CONTROLLED FLOW OVER OGEE SPILLWAY

M.A. Abourehim

Irrigation and Hydraulics Department, Faculty of Engineering,
Alexandria University, Alexandria, Egypt.

ABSTRACT

Characteristics of flow, controlled by a plane vertical gate mounted on the ogee spillway crest, are investigated. Two types of shaped crest spillway are considered; standard and the elliptical shape. Study, in the present paper, includes; the discharge characteristics, the depth of flow behind spillway, and the pressure distributions along spillway face. The limits of controlled flow are fixed. A discharge equation is then developed as a function of spillway geometry, gate opening, and the head on the spillway. Both discharge and velocity coefficients are experimentally evaluated using the experimental measurements, conducted on spillway models of different heights. The toe depth is evaluated, by solving the specific energy equation, taking into account the energy loss. The predicted equation for the toe depth is checked using the experimental data. Pressures along the spillway face were measured using piezometers. Pressures distribution are presented in graphics as a function of the design head and height of gate opening. Analysing the pressure distribution, the maximum negative pressure on the crest and the pressure force on the downstream face are evaluated.

Keywords: Spillway, Ogee weir, Controlled flow.

NOTATIONS

C_d	discharge coefficient,
C_v	Velocity coefficient,
d	Height of gate opening,
E_o	Total energy, $E_o = P + H_2 + \frac{\alpha_1 v_a^2}{2g}$,
F	Pressure force acting on the downstream face of spillway,
g	Gravity acceleration,
H	Head of ungated flow over spillway,
H_c	Head related to the center line of gate opening,
H_D	Design head of spillway,
H_1	Head related to the top of gate opening,
H_2	Head related to the bottom of gate opening,
h	Height of uncontrolled flow at the crest apex,
h_L	Energy loss,
P	Height of spillway crest,
p	pressure on spillway surface,
q	Passing discharge per unit length,
v_a	Velocity of the approaching flow,
v	Velocity of flow at spillway toe,
y	Depth of flow at spillway toe,
y_c	Critical depth,
α_1, α_2	Energy correction coefficients,
η	Coefficient of head loss,
γ	Specific weight of water.

1. INTRODUCTION

Although the ogee spillway has been the subject of more researches than perhaps any other hydraulic structures, there are still some deficiencies in the knowledge of its functioning.

Spillways are usually equipped with either plane or radial gates. The gates may be operated to be fully or partially opened, as shown in Figure (1). Under flood conditions, which might cause unexpected damage, gates will be completely open to release excess discharges. In this case, the spillway becomes ungated and the flow will be uncontrolled. However in most cases, gates should be partially opened to regulate outlet discharges. In the latter case, the spillway is gated and the flow will be controlled.

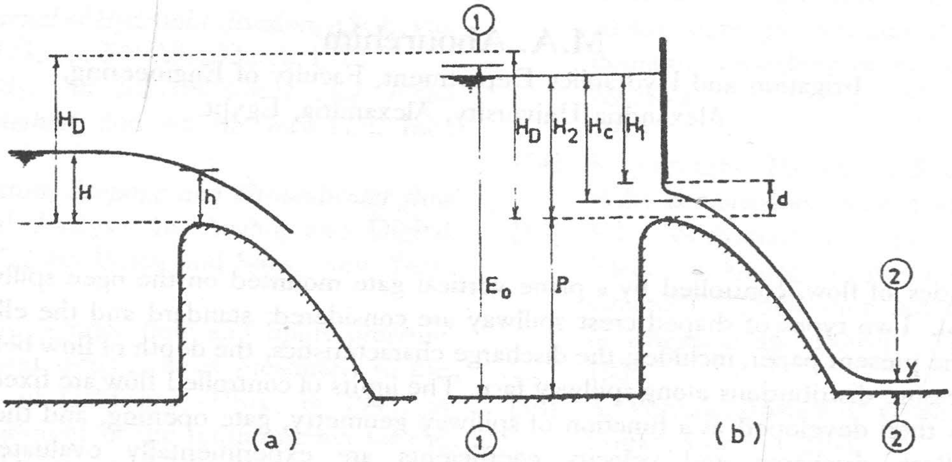

Figure 1. Definition sketch for flow over ogee spillway;
(a) Ungated spillway flow,
(b) Gated spillway flow.

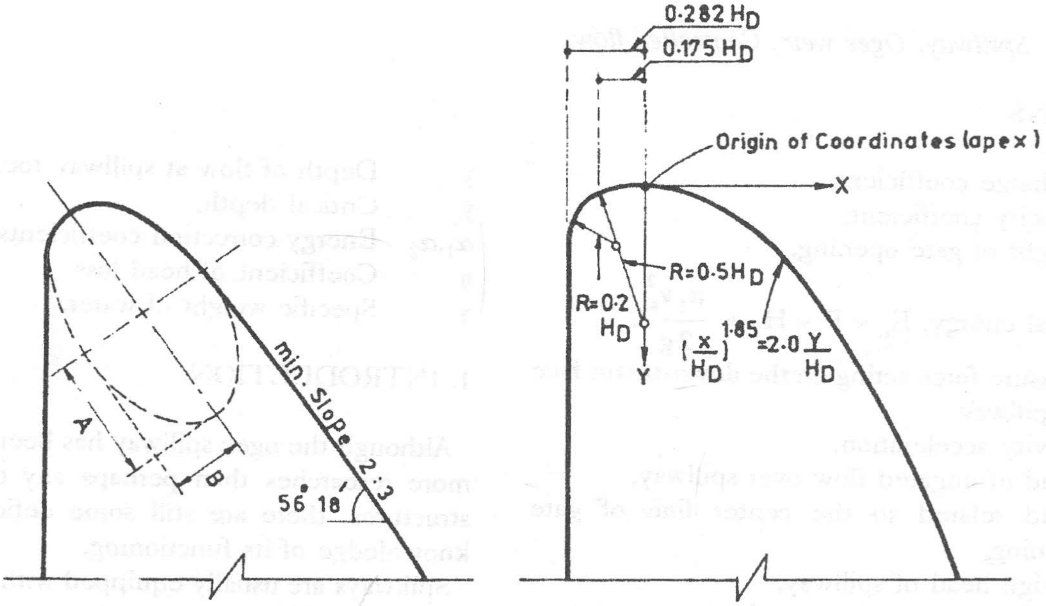

Figure 2. Standard spillway crest;
(a) Elliptical shape, A = 2 B,
(b) Standard shape, (USACE).

The crest of the ogee spillway is formed to take an elliptical shape [1], as shown in Figure (2-a). However, the most common shape, for ogee spillway crest, is the standard shape, which is designed according to the specifications of the U.S. Army corps of Engineer (USACE) [2,3,4]. In the standard shape, the crest geometry consists of a three-arc upstream quadrant and a power function at the downstream quadrant, as show in Figure (2-b).

The characteristics of flow over ungated spillway are extensively studied and well-understood. Review of these studies has been given in Refs. [5,6]. A little work has been presented for the mechanism of flow over gated spillway. The presence of gate, particularly for small openings, produces orifice flow type . Adherence of gate flow to the nappe profile results negative pressure on spillway crest, specially for heads greater than the design heads. To avoid this negative pressure, it was recommended to broaden the spillway crest [7]. However, this will reduce the discharge capacity for ungated flow condition. Further reduction in magnitude of the negative pressure can be achieved by moving the gate sill, by a distance of 0.2 H_2, downstream the crest apex [8], where H_2 is the head related to the bottom of gate opening. For a relative head H_2/H_D=0.2, the discharge coefficient value ranges from 0.55 to 0.7 [9,10]. The discharge characteristics of standard spillway, controlled by plane gate, have been investigated by Hager [11]. the hydraulics equations used by the NWS DAMBRK program in determining spillway discharge are presented and discussed by Wortman [12].

In previous studies, the depth of flow behind spillway is obtained by solving the energy equation using trail and error procedure which causes difficulties [1]. Some methods solve the energy equation using graphs for specific values of velocity coefficient C_v [13]. The interpolation between these specific values of C_v leads to inaccuracy in the calculated values of the toe depth. The depth of flow at spillway toe should be accurately estimated, since it influences various downstream conditions such as; velocity, Froude number, type of the formed jump, and the design of any required measures for energy dissipation. Noting that, an error of ± 10% in the evaluation of the toe depth, results a deviation of 15-20% in the Froude number value.

It is obvious from the previous studies, for gated spillway, that the pressure along the spillway face has not been sufficiently investigated. Since the stability of spillway is affected by the total pressure forces, acting on the spillway surface, these forces should be evaluated.

Therefore, the present study mainly aims to establishing a criteria to evaluate the pressure forces on spillway face, and to accurately estimate the flow depth at the toe of gated ogee spillway.

2. THEORETICAL STUDY

Theoretical study is intended to obtain a simple equation for the discharge over the spillway. Also, the study aims at calculating the toe depth using an analytic solution for the energy equation.

2.1 Discharge Equation

Referring to Figure (1), The governing equation, for the discharge through partially raised gate, can be derived by integrating the flow rate over the orifice opening to produce,

$$q = \frac{2}{3}C_d\sqrt{2g}\,(H_2^{3/2} - H_1^{3/2}), \qquad (1)$$

where,

- q is the discharge per unit length of crest,
- C_d is the coefficient of discharge,
- H_1 and H_2 are total heads related to the top and bottom of the gate opening, respectively.

Substituting for, $H_1 = H_c\,(1 - \frac{d}{2H_c})$,

and $H_2 = H_c\,(1 + \frac{d}{2H_c})$ in Eq (1) yields,

$$q = \frac{2}{3}\sqrt{2g}\,H_c^{3/2}\left[\left(1+\frac{d}{2H_c}\right)^{3/2} - \left(1-\frac{d}{2H_c}\right)^{3/2}\right], \qquad (2)$$

where,

- d is the height of gate opening,
- H_c is the head measured to the center of the gate opening.

Analysing the term $(1 \pm \frac{d}{2H_c})^{3/2}$ using the binomial theorem, we get;

$$(1 \pm \frac{d}{2H_c})^{3/2} = 1 \pm \frac{3}{2}(\frac{d}{2H_c}) + \frac{3}{8}(\frac{d}{2H_c})^2 \mp \frac{1}{16}(\frac{d}{2H_c})^3 + ...,$$

Hence, $$q = C_d \cdot d\sqrt{2gH_c}[1 - \frac{1}{96}(\frac{d}{H_c})^2]. \quad (3)$$

Substituting for $H_c = H_2 - d/2$ in Eq (3) yields,

$$q = C_d\sqrt{2g}H_2^{3/2} \cdot \frac{d}{H_2}\sqrt{1 - \frac{d}{2H_2}}\left[1 - \frac{1}{96}\left(\frac{\frac{d}{H_2}}{1 - \frac{d}{2H_2}}\right)^2\right], \quad (4)$$

or $$q = C_d\sqrt{2g}H_2^{3/2}\frac{d}{H_2}\sqrt{1 - \frac{d}{2H_2}}\, K, \quad (5)$$

where, $$K = \left[1 - \frac{1}{96}\left(\frac{\frac{d}{H_2}}{1 - \frac{d}{2H_2}}\right)^2\right]. \quad (6)$$

2.2 *Depth of flow at the spillway toe*

The depth of flow at the toe is the contracted depth behind the spillway. The toe depth, y, can be obtained via applying the energy equation at section 1-1, and section 2-2 where the contraction of flow occurs. To get an accurate estimation for the toe depth, the energy loss h_L between the above two sections should be taken into consideration.

Equating the specific energy equations at section 1-1, and 2-2, taking the downstream bed as a datum we get,

$$E_o = y + \frac{\alpha_2 v_2^2}{2g} + h_L. \quad (7)$$

The energy loss, h_L, on spillway surface can be expressed as, $h_L = \eta \frac{v^2}{2g}$,

where, v is the supercritical velocity at the end of the spillway, and η is head loss coefficient. Then Eq (7) becomes,

$$E_o = y + \frac{v_2^2}{2g}(\alpha_2 + \eta), \quad (8)$$

where, $$E_o = P + H_o,$$

$$H_o = H_2 + \frac{\alpha_1 v_a^2}{2g} = H_2 + \frac{\alpha_1 q^2}{2g(P + H_2)^2}.$$

Considering $\alpha_1 = \alpha_2 = 1.0$, and relating the coefficient η to the velocity coefficient C_v, which is the ratio of actual to theoretical velocity, by

$$C_v = \frac{1}{\sqrt{1 + \eta}}.$$

Substituting for C_v, and v in Eq (8), we get;

$$E_o = y + \frac{q^2}{2gy^2C_v^2}, \quad (9)$$

For rectangular section $y_c^3 = \frac{q^2}{g}$, where, y_c is the critical depth. Substituting for y_c in Eq (9), one get,

$$y^3 - E_o y^2 + \frac{y_c^3}{2C_v^2} = 0. \quad (10)$$

Eq. (2) is a cubic equation for y. Using Cardan's solution [14], Eq (10) has the following three roots;

$$y_1 = \frac{E_o}{3}\left[1 + 2\cos\frac{\theta}{3}\right], \quad (11\text{-a})$$

$$y_2 = \frac{E_o}{3}\left[1 - 2\cos(\frac{\theta}{3} + \frac{\pi}{3})\right], \quad (11\text{-b})$$

and $$y_3 = \frac{E_o}{3}\left[1 - 2\cos(\frac{\theta}{3} - \frac{\pi}{3})\right]. \quad (11\text{-c})$$

Equation (11-c) gives a negative values for y, while Eqs. (11-a), and (11-b) give values of $y_1 > y_c$ and $y_2 < y_c$, respectively. The depth y_1 represents the toe depth in submerged flow condition, while y_2 is the depth of free flow condition.

Considering the free flow condition, the flow depth at the spillway toe, y, can be calculated as,

$$y = \frac{E_o}{2}\left[1-2\cos\left(\frac{\theta}{3}+\frac{\pi}{3}\right)\right], \tag{12}$$

where, $$\cos\theta = 1 - 1\frac{6.75}{C_v^2}\left(\frac{y_c}{E_o}\right)^3. \tag{13}$$

3. EXPERIMENTAL ARRANGEMENTS

Experiments were performed in a horizontal rectangular flume of 9.00 m length and 0.40 m width. The flume was fabricated from perespex sheets supported by a steel frame. Coated wooden models, for the ogee weir, were inserted into the flume. The gate was prepared from perespex and positioned to be moved vertically, where the gate lip is rested on the apex of crest at the closed position. The gate lip was finished sharp edged. The gate opening could accurately be read with a meter fixed on the gate. Depths of flow were measured using point gauge. Discharges w re measured by a V-notch weir. As shown in Fig (2), two different shaped crest spillway models, ..th vertical upstream face, were used. The first has an elliptical crest shape with axes ratio of 2:1 constructed, according to the data reported in Ref. [1], for a design head H_D = 9.0 cm. The second model was shaped, according to the USACE standard shape, for a design head H_D = 7.0 cm. For each shape two heights, of the spillway crest P, were used; 20.80 and 14.90 cm for the first model, and 19.5, and 17.5 a cm for the second.

Pressures on spillway surface were measured, for the second shape only, by piezometers. Seventeen piezometers were installed at right angle to the spillway surface at its mid-length. A copper tubes of 1.0 mm internal diameter were used for the opening on the spillway surface. The copper tubes were connected, using polythene tubes, to open tubes manometers mounted on a vertical board.

Experiments were conducted by allowing the flow to pass through gate opening. Passing discharges ranged from 100 to 1250 cm^3/sec/cm, The ratios of H_2/p, and d/H_D ranged from 0.1 to 1.0, and from 0.1 to 1.5, respectively. For each discharge, the gate opening was increased according to a relative height,d/H_2, ranges from 0 to 0.71. For each gate opening the head H_2 and the toe depth y were measured. Pressure on spillway face were measured according to a relative head H/H_D = 0.5, 0.75, 1.0, 1.25, 1.5, 1.75 and 2.0.

4. RESULTS AND DISCUSSION

4.1 Discharge Characteristics

(i) The upper limit of controlled flow

It is important for application purposes, to determine the limits within which the flow is considered controlled. This mainly depends on the maximum height of the gate opening conditioned with gated flow. For this purpose, a set of experiments were conducted on ungated spillway models as shown in Figure (1-a). The depth of flow at the apex, h, was measured, for different discharges, and related to the head on spillway, H as depicted in Table (1). The average value of h/H is found to be 0.765. Thus, controlled flow conditions are valid for values of $d/H_2<0.765$. On the other hand, ungated flow conditions prevail whenever $d/H_2>0.765$.

Table (1) values of flow depth at the crest apex,h.

No	q, cm^3/sec. /cm.	H, cm.	h, cm.	h/H
1	115.1	3.36	2.53	0.753
2	173.5	4.28	3.23	0.755
3	225.4	5.00	3.75	0.750
4	281.1	5.66	4.40	0.777
5	340.5	6.34	4.85	0.765
6	403.2	7.03	5.35	0.761
7	444.3	7.50	5.75	0.767
8	489.2	8.00	6.15	0.769
9	557.3	8.63	6..65	0.771
10	654.6	9.37	7.15	0.763
11	770.8	10.28	7.80	0.759
12	881.6	11.10	8.50	0.766
13	991.4	11.80	9.10	0.771
14	1102.2	12.53	9.60	0.766
15	1213	13.32	10.25	0.770
16	1327	14.00	10.85	0.775

(ii) *Discharge equation*

Referring to Eq (5), the value of $K \rightarrow 1.0$ for very small values of d/H_2 (d/H_2 <0.2). Hence the discharge equation may be expressed as

$$q = C_d \sqrt{2g} H_2^{3/2} . \frac{d}{H_2} \sqrt{1 - \frac{d}{2H_2}} \tag{14}$$

Considering the upper limit of controlled flow for which $d/H_2 = 0.765$, the value of k equals 0.985, resulting a maximum deviation of 1.5% between Eqs. (4) and (14). Therefore, Eq (14) may recommended as a discharge equation due to its simplicity in application. Noting that, the discharge equation predicted by Hager [11] results more than 2% deviation comparing with equation (4).

(iii) *Discharge coefficient*

The data of 140 experimental tests, are used to evaluate the discharge coefficient for controlled spillway flow . Data includes wide ranges of different factors involved in the discharge equation. Thus, H/H_D ranges from 0.5 to 2.0 d/H_2 varies between 0.05 and 0.71, and H_2/p ranges from 0.1 to 1.0.

The discharge coefficient is mainly affected by the values of the design head H_D and the height of gate opening d. The spillway crest is shaped according to the value of the design head. Hence, the discharge coefficient will be evaluated as a function of the ratio d/H_D.

The measured values of head H_1 and H_2 were substituted in Eq (1) to get the experimental values of the discharge coefficient C_d. The obtained values of C_d are plotted against values of the ratio d/H_d, as shown in Figure (3). An empirical equation describing the above relation is obtained, with correlation coefficient R = 0.975, in the form

$$C_d = 0.75 \left(\frac{d}{H_D}\right)^{0.081} \tag{15}$$

4.2. *Velocity coefficient*

To get the value of the toe depth, using Eqs. (12), and (13), the velocity coefficient C_v has to be known a prior. Values of the velocity coefficient, in previous studies, were roughly evaluated. Two ranges, for the coefficient C_v, are only given by Pavlovsky [13]; for gated ogee spillway C_v varies from 0.85 to 0.95, while it ranges between 0.97 and 1.0 for deep orifices. Using average values, for the above two ranges, leads to inaccurate evaluation of the toe depth.

In the present work, the velocity coefficient is experimentally evaluated, since it is difficult to be evaluated theoretically. Values of C_v, among other factors, are mainly affected by height of gate opening d, spillway height P, and head on weir H_2. Using experimental measurements of q, H_2, and y for different spillway heights and gate openings, values of C_v could be obtained from Eq. (9), where,

$$C_v = \frac{q}{y\sqrt{2g(E_o - y)}}$$

Analysis of the obtained experimental values of C_v showed that evaluation of coefficient C_v includes three ranges. Thus,

(i) For small values of gate opening height, $d/H_2 \leq 0.2$, values of the coefficient C_v are very close to unity, i.e., $C_v = 1.0$.

(ii) For moderate values of gate opening, $0.2 < d/H_2 \leq 0.45$, values of C_v vary from 0.98 to 1.0.

(iii) For values of $d/H_2 > 0.45$, the coefficient C_v mainly depends on the head H_2.

For the latter range, the values of C_v are plotted versus the ratio H_2/P as shown in Figure (4). Empirical equation, describing the above relation with correlation coefficient R = 0.97, is obtained in the form;

$$C_v = 0.98 + 0.06 \, Ln \frac{H_2}{P} \tag{16}$$

Equation (16) is valid whenever $0.1 \leq H_2/P \leq 1.0$, and $d/H_2 > 0.45$

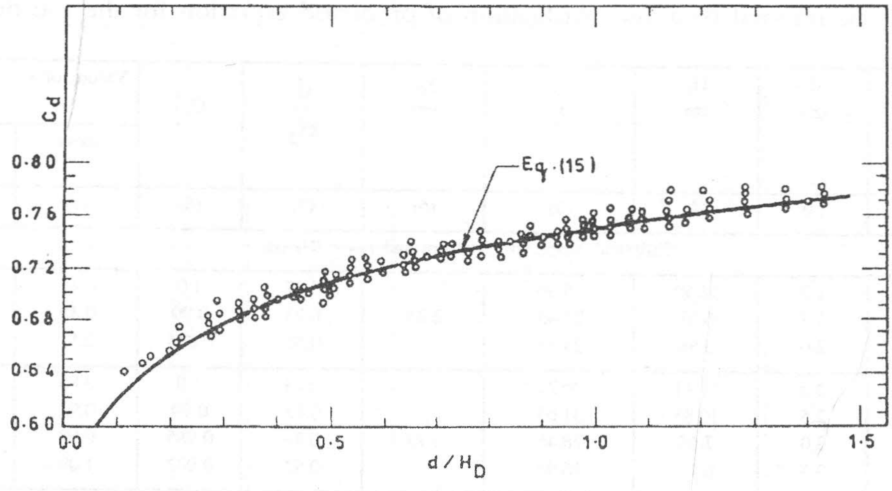

Figure 3. C_d —— $\frac{d}{H_D}$ relationship for the following conditions; H/H_D=0.5 to 2.0, d/H_2=0.05 to 0.71, and H_2/P=0.1 to 1.0.

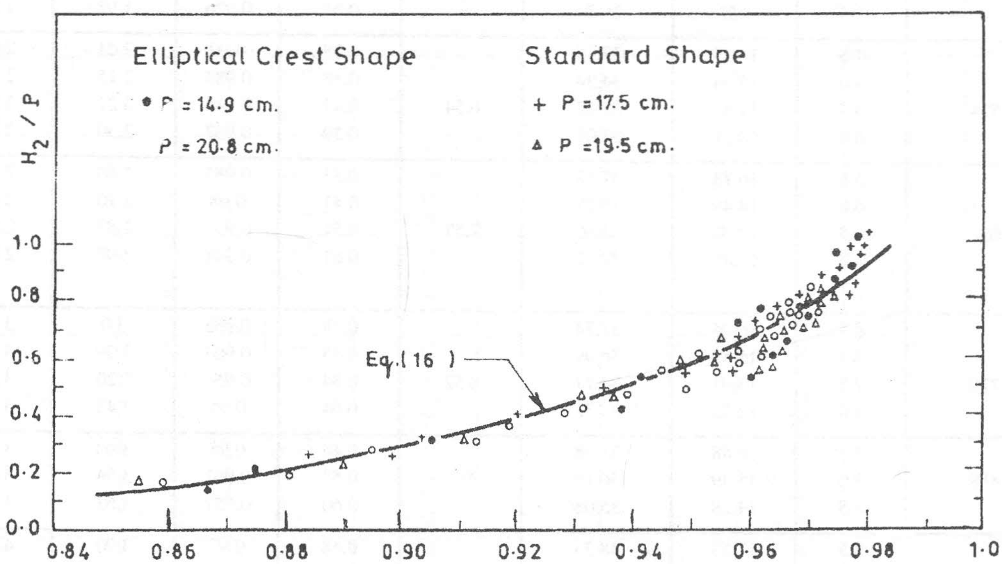

Figure 4. C_v —— $\frac{H_2}{p}$ relationship for values of $0.45 \leq d/H_2 \leq 0.71$.

4.3 *Flow depth behind the spillway*

Eq (12), intended for evaluating the toe depth y, was verified. The values of velocity coefficient were calculated via Eq. (16). The experimental values of P, H_2, y_c, and C_v were substituted in Eq (12). The resulting values of y are compared to the measured ones. As shown in Table (2), good agreement is obtained with maximum deviation of ± 3.0%.

Table 2. Experimental data and verification of predicted equation for the toe depth, y.

No	q, cm^3/sec/cm	d, cm	H_2 cm	E_o, cm	y_c, cm	$\frac{d}{H_2}$,	C_v*	Values of y meas.	Values of y cal.	Values of y %dev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Elliptical shape, p =20.8 cm and H_D = 9.0 cm										
1	106	1.0 1.5 2.0	14.50 6.63 3.58	35.30 27.43 24.38	2.25	0.07 0.23 0.56	1.0 0.99 0.874	0.41 0.45 0.57	0.40 0.46 0.56	-2.5 2.2 -1.8
2	226	2.0 2.5 3.0 3.5	15.41 10.85 7.65 6.13	36.21 31.65 28.45 26.93	3.73	0.13 0.23 0.39 0.57	1.0 0.99 0.985 0.907	0.85 0.93 0.98 1.08	0.86 0.93 1.0 1.11	1.2 0.0 2.0 2.7
3	332	3.0 3.50 4.0 4.50	14.52 11.55 9.21 7.74	35.32 32.35 30.01 28.54	4.83	0.21 0.30 0.43 0.58	0.99 0.985 0.98 0.921	1.32 1.36 1.46 1.53	1.30 1.37 1.43 1.57	-1.5 0.7 -2.0 2.6
4	423	4.0 4.5 5.0 5.5	13.85 11.22 9.69 8.50	34.65 32.02 30.49 29.30	5.67	0.29 0.40 0.52 0.65	0.99 0.98 0.934 0.926	1.70 1.80 1.92 1.97	1.68 1.77 1.91 1.97	-1.2 -1.7 -0.5 0.0
5	524	4.5 5.0 5.5 6.0	16.39 13.54 11.82 10.21	37.19 34.34 32.62 31.01	6.54	0.28 0.37 0.47 0.59	0.99 0.985 0.946 0.937	2.01 2.15 2.21 2.30	2.01 2.12 2.27 2.36	0.0 -1.4 2.7 2.6
6	652	5.5 6.0 6.5 7.0	16.73 14.49 12.80 11.42	37.53 35.29 33.60 32.22	7.57	0.33 0.41 0.51 0.61	0.985 0.98 0.951 0.944	2.60 2.70 2.87 2.97	2.53 2.63 2.79 2.88	-2.7 -2.6 -2.8 -3.0
7	779	6.5 7.0 7.5 8.0	16.94 15.59 13.97 12.52	37.74 36.09 34.77 33.32	8.52	0.38 0.45 0.54 0.64	0.985 0.963 0.956 0.95	3.0 3.09 3.20 3.42	3.03 3.18 3.28 3.38	1.0 2.9 2.5 -1.0
8	888	7.5 8.0 8.5	16.88 15.39 14.28	37.68 36.19 35.08	9.3	0.44 0.52 0.60	0.98 0.962 0.957	3.40 3.54 3.63	3.50 3.65 3.74	3.0 3.0 3.0
9	1008	8.5 9.0 9.5	17.53 15.84 14.71	38.33 36.64 35.51	10.12	0.48 0.57 0.65	0.97 0.964 0.959	3.90 4.05 4.27	4.01 4.14 4.25	2.8 2.2 0.5
10	1068	9.5 10	16.15 15.23	36.95 36.03	10.52	0.59 0.66	0.965 0.961	4.31 4.46	4.38 4.47	1.6 0.2
1	2	3	4	5	6	7	8	9	10	11
Elliptical shape, p = 14.9 cm, and H_D = 9.0 cm										
1	135	1.0 1.5 2.0 2.5	21.98 9.46 6.22 4.49	36.88 24.36 21.12 19.39	2.65	0.05 0.16 0.32 0.56	1.0 1.0 0.985 0.91	0.49 0.65 0.67 0.79	0.50 0.63 0.69 0.78	2.0 -3.0 3.0 -1.3

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
2	265	2.0	20.78	35.68	4.15	0.10	1.0	1.0	1.02	2.0
		2.5	13.73	28.63		0.18	1.0	1.14	1.16	1.8
		3.0	10.0	24.9		0.30	0.99	1.21	1.24	2.5
		3.5	7.73	22.63		0.45	0.941	1.34	1.38	3.0
3	403	3.0	20.72	35.62	5.49	0.15	1.0	1.52	1.56	2.6
		3.5	16.30	31.2		0.22	0.99	1.64	1.69	3.0
		4.0	12.48	27.38		0.32	0.985	1.78	1.83	2.8
		4.5	10.48	25.38		0.43	0.98	1.87	1.92	2.7
		5.0	8.87	23.77		0.56	0.949	2.01	2.06	2.5
4	510	4.0	20.06	34.96	6.42	0.20	1.0	1.95	2.0	2.6
		4.5	15.87	30.77		0.28	0.99	2.12	2.17	2.4
		5.0	12.89	27.79		0.39	0.985	2.31	2.31	0.0
		5.5	11.23	26.13		0.49	0.963	2.^T1	2.45	1.7
		6.0	9.74	24.64		0.62	0.955	2.58	2.56	-0.8
5	647	5.0	19.33	34.23	7.53	0.26	0.99	2.56	2.63	2.7
		5.5	16.55	31.45		0.33	0.985	2.70	2.77	2.6
		6.0	14.12	29.02		0.43	0.98	2.90	2.92	0.7
		6.5	12.75	27.65		0.51	0.955	3.0	3.09	3.0
6	800	6.0	20.38	35.28	8.67	0.29	0.99	3.27	3.22	-1.5
		6.5	17.95	32.85		0.36	0.985	3.47	3.38	-2.6
		7.0	15.59	30.49		0.45	0.98	3.65	3.55	-2.7
		7.5	14.42	29.32		0.52	0.978	3.7	3.64	-1.6
7	936	7.0	20.67	35.59	9.63	0.34	0.985	3.75	3.81	1.6
		7.5	18.75	33.65		0.40	0.985	3.9	3.94	1.0
		8.0	15.59	31.66		0.48	0.98	4.01	4.11	2.5
		8.5	14.42	30.25		0.55	0.98	4.17	4.23	1.4
1	2	3	4	5	6	7	8	9	10	11
Standard shape, p = 19.5 cm, H_D = 7.0 cm										
1	108	1.0	14.65	34.15	2.28	0.07	1.0	0.43	0.42	-2.3
		1.5	6.67	26.17		0.22	0.99	0.5	0.49	-2.0
		2.0	4.0	23.5		0.50	0.98	0.58	0.57	-1.7
2	237	.0	16.19	35.69	3.85	0.12	1.0	0.90	0.91	1.0
		2.5	11.04	30.54		0.23	0.99	0.97	0.99	2.0
		3.0	7.94	27.44		0.38	0.985	1.03	1.06	2.9
		3.5	6.44	25.94		0.54	0.914	1.15	1.17	1.7
3	347	3.0	15.67	35.17	4.97	0.19	1.0	1.32	1.35	2.3
		3.5	11.82	31.32		0.3	0.99	1.41	1.45	2.8
		4.0	9.36	28.68		0.43	0.98	1.50	1.53	2.0
		4.5	7.99	27.49		0.56	0.926	1.62	1.67	3.0
4	462	4.0	15.05	34.55	6.01	0.27	0.99	1.79	1.84	2.8
		4.5	12.72	32.22		0.35	0.985	1.93	1.92	-0.5
		5.0	10.68	30.18		0.47	0.944	2.03	2.08	2.5
		5.5	9.36	28.86		0.59	0.936	2.13	2.16	1.4
5	559	5.0	14.42	33.92	6.92	0.35	0.985	2.27	2.33	2.6
		5.5	12.71	32.21		0.43	0.980	2.34	2.41	3.0
		6.0	11.03	30.53		0.54	0.946	2.49	2.57	3.0
		6.5	9.95	29.45		0.65	0.94	2.57	2.65	3.0
6	676	6.0	14.67	34.17	7.75	0.41	0.98	2.79	2.78	-0.4
		6.5	13.29	32.79		0.49	0.957	2.89	2.92	1.0
		7.0	11.82	31.32		0.59	0.95	3.09	3.02	-2.3
		7.5	11.13	30.63		0.67	0.946	3.16	3.07	-3.0
		7.0	14.96	34.46		0.47	0.98	3.25	3.25	0.0
		7.5	13.59	33.09		0.55	0.958	3.35	3.39	1.2
7	784	8.0	12.31	31.81	8.56	0.65	0.952	3.45	3.50	1.5
8	892	8.0	15.12	34.62	9.33	0.53	0.965	3.66	3.76	2.7
		8.5	14.15	33.65		0.6	0.961	3.75	3.84	2.4
		9.0	12.70	32.20		0.71	0.954	3.87	3.98	2.8
9		9.0	14.98	34.48		0.60	0.964	4.17	4.2	0.7
	986	9.5	13.93	33.43	9.97	0.68	0.96	4.3	4.3	0.0
10	1120	10	15.75	35.25	10.85	0.64	0.967	4.59	4.73	3.0

Table 2. Continue.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Standard shape, p= 17.5 cm, and H_D = 7.0 cm										
1	132	1.0	18.26	35.76	2.61	0.06	1.0	0.50	0.50	0.0
		1.5	9.55	27.05		0.16	1.0	0.59	0.58	-1.7
		2.0	5.89	23.39		0.34	0.985	0.65	0.63	-3.0
		2.5	4.47	21.97		0.56	0.913	0.70	0.68	-2.9
2	247	2.0	17.12	34.62	3.96	0.12	1.0	0.94	0.96	2.0
		2.5	11.98	29.48		0.21	0.99	1.04	1.06	2.0
		3.0	8.50	26.0		0.35	0.985	1.15	1.13	-1.7
		3.5	7.01	24.51		0.50	0.925	1.24	1.25	0.8
		4.0	5.78	23.28		0.69	0.914	1.29	1.30	0.8
3	390	3.0	17.46	34.96	5.37	0.17	1.0	1.48	1.52	2.7
		3.5	14.67	32.17		0.24	0.99	1.57	1.61	2.5
		4.0	11.59	29.09		0.35	0.985	1.68	1.71	1.8
		4.5	9.70	27.2		0.46	0.945	1.80	1.85	2.8
		5.0	8.61	26.11		0.58	0.937	1.86	1.91	2.7
4	507	4.0	17.34	34.48	6.40	0.23	0.99	1.98	2.03	2.5
		4.5	14.83	32.33		0.30	0.99	2.04	2.10	3.0
		5.0	12.65	30.15		0.40	0.985	2.14	2.20	2.8
		5.5	10.7	28.2		0.51	0.95	2.30	2.37	3.0
		6.0	9.66	27.16		0.62	0.944	2.38	2.44	3.0
5	652	5.0	18.36	35.86	7.57	0.27	0.99	2.51	2.58	2.8
		5.5	15.61	33.11		0.35	0.985	2.63	2.71	3.0
		6.0	14.34	31.84		0.42	0.98	2.72	2.79	3.0
		6.5	12.55	30.05		0.52	0.96	2.89	2.95	2.0
		7.0	11.48	28.98		0.61	0.955	3.0	3.03	1.0
		7.5	10.53	28.03		0.71	0.95	3.10	3.11	0.3
6	751	6.0	17.83	35.33	8.32	0.34	0.985	3.05	3.03	-0.7
		6.5	15.48	32.98		0.42	0.98	3.15	3.17	0.6
		7.0	14.14	31.64		0.50	0.967	3.26	3.30	1.2
		7.5	12.60	30.10		0.60	0.96	3.36	3.42	1.8
		8.0	11.55	29.05		0.69	0.955	3.46	3.52	1.7
7	896	7.0	18.30	35.80	9.35	0.38	0.985	3.70	3.62	-2.0
		7.5	16.82	34.32		0.45	0.978	3.85	3.74	-3.0
		8.0	15.16	32.66		0.53	0.971	3.91	3.88	-0.8
		8.5	13.94	31.44		0.61	0.966	4.02	4.00	-0.5
		9.0	12.78	30.28		0.70	0.961	4.16	4.11	-1.2
8	1005	8.0	18.13	35.63	10.1	0.44	0.985	4.0	4.10	2.5
		.5	16.65	34.15		0.51	0.977	4.19	4.25	1.4
		9.0	15.23	32.73		0.59	0.972	4.30	4.39	2.0
		9.5	14.01	31.51		0.68	0.967	4.45	4.52	1.6
9	1120	9.0	17.98	35.48	10.85	0.50	0.98	4.51	4.64	2.9
		9.5	16.79	34.29		0.57	0.978	4.62	4.76	3.0
		10.0	15.81	33.31		0.63	0.974	4.75	4.87	2.5
		10.5	14.40	31.90		0.69	0.968	4.9	5.04	2.9

* $C_v = 1.0$, for $d/H_2 \leq 0.2$,

$C_v = 0.98$ to 1.0, for $0.2 < d/H_2 \leq 0.4$,

$$C_v = 0.98 + 0.06 \ln \frac{H_2}{p}, \text{ for } d/H_2 > 0.45.$$

Another procedure may be behaved to obtain an approximate determination for the toe depth. Rearranging Eq (10) to be written in the form;

$$y^2\left(1-\frac{y}{E_o}\right)=\frac{y_c^3}{2C_v^2E_o}$$

Analysis of experimental results showed that, the term (1- y/E_o) $\rightarrow$1.0 when E_o/y_c > 10. Hence, Eq (10) can be simplified resulting an approximate equation for y in the form;

$$y=\frac{y_c}{C_v}\sqrt{\frac{y_c}{2E_o}}\,,\qquad \frac{y_c}{E_o}<0.1 \qquad (17)$$

4.4. *Pressure distribution on spillway surface*

The pressure distribution along the spillway surface is considered for specific values of the relative head H/H_d equal 0.5, 0.75, 1.0, 1.25, 1.5, 1.75, and 2.0. For each of the former values, pressure head P/γ was measured for different relative heights of the gate opening d/H_D as shown in Figure (5-a,b......g). The pressure distribution on spillway surface may be classified into three zon pressure on the upstream face, on the crest, and the downstream face.

The pressure distribution on the vertical upstream face does not change much from the normal linear distribution for small values of d/H_D as well as for great values of H_2/H_D. Thus, for cnstant value of H_2/H_D = 2.55, the deviation of pressure from the hydrostatic values equals zero when $H/H_D \le 1.0$. For $H/H_D > 1.0$, the maximum deviation is 2.5%. Considerable deviation is resulted for large values of d/H_D specially when $H/H_D > 1.25$.

The values of pressure on the crest depends upon d/H_D and H/H_D. As shown in Figure (5), when the gate is operated with small openings under high heads, negative pressure occurs on the crest. This negative pressure acts on the crest behind the gate whenever $H/H_D \le 1.0$., while it is exerted immediately below the gate for values of H/H_D > 1.0. For the same value of H/H_D, the negative pressure decreases as d/H_D increases. On the other hand, the negative pressure increases as H/H_D increases. For a constant value of H_2/H_D = 2.55, in the all considered values of $0.5 \le H/H_D \le 2.0$, the maximum values of relative negative pressure on the crest are plotted versus the corresponding values of d/H_D as shown in Figure (6).

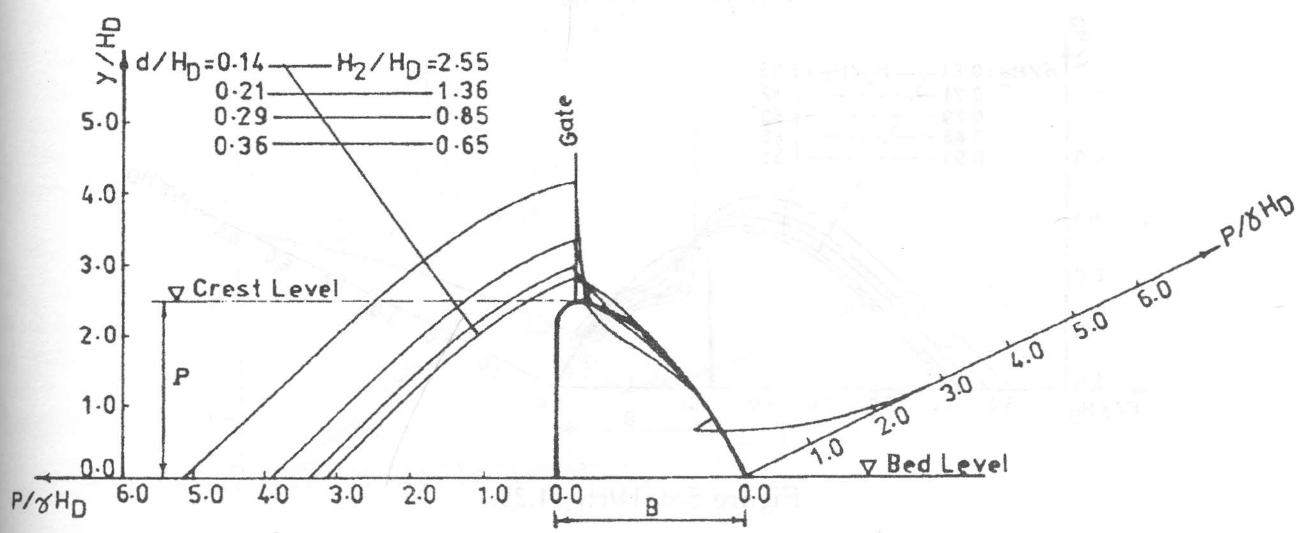

Figure 5. Pressure distribution on spillway surfac for: P-17.5 cm, H_D=7.0 cm, P/H_D=2.5, and B/H_D=2.7, (a) H/H_D=0.5.

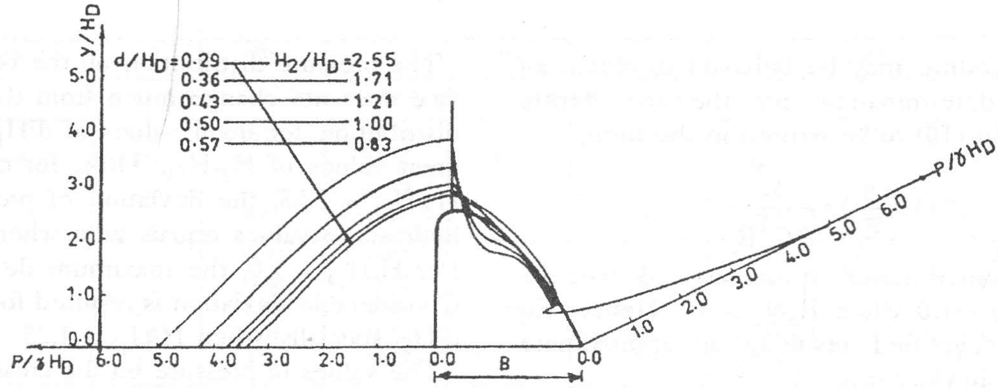

Figure 5-b. H/H_D=0.75.

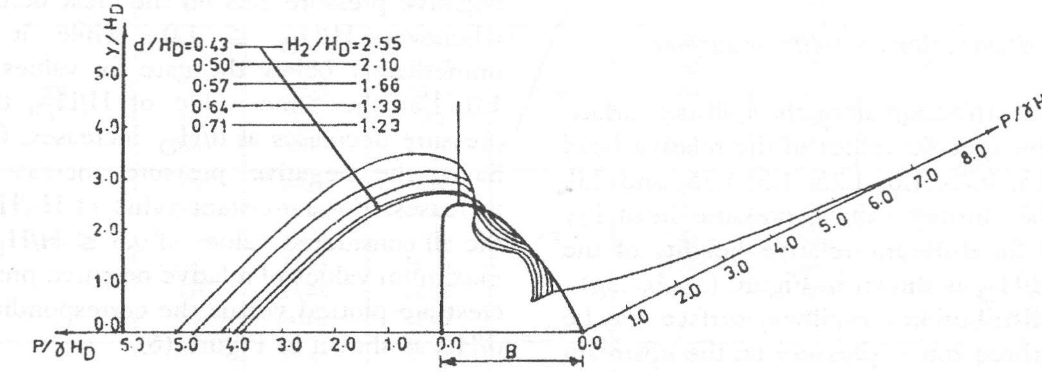

Figure 5-c. H/H_D=1.0.

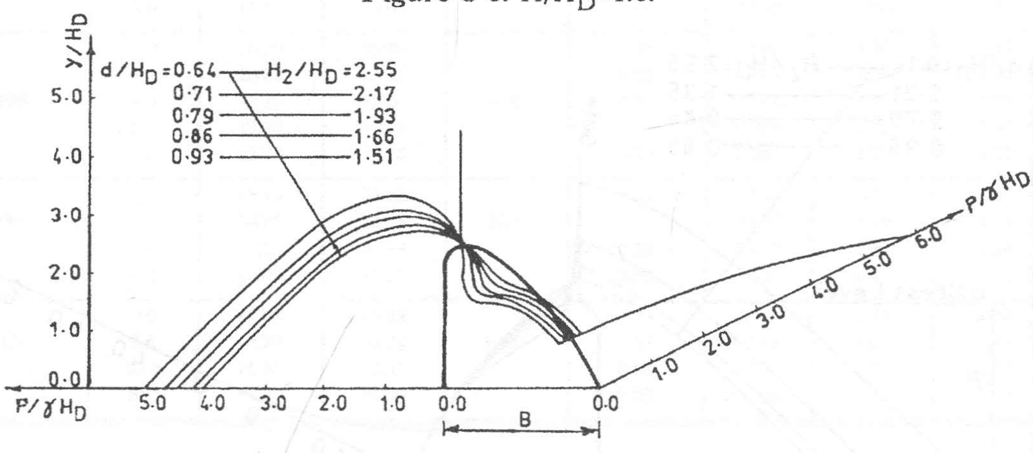

Figure 5-d. H/H_D=1.25.

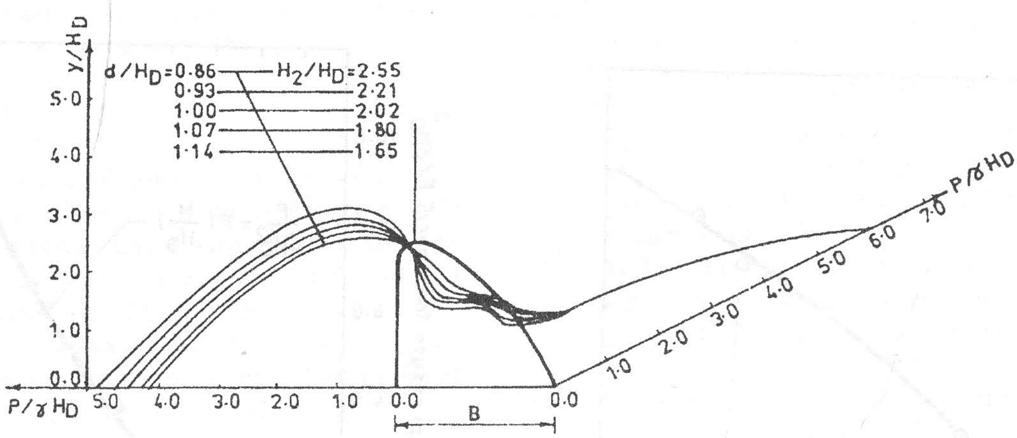

Figure 5-e. H/H_D=1.25.

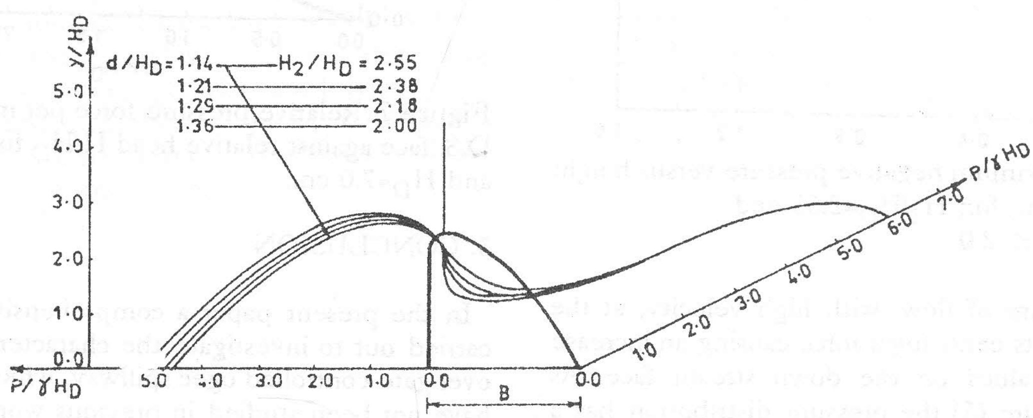

Figure 5-f. H/H_D=1.75.

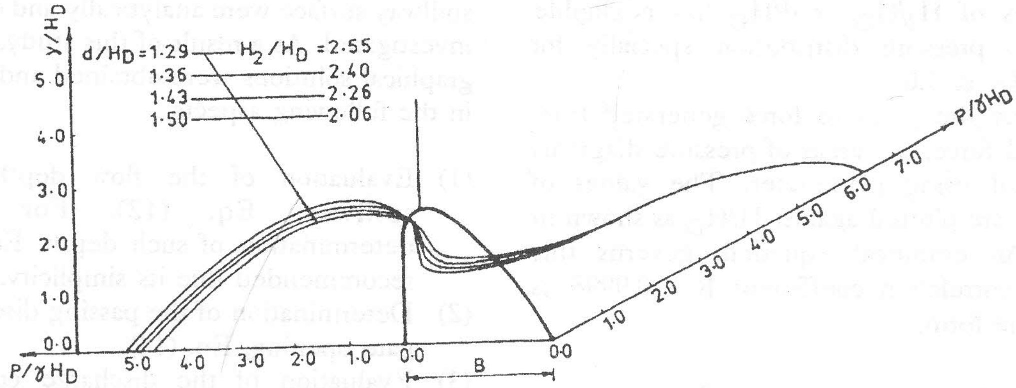

Figure 5-g. H/H_D=2.0

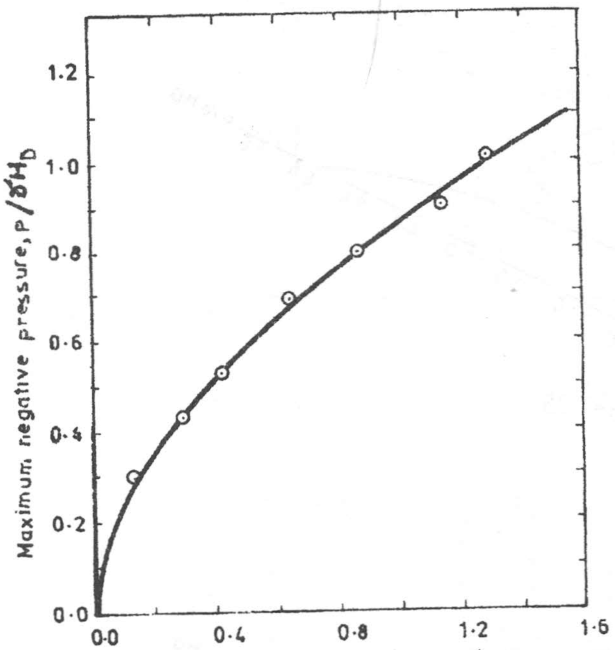

Figure 6. Maximum negative pressure versus height of gate opening for; H_2/H_D=2.55 and $0.5 \leq H/H_D \leq 2.0$.

The curvature of flow, with high velocity, at the toe zone results centrifugal force causing an increase in pressure values on the down stream face. As shown in Figure (5) the pressure distribution has a maximum value at the toe. For values of $H/H_D \leq 1.0$, the pressure line is concave while it is convex when H_2/H_D > 1.0. On the other hand, the variation of relative values of H_2/H_D or d/H_D has negligible effect on the pressure distribution specially for values of $H/H_D \leq 1.0$.

To determine the pressure force generated from the centrifugal force, the areas of pressure diagrams were measured using planimeter. The values of pressure force are plotted against H/H_D as shown in Figure (7). An empirical equation, governs this relation with correlation coefficient R = 0.9998, is obtained in the form;

$$F = \gamma H_D^2 \left[1.84 \left(\frac{H}{H_D} \right)^2 + 0.43 \left(\frac{H}{H_D} \right) \right] \quad \text{t/m}' \quad (18)$$

Equation (18) can be used to calculate pressure force for values of H/H_D up to 2.0.

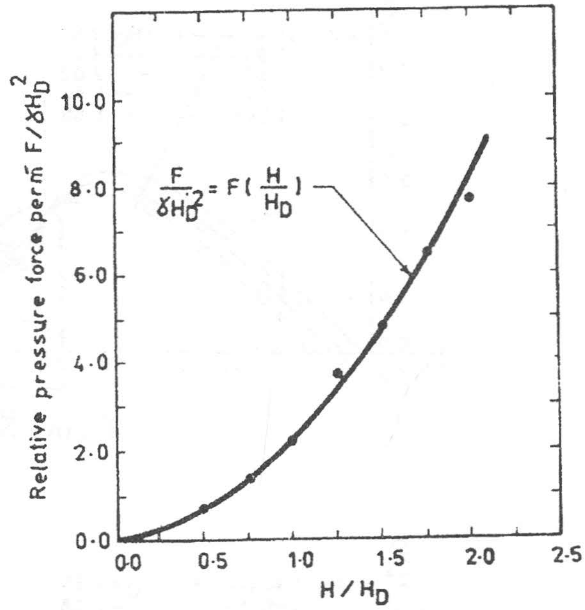

Figure 7. Relative pressure force per m' on spillway D.S. face against relative head H/H_D for; P=17.5 cm, and H_D=7.0 cm.

5. CONCLUSION

In the present paper a comprehensive study was carried out to investigate the characteristics of flow over gate-controlled ogee spillway, which sufficiently have not been studied in previous works.

The discharge characteristics, the depth of flow behind the spillway, the velocity coefficient, the discharge coefficient, and the pressure along the spillway surface were analytically and experimentally investigated. As a result of this study, analytical and graphical solutions were obtained and could be used in the following aspects;

(1) Evaluation of the flow depth behind the spillway, Eq. (12). For approximate determination of such depth, Eq (17) may be recommended due its simplicity.
(2) Determination of the passing discharge through gate opening, Eq. (14).
(3) Evaluation of the discharge coefficient as a function of gate opening and the design head, Eq (15).
(4) Estimation of the velocity coefficient at the contracted section of flow behind the spillway, Eq (16).

(5) Determination of the pressure force exerted on the downstream face, Eq (18).

REFERENCES

[1] P.G. Kiselov, *Reference Book For Hydraulic Calculations*, in Russian, Energia, Moscow, 1974.

[2] Overflow Spillway Crest with adjacent concrete sections, Hydraulic Design Chart 111-311, *USACE*, Vicksburg, Miss, 1964.

[3] Overflow Spillway Crest with adjacent embankment sections, Hydraulic Design Chart 111-313, *USACE*, Vicksburg, Miss, 1964.

[4] Overflow Spillway Crest, Hydraulic Design Chart 111-2/1, *USACE*, Vicksburg, Miss, 1971.

[5] V.T. Chow, *Open Channel Hydraulics*, McGraw-Hill Book Company, New York, 1959.

[6] M.A. Abourehim, "Characteristics of Flow Over Ungated Ogee Spillway" *Proc. Alex. Eng. Journal*, No. 3, sec, pp. 115-124, July, 1991.

[7] J.A. Roberson, J.J. Cassidy and M.H. Chaudhry, *Hydraulic Engineering*, Houghton Mifflin Company, Boston, 1988.

[8] P. Novak, A.I.B., Moffat and C. Nalluri, and R. Narayanan, *Hydraulic Structures*, UNWIN HYMAN, London, 1990.

[9] A. Lencastre, *Handbook of Hydraulic Engineering*, Chichester: Ellis Horwood, 1987.

[10] W.H. Hager, Discharge Characteristics of gated standard spillway. *Water power & Dam Construction*, 40 (1), 1988.

[11] W.H. Hager: Plane Gate on Standard Spillway", J. of the Hydraulic Engineering, *Proc. ASCE*, vol. 1114, No. 11, November, 1988.

[12] R. Wortman, "Spillway Discharge Calculations in NWS DAMBRK, J. of the Hydraulic Engineering, *Proc. ASCE*, vol. 115, No. 7, July, 1989.

[13] R.R.. Chugaev, *Hydraulics*, in Russian, Energoizdat, Leningrad, 1982.

[14] G.A. Korn, and T.M. Korn, *Mathematical Handbook*, for Scientist and Engineer, McGraw-Hill Book Company, New York, 1968.