0.587*	0.652	5.03	0.591*	0.655	5.03	0.593*	0.657
5.52	0.618*	0.691	5.54	0.616*	0.690	5.52	0.617*	0.690	5.52	0.625*	0.697	5.52	0.633*	0.704
6.01	0.639*	0.722	6.04	0.634*	0.719	6.01	0.639*	0.722	6.01	0.646*	0.728	6.01	0.651*	0.732
6.54	0.666**	0.760	6.54	0.669**	0.762	6.54	0.678**	0.770	6.54	0.671**	0.764	6.54	0.678**	0.770
6.99	0.698**	0.799	6.91	0.696**	0.795	7.13	0.700**	0.805	6.99	0.698**	0.799	6.99	0.705**	0.805
7.55	0.723**	0.835	7.48	0.721**	0.832	7.55	0.730**	0.841	7.55	0.733**	0.843	7.55	0.740**	0.849
8.07	0.753**	0.875	8.07	0.751**	0.873	8.07	0.757**	0.878	8.07	0.759**	0.880	8.07	0.778**	0.895
8.56	0.772**	0.905	8.68	0.778**	0.913	8.56	0.775**	0.907	8.56	0.787**	0.916	8.56	0.795**	0.923
9.02	0.796**	0.937	9.02	0.799**	0.940	9.02	0.798**	0.939	9.02	0.807**	0.946	9.02	0.813**	0.951
9.46	0.813**	0.964	9.46	0.816**	0.967	9.46	0.814**	0.965	9.46	0.820**	0.970	9.46	0.831**	0.978
9.98	0.842**	1.003	9.98	0.841**	1.002	9.98	0.836**	0.998	9.98	0.852**	1.01	9.98	0.857**	1.014
10.58	0.859**	1.035	10.43	0.854**	1.026	10.48	0.862**	1.034	10.48	0.866**	1.036	10.58	0.874**	1.046
10.91	0.880**	1.061	10.96	0.877**	1.060	10.95	0.869**	1.054	10.95	0.896**	1.074	10.95	0.895**	1.073
11.50	0.902**	1.096	11.46	0.898**	1.092	11.50	0.905**	1.098	11.50	0.910**	1.102	11.50	0.918**	1.107
12.02	0.909**	1.119	11.97	0.901**	1.112	12.02	0.913**	1.121	12.02	0.921**	1.127	12.02	0.929**	1.132
12.60	0.932**	1.155	12.51	0.926**	1.147	12.51	0.926**	1.147	12.51	0.941**	1.157	12.51	0.939**	1.156
13.07	0.952**	1.184	12.99	0.954**	1.183	12.99	0.953**	1.182	12.99	0.954**	1.183	12.99	0.955**	1.184
13.53	0.958**	1.205	13.53	0.970**	1.213	13.53	0.974**	1.215	13.53	0.982**	1.220	13.53	0.992**	1.227

Note: $H_d = 0.8$ ft.

* Measured head on crest excluding velocity head measured $4.0 \times H_d$ upstream of crest.

** Represents average of six readings of stage.

Table 4

Basic Data, $P/H_d = 1.0$, No Piers

Upstream Face Slope														
Vertical			4V on 1H			2V on 1H			1V on 1H			2V on 3H		
Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft
1.50	0.307*	0.312	1.51	0.305*	0.310	1.51	0.305*	0.310	1.51	0.307*	0.312	1.51	0.308*	0.313
2.01	0.366*	0.373	2.01	0.367*	0.374	2.01	0.362*	0.369	2.02	0.366*	0.373	2.01	0.365*	0.372
2.50	0.414*	0.425	2.50	0.414*	0.425	2.50	0.412*	0.423	2.50	0.415*	0.426	2.50	0.416*	0.427
2.99	0.457*	0.471	3.03	0.461*	0.475	2.98	0.447*	0.461	2.98	0.459*	0.473	2.99	0.460*	0.474
3.51	0.501*	0.519	3.52	0.506*	0.524	3.52	0.500*	0.518	3.54	0.509*	0.527	3.54	0.511*	0.529
3.99	0.541*	0.563	3.98	0.540*	0.562	4.00	0.538*	0.560	3.98	0.545*	0.567	3.98	0.545*	0.567
4.50	0.579*	0.605	4.52	0.579*	0.606	5.50	0.575*	0.602	4.49	0.585*	0.611	4.50	0.585*	0.611
5.05	0.619*	0.651	5.05	0.618*	0.650	5.03	0.612*	0.644	5.07	0.626*	0.657	5.06	0.625*	0.656
5.53	0.653*	0.689	5.52	0.648*	0.684	5.49	0.646*	0.682	5.53	0.657*	0.693	5.52	0.659*	0.695
6.03	0.686*	0.727	6.01	0.683*	0.724	5.98	0.678*	0.719	6.02	0.691*	0.732	6.01	0.691*	0.731
6.54	0.716**	0.762	6.54	0.719**	0.765	6.54	0.716**	0.762	6.61	0.724**	0.771	6.69	0.728**	0.776
6.99	0.746**	0.797	6.99	0.746**	0.797	7.06	0.752**	0.803	7.06	0.754**	0.805	6.99	0.747**	0.798
7.68	0.785**	0.843	7.55	0.772**	0.829	7.55	0.773**	0.830	7.61	0.786**	0.843	7.55	0.781**	0.838
8.09	0.810**	0.873	8.07	0.803**	0.866	8.07	0.801**	0.864	7.94	0.807**	0.868	8.13	0.808**	0.872
8.56	0.835**	0.903	8.56	0.830**	0.899	8.56	0.828**	0.897	8.62	0.841**	0.910	8.56	0.837**	0.905
9.02	0.858**	0.932	9.02	0.860**	0.933	8.91	0.858**	0.930	9.02	0.861**	0.934	8.96	0.864**	0.936
9.46	0.882**	0.961	9.46	0.881**	0.960	9.51	0.877**	0.957	9.46	0.886**	0.964	9.46	0.890**	0.968
9.98	0.907**	0.992	10.01	0.906**	0.992	9.98	0.904**	0.989	10.08	0.917**	1.003	9.98	0.914**	0.998
10.50	0.933**	1.024	10.48	0.927**	1.019	10.48	0.929**	1.020	10.53	0.937**	1.028	10.48	0.940**	1.03
10.91	0.954**	1.050	10.95	0.950**	1.047	11.00	0.958**	1.055	11.00	0.963**	1.060	10.95	0.960**	1.056
11.59	0.980**	1.085	11.50	0.980**	1.084	11.50	0.974**	1.079	11.54	0.990**	1.093	11.50	0.993**	1.095
12.02	1.002**	1.113	12.02	0.999**	1.110	12.06	1.003**	1.114	12.06	1.010**	1.120	12.02	1.013**	1.122
12.47	1.025**	1.141	12.51	1.022**	1.139	12.43	1.023**	1.139	12.56	1.033**	1.150	12.51	1.033**	1.149
12.99	1.042**	1.166	12.99	1.039**	1.163	12.91	1.041**	1.163	13.07	1.056**	1.179	13.03	1.053**	1.176
13.53	1.068**	1.199	13.53	1.064**	1.195	13.45	1.058**	1.188	13.60	1.079**	1.209	13.53	1.082**	1.211

Note: $H_d = 0.8$ ft.
* Measured head on crest excluding velocity head measured $4.0 \times H_d$ upstream of crest.
** Represents average of six readings of stage.

Table 5

Basic Data, $P/H_d = 2.0$, No Piers

Upstream Face Slope					
Vertical			1V on 1H		
Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft
1.52	0.316*	0.318	1.49	0.311*	0.313
2.00	0.372*	0.375	2.01	0.373*	0.376
2.50	0.424*	0.428	2.49	0.424*	0.428
2.98	0.466*	0.471	3.00	0.473*	0.478
3.52	0.517*	0.524	3.54	0.519*	0.526
3.96	0.554*	0.562	3.98	0.564*	0.572
4.47	0.592*	0.602	4.50	0.602*	0.612
5.03	0.635*	0.648	5.00	0.633*	0.645
5.53	0.669*	0.684	5.51	0.672*	0.687
6.00	0.707*	0.724	5.98	0.710*	0.727
6.54	0.739**	0.758	6.66	0.752**	0.772
6.99	0.770**	0.792	6.97	0.772**	0.793
7.61	0.809**	0.834	7.41	0.797**	0.821
8.07	0.832**	0.859	8.04	0.838**	0.865
8.68	0.868**	0.899	8.60	0.876**	0.906
9.02	0.895**	0.928	9.02	0.897**	0.929
9.44	0.913**	0.948	9.49	0.924**	0.959
9.88	0.946**	0.983	9.98	0.955**	0.993
10.42	0.968**	1.009	10.53	0.986**	1.027
10.86	0.990**	1.034	10.91	1.007**	1.051
11.62	1.030**	1.079	11.54	1.041**	1.088
11.85	1.046**	1.096	11.97	1.064**	1.114
12.43	1.072**	1.126	12.47	1.075**	1.129
13.07	1.106**	1.164	12.95	1.109**	1.166
13.58	1.127**	1.189	13.49	1.135**	1.196

Note: $H_d = 0.8$ ft.
* Measured head on crest excluding velocity head measured $4.0 \times H_d$ upstream of crest.
** Represents average of six readings of stage.

Table 6

Basic Data, $P/H_d = 0.25$, with Piers

Upstream Face Slope: Vertical, Q cfs	Vertical, H ft	Vertical, H_e ft	1V on 1H, Q cfs	1V on 1H, H ft	1V on 1H, H_e ft
1.51	0.333	0.353	1.52	0.330	0.350
2.01	0.392	0.421	2.02	0.392	0.421
2.50	0.443	0.481	2.52	0.444	0.482
2.98	0.488	0.535	2.98	0.483	0.530
3.52	0.533	0.590	3.52	0.526	0.584
3.98	0.564	0.632	3.98	0.559	0.627
4.50	0.599	0.678	4.50	0.598	0.677
5.03	0.632	0.723	5.03	0.634	0.724
5.52	0.664	0.766	5.52	0.662	0.764
6.01	0.689	0.803	6.01	0.682	0.798
6.54	0.717	0.844	6.54	0.715	0.842
6.99	0.739	0.877	6.99	0.741	0.878
7.55	0.759	0.913	7.55	0.758	0.913
8.07	0.789	0.955	8.07	0.789	0.955
8.56	0.813	0.991	8.56	0.800	0.982
9.02	0.837	1.025	9.02	0.833	1.023
9.46	0.842	1.047	9.46	0.847	1.050
9.98	0.850	1.075	9.98	0.853	1.076
10.48	0.886	1.118	10.48	0.871	1.109
10.96	0.902	1.148	10.95	0.904	1.149
11.50	0.917	1.181	11.50	0.927	1.186
12.10	0.945	1.223	12.02	0.946	1.220

Table 7

Basic Data, $P/H_d = 0.50$, with Piers

Upstream Face Slope

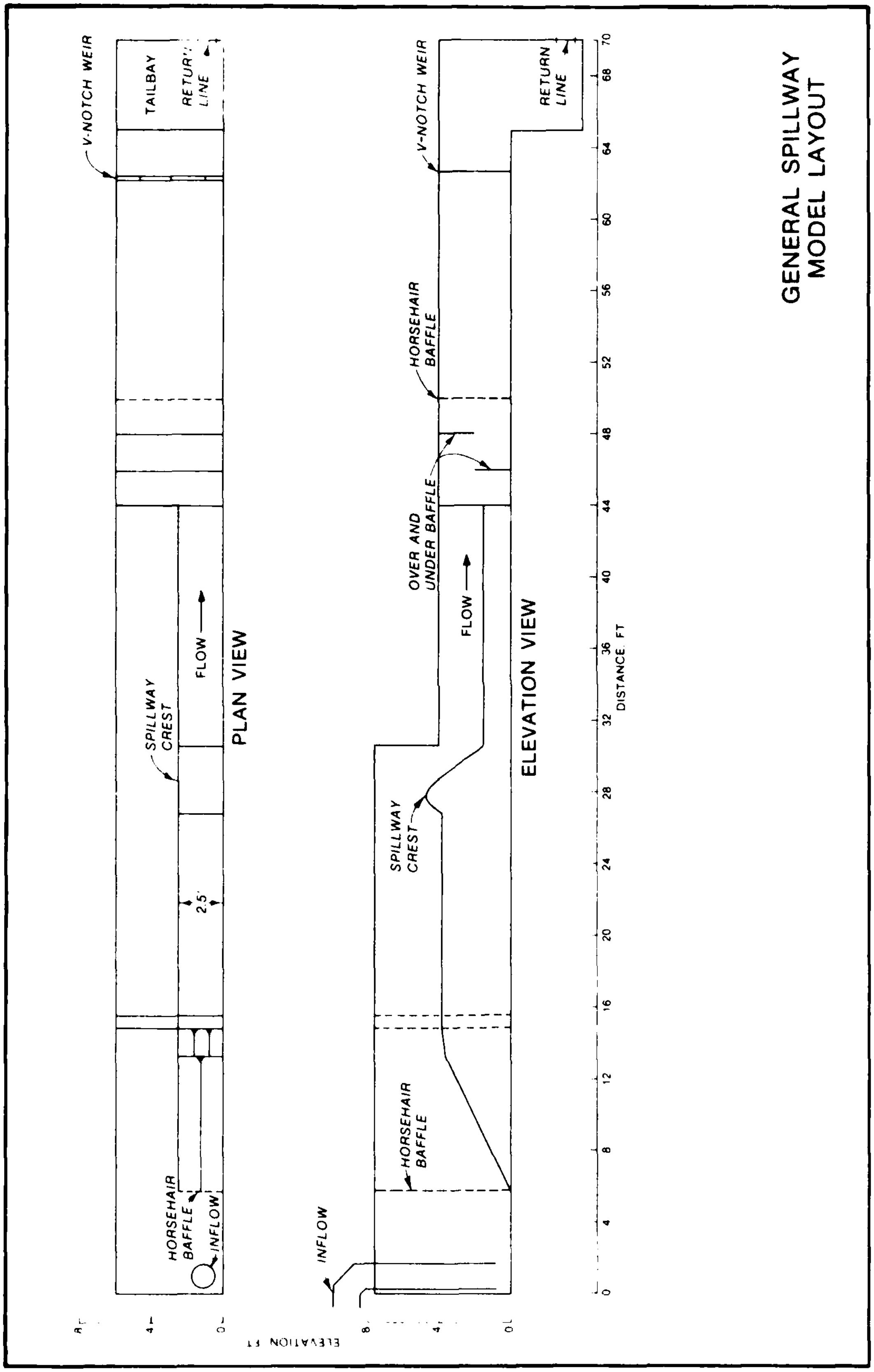
Vertical		
Q cfs	H ft	H_e ft
1.51	0.343	0.353
2.01	0.406	0.421
2.50	0.465	0.486
2.98	0.524	0.550
3.52	0.566	0.599
3.98	0.605	0.644
4.50	0.640	0.687
5.03	0.677	0.731
5.52	0.705	0.767
6.01	0.742	0.811
6.54	0.766	0.844
6.99	0.794	0.879
7.48	0.823	0.916
8.07	0.852	0.955
8.62	0.885	0.997
9.02	0.912	1.030
9.46	0.934	1.059
9.98	0.952	1.088
10.43	0.978	1.121
11.02	1.006	1.159
11.50	1.026	1.188
12.10	1.049	1.222
12.56	1.079	1.258
12.91	1.092	1.278
13.49	1.110	1.309

1V on 1H		
Q cfs	H ft	H_e ft
0.50	0.185	0.187
0.74	0.233	0.236
1.00	0.276	0.281
1.25	0.318	0.326
1.51	0.352	0.362
2.02	0.414	0.429
2.50	0.470	0.491
2.98	0.514	0.540
3.45	0.547	0.580
4.00	0.598	0.638
4.56	0.636	0.684
5.05	0.673	0.728
5.53	0.705	0.767
5.96	0.734	0.803
6.46	0.770	0.846
6.99	0.803	0.887
7.53	0.834	0.927
7.94	0.849	0.950
8.56	0.889	0.999
9.02	0.915	1.032
9.51	0.937	1.063
9.98	0.960	1.094
10.38	0.978	1.119
10.95	1.007	1.158
11.85	1.035	1.205
12.06	1.052	1.224
12.50	1.067	1.248
13.07	1.080	1.274
13.53	1.107	1.308

Table 8

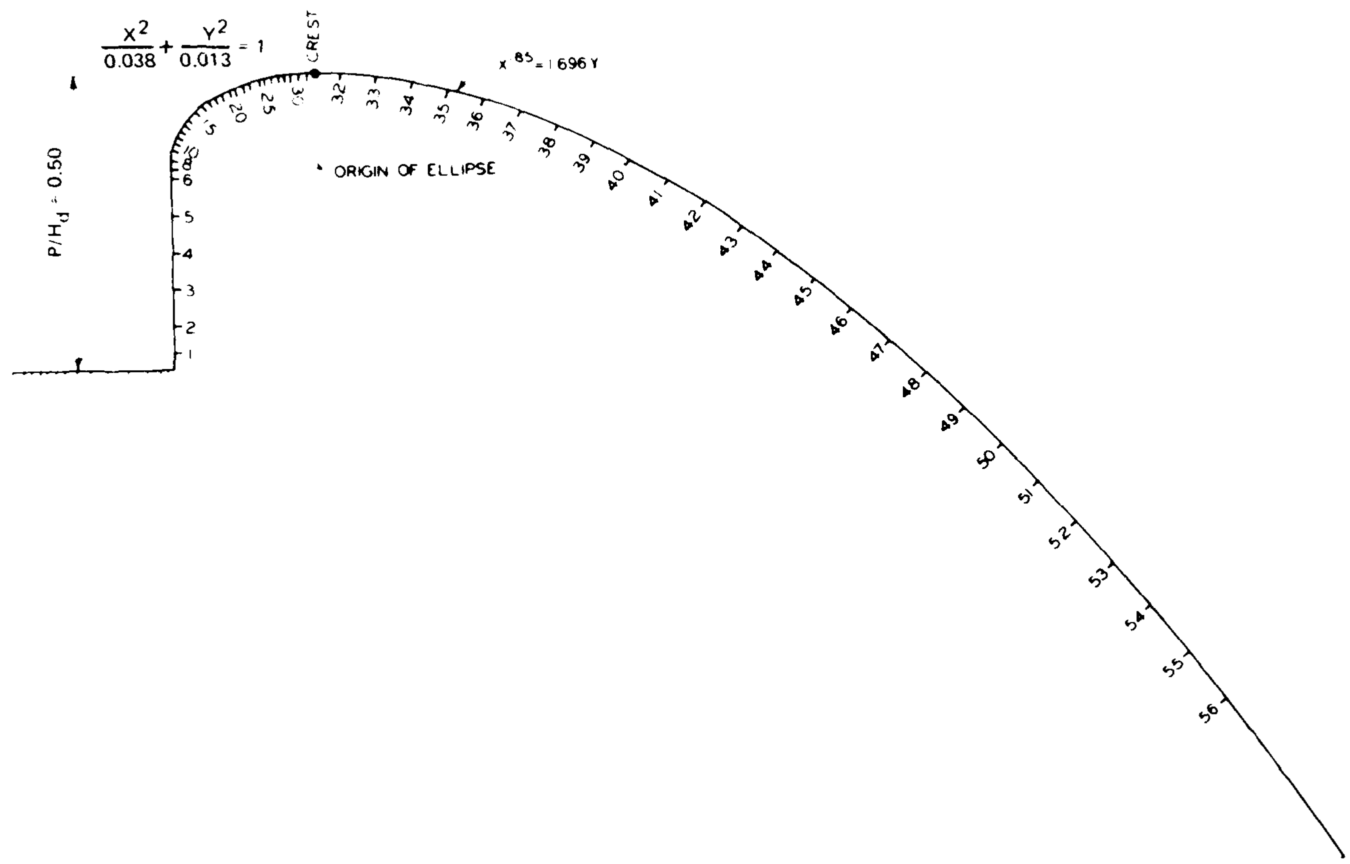
Basic Data, $P/H_d = 1.0$, with Piers

Upstream Face Slope					
Vertical			1V on 1H		
Q cfs	H ft	H_e ft	Q cfs	H ft	H_e ft
1.49	0.340	0.344	1.52	0.350	0.354
2.03	0.414	0.421	2.02	0.418	0.425
2.51	0.468	0.478	2.52	0.474	0.484
3.03	0.521	0.534	2.98	0.521	0.534
3.54	0.560	0.577	3.52	0.574	0.590
3.98	0.611	0.631	3.98	0.613	0.633
4.51	0.652	0.676	4.50	0.661	0.685
5.03	0.693	0.721	5.03	0.702	0.730
5.46	0.730	0.762	5.52	0.738	0.770
6.00	0.767	0.803	6.01	0.772	0.808
6.38	0.793	0.833	6.54	0.811	0.852
6.99	0.837	0.882	6.99	0.845	0.890
7.27	0.862	0.910	7.55	0.881	0.931
7.94	0.882	0.937	8.07	0.909	0.964
8.91	0.954	1.018	8.56	0.938	0.998
9.24	0.973	1.041	9.02	0.971	1.036
9.57	0.994	1.065	9.46	0.994	1.063
9.98	1.007	1.083	9.98	1.024	1.098
10.48	1.041	1.122	10.48	1.054	1.133
10.95	1.068	1.153	10.95	1.078	1.163
11.41	1.086	1.177	11.50	1.108	1.198
11.97	1.117	1.214	12.02	1.124	1.221
12.43	1.132	1.235	12.51	1.162	1.263
12.99	1.152	1.262	12.99	1.181	1.288
13.53	1.174	1.291	13.53	1.211	1.324



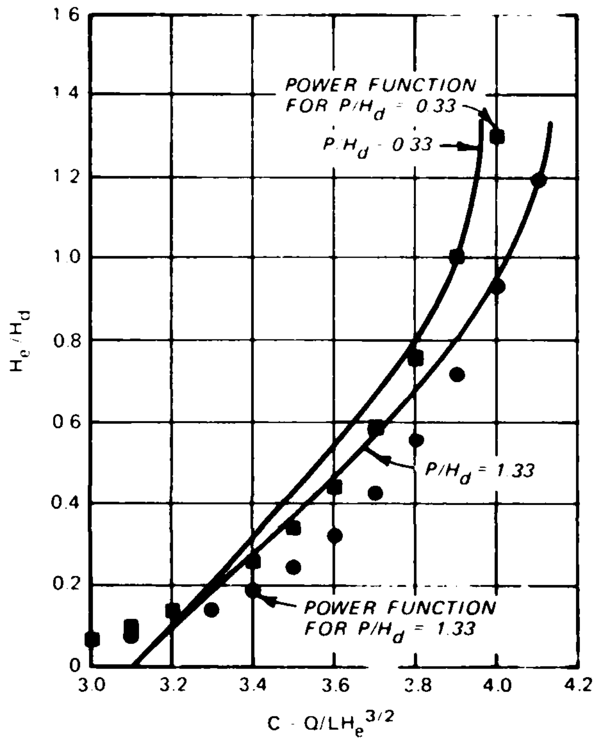
GENERAL SPILLWAY MODEL LAYOUT

PLATE 1

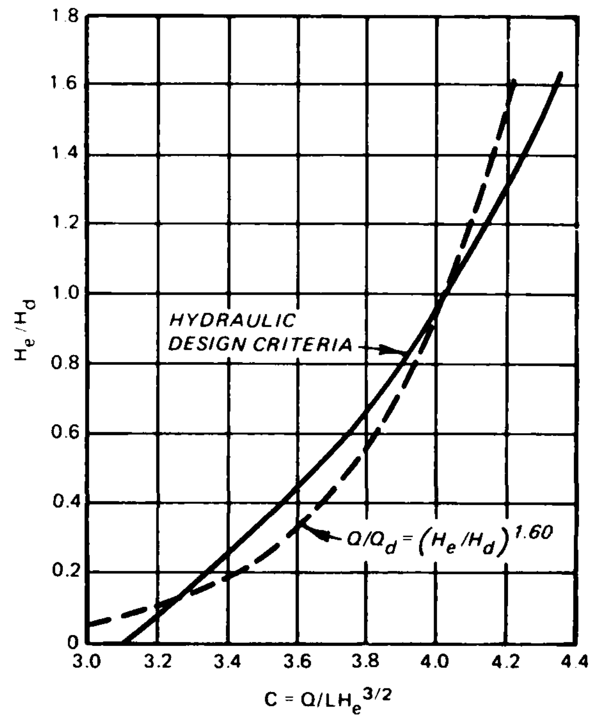


GENERAL SPILLWAY
CREST DETAILS AND PIEZOMETER LOCATIONS
$P/H_d = 0.50 \quad H_d = 0.80$ FT
VERTICAL UPSTREAM FACE

PLATE 2



a. FROM HDC 122-1, LOW OGEE CREST

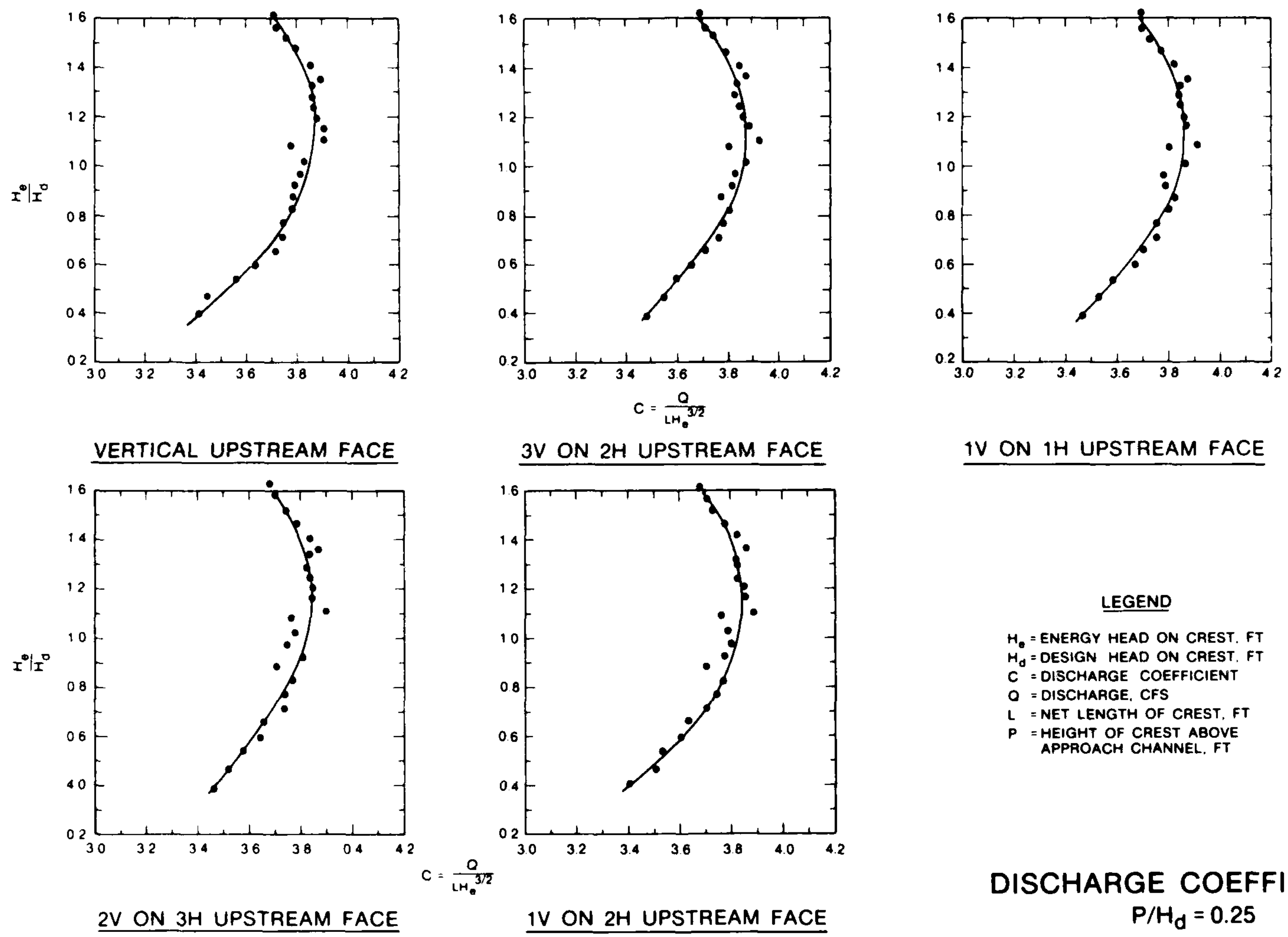


b. FROM HDC 111-3, HIGH OVERFLOW DAMS

NOTE:
H_e ENERGY HEAD ON CREST, FT
H_d DESIGN HEAD ON CREST, FT
C DISCHARGE COEFFICIENT
C_d DESIGN DISCHARGE COEFFICIENT
Q DISCHARGE, CFS
Q_d DESIGN DISCHARGE, CFS
L NET LENGTH OF CREST, FT
P HEIGHT OF CREST ABOVE APPROACH CHANNEL, FT

DISCHARGE AT HEADS OTHER THAN DESIGN HEAD

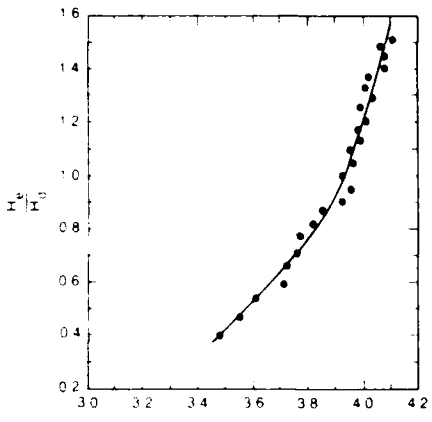
HDC POWER FUNCTION



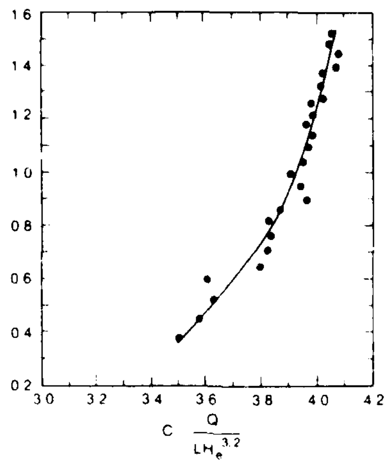
LEGEND

H_e = ENERGY HEAD ON CREST, FT
H_d = DESIGN HEAD ON CREST, FT
C = DISCHARGE COEFFICIENT
Q = DISCHARGE, CFS
L = NET LENGTH OF CREST, FT
P = HEIGHT OF CREST ABOVE APPROACH CHANNEL, FT

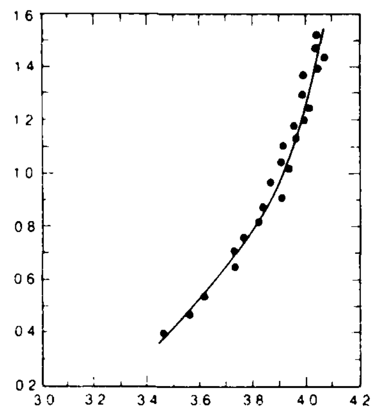
DISCHARGE COEFFICIENTS
$P/H_d = 0.25$



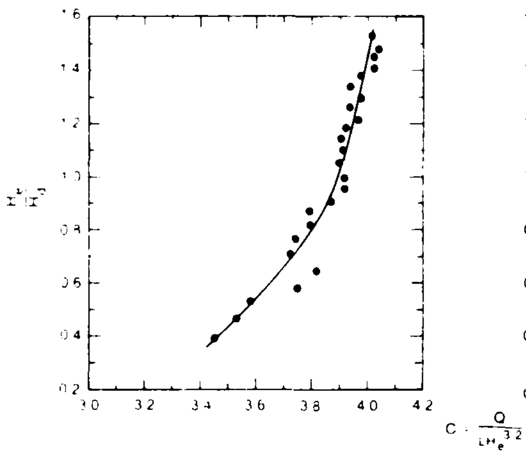
VERTICAL UPSTREAM FACE



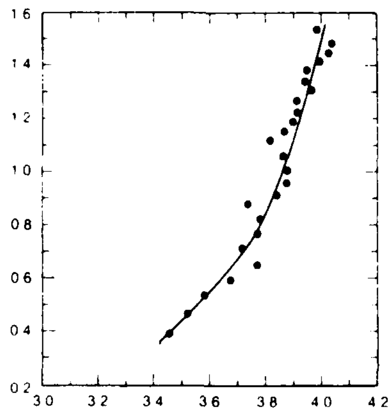
4V ON 1H UPSTREAM FACE



3V ON 2H UPSTREAM FACE



1V ON 1H UPSTREAM FACE



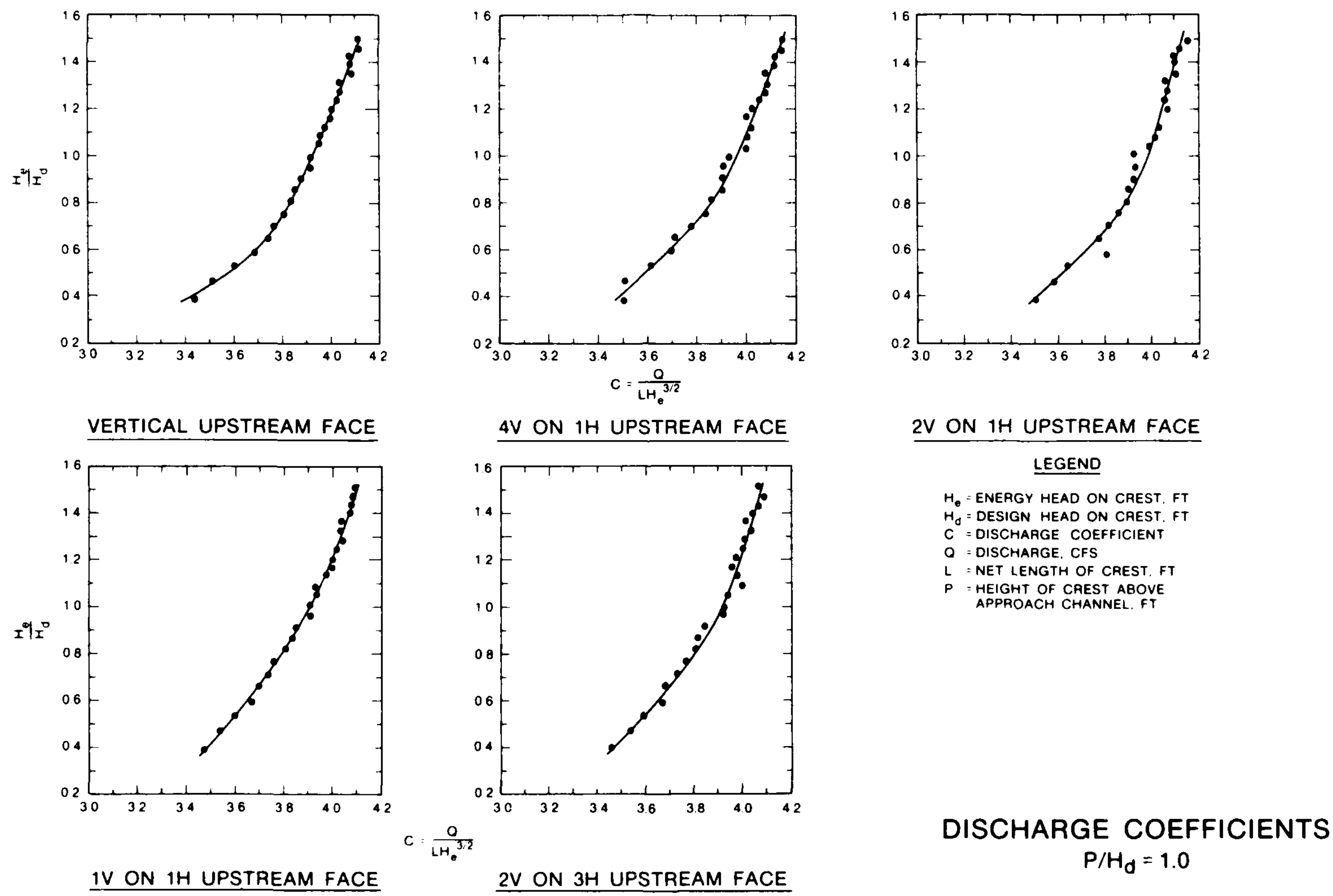
2V ON 3H UPSTREAM FACE

LEGEND

H_e ENERGY HEAD ON CREST, FT
H_d DESIGN HEAD ON CREST, FT
C DISCHARGE COEFFICIENT
Q DISCHARGE, CFS
L NET LENGTH OF CREST, FT
P HEIGHT OF CREST ABOVE APPROACH CHANNEL, FT

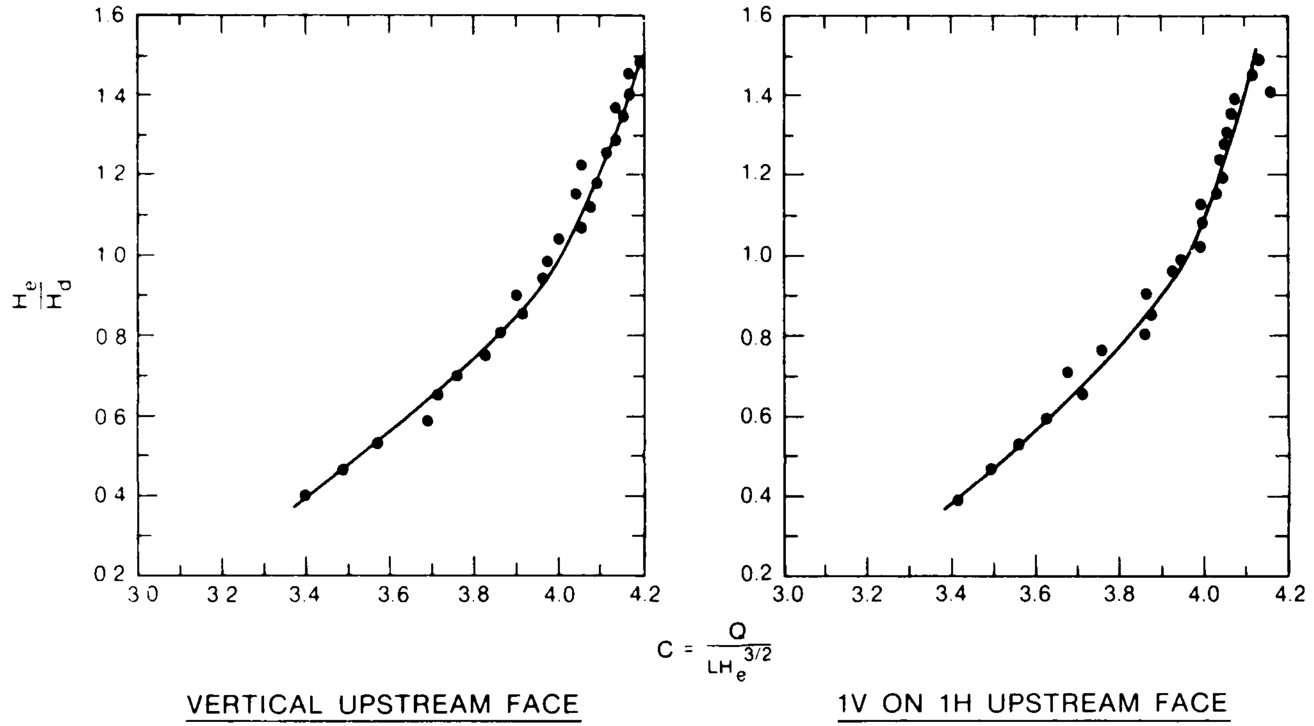
DISCHARGE COEFFICIENTS

$P/H_d = 0.50$



DISCHARGE COEFFICIENTS

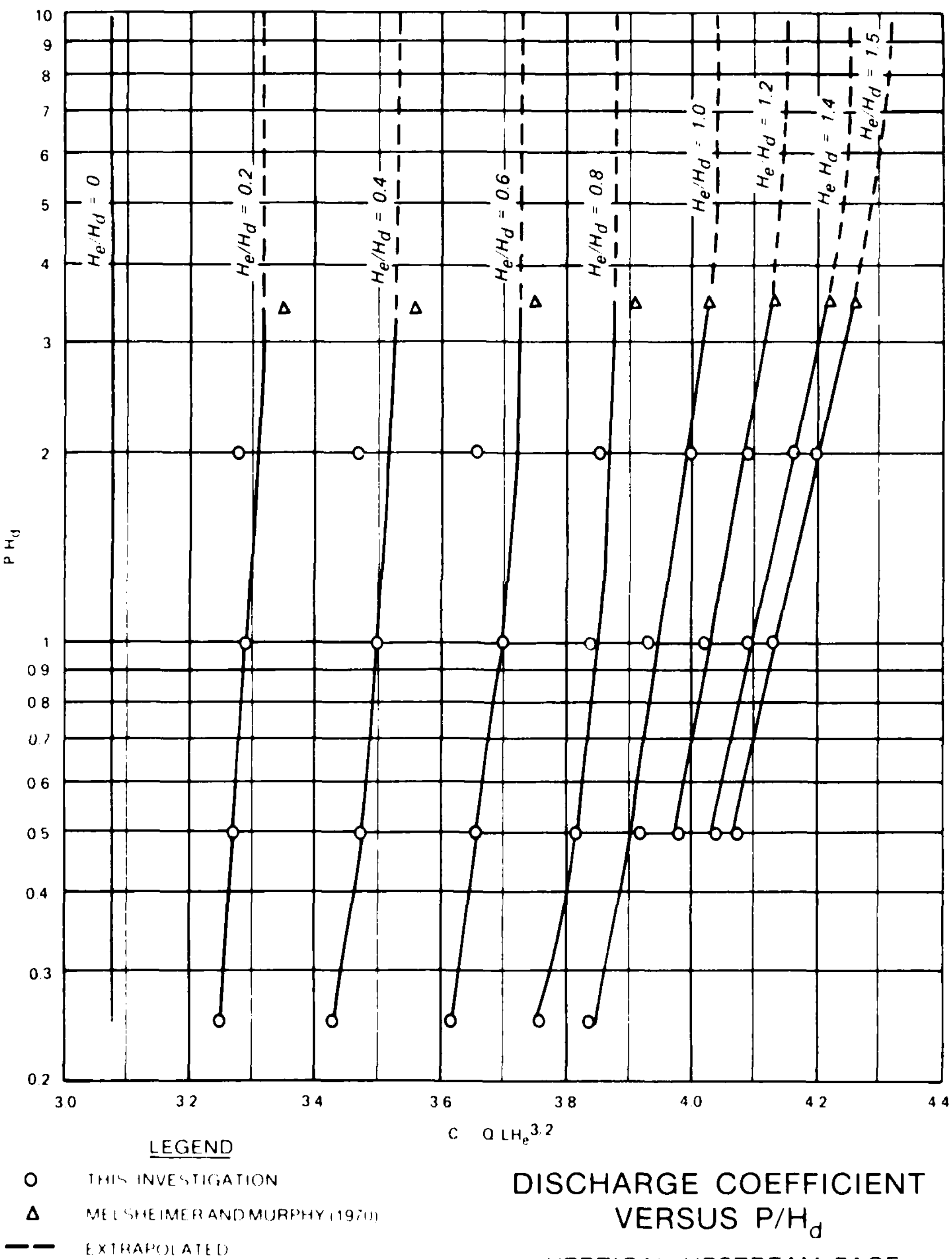
$P/H_d = 1.0$



LEGEND

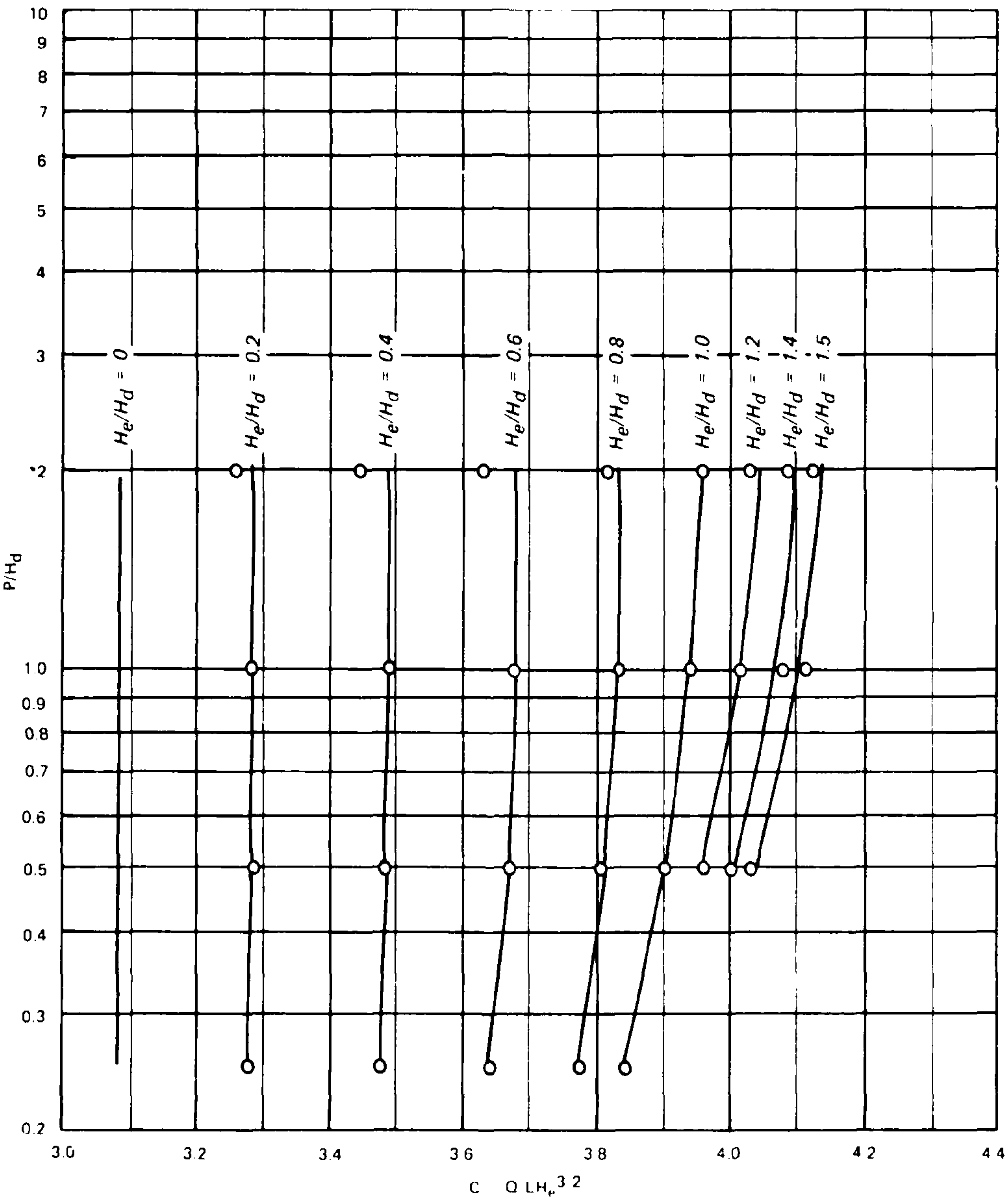
H_e = ENERGY HEAD ON CREST, FT
H_d = DESIGN HEAD ON CREST, FT
C   DISCHARGE COEFFICIENT
Q   DISCHARGE, CFS
L   NET LENGTH OF CREST, FT
P   HEIGHT OF CREST ABOVE APPROACH CHANNEL, FT

DISCHARGE COEFFICIENTS
$P/H_d = 2.0$



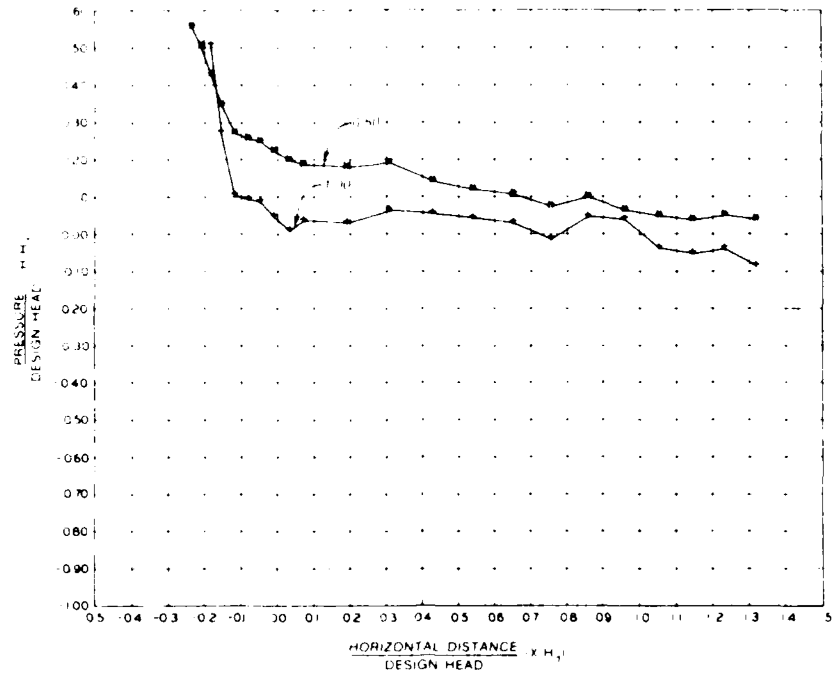
DISCHARGE COEFFICIENT VERSUS P/H_d

VERTICAL UPSTREAM FACE

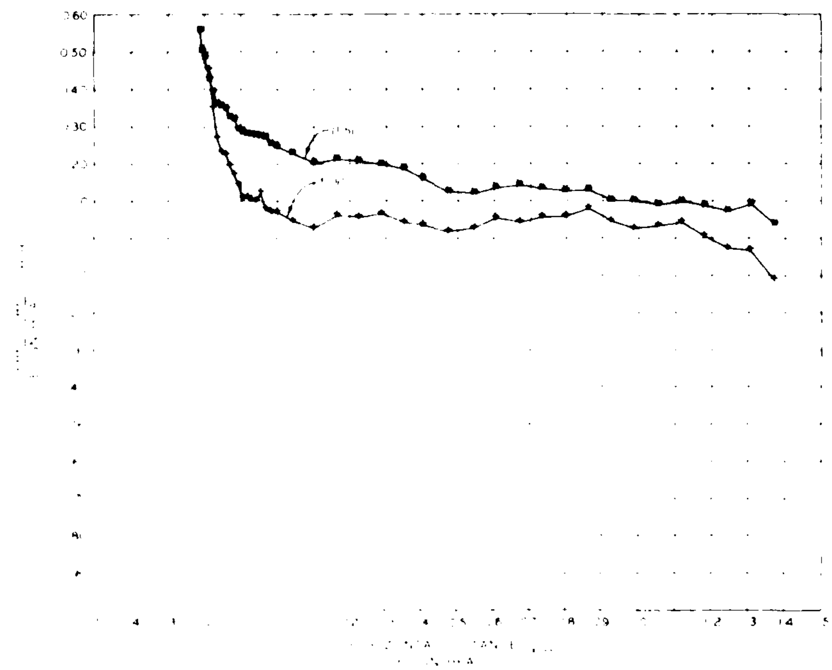


DISCHARGE COEFFICIENT
VERSUS P/H_d

1V ON 1H UPSTREAM FACE



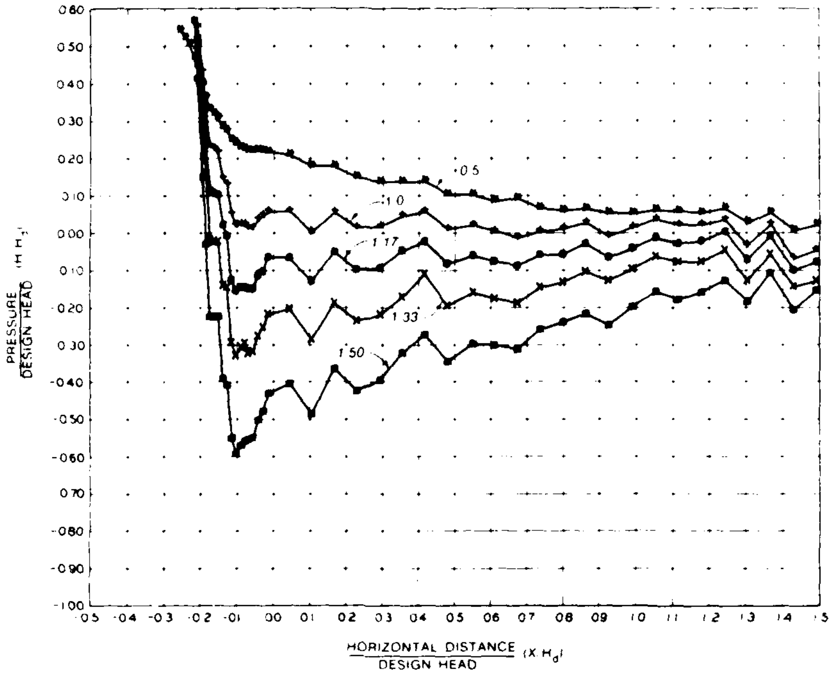
IV ON IH UPSTREAM FACE



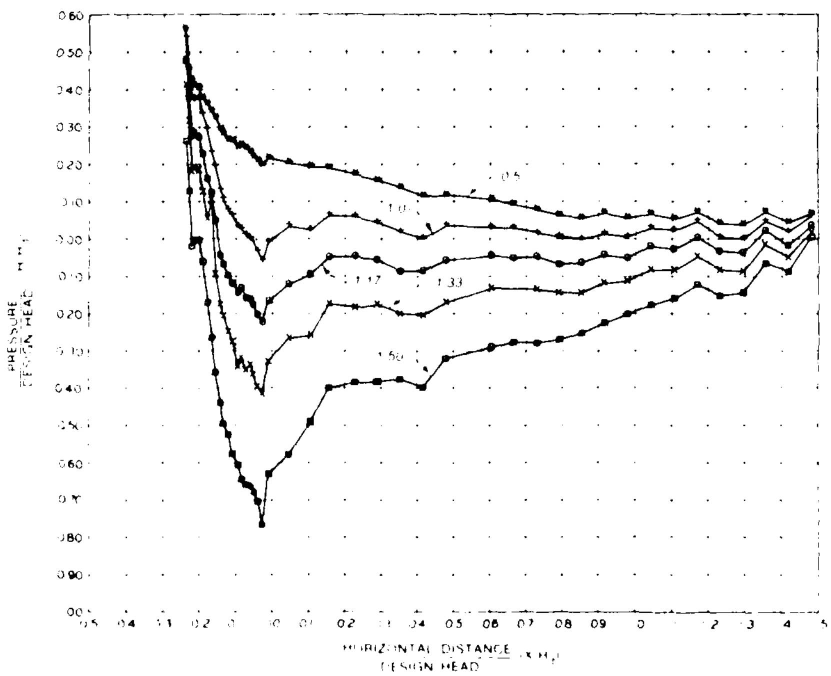
SPILLWAY CREST PRESSURES

NO PIERS

P H, 0.25



1V ON 1H UPSTREAM FACE

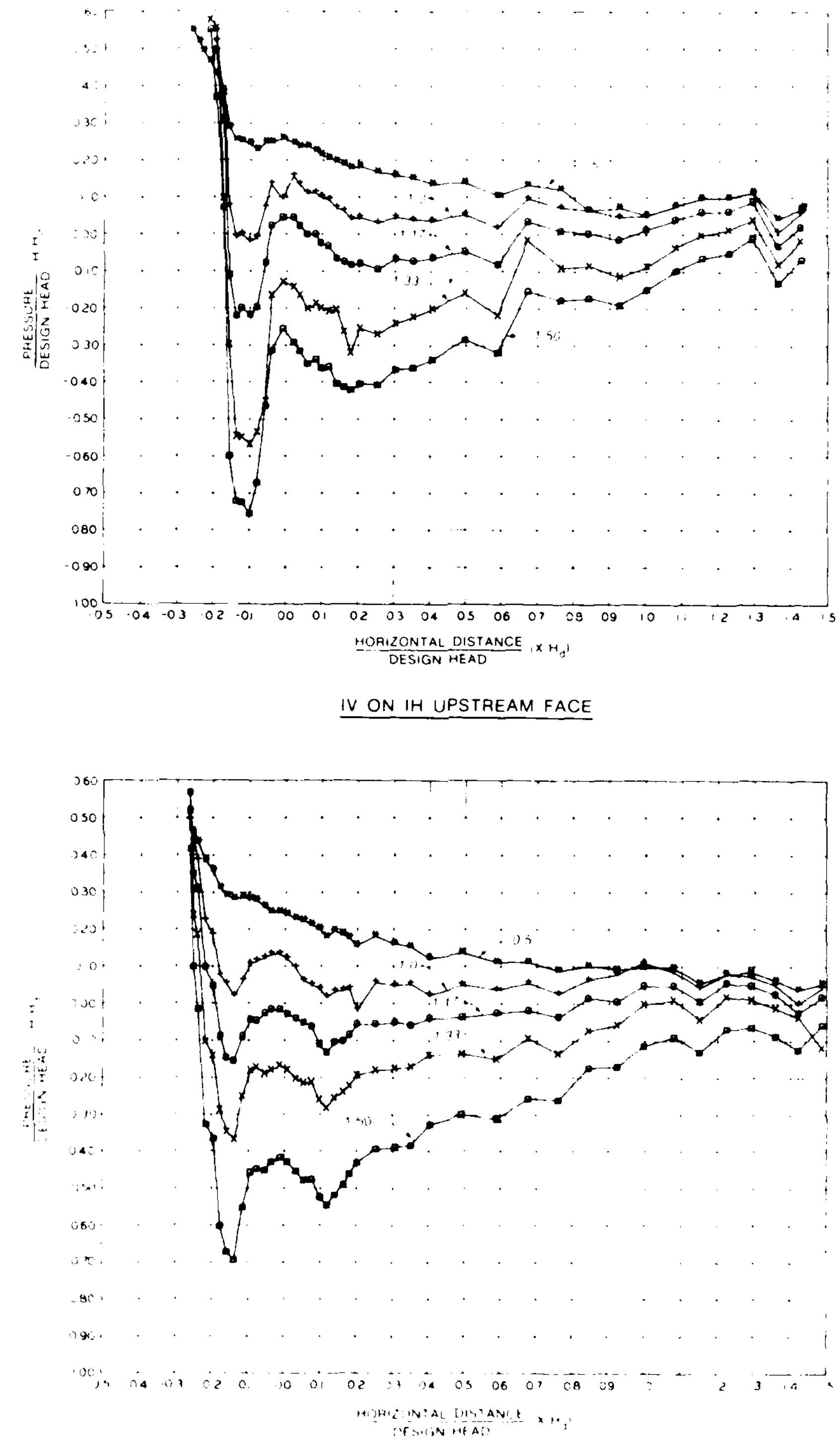


VERTICAL UPSTREAM FACE

SPILLWAY CREST PRESSURES

NO PIERS

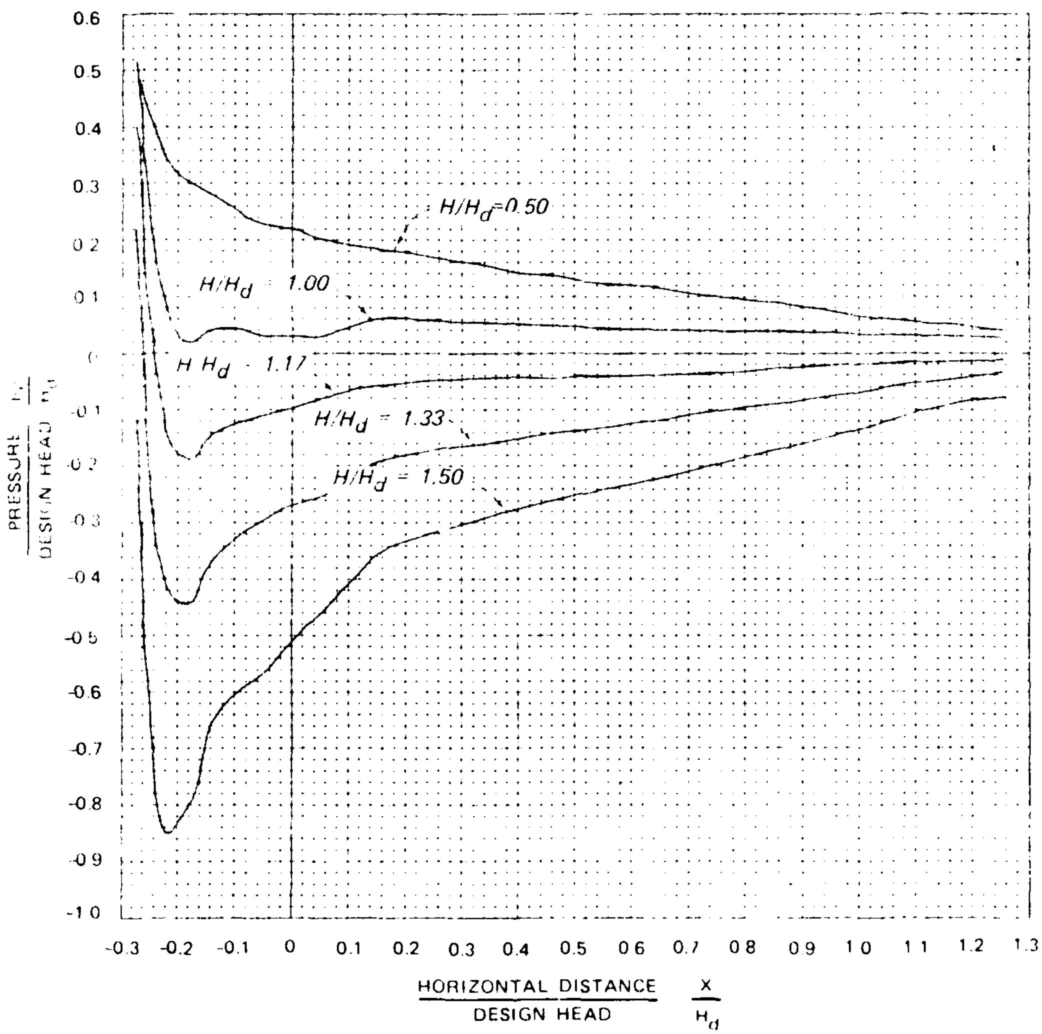
P/H_d 0.50



IV ON IH UPSTREAM FACE

VERTICAL UPSTREAM FACE

SPILLWAY CREST PRESSURES
NO PIERS
P/H_d 1.0



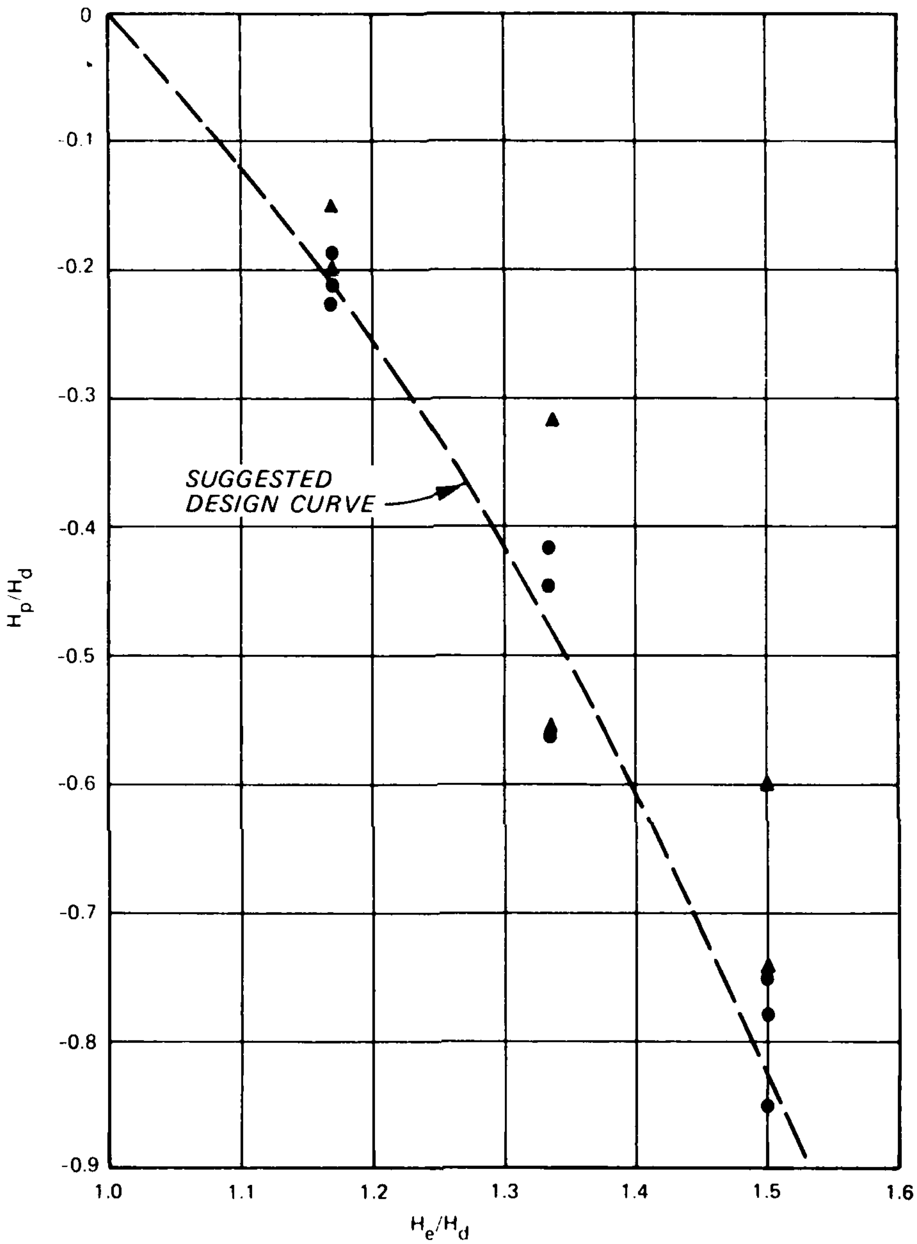
CREST PRESSURES

WITHOUT PIERS

P/H_d 3.4

VERTICAL UPSTREAM FACE

(FROM MELSHEIMER AND MURPHY (1970))



LEGEND

● VERTICAL UPSTREAM FACE

▲ 1V ON 1H UPSTREAM FACE

H_p - PRESSURE ON CREST, FT

H_d - DESIGN HEAD, FT

H_e - ENERGY HEAD, FT

P APPROACH DEPTH, FT

MAXIMUM NEGATIVE PRESSURE VS H_e/H_d

UNCONTROLLED CREST

$P/H_d \geq 0.50$

$\frac{P}{H_d}$ 0.25

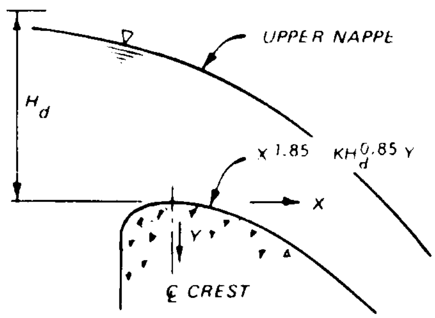
$\frac{H_e}{H_d}$	0.50	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
1.0	-0.452	0.768
-0.8	0.452	-0.759
0.6	-0.446	-0.750
-0.4	-0.435	-0.735
-0.2	-0.414	-0.712
0.0	0.378	-0.678
0.2	-0.319	-0.629
0.4	-0.233	-0.550
0.6	-0.120	-0.453
0.8	0.020	-0.331
1.0	0.188	-0.172
1.2	0.375	0.008
1.4	0.578	0.212

$\frac{P}{H_d} = 0.50$

$\frac{H_e}{H_d}$	0.50	1.0	1.50
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.467	-0.849	-1.169
-0.8	-0.463	-0.840	-1.161
0.6	-0.452	-0.822	1.141
0.4	-0.436	-0.796	1.114
-0.2	-0.409	-0.764	-1.073
0.0	-0.365	-0.714	-1.016
0.2	-0.297	-0.647	-0.953
0.4	-0.199	-0.557	-0.866
0.6	-0.076	-0.449	-0.770
0.8	0.071	-0.307	-0.641
1.0	0.244	-0.140	-0.481
1.2	0.445	0.059	-0.300
1.4	0.661	0.278	-0.100

$\frac{P}{H_d}$ 1.0

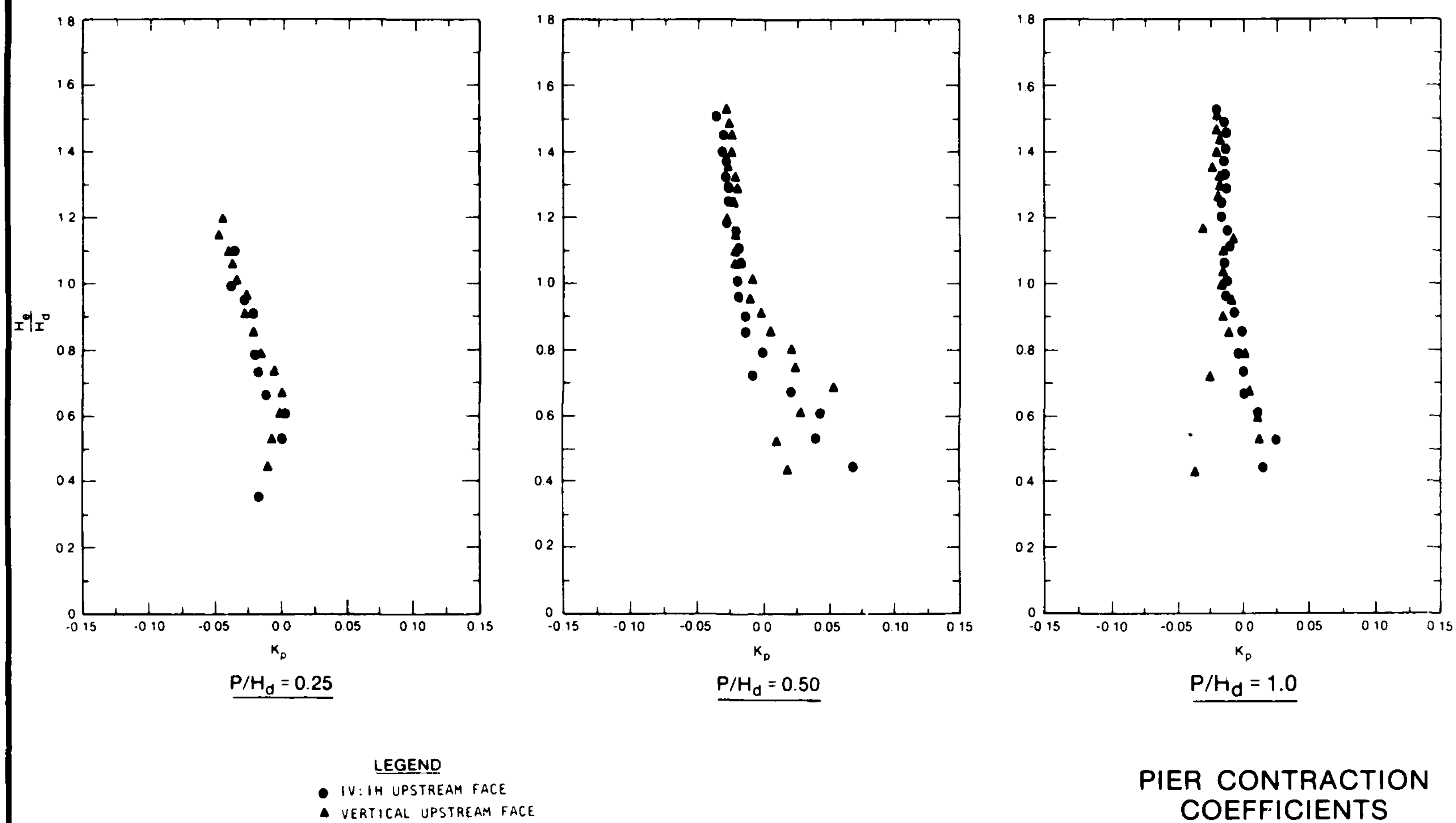
$\frac{H_e}{H_d}$	0.50	1.0	1.50
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.479	-0.897	-1.259
-0.8	-0.472	-0.879	-1.241
-0.6	-0.462	-0.857	-1.213
-0.4	-0.445	-0.829	-1.178
-0.2	-0.419	-0.792	-1.130
0.0	-0.377	-0.742	-1.073
0.2	-0.318	-0.677	-1.000
0.4	-0.219	-0.579	-0.912
0.6	-0.102	-0.465	-0.811
0.8	0.041	-0.328	-0.692
1.0	0.218	-0.160	-0.542
1.2	0.412	0.033	-0.367
1.4	0.629	0.243	-0.175



DEFINITION SKETCH

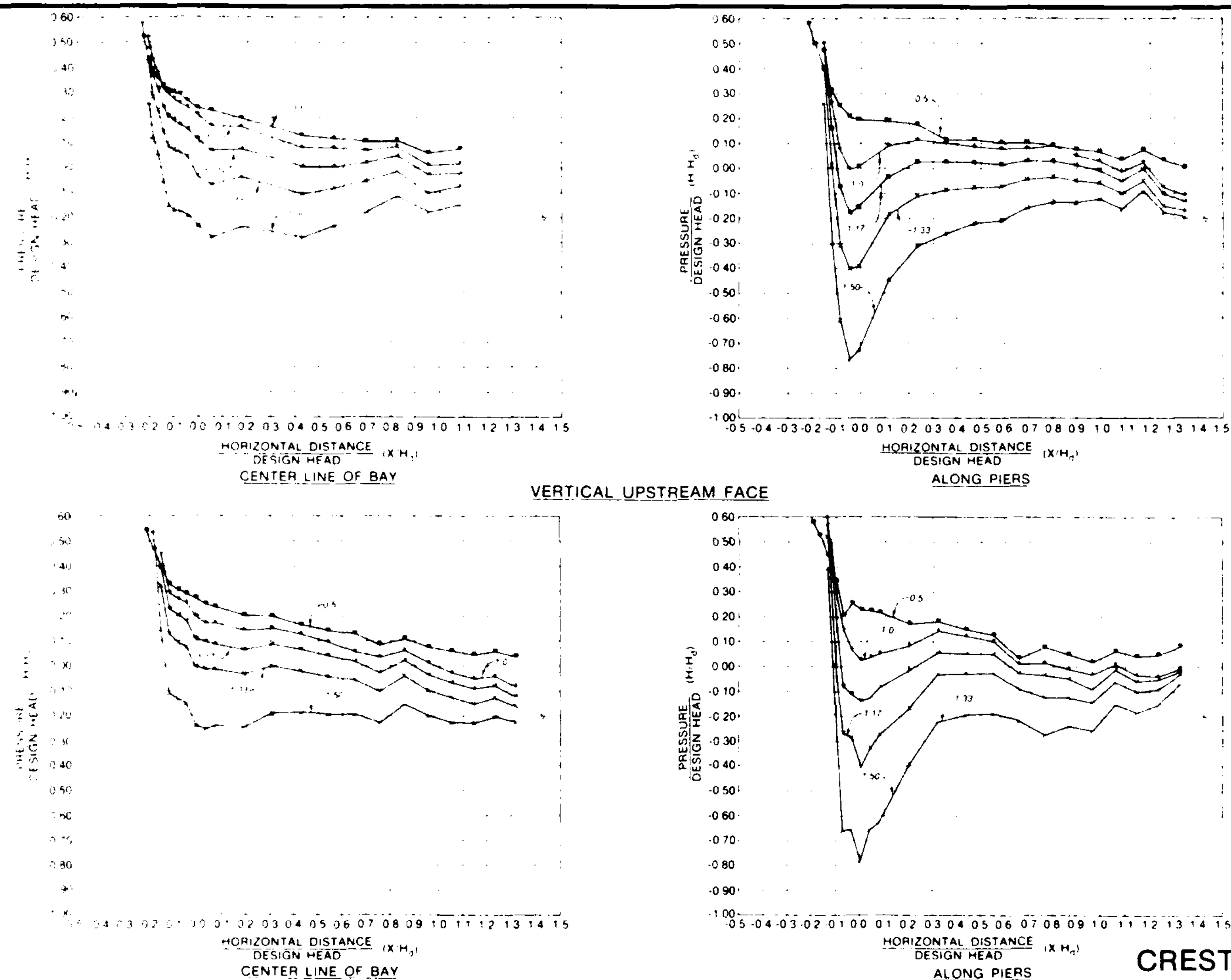
WATER-SURFACE PROFILES

UNCONTROLLED CREST



PIER CONTRACTION COEFFICIENTS

$P/H_d = 0.25,\ 0.50,\ \text{AND}\ 1.0$

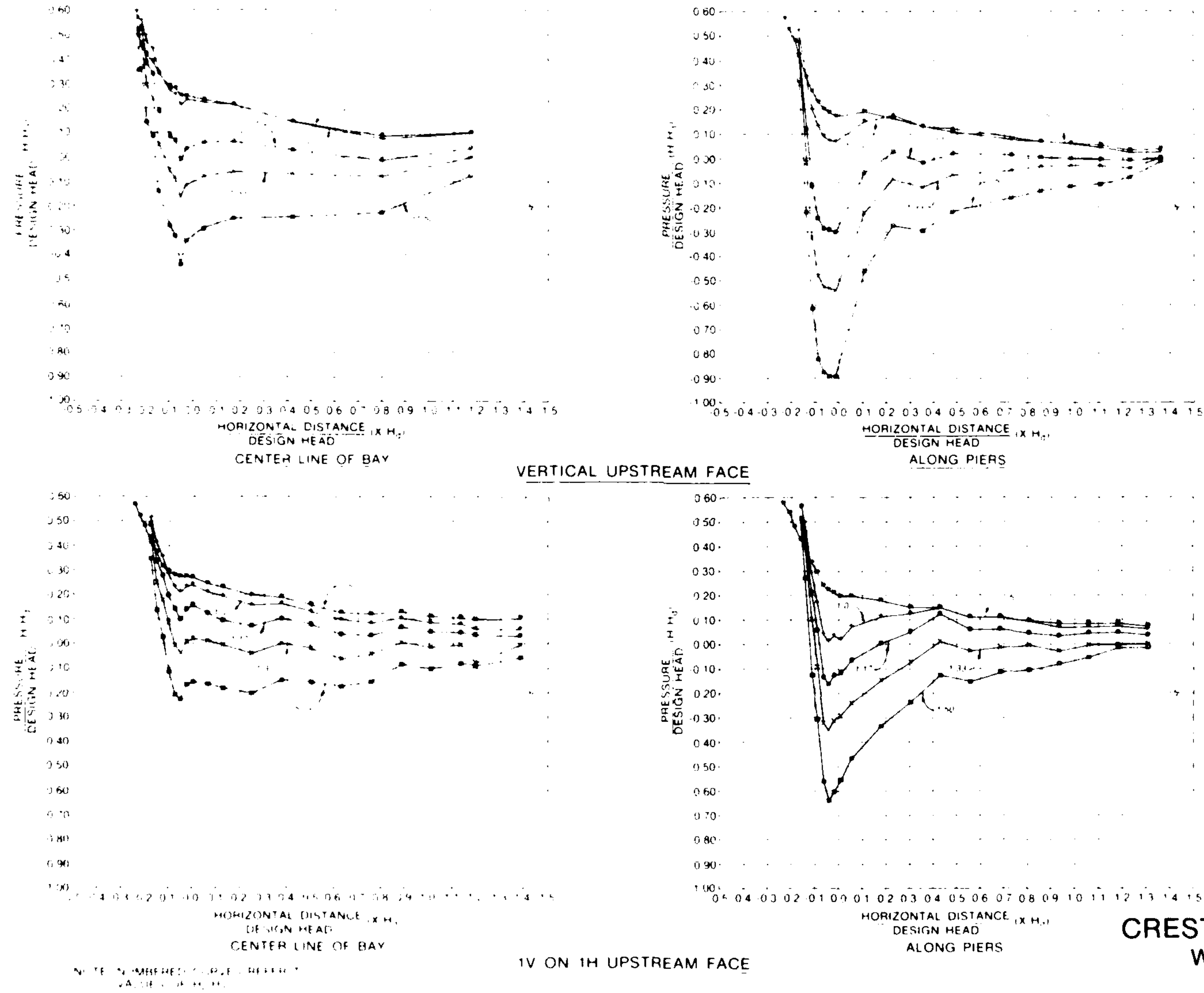


VERTICAL UPSTREAM FACE

1V ON 1H UPSTREAM FACE

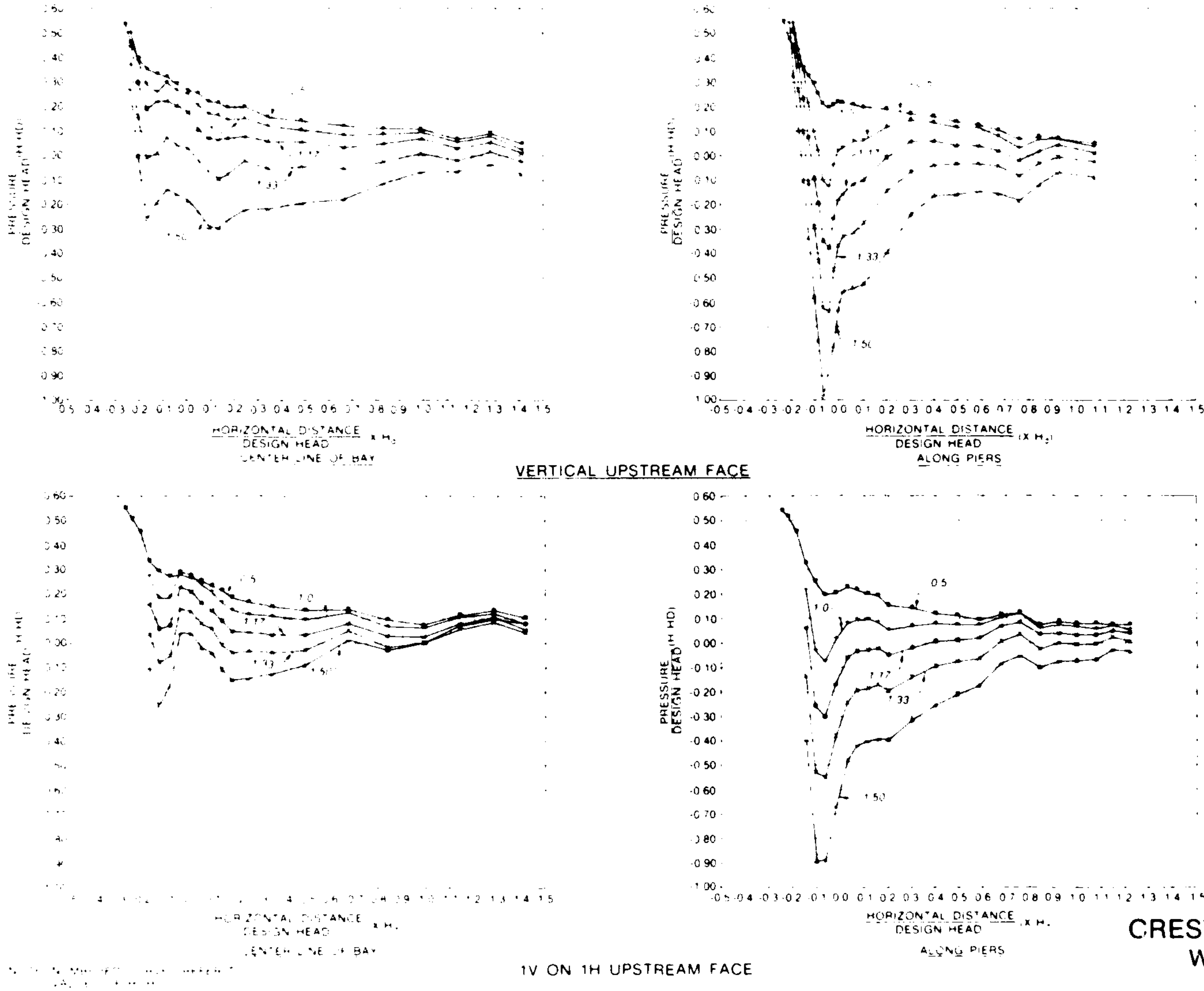
NOTE: NUMBERED CURVES REFER TO VALUES OF H/H_d

CREST PRESSURES
WITH PIERS
$P/H_d = 0.25$

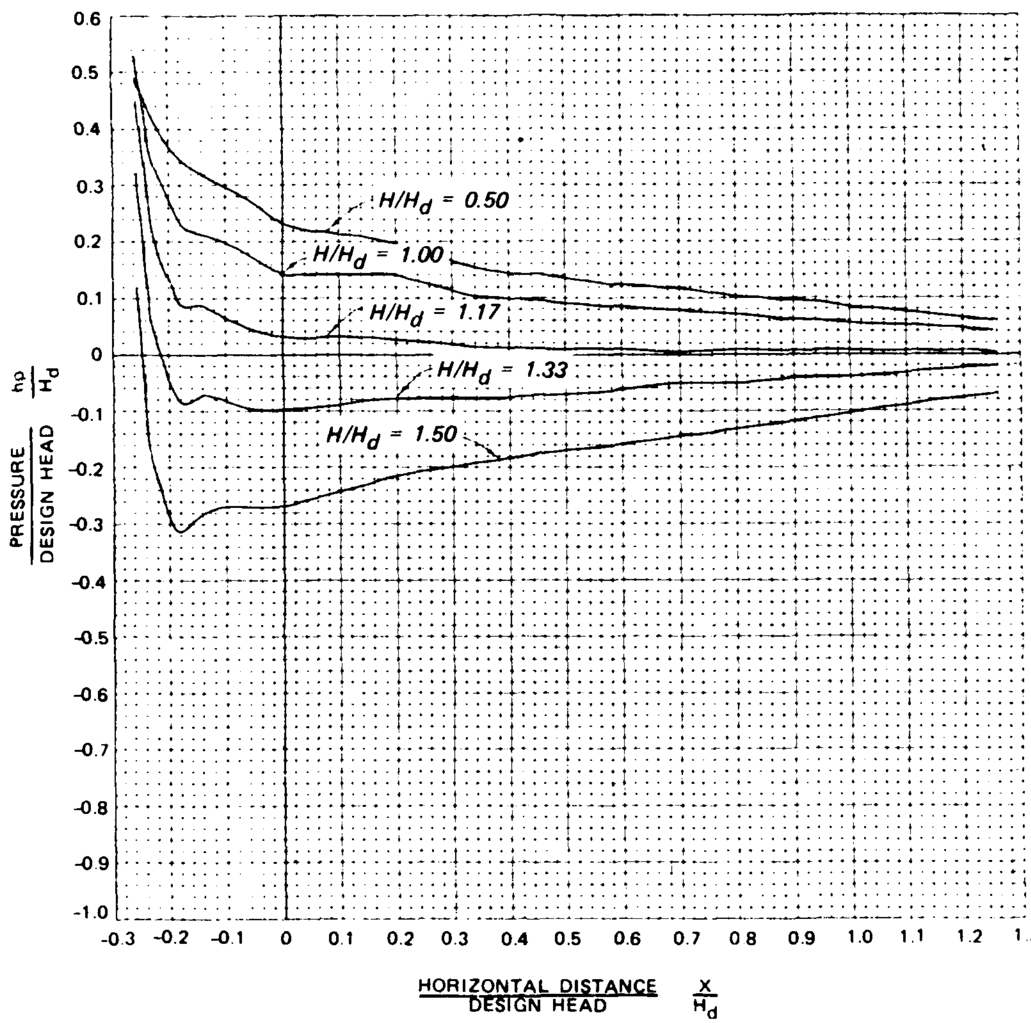


CREST PRESSURES
WITH PIERS
$P/H_d = 0.50$

PLATE 18



CREST PRESSURES
WITH PIERS
$P/H_d = 1.0$



CRHEST PRESSURES

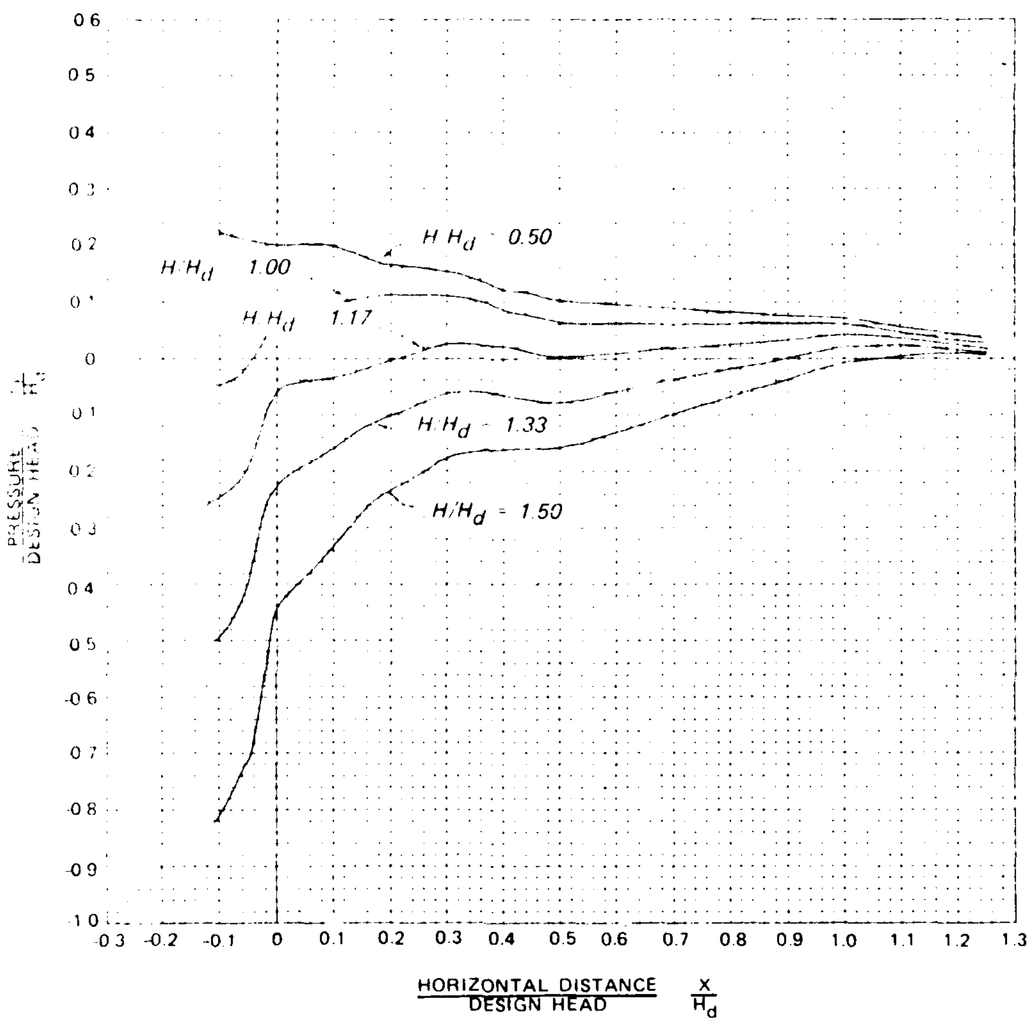
CENTER LINE OF GATE BAY

$P/H_d = 3.4$

VERTICAL UPSTREAM FACE

(FROM MELSHEIMER AND MURPHY (1970))

PLATE 20



CREST PRESSURES

ALONG PIER

P/H_d 3.4

VERTICAL UPSTREAM FACE

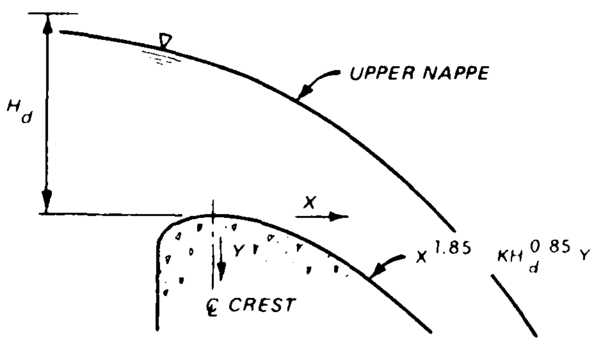
(FROM MELSHEIMER AND MURPHY (1970))

CENTER LINE OF GATE BAY

$\frac{H_e}{H_d}$	0.50	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
-1.0	-0.469	-0.850
-0.8	-0.469	-0.848
-0.6	-0.464	-0.839
-0.4	-0.454	-0.823
-0.2	-0.438	-0.796
0.0	-0.405	-0.758
0.2	-0.358	-0.715
0.4	-0.260	-0.640
0.6	-0.151	-0.553
0.8	-0.018	-0.448
1.0	0.135	-0.303
1.2	0.315	-0.135
1.4	0.528	+0.045

ALONG PIERS

$\frac{H_e}{H_d}$	0.50	1.0
$\frac{X}{H_d}$	$\frac{Y}{H_d}$	
-1.0	-0.469	-0.838
-0.8	-0.469	-0.835
-0.6	-0.466	-0.833
-0.4	-0.469	-0.835
-0.2	-0.488	-0.894
0.0	-0.414	-0.900
0.2	-0.286	-0.756
0.4	-0.175	-0.615
0.6	-0.066	-0.471
0.8	+0.061	-0.311
1.0	+0.209	-0.139
1.2	+0.378	+0.044
1.4	+0.577	+0.250



DEFINITION SKETCH

WATER-SURFACE PROFILES

CONTROLLED CREST

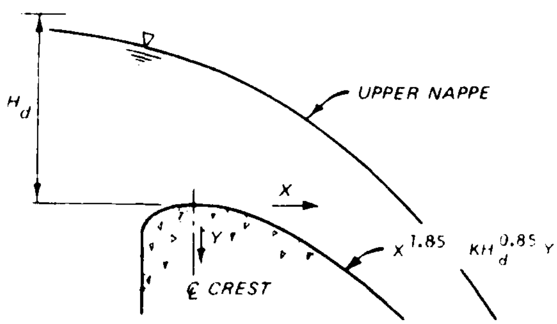
$P/H_d = 0.25$

H_e/H_d	0 50	1 0	1.5
X/H_d	Y/H_d		
1 0	0 483	-0 894	-1.254
0 8	0 479	0 886	-1.244
0 6	0 471	0 871	-1.229
0 4	0 454	-0 851	-1.208
0 2	0 429	0 824	-1.183
0 0	-0 388	-0 783	-1.149
0 2	-0 329	-0 728	-1.099
0 4	0 241	-0.655	-1.034
0 6	-0.123	0 570	-0.951
0 8	+0.019	-0 458	-0.856
1 0	+0.198	-0 300	-0.753
1 2	+0 394	-0.104	-0.631
1 4	+0 613	+0.119	-0.426

CENTER LINE OF GATE BAY

H_e/H_d	0.50	1.0	1.5
X/H_d	Y/H_d		
-1.0	-0.483	-0.889	-1.257
-0.8	-0.481	-0.880	-1.248
-0.6	-0.477	-0.869	-1.233
-0.4	-0.480	-0.880	-1.243
-0.2	-0.467	-0.917	-1.338
-0.15	-0.450	-0.910	-1.373
0.0	-0.356	-0.825	-1.324
0.2	-0.252	-0.677	-1.176
0.4	-0.159	-0.541	-1.029
0.6	-0.055	-0.414	-0.885
0.8	0.081	-0.258	-0.735
1.0	0.256	-0.089	-0.566
1.2	0.477	0.105	-0.383
1.4	0.672	0.319	-0.188

ALONG PIERS



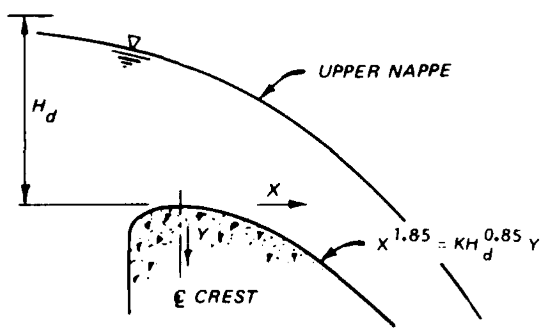
DEFINITION SKETCH

WATER-SURFACE PROFILES

CONTROLLED CREST
$P/H_d = 0.50$

CENTER LINE OF GATE BAY			
$\frac{H_e}{H_d}$	0.50.	1.0	1.5
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.494	-0.939	-1.311
-0.8	-0.488	-0.925	-1.300
-0.6	-0.483	-0.913	-1.275
-0.4	-0.476	-0.888	-1.248
-0.2	-0.445	-0.855	-1.210
0.0	-0.403	-0.808	-1.162
0.2	-0.335	-0.743	-1.102
0.4	-0.240	-0.666	-1.029
0.6	-0.116	-0.573	-0.938
0.8	+0.029	-0.454	-0.833
1.0	0.201	-0.291	-0.707
1.2	0.403	-0.086	-0.562
1.4	0.626	+0.150	-0.395

ALONG PIERS			
$\frac{H_e}{H_d}$	0.50	1.0	1.5
$\frac{X}{H_d}$	$\frac{Y}{H_d}$		
-1.0	-0.489	-0.933	-1.311
-0.8	-0.483	-0.925	-1.300
-0.6	-0.475	-0.918	-1.280
-0.4	-0.488	-0.931	-1.313
-0.2	-0.463	-0.935	-1.375
-0.15	-0.438	-0.915	-1 383
0.0	-0.369	-0.829	-1.315
0.2	-0.264	-0.695	-1.171
0.4	-0.170	-0.571	-1.023
0.6	-0.063	-0.441	-0.882
0.8	0.069	-0.298	-0.730
1.0	0.234	-0.128	-0.555
1.2	0.431	0.065	-0.362
1.4	0.651	0.278	-0.140



DEFINITION SKETCH

WATER-SURFACE PROFILES

CONTROLLED CREST

$P/H_d = 1.0$

END

FILMED

8-85

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