EFFECT OF SUBMERGENCE ON DISCHARGE COEFFICIENTS OF DIFFERENT SPILLWAY WEIRS

A Dissertation
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MASTER OF ENGINEERING
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WATER RESOURCES DEVELOPMENT

BY
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CERTIFICATE

Certified that the dissertation entitled "EFFECT OF SUBMERGENCE ON DISCHARGE COEFFICIENTS OF DIFFERENT SPILLWAY WEIRS" which is being submitted by Shri S.K.Mohanty in partial fulfilment for the award of the Degree of MASTER OF ENGINEERING IN WATER RESOURCES DEVELOPMENT of the University of Roorkee is a record of candidate's own work carried by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for 9th months from 1.1.71 to 30.9.1971 for preparation of this dissertation.

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(SARAT KUMAR MOHANTY)

SYNOPSIS

Submerged flow over weirs is a common phenomenon during high floods. The discharge passing over the weirs reduces considerably because of the effect of submergence. The reduction in discharge quantity is evaluated by arbitrarity chosing some value of the coefficient of Discharge which normally gives much error in the stream flow data.

The work done in this field has been reviewed for different types of weirs both for the purpose of laboratory measurement and field measurement. Unfortunately the volume of Literature in this subject is rather not much considering the importance of the subject.

The relation between the discharge ratio and the submergence ratio has been analysed for different types of weirs taking the experimental data of some eminent hydraulic engineers. The analysis suggests distinctly different trends for both sharp crested and thick weirs e.g. ogee, broad crested etc.

Separate formulae corelating discharge ratio and submergence ratio for both the sharp crested and thick weirs have been proposed for use of field engineers.

NOTATIONS

(All Units are in F.P.S. System Unless otherwise stated)

a	Side slope of the trapezoidal Channel, Horizontal to vertical.
a'	Side slope of the trapezoidal notch, Horizontal to Vertical.
a_1	Weir area corresponding to head H_1
a_2	Weir area corresponding to head H_2
b	Base width of trapezoidal Channel
b'	Base width of trapezoidal notch.
$C, C_1 \& C_2$	Coefficient of discharge or constants used in general equations.
C_{d1}	Coefficient of discharge due to weir flow
C_{d2}	Coefficient of discharge due to orifice flow
C_F	Discharge coefficient proportional to Froude's number
d	Depth of tail water below the weir
d_1	Height of V notch
F	Area of water way in the trapezoidal channel
F'	Area of water way in the trapezoidal notch
g	Acceleration due to gravity
H_1	Head over crest upstream of weir, $H_1 = H_1' + h_a$
H_2	Head over crest downstream of weir, $H_2 = H_2' + h_a$
H_1'	Upstream depth of water over crest
H_2'	Downstream depth of water over crest
h_a	Head due to velocity of approach
h_d	Difference between T.E.L. Upstream and T.E.L. Downstream, $H_1 - H_2$
h_d'	$H_1' - H_2'$, Difference in head water and tail water levels.

K	Constant used in some equations
L	Length of the weir
L'	Effective length of the weir after deducting end contraction effect
'm'	Used as exponent by Villemonte
n	Number of end contractions or exponent
P	Height of the weir from Channel bed up to crest level
Q	Discharge under submerged condition per Unit length
Q'	Total discharge over the weir in submerged condition
Q_1	Discharge under free condition for head H_1 per unit length
Q'_1	Total discharge over weir for head H_1 in free condition
Q_2	Discharge under free condition at head H_2 per unit length.
Q/Q_1	Discharge ratio
S	Submergence factor $= \dfrac{a_2\sqrt{H_2}}{a_1\sqrt{H_1}}$
S_1	Submergence ratio H_2/H_1
V	Velocity of approach
θ	Angle of V notch
μ	Coefficient of dynamic viscosity
ρ	Mass density of water, fluid mass per unit volume
σ	Surface tension force per unit length
η	Reduction factor due to effect of submergence. given by Anderson & Dahl $= 0.392 + 2.00\frac{h_d}{H_1} - 2.31\left(\frac{h_d}{H_1}\right)^2 + 0.915\left(\frac{h_d}{H_1}\right)^3$

CONTENTS

CHAPTER-1

INTRODUCTION

CHAPTER - 1

INTRODUCTION

1.0. GENERAL

1.1. A weir, as it is normally understood is a barrier across a river or a stream for the purpose of diverting part or all of the water in to a channel which takes off from the upstream side for irrigation or Power generation. In India, the weirs have been constructed on several rivers of north and south for more than a century. With more stress on agriculture and irrigation more nos. of weirs are being constructed in the country at the present moment. Weirs are normally of low height. It creates a small pool of water at the upstream during the period of normal flow and the excess water trickles over the top of the weir, known as the crest. During the monsoon period, the flow passes over the weir submerging the entire structure. The presence of the structure is only felt by a slight depression of the water level there (Fig.1).

1.2. Existing weirs are also being utilised for the measurement of stream flow. The usual formula is $Q_1 = C.L'.H_1^{3/2}$ where C is a coefficient which depends on the shape of the weir profile, H_1 is the head over crest, L' is the effective length of weir and Q_1, the discharge required. For normal flows, a fairly accurate estimation of stream flow data is obtained from the weir formula.

1.3. As the head over crest increases due to flood flow the water level downstream of the weir also increases. As this process goes on, the downstream water level or tail water level submerges the crest of the weir by a considerable height. This phenomenon is known as submerged flow over the weir. In this condition the stream flow data becomes unreliable because of the arbitrarily chosen value of the coefficient of discharge in the weir formula. Although many attempts have been made to study this aspect of hydraulics , People have been partially successful because of many uncertain factors. A little more about this aspect of effect of submergence on the coefficient of discharge in a weir will be studied by the author on different types of weirs.

1·4·0 DIFFERENT TYPES OF WEIRS

1·4·1 - In the hydraulics laboratories, the weirs are most extensively used for measurement of flows. A weir is a notch

The different types of weirs are illustrated in Fig.3.

trapezoidal shaped etc. are put in this category.

The approach channel is the channel leading up to the weir and the mean velocity in this channel is called velocity of approach. If the nappe discharges in to air the weir has free discharge. If the discharge is partially under water the weir is said to be submerged or drowned.

2.2 Weirs can also be classified in a different way.

A. According to their shape at right angles to flow. In this category are rectangular, triangular, parabolic, proportional, trapezoidal, circular, cuspidate parabolic etc. All these are in the category of sharp crested weirs.

B. According to their shape in the direction of flow. The types of weirs in this type are broad crested weirs, ogee type etc. The different types of weirs classified under A & B above are shown in figure 3.

1.5.0 ~~2.3~~ Definitions:

Suppressed Weir:

A sharp crested measuring notch whose sides are flush with the channel, thus eliminating (suppressing) end contraction of the overflowing water. The weir may be suppressed on one end, two ends bottom or any combination of them.

Contracted Weir:

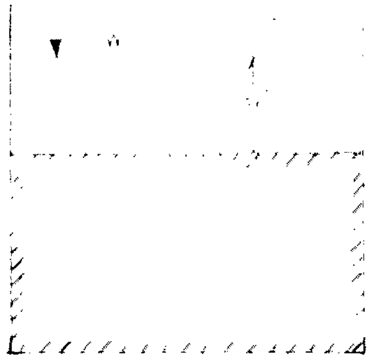
A sharp crested measuring notch with sides designed to produce a contraction in the area of the overflowing water.

Notch:

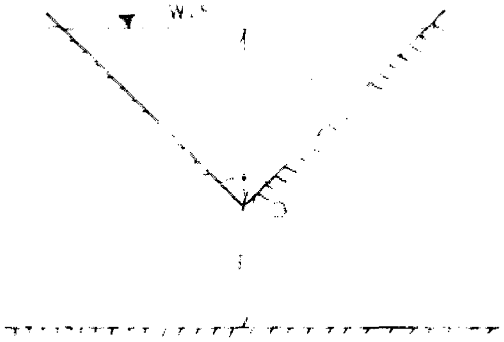
The opening in a weir for the passage of water.

Rectangular Weir:

A sharp crested measuring weir with a rectangular notch. It may be contracted or suppressed.



RECTANGULAR SUPPRESSED WEIR



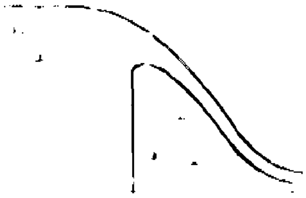
TRIANGULAR WEIR

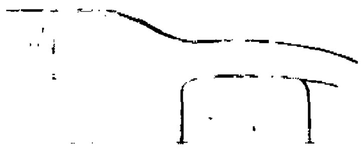
PARABOLIC WEIR

PROPORTIONAL WEIR

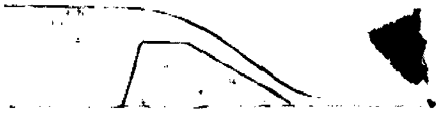
A) ACCORDING TO SHAPES OF WEIRS

FIG. 3 DIFFERENT TYPES OF WEIRS





BROAD CRESTED



TRAPEZOIDAL

B) ACCORDING TO SHAPE IN THE FLOW DIRECTION

Broad Crested Weir:

A spillway or a measuring device in which the nappe is supported for an appreciable length (much larger than the height of the nappe over the crest) at the throat in the direction of flow.

Triangular Weir or V.Notch:

A contracted sharp crested measuring weir notch with sides that form an angle with its apex downward; the crest is the apex of the angle.

Free Overfall Weir:

A weir that is not submerged. In case of sharp edged weirs, there shall be free access of air below the nappe of falling water, and the tail water is below the bottom of the notch. In broad crested weir, the tail water is below the crest and the flow is in no way affected by the elevation of the tail water.

Ogee Weir:

A weir having crest of ogee shape.

Parabolic Weir:

A measuring weir whose notch is bounded on the sides by parabolas.

Circular Weir:

A measuring weir having circular crest.

Proportional Weir:

A weir in which the rate of flow is simply proportional to the head over the weir.

Coefficient of Discharge:

A coefficient by which the theoretical discharge of water through orifices, weirs or other hydraulic structure must be multiplied to obtain the actual discharge .

Velocity of Approach:

The mean velocity in the stream immediately upstream of a weir or a dam or other structure.

Velocity Head:

The theoretical vertical height through which a liquid body may be raised due to its kinetic energy . It is equal to the square of the velocity divided by twice the acceleration due to gravity.

2.4 The following are the discharge formulae for some of the different types of weirs:

1. Rectangular $Q = \frac{2}{3} C_{d1} L' \sqrt{2g} \left[(H+h_a)^{3/2} - h_a^{3/2} \right]$

2. Triangular $Q = \frac{8}{15} C_{d1} \sqrt{2g} \tan \frac{\theta}{2} \quad d_1^{5/2}$

3. Broad Crested Weirs: $Q = CL' \left[(H + h_a)^{3/2} - h_a^{3/2} \right]$

4. Ogee Shaped Weir: $Q = C.L' \left[(H + h_a)^{3/2} - h_a^{3/2} \right]$

Where 'C' is the coefficient of discharge which is different in different weirs.

1.6.0 SCOPE OF DISSERTATION

1.6.1 Many attempts by different questers have been made to piece together experimental data from various sources on flow over submerged weirs. As submerged flow at its best is unstable, it is not difficult to understand why these attempts have been only partially successful. Even if some experimental data can be pieced together its scope would not be sufficient to represent the picture as a whole. Therefore an attempt is made to pool all available data on the submerged weirs and to analyse it to derive some statistical and graphical correlations to aid in the design of submerged dams and also to get general idea about the correct measurement of flow under submerged conditions. Study has been made to probe if a general equation can represent all types of weirs under submerged conditions. It is a well known fact that the coefficient of discharge in a weir is affected by many factors. Out of which the effect of submergence is one of the major factor. In this dissertation attempts are made to study the effect of submergence only on the coefficient of discharge.

~~1.6.~~ 1·7·0 ~~Scope Of Future Work:~~

~~There is a lot of scope to do future work in this subject. The co-relation of the submergence and the discharge ratios may be done for a wide range of discharges over different widths of the flume. From the data analysed, though various discharge rates are allowed over the weirs no definite trend, as to whether the rate of flow has any bearing on the corelation of Q/Q_1 and S_1, could be established. The aspect of scale effect also could not be studied. For very low range of discharges the viscosity and surface tension effects may be Predminent and these must also be considered. Further studies in these regard are needed.~~

More studies may also be made to analyse the effect of the height of the weir. From a very low value of P/H_1 to high values the effect of this non-dimensional parameter on the discharge ratio may be established.

The geometry of the weir is another very important factor. Although some studies have been made on ogee type crests very few studies have been made on broad crested weirs. Different shapes of broad crested weirs and other weirs over a wide range of discharges may be subjected to model testing up to almost 100 percent submergence. This may be done in the same manner as that of James. G.Woodburn in the non modular range of flow.

CHAPTER-2

REVIEW OF WORK DONE

CHAPTER- 2

REVIEW OF WORK DONE

2.0 INTRODUCTION

Submerged flow, defined as the condition that exists when the tail water at a dam or a weir is at an elevation higher than that of the crest, often occurs during floods at low head dams and weirs. Submerged weirs conserve elevation and should have wide application in the design of gravity systems where savings in head loss mean saving in construction costs.

2.0.1 Despite the considerable importance of the problem, published data on submerged flow over dams are exceedingly rare. Therefore engineers hesitate to use submerged weirs for liquid measurement and control in open channels because of lack of design and performance data. These early theories satisfied the limited experimental data of the persons postulating them but failed to ~~convince~~ corelate with the other experimental work done later on to check the theories.

~~2.0.2 It may be mentioned here that a lot of work has been done for free flow over different types of weirs. These works are not reviewed here being beyond the scope of this dissertation.~~

2.1 The formula which is being used to determine the approximate discharge over submerged weirs is as follows:

$$Q = Cd_1 \cdot \frac{2}{3} L \sqrt{2g}\,(H_1 - H_2)^{3/2} + Cd_2\, L\, H_2 \sqrt{2g\,(H_1 - H_2)}$$

(Ref. Hydraulics by Prof. N.S. Govind Rao)

In order to derive the expression it was necessary to make some simplifying assumptions. Possibly one such assumption was first made by Nancay Dubuat (Principles d' Hydraulique Vol. 1F. p. 203,1816). He considered the flow over submerged weir to be made up of two parts.

(1) a free flow with effective head equal to the difference in upstream and downstream levels, and

(2) a submerged orifice flow like wise operating under a head equal to the difference in upstream and downstream levels.

The difficulty with the above equation is in the correct assumption of the values of C_{d1} and C_{d2} which are only to be obtained from model experiments. For practical problems where approximate discharge calculations are required a value of 0.62 for C_{d1} and 0.9 for C_{d2} is usually assumed. Ref. Hydraulics by Prof. N.S. Govind Rao

2.2.0. Sharp Crested Weirs:

2.2.1.0. Probably the first man to attempt measurement of flow under submerged condition was Joannis Poleni[5], Professor at the University of Padua, Italy in 1717. He wrote a book DE MOTU AQUARE MIXTO on the flow of water through submerged rectangular notches. A translation of this work into Italian was published about 1767, but Poleni's work has rarely been mentioned subsequently. Yet his experiments and analysis merit careful study today.

2.2.1.1. Poleni's experiments were conducted with an apparatus (Fig.4) at a stream 12 ft. (3.66 m) wide and 3 feet (0.915 m) deep. The weir tank P, was 30 Pouces in diameter.......

(old french measures : 1 pied = 12 pouces = 144 lignes = 1.066 ft. = .325 m.). A rectangular notch IGEL, was set vertically with its crest GE at different elevations with reference to the tail water. Rates of flow were measured by means of an orifice tank S which in turn was supplied with water from another large tank T through spigots Y. Tank S was 42 pouces in diameter and 21 pouces in depth below the level of an overflow M. The bottom of this tank was provided with sixteen orifices, K, each 8 lignes in diameter (1.8 cm.).

In Poleni's first experiment, notch IGEL was 31/2 lignes (1.38" , 3.5 cm.) wide . Three orifices discharged water in to the weir tank under a head of 21 pouces (22.4" , .569 m.)

The water level inside the tank P under these conditions was 35/4 lignes (0.78 inch , 1.99 cm.) above tail water. Poleni systematically varied width of notch, submergence of sill below tail water level and rate of flow. The data summarized in Table 1 may well be the first to be obtained in systematic experiments on submerged weirs.

Head on the weir notch versus a factor proportional to rate of flow per unit width of notch discharging freely are shown in figure 5. The observations deviate on the average less than 2% from the equation $KQ/B = 0.080\, H_1'^{1.5}$ where,

N = number of orifices open

$KQ = 100\, N$

B = Width of notch in lignes

H_1' = Head water depth in lignes

Q = Cu.ft. per sec.

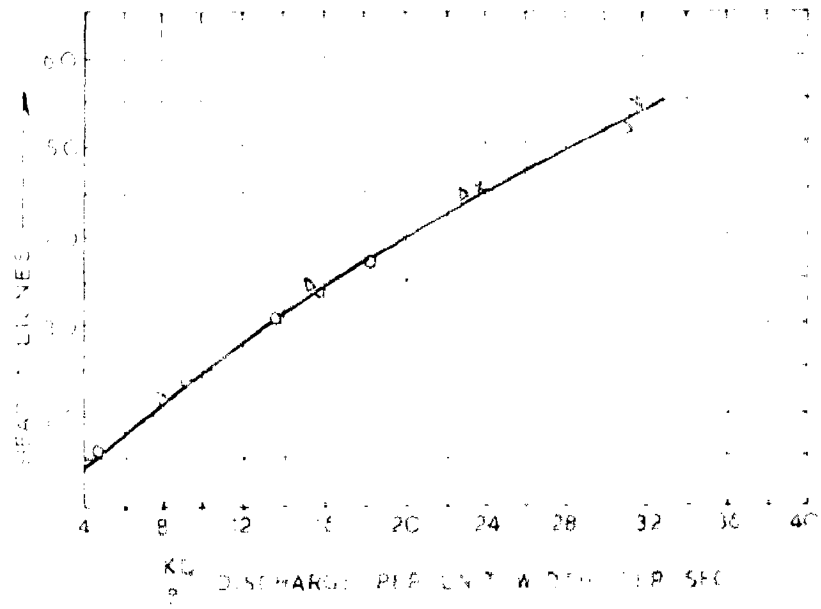


FIG 3 DISCHARGE VS HEAD - POLENI'S EXPERIMENT

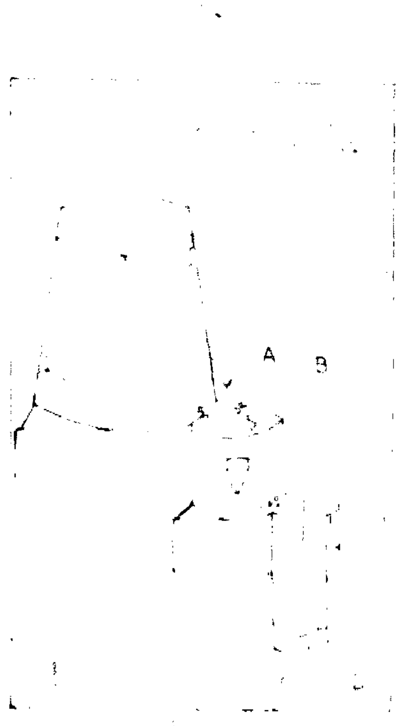
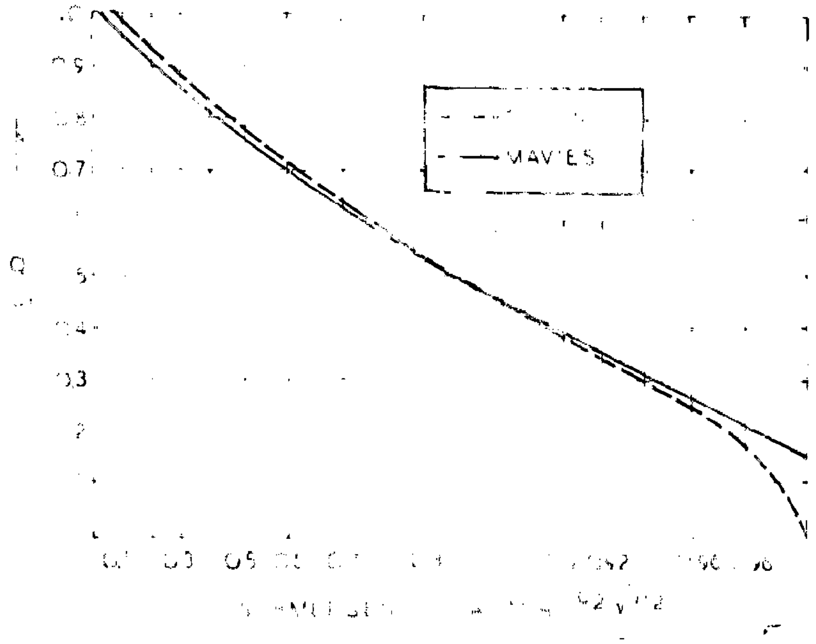


FIG 4 POLENI'S EXPERIMENT



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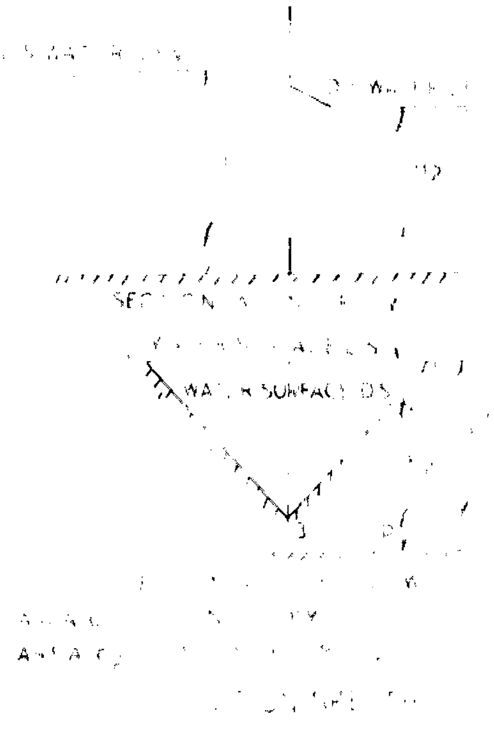


TABLE-1

SUMMARY OF EXPERIMENTS ON FLOW THROUGH RECTANGULAR NOTCHES BY JOANNIS POLENI, 1717

(12 Pouces = 144 lignes = 1.066 feet = 0.325 m)

B= Width of notch in lignes 1	H_2'=tail water depth in lignes. 2	N = no. of orifices Open. 3	$H_1'-H_2'$ = Head water tail water differential in lignes. 4
31/2	55	3	35/4
31/2	55	6	25
31/2	55	9	42
31/2	55	12	58
31/2	55	15	294/4
31/2	108	3	11/4
31/2	108	6	41/4
31/2	108	9	84/4
31/2	108	12	126/4
31/2	108	15	43
38	65/4	3	10
38	65/4	6	85/4
38	65/4	9	33
38	65/4	12	42
38	65/4	15	203/4
38	48	8	12
38	48	16	119/4
79	35	5	10/4
79	35	15	63/4

Contd..../

1	2	3	4
88	0(Free)	4	15
88	0(Free)	8	23
88	0(Free)	12	122/4
88	0(Free)	16	37
52	0(Free)	4	85/4
52	0(Free)	8	34
52	0(Free)	12	177/4
52	0(Free)	16	209/4
38	0(Free)	3	88/4
38	0(Free)	6	133/4
38	0(Free)	9	179/4
38	0(Free)	12	54
38	0(Free)	15	63

2.2.1.2. Poleni's Conclusions:

It was clear to Poleni that velocity of efflux was proportional to square root of the head on an orifice discharging freely. He showed experimentally that free discharge through a rectangular weir varied as the three-halves power of the head. He reasoned that velocity distribution in the section of the notch between head water and tail water level was parabolic and velocity in part below tail water level was uniform.

Hence the discharge factor for a rectangular weir notch according to Poleni's analysis can be expressed as $\frac{Q}{Q_1} = \left(1 + \frac{H_2}{2H_1} \right) \sqrt{1 - \frac{H_2}{H_1}}$.

(This was done by F.T. Mavies Ref.5).

2.2.1.3. An analysis of Poleni's experiments on the effect of submergence of flow through rectangular notches is given in figure 6. Data was Plotted by Mavies with discharge factor $\frac{Q}{Q_1}$ as ordinates and submergence factors $S = \frac{a_2\sqrt{H_2}}{a_1\sqrt{H_1}}$ as abscissa. Mavies found that the curve based on Poleni's data tallied accurately with that of his own curve based on tests conducted in 1948 at the carnegie Institute of Technology. This will be discussed later.

Poleni's curve can be represented by the equation

$$\frac{Q}{Q_1} = (1 + 0.5\, S^{2}) \sqrt{1 - S^{2}}$$

Where Q = submerged flow

Q_1 = corresponding free flow for head H_1

& S = submergence factor = $\frac{a_2\sqrt{H_2}}{a_1\sqrt{H_1}}$

2.2.2.0. Some experiments were conducted in 1877 by A.Fteley and F.P.Stearns[6] for ascertaining correctly the gaugings made for conveying the water from Sudbury river to Boston for a number of years. Most of their studies were for free flow conditions over sharp crested weirs. However some experiments were hastily done by them under submerged conditions.

2.2.2.1. The experiments were done first with 5 ft(1.53 m) long weirs with depth varying from 0.07 to 0.83ft(2.14 cm to 25.3 cm). Subsequently a second weir was tried with length 19'(5.79 m) and head range 0.47 to 1.63 ft.(0.1435 m to 0.497 m).

It was found by the investigators that

$$Q = C\,L\,\left(H_1' + \frac{H_2'}{2} \right) \sqrt{h_d'} \quad \text{where}$$

C = Experimental coefficient which was found to vary with $\frac{H_2'}{H_1'}$

L = Length of weir

H_1' = Upstream water level - crest level

H_2' = Downstream water level - crest level

h_d' = Upstream water level - Downstream water level.

The coefficient C which depended on H_2' / H_1' ratio was determined from figure 7. The heads H_1' & H_2' were also to be corrected to include the velocity of approach. The above formula was not applicable for H_2'/ H_1' less than 0.08.

2.2.3.0. J.B.Francis[7] also made some experiments in 1883 to compute the coefficients C and C' in the general submerged flow formula which can be written as

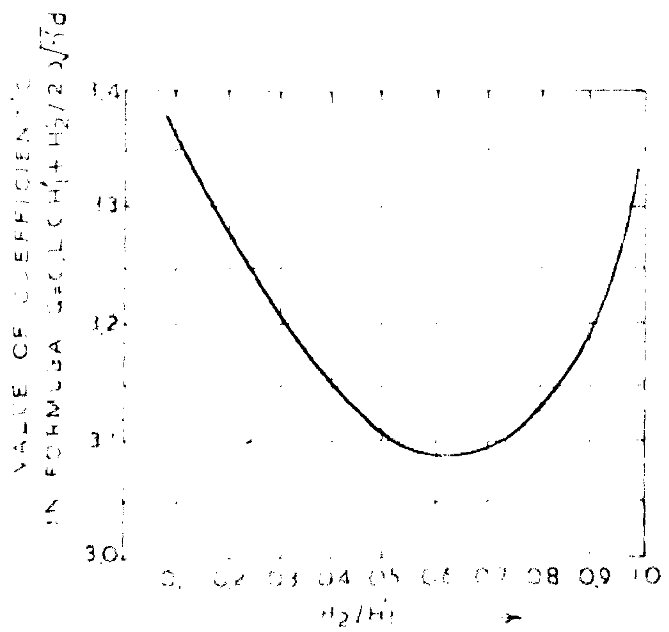
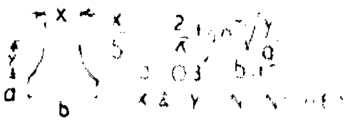
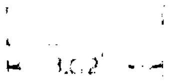
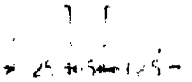


FIG 7 RELATION BETWEEN H2/H1 & COEFFICIENT OF DISCHARGE

(CARTER, STEELEY & STEARN [illegible])

WEIR NO	DESCRIPTION	SHAPE & SIZE	HEAD	FREE FLOW Q FORMULA	EXPON	NO. OF TESTS
[illegible]	SYMMETRICAL PROPORTIONAL	 X & Y IN INCHES	1.0*	Q = 0.8 [illegible]	1.0	4[illegible]
2	RECTANGULAR FULL WIDTH		2	$Q = 3.3[illegible]^{1.45}$	[illegible]	[illegible]
3	RECTANGULAR CONTRACTED		1	$Q_1 = 3.0[illegible]^{1.45}$	1.45	38
4	DO		.25	$Q = 3.0[illegible]^{1.45}$	.45	9
5	PARABOLIC	 $y = 3x^2$ IN FT	.83	$Q = 2.04[illegible]^{1.98}$	2.0	3[illegible]
6	[illegible] ANGULAR	 $y = x$ [illegible]	[illegible]	[illegible]	2.5	40
[illegible]	[illegible]	 y [illegible]	.83	$Q = [illegible]4[illegible]^{3.1[illegible]}$	[illegible]	[illegible]

[illegible]

$$Q = C_{d_1}\frac{2}{3}\cdot L\sqrt{2g}\,(H_1 - H_2)^{\frac{3}{2}} + C_{d_2}\cdot L\,H_2\sqrt{2g(H_1-H_2)}$$

$$= C.L.\,(H_1 - H_2)^{\frac{3}{2}} + C'.L\sqrt{H_1-H_2}\cdot H_2$$

The first term of the right side expression is the usual formula for computing the flow over a weir in which there is no obstruction to the flow from the water on the downstream side using for the depth of the weir $H_1 - H_2$ i.e. the part of the whole depth H_1 which is above the level of water on the downstream side is treated as a regular weir.

The second term is the usual formula for the flow through an orifice of the length L and height H_2 through which water is discharged under a head $H_1 - H_2$.

1.71 in metric units

2.2.3.1. Francis took C = 3.33 (fps units) which was found out by experiments given in Lowell hydraulic Experiments(1848).

8.29 in metric units

The term C' was found to be equal to 4.5988(fps units) by experiments conducted by him. The formula could also be expressed as follows:

$$Q = 3.33\,L\sqrt{H_1 - H_2}\,(H_1 + 0.381\,H_2) \quad (\text{f.p.s units})$$

If n = no. of end contractions

$$Q = 3.33\,(L - 0.1\,n\,H_1)\sqrt{H_1 - H_2}\,(H_1 + 0.381\,H_2)$$

(f.p.s. units).

2.2.4.0 James R. Villemonte[8] had conducted experimental work at the hydraulics laboratory of the Pennsylvania state college based on an application of the super-position principle. He obtained a general discharge formula for submerged sharp crested weirs. Results of the experiments also proved that:-

(1) Triangular and parabolic weirs are more accurate measuring devices than proportional and rectangular types and

(2) That sharp crested submerged weirs can be used in practice with confidence, if certain design and operational specifications are satisfied.

2.2.4.1. As a first approximation to the proper form of submerged discharge Q, as a function of the upstream and downstream heads H_1 & H_2 above the weir crest a simplifying assumption can be made. Assuming that the net flow over the weir is the difference of the free flow discharge due to head H_1 minus the free flow discharge due to head H_2, then

$$Q = Q_1 - Q_2 \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

This assumption implies that the head H_2 does not directly affect the flow of water due to H_1 and likewise, that head, H_1 does not prohibit counterflow due to H_2. Its use is thus equivalent to the application of the Principle of super position which is frequently used in evaluating the combined effect of several independent conditions.

The equation $\frac{Q}{Q_1} = 1 - \frac{Q_2}{Q_1}$ should not be expected to give a quantitative relation for determining Q, since interaction and other perturbing influences have been entirely neglected. These effects would probably contribute additional higher order terms in $\frac{Q_2}{Q_1}$.

The experimental tests by Villemonte show that Q/Q_1 is related functionally to $1 - \frac{Q_2}{Q_1}$ but that the appropriate function is not the simple linear one of equation $\frac{Q}{Q_1} = 1 - \frac{Q_2}{Q1}$ ...(2)

The results show that this relation ship may be expressed in the form

$$\frac{Q}{Q_1} = f\left(1 - \frac{Q_2}{Q_1}\right) \overset{=}{+} K\left(1 - \frac{Q_2}{Q_1}\right)^m \quad \ldots\ldots \quad (3)$$

or since $Q_1 = C_1 H_1^{n1}$ and $Q_2 = C_2 H_2^{n2}$

$$\text{then } \frac{Q}{Q_1} = K\left(1 - \frac{C_2 H_2^{n2}}{C_1 H_1^{n1}}\right)^m \quad \ldots\ldots \quad (4)$$

where K and m are constants to be obtained from experimental data and the C's and n's are the coefficients and exponents that appear in the free flow discharge equation obtained from previous calibration or by use of an appropriate standard weir formula. The values of C's and n's for the weirs tested by Villemonte are shown in figure 8.

For any given type of weir the coefficients C_1 and C_2 and exponents n_1 and n_2 should be equal. Then the equation (4) reduces to $\frac{Q}{Q1} = K(1 - S_1^n)^m$ where S_1 is the submergence ratio $= \frac{H2}{H_1}$. The constant K and the exponent m which account for the interaction effects were determined separately for each weir type by the algebraic method of averages. The results for the seven weir types tested by Villemonte when averaged arithmatically showed K equal to 1.00 and 'm' equal to 0.385 for a practical submergence range of 0.00 to 0.90. The maximum variation of each from the average is less than 1 percent.

Thus, the general discharge equation for all sharp crested weirs with crest curvatures expressed by continuous single valued functions is

$$Q = Q_1 (1-S_1^n)^{0.385} \quad \ldots\ldots\ldots\ldots \quad (6)$$

For all sharp crested weirs regardless of crest curvature the more general form of equation (3) should be used. Assuming the same constants to apply the equation for this case is

$$Q = Q_1 \left(1 - \frac{Q_2}{Q_1} \right)^{0.385} \quad \ldots\ldots\ldots \quad (7)$$

2.2.4.2. Experimental Test:

The seven experimental weirs described in fig.8 were mounted in turn mid way in a steel flume 3 ft.(0.91 m) wide 3 ft. (0.91 m) deep and 25 ft. (7.6 m) long. Discharge was measured by a calibrated standard 90° triangular sharp crested weir placed near the upstream end of the flume, and the degree of submergence was controlled by three 6 inch (15.25 Cm) vertical outlets passing through the flume floor at the downstream end and adjustable in elevation.

The same testing procedure was used for all weirs. After calibrating each weir for free flow by comparision with the triangular weir, the discharge was held constant and the degree of submergence changed 10 to 15 times from 0.00 to about 0.95. Upstream and downstream head observations were taken for each submergence setting using point gages reading in 4 in(10 cm) stilling tubes. For each type of weir the same procedure was followed using three additional constant discharge rates. Detailed observations were recorded on the behaviour of the nappe and downstream flow conditions.

The experimental results show that the dimensionless terms $\frac{Q}{Q_1}$ and $1 - \frac{Q2}{Q_1}$ have a consistent functional relation for all sharp crested weir types. A logarithimic plotting of simultaneous values of Q/Q_1 and $1-S_1^n$ for each experimental weir (Fig.9) shows the general conformity of all data with equation 5. The constant K and exponent m were evaluated as 1.00 & 0.385 respectively.

It should be noted that as S_1 increases the term $1-S_1^n$ decreases. All tests at degree of submergence ratio S_1 0.90 were excluded from the calculations so that the results would typify more closely practical submergence conditions.

The confirmity of each test with equation (6) is illustrated in Fig.10. The degree of submergence is plotted against the percentage correction that must be applied to Q computed by equation 6 to give the measured discharge rate. Average curves are shown for each constant discharge series, together with their ranges in P/H_1 and equivalent free flow heads H_{01}. The point of transition from surface to plunging nappe conditions is also indicated.

2.2.4.3. Conclusions:

Results of experiments conducted by Villemonte agree favourably with those of earlier workers.Values of Q/Q_1 found by equation 6 are compared in Table 2 with a summary of existing data on rectangular notches compiled by Vennard, Weston & Stevens.

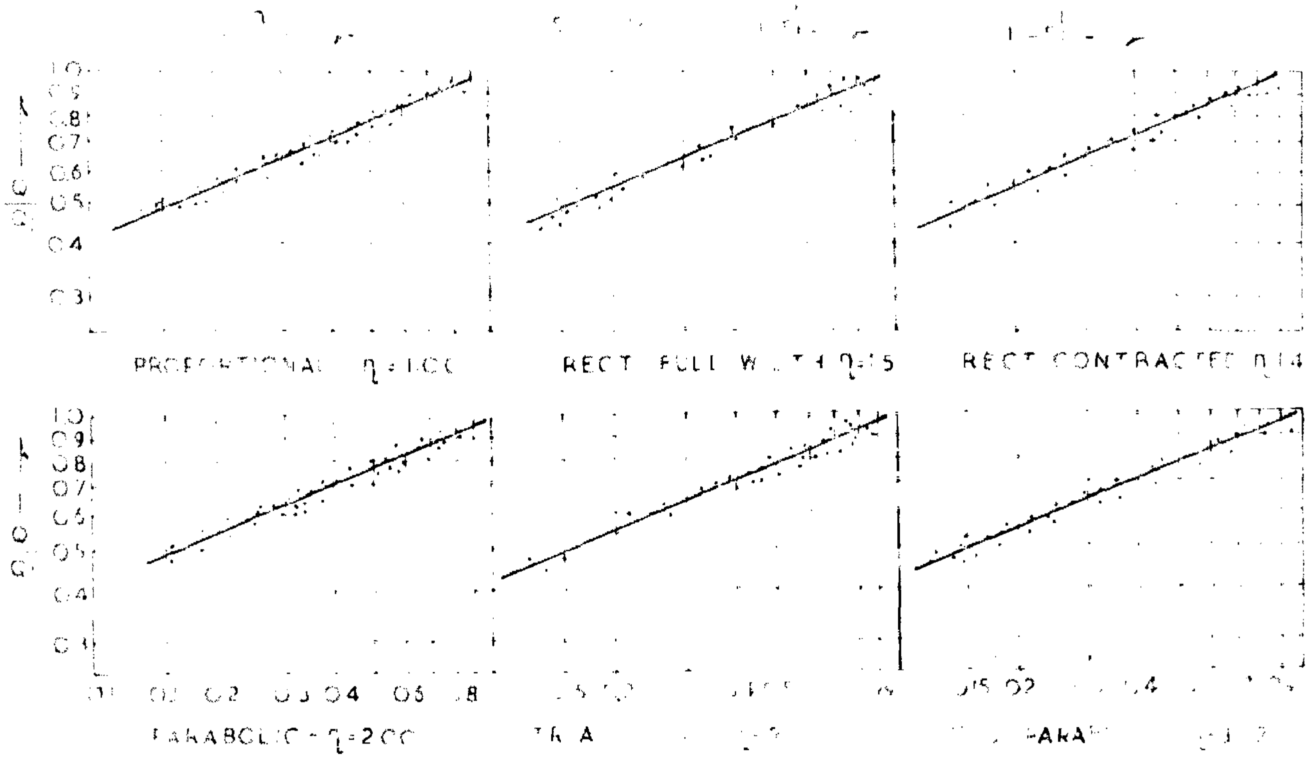


FIG 9 RELATION OF DISCHARGE AND SUBMERGENCE

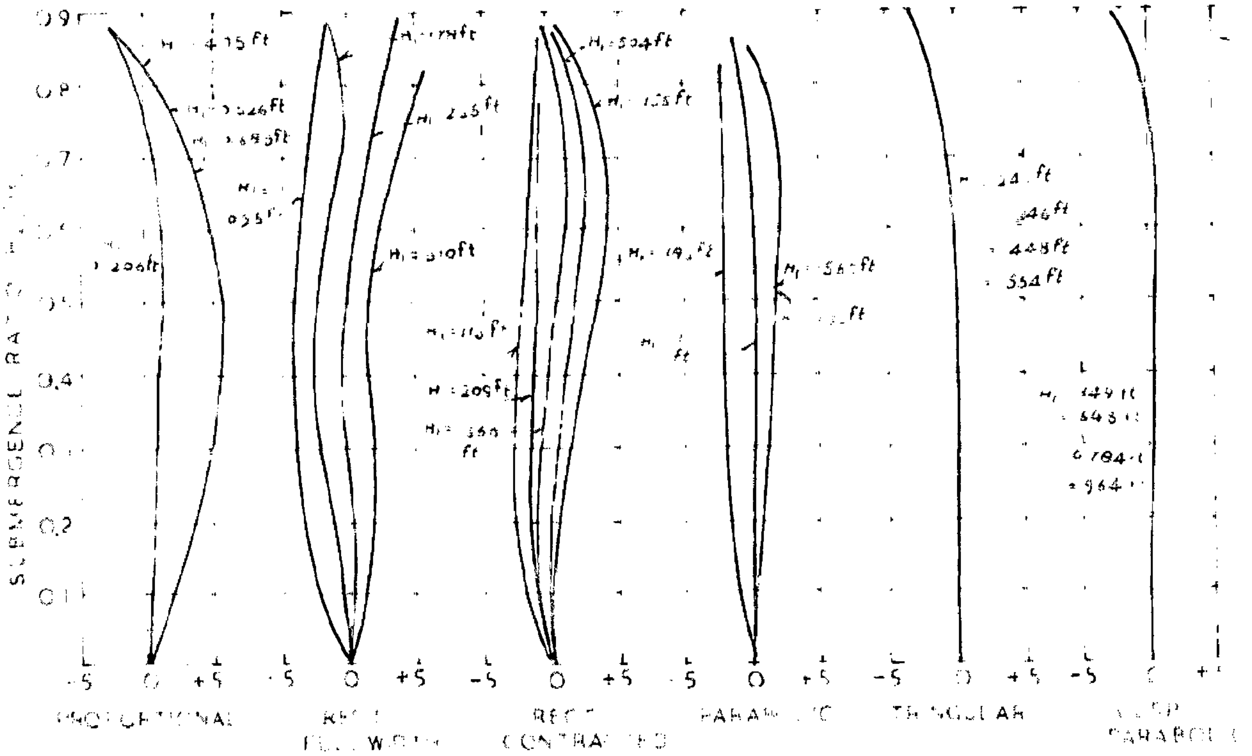


FIG. 10 PERCENTAGE CORRECTION [illegible] RESULTS

CASTER [illegible] 8

TABLE -2

VALUES OF Q/Q_1 FOR THREE COMMON SHARP CRESTED WEIR NOTCHES BY EQUATION-6 SHOWING COMPARISION WITH RESULT OF OTHER WORKERS.

Degree of Submergence	Proportional notch equation 6	Equation-6	Cone	Cox	Rectangular notches Francis Iteley Sterns	Sterns	Bazin	V notch Equation-6
1	2	3	4	5	6	7	8	9
P/H_1	-	2.10	6.0	4.30	3.10	-	6.0	-
0.10	0.96	0.99	0.99	1.00	0.99	1.00	1.00	1.00
0.20	0.92	0.96	0.95	0.96	0.96	0.97	0.97	0.99
0.30	0.87	0.93	0.93	0.92	0.92	0.93	0.93	0.98
0.40	0.82	0.39	0.89	0.88	0.88	0.89	0.89	0.96
0.50	0.77	0.85	0.85	0.83	0.83	0.84	0.84	0.93
0.60	0.70	0.79	0.79	0.77	0.77	0.78	0.79	0.88
0.70	0.63	0.71	0.71	0.70	0.69	0.70	0.72	0.82
0.80	0.54	0.62	0.62	0.60	0.59	0.59	0.63	0.72
0.90	0.41	0.48	0.46	0.45	0.45	0.51	0.48	0.57

Also values of Q/Q_1 for two other types of sharp crested weirs are given.

As a result of the study the following conclusions were drawn by Villemonte.

1. The experimental work indicates that Q/Q_1 and $1-\frac{Q_2}{Q_1}$ have the same functional relationship for each weir tested. In addition it is reasonable to expect that this relationship should be applicable to general group of sharp crested weirs.

2. The effect of submergence on weir discharge decreases as 'n' increases but the stability and consistency of such effect are improved as 'n' increases. Therefore, triangular and parabolic weirs are more accurate when submerged than are proportional or rectangular weirs.

3. The accuracy of the submerged weirs will be improved if the tail water basin is sufficiently wide and deep to permit free circulation of water underneath the nappe. In this way interaction effects are stabilised.

4. An application of the principle of superposition might be valuable in studying the effect of submergence on other hydraulic structures.

~~2.2.5.0. When a thin-plate weir is submerged that is when the tail water rises sufficiently above the crest of the notch, head water will also rise if the rates of discharge remain constant. Conversely, if head water level is fixed above a thin plate weir, rate of discharge will diminish as tail water rises above the crest. Discharge over a submerged weir is expressed as a fraction of discharge over the same weir if it were unsubmerged but functioning at the same head water depth. This fraction of discharge Q/Q_1 is a function of the submergence factor~~

$$S = \frac{\cancel{Q_2}\, a_2\sqrt{H_2}}{\cancel{Q_1}\, a_1\sqrt{H_1}}$$ ~~in which~~

~~H_1 = Head water depth above crest.~~

~~H_2 = Tail water depth above crest~~

~~a_1 = Area of segment of weir notch below the level of the head water pool.~~

~~a_2 = Area of segment of weir notch below the level of tail water pool.~~

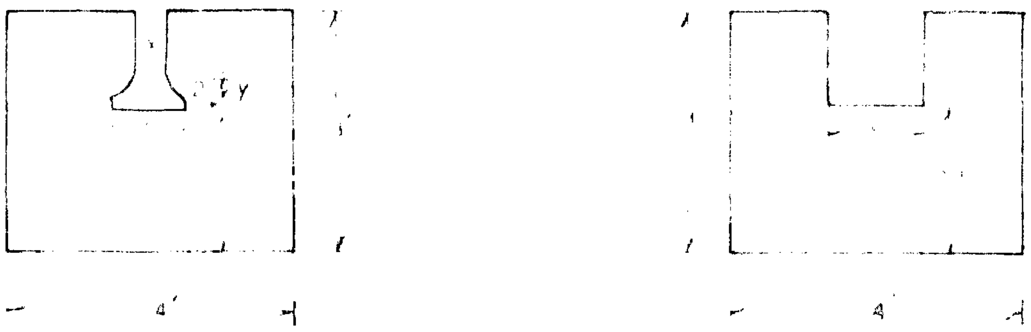
2.2.5.1. A simple formula was presented by F.T.Mavies[5], who conducted flow tests in 1948 at the Carneigh Institute of Technology on six different shapes of thin plate weirs (Fig.11a) under submerged conditions, to express relationship between the discharge factor Q/Q_1 and the submerged factor $\frac{a_2\sqrt{H_2}}{a_1\sqrt{H_1}}$ for each shape.

$$\frac{Q}{Q_1} = 1.00 - \left[0.45\,S + \frac{0.40}{(10-10S)}\right]$$

It may be mentioned that both Villemonte and Mavis are contemporary; but each gave independent formulae. Whether they used each other's data is not known.

2.2.5.2. **Experiments:**

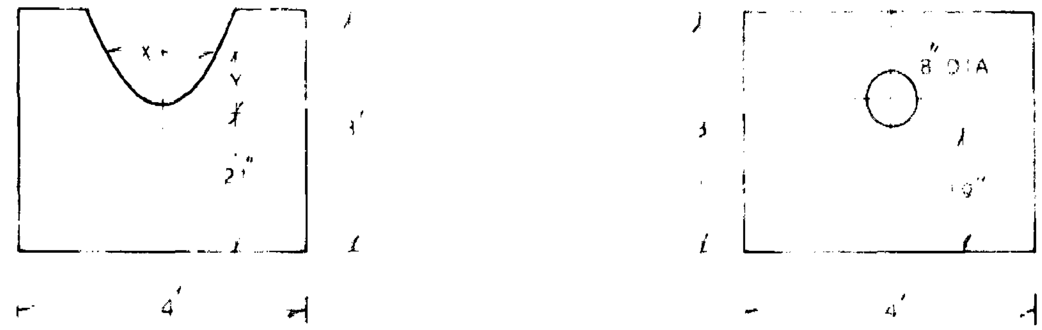
~~The tests were conducted in a tank 9.7' (2.96 m) long~~,



SUTRO

X = [illegible] (1 − 2/π ARC TAN √(Y/2)), X & Y IN INCHES

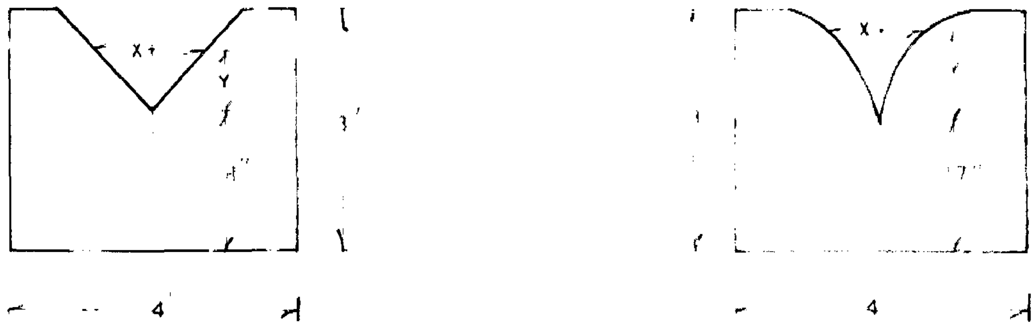
RECTANGULAR



PARABOLIC

$\frac{X}{24} = \sqrt{\frac{Y}{4}}$ X,Y IN INCHES

CIRCULAR



V-NOTCH

X=2.0 Y

[illegible] 5/4 DEG

X [illegible] 84 [illegible]

FIG. 11(a) DIFFERENT WEIR SHAPES [illegible]

[illegible] NAVIES [illegible]

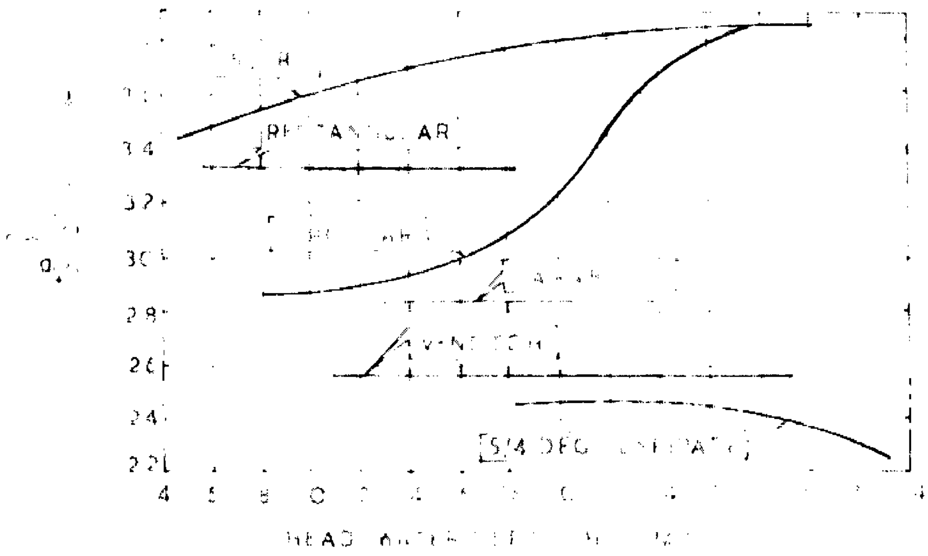
4 ft(1.22 m) wide and 3 ft.(0.91 m) deep. ~~Weir plates of 16 gage galvanised sheet metal were bolted to a vertical bulk head frame 3.2 ft.(.98 m) upstream from a tail gate 2 ft.(0.61 m) wide which controlled tail water elevations. Head water and tail water depths above the crest of each weir were read by means of open tube manometers connected to the side of the tank. A calibrated venturimeter was generally used to measure rates of flow. A calibrated V notch weir was used for measuring very small discharge.~~

The coefficient C in the equation $Q_1 = C. a_1 \sqrt{H_1}$ as a function of head for free discharge, for each of the weirs tested is shown in Fig.11(b).

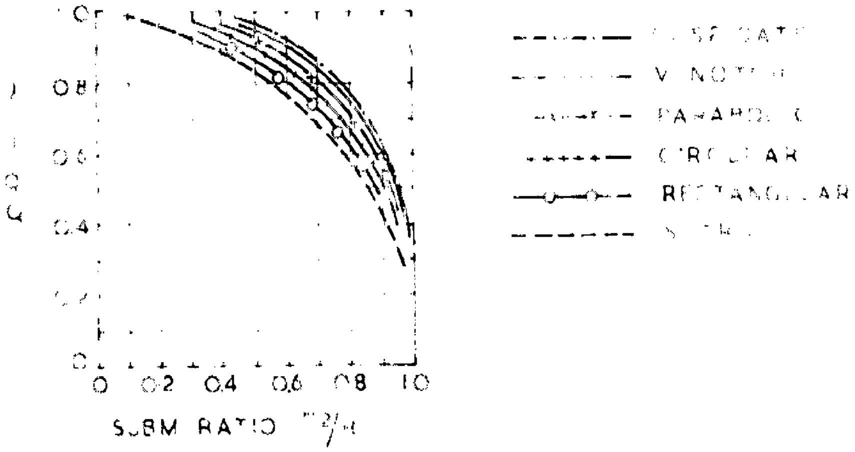
~~Tests, in general, to determine effects of submergence were conducted as follows: After rate of flow became steady the venturimeter and head water gage, and tail water gage were recorded. These observations were converted respectively in to discharge, head water depth and tail water depth.~~

2.2.5.3. Analysis Of Data:

For each observation of H_1,H_2 and Q a fictitious discharge (Q_1) was calculated from observed head water depth and coefficient of discharge for that particular weir (Fig.11(b)). The ratio Q/Q_1 is the discharge factor. For each test the corresponding submergence ratio H_2/H_1 was also determined. The discharge factor was then plotted as a function of the submergence ratio in Fig.12. It is seen from the figure that the sutro weir is most affected by submergence and the weir least affected in the cuspidate weir.



F[illegible] 5 FRE[illegible] [illegible] OF THIN PLATE WEIRS



[illegible] RELATION [illegible] & [illegible] RATIOS

[illegible]

The relations between Q/Q_1 and the submergence factor 'S' for all weirs tested are shown in figure 13. Tests of all submerged weirs follow a single pattern and one curve can be drawn to represent adequately the discharge factor as a function of the submergence factor for all weirs tested. The equation is $\frac{Q}{Q_1} = 1.00 - \left[0.45\,S + \frac{0.40}{{}_{2}(10 - 10S)}\right]$.

2.2.5.4. <u>Conclusions by Mavies:</u>

Mavies found that the results obtained by Poleni (para 2.2.1.) and himself are strikingly similar. He had restated Poleni's formula interms of the submergence factor 'S' which has been previously mentioned.

2.2.6.0. Another ~~[illegible]~~ study of the effect of submergence on trapezoidal notch was done by V.Mandrup Andersen and Niels Dahl[9] at the hydraulic laboratory of the technical university of Denmark in 1961. They observed that

$$Q = \eta \cdot C_F \cdot F \sqrt{2g\,H_1}$$

Where
Q = Submerged discharge
F = Area of the water way of the trapezoidal approach channel.
F' = Area of water way of weir notch.
C_F = Discharge coefficient with free nappe which is a function of F'/F.

*

In the above formula C_F is a function of Froude's number which is given as.. $\frac{V}{\sqrt{g H_1}}$.. It will thus be seen that $\sqrt{g\,H_1}$ will cancell at numerator and denominator leaving discharge as a function of hd/H1 which is nothing but $(1 - H_2/H_1)$ or $(1-S_1)$.

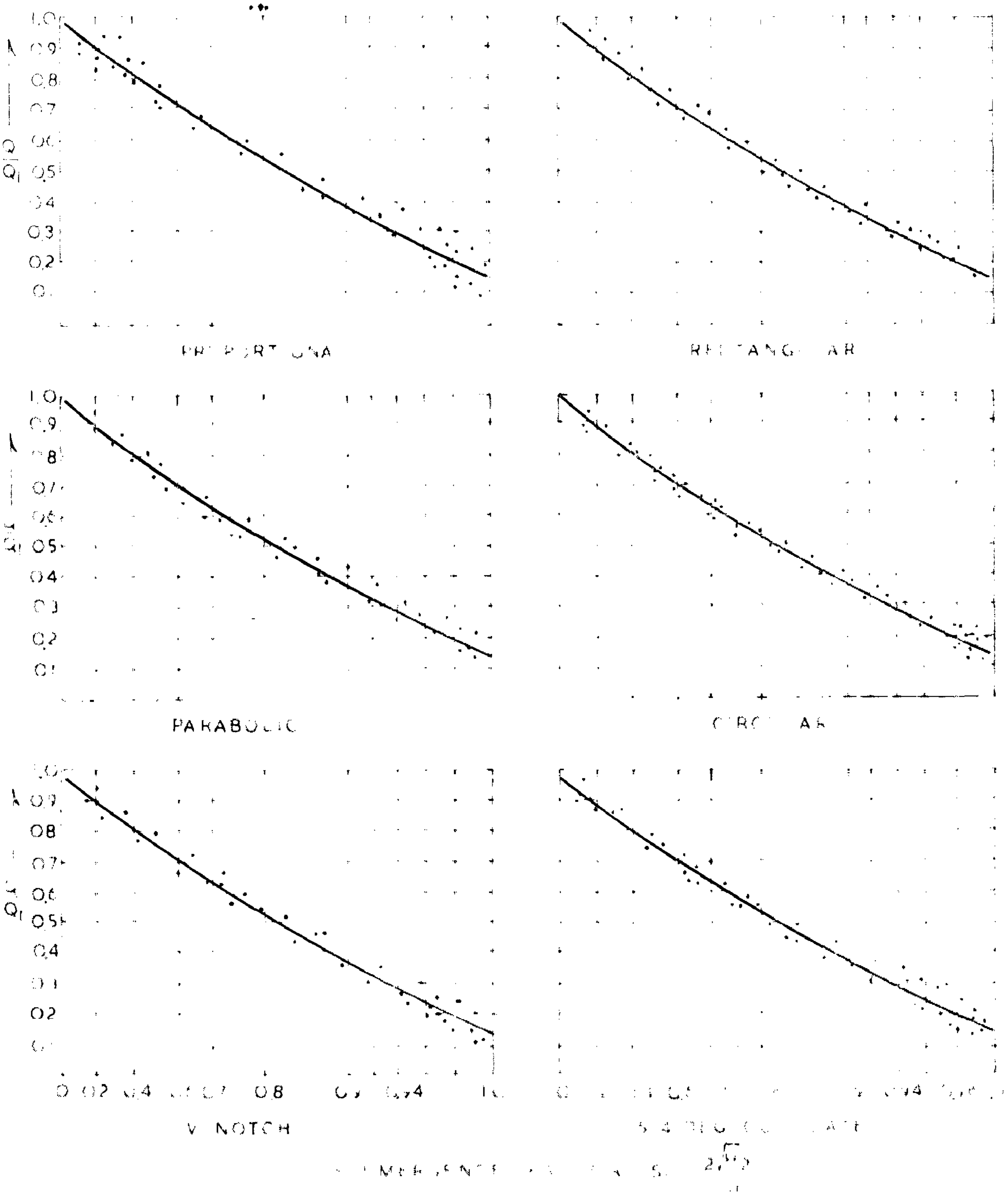


FIG. [illegible] RELATION [illegible]

[illegible]

[illegible]

2.2.6.2. Tests with weirs with a free nappe corresponding to $hd/H_1 \geq 1$ are treated first. By this arrangement a relation between the discharge coefficient C_F and the area ratio F'/F has been found (Fig.$\overset{14}{18}$). It will be seen from the Plot, a smooth curve can be drawn in such a manner that great majority of the test points have only comparatively small scatter. Only when F'/F is higher than 0.6 - 0.7 is the deviation too great. This is due to the fact that the discharge coefficient cannot be expressed by just one parameter as done here. It is also to a certain extent dependent on the geometric shape. With fair approximation this curve in figure 18 satisfies the equation

$$C_F = 0.021 + 0.505 \; (F'/F)^{1.45}$$

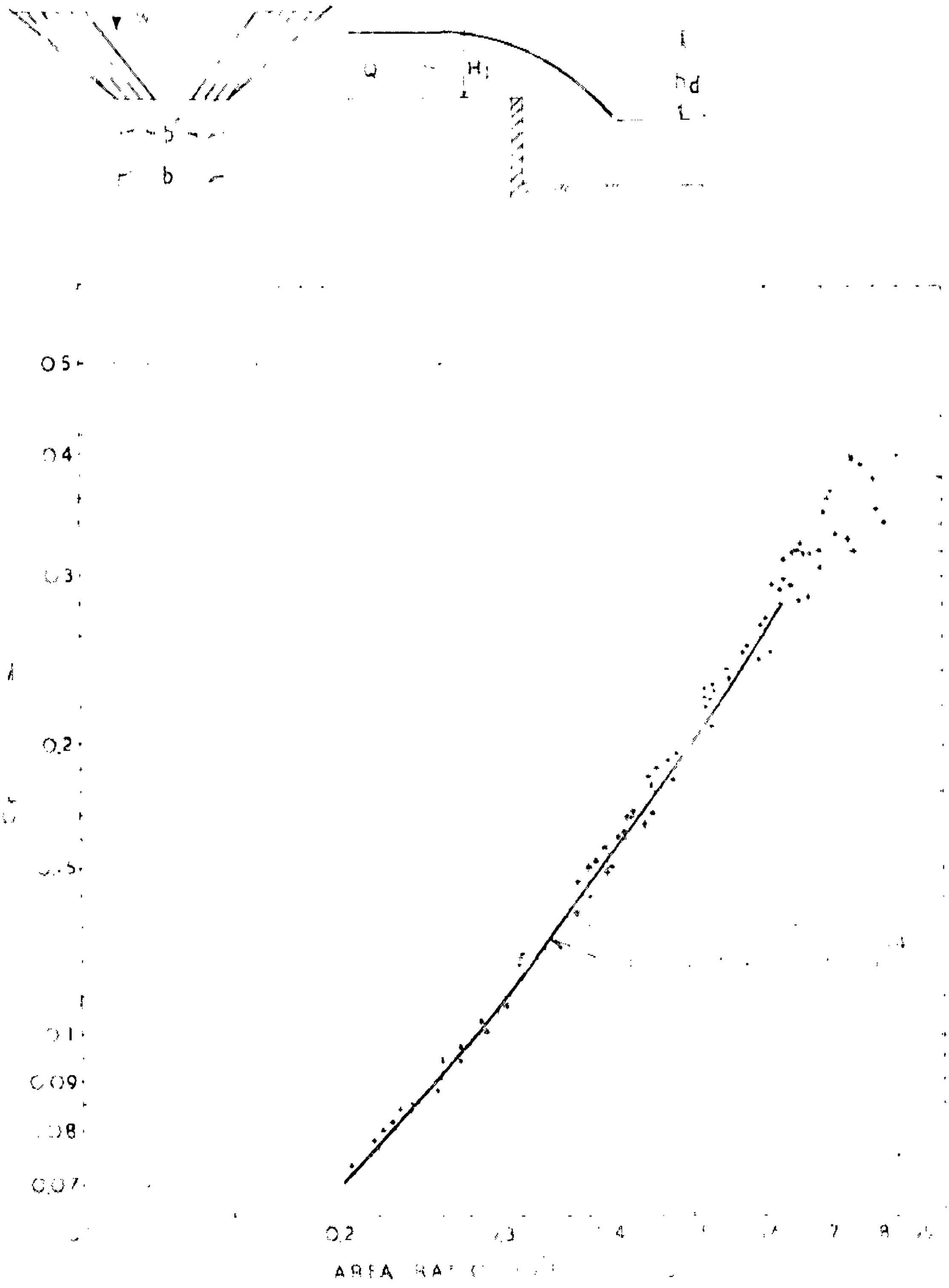
where $0.2 < F'/F < 0.6$ and $a' < 1.25$

2.2.6.3. Tests with submerged weirs were treated next and a relation between the hd/H_1 (which is $1 - H_2/H_1$) and the reduction coefficient was found on the basis of the knowledge of the discharge coefficients for weirs with free nappe.(Fig.$\overset{15}{19}$). A smooth curve which approximates the test results as far as possible is drawn. The deviations (4 to 8%) in this case also may be due to geometric shape.

The curve satisfies the equations

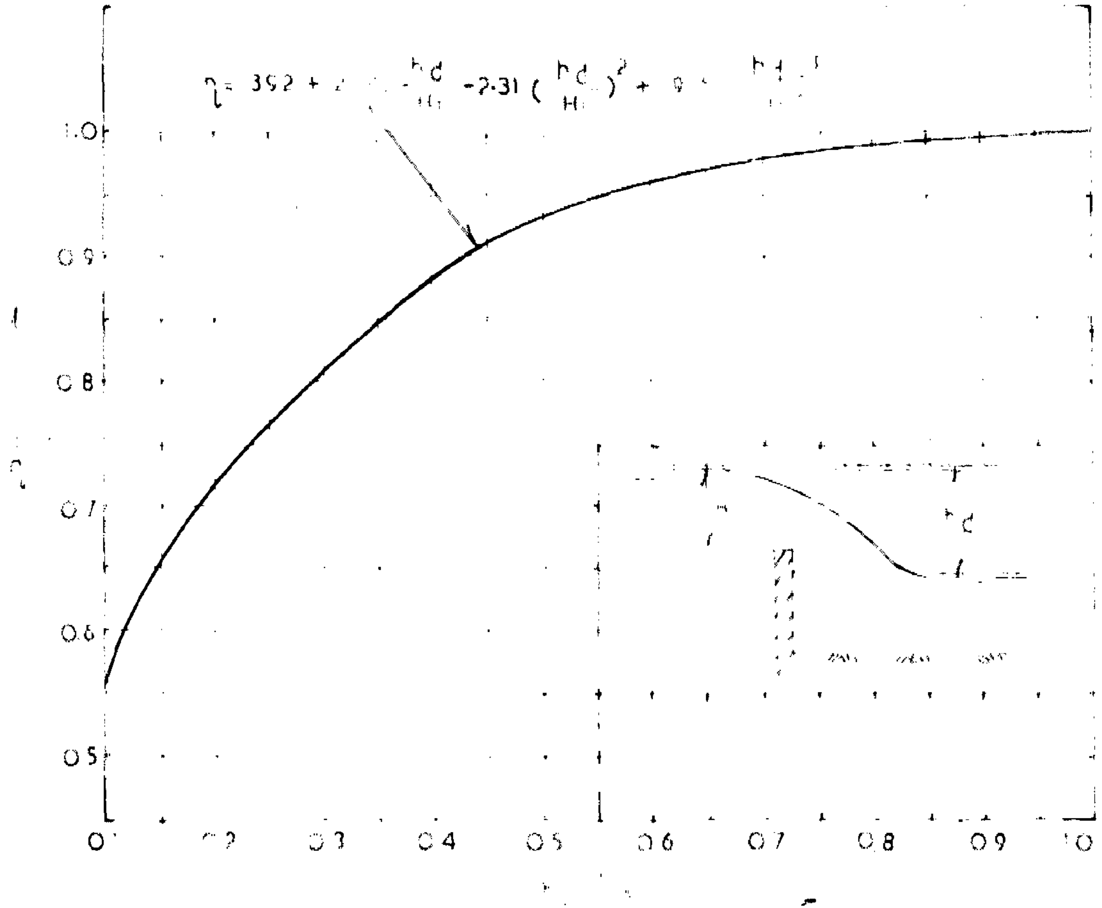
$$\eta = 0.392 + 2.00 \; hd/H_1 - 2.31 \; (hd/H_1)^2 + 0.915 \; (hd/H_1)^3$$

2.2.6.4. Hence for trapezoidal weirs the validity of the formula $Q = \eta \cdot C_F \cdot F \sqrt{2g \, H_1}$ has been proved convincingly in which η depends on the submergence ratio and C_F depends on F'/F. It has also been proved that Geometric Shape also affects the



[illegible] LATION [illegible] COEFFIC [illegible]

[illegible]



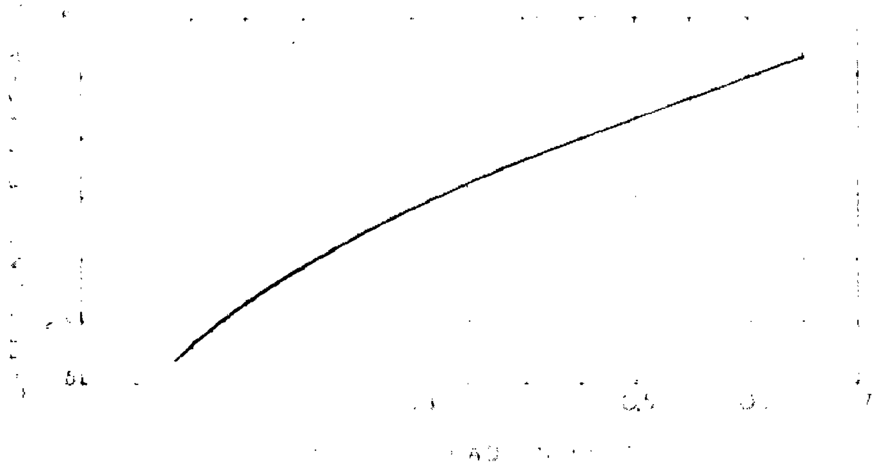
[illegible]5 RELATION BETWEEN [illegible] & [illegible]

(CARTER ANDERSON & SAN[illegible] [illegible]) results although it can be of secondary importance because of the small deviations in the test results.

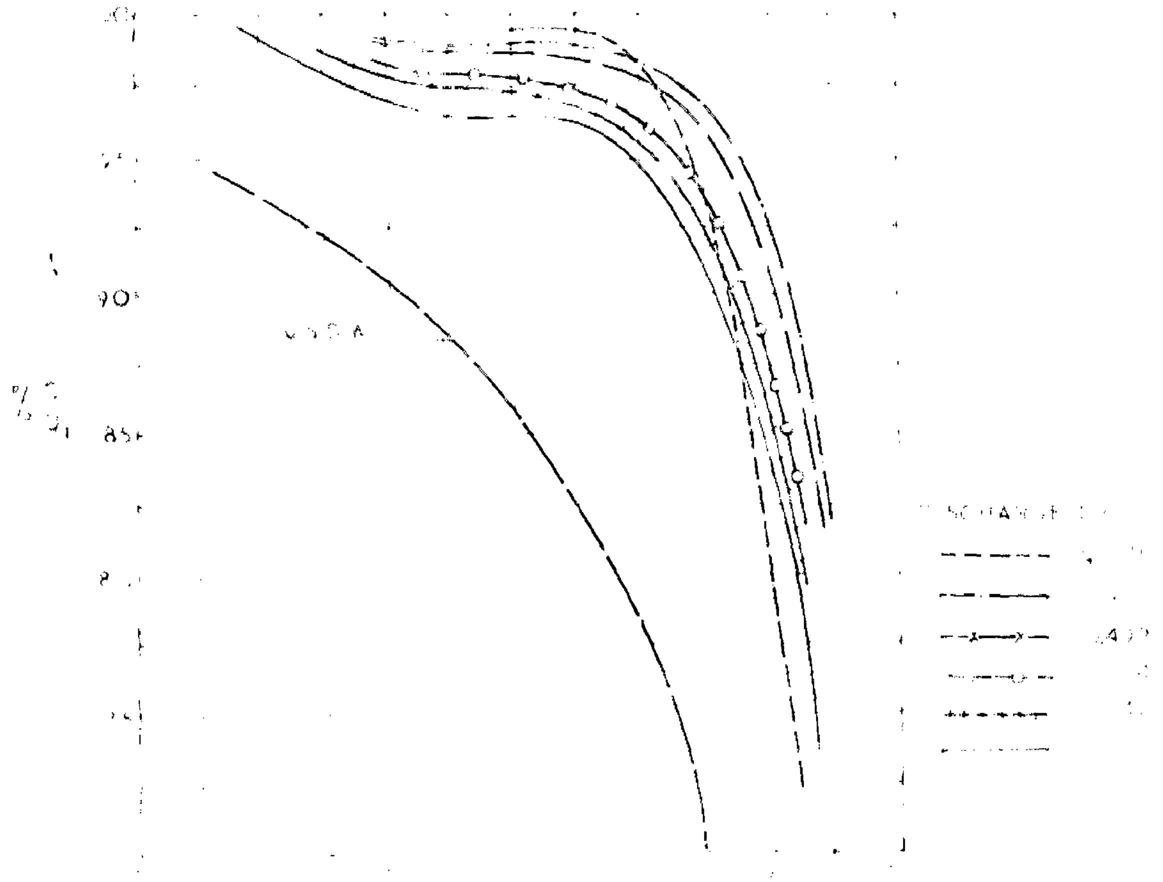
2.3.0. HIGH COEFFICIENT WEIRS(OGEE SHAPED)

2.3.1.0. Edward Soucek^{10} made a comparision between a series of current meter measurements of discharge over the low University dam (in mid western U.S.A.) and the discharge over a 1:12 scale model of the dam. The range of discharge studied was such that both free flow as well as submerged flow conditions were compared. Both the model and the prototype exhibited a considerable reduction of discharge due to submergence. However, the model tests demonstrated the inadequacy of single valued relations between the percentage submergence and the reduction in discharge capacity.

2.3.1.1. The results of the model tests for free and submerged conditions are shown in Fig.~~14~~16(a) & ~~14~~16(b). Following usual practice the diminution in discharge capacity due to submergence is indicated by curves in which the submerged discharge, expressed as a percentage of the unsubmerged discharge which would obtain under the same energy head, is plotted against the submergonce. For submergence lower than 45%, the relation between discharge and the energy head was not affected by the submergence. It is also seen that for different discharge rates on the model different curves are obtained. It is also interesting to note that the sequence of change from a lower discharge to a higher discharge is systematically from right to the left exept for the case of lowest discharge which does not confirm to the above. This may be due to the fact that the energy head over the model was only 0.1 ft in which case



[illegible]


90
85

[illegible]

Viscosity and surface tension might have played some part for this irregular behaviour. This point however could not be thoroughly investigated the Soucek.

This means that, the ratio Q/Q_1 at the same energy head is a function not only of the submergence but also of the discharge flowing over the model.

A part of the relation between submergence and the ratio Q/Q_1 obtained on the Deep waterway Dam by the U.S.Board of engineers is also shown for comparision in Fig.14.(b)(Curve U.S.D.W.). It is seen that the effect of submergence obtained by Soucek was only a small fraction of that obtained on the Deep waterways Dam. The effect of submergence on the model of the University Dam was not appreciable until the submergence exceeded 45% where as the Deep water way dam was affected by much lower submergences.

2.3.1.2. Soucek opined that the use of a typical submerged flow data can be nothing but a rough approximation. The fact that the Q/Q_1 VS H_2/H_1 curve plots to be a smooth curve, appears to have given rise to a prevalent belief that in general, the effect of submergence on the discharge over a dam can be represented by a single valued curve relating submergence and discharge ratio. Attempts to plot submerged spillway data in this manner invariably lead to a wide scatter of observation points. This in turn leads to the opinion that submerged flow over dams is a far more unstable phenomenon than is really the case.

2.3.1.3. The conclusions by soucek were that :-

1. The effect of submergence on the discharge is affected by many other factors including the shape of the spillway.

2. It is not possible to express the effect of submergence upon the discharge by a single valued relation between the submergence and the discharge ratio. In the range covered by the tests the effect of submergence is greater for higher discharges.

2.3.2.0. United States Bureau of Rreclamation[11] has conducted some experiments on ogee shaped structures and obtained very useful informations about the effect of submergence on the coefficient of discharge. The scope of their study was quite wide. In addition to the above, the studies also furnished information regarding the effect of the level of downstream apron on the coefficient of discharge and also the types of flow that existed immediately downstream of the weir crest. However only the relevant Points will be discussed here as the other subjects are beyond the scope of this dissertation.

2.3.2.1. Two small dams were subjected to the model tests. The sections of the dams and some coordinations of the ogee are shown in Fig.~~15~~17. Discharge coefficients were first determined for the free flow conditions then redetermined for the various conditions involving submergence. The difference between the two is termed "The decrease in the coefficient of discharge due to submergence".

This factor expressed in percentage of the free flow coefficient has been plotted for practically all combinations of flow which can occur on small dams with horizontal downstream aprons.

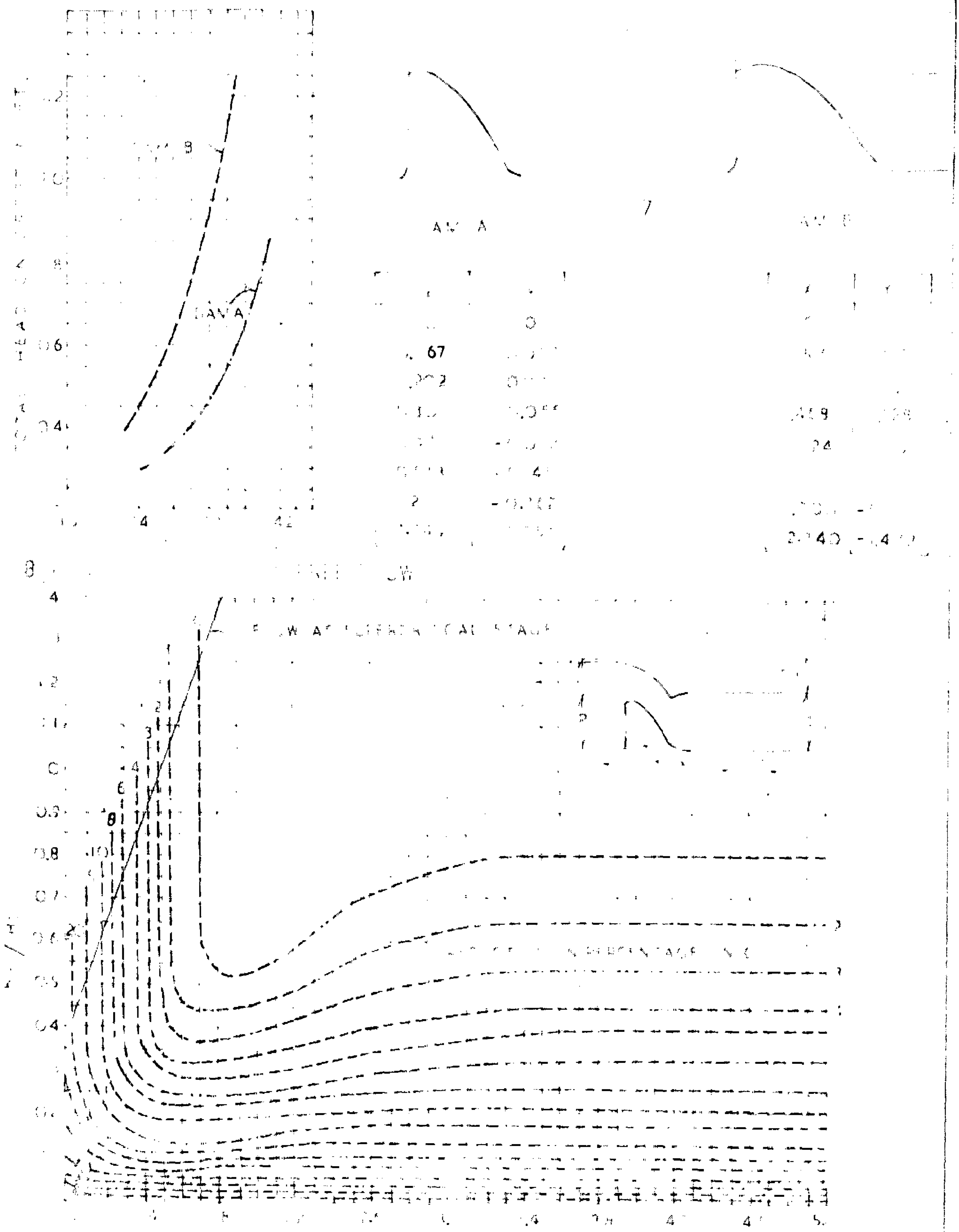
2.3.2.2. Test Procedure:

Free flow was allowed on the models and the relationship between head on crest and the coefficient of discharge was established for both the dams. This is shown in Fig.~~16~~.18

After this the movable floor at the downstream side was fixed in a particular position and a constant head was maintained on the dam. The tail water depth was then made to vary from a point of no submergence to almost 90 to 95% submergence. About 6 to 8 runs were required for this. During each run measurement of discharge and downstream water level were noted accurately. The same series of tests was repeated while a second head was maintained over the crest of the dam. These were again repeated for different other heads on the dam as well as for different floor positions at the downstream.

The coefficient of discharge was obtained for each run and the decrease in the coefficient of discharge due to submergence and also due to the presence of the downstream floor was obtained for each run by subtracting the coefficient of discharge as computed above from the free flow coefficient (Fig.~~16~~18) for a corresponding flow condition.

2.3.2.3. The results were plotted in Fig.~~17~~19. One of the finding is regarding the types of flows encountered. ~~and are shown in the figure.~~ The vertical dashed lines indicate the percentage


DAM B
DAM A
0.4
0.6
1.0
.2
4
4.2
67
0.4
0.5
0.6
0.7
0.8
0.9
8
10
8
6
4
3
2
1
.24
.459
2.40
-.4
4.
5.

[illegible]

[illegible]

decrease in the coefficient of discharge due to the effect of downstream floor where as the horizontal dashed lines indicate the decrease due to the effect of the submergence.

The curved portion indicate that the reduction in the coefficient of discharge is due to both submergence and the downstream apron.

2.3.2.4. The results obtained from model studies gave an idea about the general relationship between the degree of submergence and the reduction in the coefficient of discharge. It is seen from Fig.17 that the coefficient of discharge only gets reduced if the degree of submergence exceed beyond 45% or so.

2.4.0. BROAD CRESTED WEIRS

2.4.1.0. James G. Woodburn[12] made a number of tests during 1928-29 at the University of Michigan on broad crested weirs of various designs in a rectangular wooden flume 2 feet(0.61 m) wide. The crests varied in breadth from 10 to 15.5 ft.(2.05 m to 4.72 m) and in slopes and combinations of slopes from level to 0.085. The range of head was from 0.5 ft. (.153 m) to 1.5 ft. (.455 m) and volume of flow from 2.0 to 11.0 cusecs(0.0566 to 0.312 cumecs).

The purpose of tests was to obtain experimental data in regards to the following two advantages of the broad crested weirs over the sharp crested weirs.

1. Its adaptability to the use of the hydraulic jump to reduce to a minimum the head lost in the weir,&

2. The possibility of producing flow at critical depth with a resulting simple relation between depth and discharge which is independent of the confusing effects of velocity of approach.

2.4.1.1. Coefficient of discharge of various weir models were determined both with over fall and with the weir submerged. The data thus obtained permitted a study of the effects of submergence on the coefficient to discharge.

The tests in which the jump could not be produced, when the weir was submerged, the coefficient of the submerged weir was on the average 0.219 % less than for the weirs with free over fall. The remaining tests in which the hydraulic jump was formed on the crest under the submerged condition the coefficient of the submerged weirs was on the average 0.081% less than for weirs with free overfall.

The submergence was created by backing up the tail water to an elevation only slightly lower than the water surface at the downstream end of the weir.

2.4.1.2. Unfortunately no degree of submergence was indicated by Wood burn. By inspection of the water surface profiles given, it is estimated that where the hydraulic Jump occurs this submergence may reach 75% to 80% as a maximum with no material change in the coefficient of discharge. The ability of the broad crested weirs to operate under submerged condition, up to about 75 to 80%, with practically no change in the coefficient of discharge gives it an advantage over sharp crested weirs.

The broad crested weirs operating under a considerable degree of submergence requires the measurement of head at one point only.

However, Wood burn did not investigate the effect of higher submergence on the coefficient of discharge. At greater tail water conditions, the upstream water level will fluctuate and the coefficient of discharge will reduce.

2.4.2.0. Edwin Samuel Crump[13] devised a new method of gauging stream flow with little afflux by means of a submerged weir of triangular profile. Tests made by him on a model weir of standard profile show that the modular coefficient of discharge is sensibly constant and that the ratio Q/Q_1 i.e. actual discharge to modular discharge is a unique function for a given value of head over crest.

2.4.2.1. In the submerged condition, the discharge is a functio of two independent variables instead of one as in the case of modular range. The normal procedure is to first establish the value of the modular coefficient 'C' from which to calculate the free discharge corresponding to the upstream head over crest and then to ascertain themanner in which the reduction factor Q/Q_1 varies with different degrees of submergence.

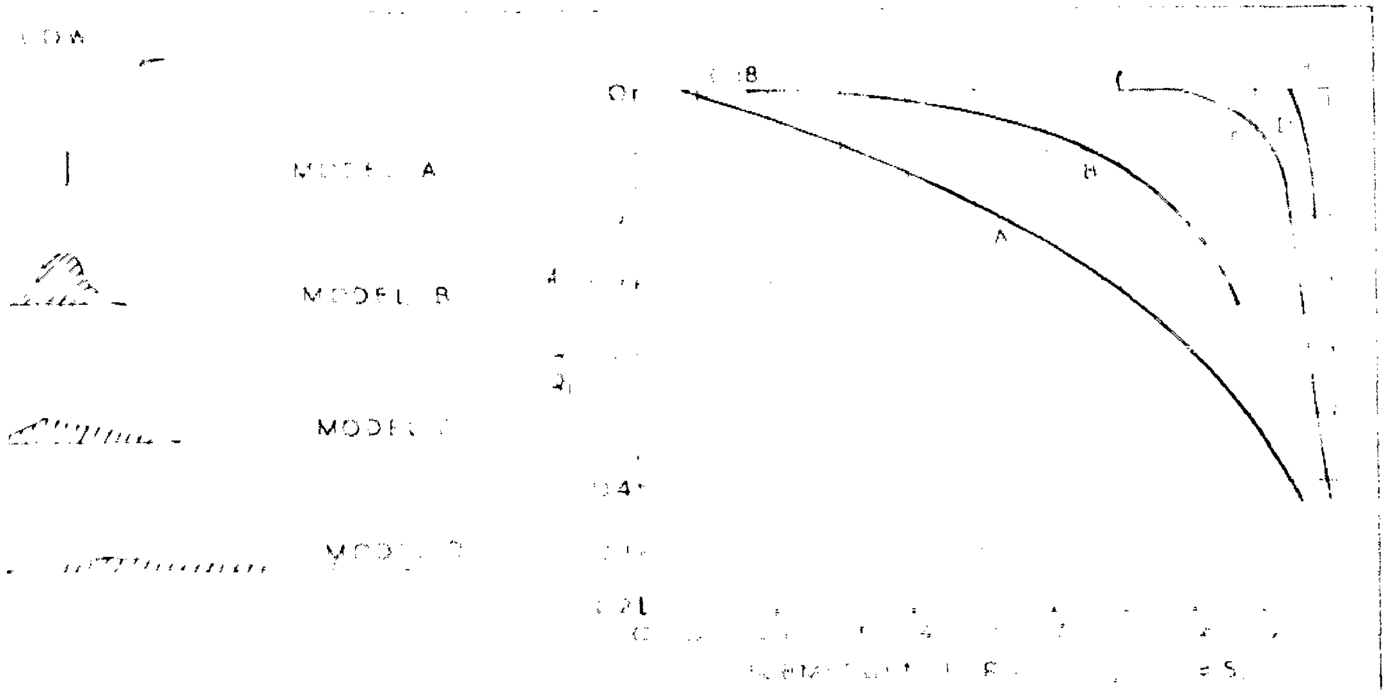
2.4.2.2. In majority of the cases the modular coefficient 'C' is not constant but increases with upstream head. If 'C' is not constant the relationship between the reduction factor Q/Q_1 and ratio of submergence is not unique but is different for different values of H_1. In seeking a form of weir likely to prove suitable for double gauging, it was realised that the best & possibly the only chance of success lay in finding a weir which avoided these difficulties and complexities. It was found that in the case of all weirs which are rounded or have a flat fact in the vicinity of the crest, 'C' increases with H_1 where as all weirs investigated by Bazin exhibited for each particular profile an approximately constant value of 'C'.

2.4.2.3. Results of model tests on triangular profile and other profiles are shown in figure 20. Curve 'A' is plotted from results of Francis and Ftely-Stearns for the sharp crested weir. A point to be noted is the very wide range in values of the modular limit.

In model 'B' the modular limit was reached at a submergence ratio of 18%. The model 'C' was Crump's Standard weir for double gauging. It is seen that this has a high modular limit of 70% and that with submergence ratio of 80% the departure from modularity is only 1%. The model 'D' which is an idealised replica of the standard design of old Indian Weirs, has a very high modular limit of about 94%.

2.4.3.0. The Irrigation research institute[14] at Roorkee had conducted model tests in order to study the relationship between Q/Q_1 and the submergence ratio under different heads for the Sarda Canal main regulator at M.2-5-440. The model was constructed to a scale 1:20 and it was run for various discharges for recording corresponding water depths at upstream and downstream ends under submerged condition.

2.4.3.1. From the observed data curves were fitted between Q/Q_1 and percentage submergence for six different depths ranging from



RELATION BETWEEN [illegible] DIFFERENT [illegible] MODELS

4 ft. to 9 ft.(1.22 m to 2.74 m) at the upstream end. A general relationship was obtained

$Q/Q_1 = 1 - C\ S_1^5$, and a relationship was also derived for C $C = 0.4613\ H_1^{0.361}$ where H_1 is between 4 ft. & 9 ft.(1.22 m to 2.74 m).

2.4.4.0. Richard A.Smith[15] conducted model testing on broad crested weirs corresponding to the inundated Levees or roadbeds, in the Louisiana State U.S.A. The purpose was to calibrate the inundated road bed, acting as a weir for two conditions of flow: first, when the weir controlled the discharge and second, when the tail water was high so that control of the discharge had passed to conditions downstream from the weir.

In the analysis of the observed data the specific objectives were three fold:-

a) To determine the criterion by which it could be ascertained when control of the regimen passed from the submerged weir to tail water conditions downstream.

b) Calibrate the head-discharge relationship for the condition within modular range i.e. within the control of the weir.

c) Co-rrelate the variables head water and tail water elevations the rate of discharge for the condition when control of the regimen was downstream from the weir.

2.4.4.1. Models were prepared with broad crested weirs having crest width of 0.5 ft.(.153 m), height 0.425 ft.(0.12 m) and in one case having upstream and downstream slopes 1 in 1 and in the other case upstream and downstream slopes 1:2. The flume was 5 ft. (1.53 m) wide and the maximum flow available was 1 cusec. The models were operated in the range of discharge (0.0254 cumecs). At each discharge rate the tail water elevation was varied through a full cycle of progressively higher and progressively lowe elevations in successive runs.

2.4.4.2. For the weir having 1:1 slope a relation was derived for Q.

$$Q = H_1^{3/2} \sqrt{g\left[0.98 - (H_2/H_1)^{2.2}\right]\left[0.663+0.334(H_2/H_1)^{2.2}\right]^3}$$

Similarly for the other weir having 1:2 slope.

$$Q = H_1^{3/2} \sqrt{g(2.86-2.96\, H_2/H_1)\,(0.027+0.991\, H_2/H_1)^3}$$

These were for the condition beyond the modular range.

2.5.0. In the previous paragraphs some notable works on submerged flow over weirs have been dealt with. It is not possible to deal with all the previous works on the subject although the important ones have been studied. Besides these investigators, some others e.g. Vennard and Weston, Cox, Koloseus, Dominguez etc. have also thrown much light on the subject. But in general the volume of literature on this subject is rather limited.

2.6.0. RECAPITULATION:

Summing up the review work it is noticed that some of the investigators have only tested sharp crested weirs of different shape e.g. Fleley and stearns, Francis, Villemonte and Mavies. The others have tested with practical profiles of weirs as they exist in actual cases i.e. ogee, Broad crested shapes etc.

2.6.1.0. Fteley and Stearns and Francis had developed empirical formulae and tested its correctness by model testing. They had not investigated about the generalisation of the discharge with respect to submergence ratio. Fteley and Stearns however, did notice the change of coefficient of discharge with respect to the submergence ratio.

2.6.1.1. Villemonte and Mavies Worked on similar lines on similar types of weirs. Each of them had developed a discharge equation for all sharp crested weirs expressed by a single valued function. Mavies had introduced the conception of submergence factor defined by $\frac{a_2\sqrt{H_2}}{a_1\sqrt{H_1}}$. However by comparing their basic formulae it is observed that the submerged factor 'S' of Mavies is functionally equivalent to Villemonte's S_1^m where S_1, is the submergence ratio H_2/H_1.

2.6.1.2. Mandrup Anderson and Dahl observed that the discharge can be found out by a single equation neglecting small effects of geometric shape. They also suggested that the reduction factor due to submergence can be realated to S_1. However their study was only on a particular type of weir i.e. a trapezoidal notch.

2.6.2.0. Edward Soucek and the U.S.B.R. made their experiments on high coefficient weirs (ogee shape). According to U.S.B.R. the coefficient to discharge is not affected up to a submergence of 45 percent. Soucek observed that a single valued relationship is not possible because of other factors coming in play.

2.6.3.0. Three investigators have tested broad crested weirs under submerged condition. Woodburn's tests were conducted in the modular range of flow although the results have shown that the state of modularity extends up to a degree of submergence of 70 to 80%. However his tests were not conducted beyond the modular range.

2.6.3.1. Crump made his studies with different shapes of weirs although weir with a triangular profile was suggested by him for discharge measurements. His study has shown that no single relationship can be derived for all types of weirs as seen from Fig.20 where different weirs have ~~a~~ dist~~r~~inctly different characteristics as regards the relationship between the discharge and the submergence ratios.

2.6.3.2. I.R.I. Roorkee also derived a general equation for the discharge relating S_1 and H_1 but their test was limited to 4' $H_1 < 9'$ and also they studied one type of weir.

2.6.3.3. Richard Smith's experiments on a typical broad crested weir revealed that the geometry also affects the discharge under submerged condition. He suggested two different equations for two weirs having different geometrical shape. It was also revealed that beyond the modular range both head water and tail water affect the discharge.

2.6.4. Thus the review has shown that all the investigators do not agree with the fact that a single valued relation can be established between the degree of submergence and the reduction in the discharge for all types of weirs.

CHAPTER-3

THEORETICAL ASPECTS

CHAPTER - 3

THEORETICAL ASPECTS

3.1.0. GENERAL

Flow over different types of weirs can be expressed by the equation

$Q_1 = C.L'. H_1^n$ in which L' is the effective length of weir and H_1 the total head over crest and 'C' a coefficient known as the coefficient of discharge. Out of the many factors that affect the coefficient of discharge; the most important is the effect of submergence which is being discussed in this dissertation. However it will be worth while to mention also the other factors that affect the coefficient of discharge.

3.2.0. FACTORS AFFECTING 'C'

3.2.1. The effect of depth of approach:

The nearer the sill is to the bottom of the channel the less the contraction at the sill and if the depth P is small compared to H_1 the diminution of the contraction will considerably affect the flow. For sharp crested weirs U.S.B.R. observed that if $P > 0.2\ H_1$, the flow is not affected. For the ogee crest, at $P/H_1 =$ about 2.5 the coefficient does not change taking the head over the crest same as the design head for which the ogee profile is made.

3.2.2. Effect Of Heads Differing From Design Head:

When the ogee crest is formed to a shape differing from the ~~ideal~~ designed shape i.e. when the crest has been shaped for a head larger or smaller than the one under consideration, the coefficient of discharge will be affected.

In ~~sharp crested~~ ogee weirs it is observed that for larger head over the crest the coefficient increases.

3.2.3. Effect Of Upstream Face Slope:

For small ratios of P/H_1, sloping the upstream face of the over flow results in an increase of the coefficient. For large ratios it will decrease the coefficient of discharge.

3.2.4. Effect Of Downstream Apron Interference And Downstream Submergence:

The vertical distance from the crest of the overflow to the downstream apron and the depth of flow in the downstream channel are factors which alter the coefficient of discharge. Depending on the relative positions of the apron and the downstream water surface five ~~district~~ distinct characteristics of flow occur. Ref. USBR curves Fig 19

1. Flow continues at supercritical stage.
2. A partial or incomplete hydraulic jump occurs immediately downstream from the crest.
3. A true hydraulic jump occurs.
4. A drowned jump occurs in which the high velocity jet follows the face of the overflow and then continue in an erratic and fluctuating path for a long distance under and through the slower water.
5. No jump occurs. The jet ~~beams~~ breaks away from the face of the overflow and slide along the surface for a short distance and then erratically mingle with the slow moving water underneath.

In the first three cases the reduction in coefficient of discharge is mainly due to the ~~leave~~ back pressure effect of the downstream apron and is independent of any submergence effect due to tail water. The parameter $(h_d + d)/H_1$ is less than 1.7 in the above case. However if $\frac{h_d + d}{H_1 + a}$ is more than 1.7 the downstream apron has not effect of the reduction of the coefficient of discharge.

In such case the decrease is only due to the effect of submergence, U.S.B.R. observed in their experiments on ogee shaped weir that up to submergence of about 45%, there is no appreciable reduction in discharge.

3.2.5. Effect Of Geometry:

Geometry plays an important role in affecting the coefficient of discharge. For this reason different types of weirs have different coefficients. The value of 'C' changes for each type of shape of the weir profile.

3.2.6. Other factors such as approach conditions, effect of piers etc. also affect the discharge coefficients.

3.3.0. DIMENSIONLESS FACTORS IN SUBMERGED FLOW

~~For submerged flow determination of coefficient of discharge is a complex process.~~ In the previous chapter it is shown that various investigators have tried to determine the coefficient by conducting model tests. The experimental coefficient which is obtained from tests is a function of the non-dimensional factors associated with the problem of flow over weirs under submerged condition. The non-dimensional constants are very important as it enables the behaviour of problems of similar type to be predicted providing the linear dimensions are geometrically similar. It will be useful to determine the non-dimensional factors involved in the phenomenon. If co-relation is made between these factors then the problem will have wide application.

3.4.0 The flow over a weir in a submerged condition, provided the height of weir is sufficient as specified, depends upon the following factors:

H_1 & H_2	the upstream and downstream heads over the crest.
g	acceleration due to gravity
ρ	mass density of water
μ	Dynamic viscosity of water

Geometry of the weir will also be a parameter. The non-mensionless parameter will be a simple ratio of the two portant geometrical parameters.

i.e. 7 - 3 = 4 groups. By dimensionless analysis it can be shown that the four independent dimensionless groups governing the problem are:

1. Reynolds Number $\frac{V H_1 \rho}{\mu}$
2. Froude Number $\frac{V}{\sqrt{gH_1}}$
3. Weber Number $\frac{\rho V^2 H_1}{\sigma}$

4. Submergence ratio $\frac{H_2}{H_1}$

* 3.5.0 ~~In problems of weirs as used in the field the effects~~

* For conditions where the value of Reynold's no is more than $.5\times10^6$ as is usually experienced in the proto-type, the viscous and surface tension effects are negligible. The Reynold's and weber nos. can therefore be dropped out of the problem.

[illegible] thus only parameter $\frac{H_2}{H_1}$, is the most important in submerged flow problems. From the model tests co-relation studies can be made between the discharge ratio and the submergence ratio. The corelation can give directly the reduction in discharges for different degree of submergence.

3.8.0 The discharge per unit length in submerged condition can be expressed as $Q = C_s \cdot H_1^n$, in which C_s is the reduced value of coefficient of discharge due to effect of submergence. Under the same head H_1 in free flow condition, the discharge $Q_1 = C\,H_1^n$, in which 'C' is the coefficient in that condition.

Hence $\frac{Q}{Q_1} = \frac{C_s}{C}$ (According to Villemonte)

CHAPTER - 4

COLLECTION OF DATA

4.1.0 GENERAL

Possibilities of doing experimental work, different type of submerged weir were explored. The studies necessitated arrangements for volumetric measurements of discharge for correct evaluation of the discharge factor. It was found that the tank required was of quite big dimensions even to store water for a minimum period of five minutes. Very small size tanks were available in the laboratory which could not serve the purpose. Manufacture of new tank of bigger dimension would have meant time and cost. By reducing the discharge over the weir the tank size could be controlled but the effect of viscosity and surface tension would have become more dedominant, there by complicating the study of submergence effect on discharge factor. It was, therefore, considered desirable to depend on the actual experimental data on submerged weirs collected by different eminent hydraulic engineers of the world. This data apart from being accurate would also serve as a broader base for evolving suitable relations for the engineering proffession.

4.1.1 ~~The coefficient of discharge is obtained by model testing from corelation of non-dimensional factors.~~ In submerged flow problems, the submergence ratio is the principal factor which is corelated with the coefficient ~~of discharge~~. In order to establish the relationship between S_1 and $\frac{Q}{Q1}$, experimental data has been collected

4. Submergence ratio $\frac{H_2}{H_1}$

* 3.5.0 ~~In problems of weirs as used in the field the effects~~ of viscosity and surface tension are not at all important. These variables will be governing when there is very little flow over the weirs, but in submerged weirs these will have

* negligible ~~no~~ effects. So these factors can be ignored.

3.6.0 The Froude's number for submerged flow phenomenon over weirs is also less than one. As the degree of submergence increases the Froude's number also gets reduced. slightly As such no suitable co-relation can be made between the discharge ratio and Froude's number. (Ref. Para 2.2.6.0)

3.7.0 Thus the only parameter $\frac{H_2}{H_1}$, is the most important in submerged flow problems. From the model tests co-relation studies can be made between the discharge ratio and the submergence ratio. The corelation can give directly the reduction in discharges for different degree of submergence.

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CHAPTER-4

COLLECTION OF DATA

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4.1.1 ~~The coefficient of discharge is obtained by model testing from corelation of non-dimensional factors.~~ In submerged flow problems, the submergence ratio is the principal factor which is corelated with the coefficient ~~of discharge~~. In order to establish the relationship between S_1 and $\frac{Q}{Q1}$, experimental data has been collected from various investigators working on various types of weirs. The purpose is to see if all the different types of weirs can be corelated by a single valued function. Other investigators have found out common relationship for a similar type of weir e.g. Mavies & Villemonte for thin plate weirs. In this analysis, data has been collected for different types of weirs i.e. sharp crested weirs, broad crested weirs, triangular weirs, trapezoidal notches and ogee shaped weirs. It will be useful to mention in brief the model set up utilised by the investigators in doing their experiments.

4.2.0 SHARP CRESTED WEIRS

4.2.1 J.B.Francis made experiments on the flow of water over a submerged weir. with no ~~entr~~ end contractions His equation has been described in para 2.2.3.1 . The apparatus arranged for testing a turbine water wheel afforded the opportunity for testing his submerged flow equation. ~~During the experiments the wheel was prevented from revolving, the brake being screwed up tight and securely blocked.~~ The regulating gate was opened to admit a quantity of water to give the desired depth on the weirs, ~~when there was no obstruction to the flow from the water on the downstream side of the weirs~~, The quantity of water flowing over the weirs was then gauged by the usual weir formula , ~~the level of the water on downstream side was then raised, a little at a time and maintained long enough at any desired height to allow obstructions to be made.~~ There was a waste weir in the canal into which the waters discharged over the weirs flowed which afforded ample means for regulating the height of water on the downstream side of the weirs. ~~When the water is raised on the downstream side of the weirs to a height to obstruct the flow over the weirs its effect is to raise the height of the water on the upstream side of the weirs without any change in the quantity of water flowing over the weirs, and the experiments consisted in observing the heights of the water above and below under these circumstances.~~

The data collected for free flow condition are appended in appendix A-1. The data for submerged flow condition are appended in appendix B -1~~2~~. The upstream water level, downstream water level and the discharge passing over the weirs were only extracted for the purpose of this study.

The other pertinent points of the experiment were:

1. Length of the weir = 22.2 ft. (6.77 m)
2. Height of the crest above channel bed = 2.0 ft. (0.61 m.)

4.2.2 Fteley and Stearns also had done experiments on submerged sharp crested weirs in 1877.

The length of the weir was 5 ft. (1.53 m.) and depths over the crest varying from 0.07 ft. (2.14 cm) to 0.8 ft. (0.244 m). The mean depth of the channel below the crest was 3.17 ft. (0.96 m). The upstream and downstream water levels were measured by hook gauges. The model had no end contractions.

The discharge records under free flow condition have been appended in appendix A-2.

The level of the water on the downstream side was controlled by stop planks placed in a small dam just below the weir. The downstream water level was noted at a distance away from the crest to avoid the turbulence. The submerged flow data are recorded in appendix B - 2. The upstream and downstream water levels and the measured discharge were only taken for the purpose of this study.

4.2.3 V.Mandrup Anderson & N.Dahl had conducted model tests on trapeziodal notches in submerged condition.

The experiments were carried out in a 2m wide hydraulic flume. In this flume a canal with trapezoidal cross section was prepared. The dimensions of the trapezoidal cross section were:

Width of bottom = b = 0.60 m (1.97 ft)
Slope of the sides a = 1.5
Depth of water 0.1 m to 0.3 m. (3.94 inches to 11.82 inches)

At a distance 10 m from the inlet of the approach canal a trapezoidal test weir was errected. Behind the weir the bottom of the tail water canal was located at 0.3 m below the weir crest and the bottom of the approach canal. The bottom width of the weir and side slopes, a', could be varied by making suitable arrangements.

The model was supplied with water from a tank about 5 m (16.5 ft.) above the model level. The discharge was controlled by a sluicing valve. The discharge measurement was made by a calibrated triangular weir in the return flume and also the water levels were measured by suitable arrangements.

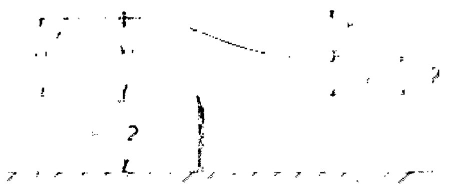
Test results for one setting of the weir having a' = 0.5 and b' = 0.1 m (3.94 in.) are appended in appendix A.7 and B.7 for free flow and submerged flow conditions respectively.

4.3.0 THICK WEIRS (WEIRS NOT SHARP CRESTED)

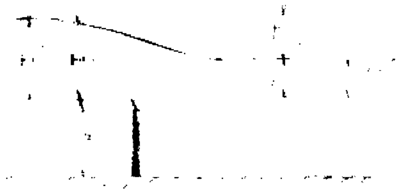
4.3.1 R.A.Smith had made experiments on broad crested weirs (Fig.21) . Model studies were conducted for two models having different u·s & d·s ~~side~~ slopes . One had slopes of 1 on 1 and the other had both u/s and d/s slopes of 1 vertical on 2 horizontal ~~with the upstream and downstream slopes same in both cases.~~ Length of the weir normal to the direction of flow was 4.95 ft.(1.51 m), width of weir crest parallel to the direction of flow, 0.5 ft.(0.153 m) ; height of weir crest above the bottom of channel , 0.425 ft.(0.13 m.) .

Head water and tail water levels were controlled by a supply valve and tail gate respectively. Head water and tail water elevations were observed by means of 1/2" (1.25 cm) piezometer connection to stilling wells equipped with hook gauges. Discharge was observed by means of a weighing tank and electric stop clock which read directly to 1/10 second. The model was operated throughout the range of discharge capacity available upto 1.00 cfs.(0.0283 cumec). At each rate of discharge, the tail water elevation was varied through a full cycle of progressively higher and progressively lower elevations in successive runs.

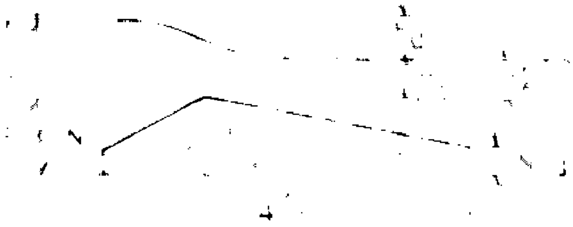
The observed data for free flow and submerged conditions respectively recorded in appendix A.4 and B.4



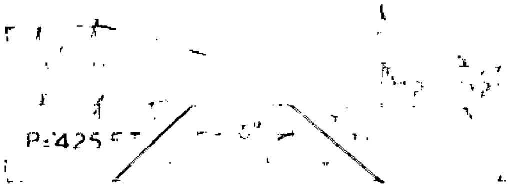
[illegible] SHARP CRESTED WEIR



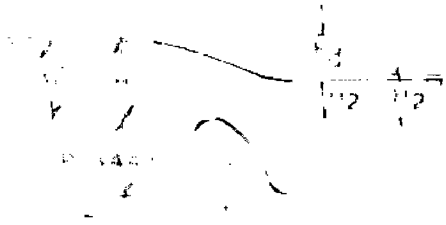
[illegible] & STEARNS' SHARP CRESTED [illegible]



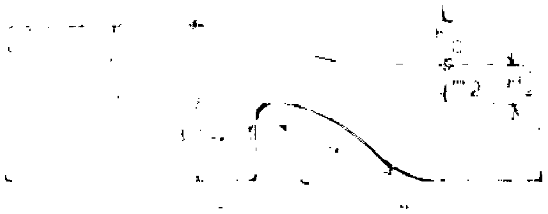
[illegible] TRIANGULAR WEIR



SMITH'S BROAD CRESTED WEIR



USBR DAM A OGEE



USBR DAM B - OGEE

[illegible] MODELS OF WEIR EXPERIMENTS [illegible]

4.3.2 E.S.Crump made his experiments over a Weir with triangular profile shape weir (Fig. 21). The model was fitted into a rectangular flume 22" wide (0.508 m) and 13 inches deep (0.33 m) with its crest approximately 3 inches above the steel floor of the flume . The weir block had an upstream face at an inclination of 1 in 2 and a downstream face at an inclination of 1 in 5 giving a crest angle of 142° - 7'. The block was truncated to give an overall length of 14 inches with two vertical faces : one, 4 inches (0.102 m) upstream of the crest, the other 10 inches (0.254 m) downstream of the crest. The machined brass angle plate, forming the crest of the weir, had eight small holes of 1/16" (1.5 mm) diameter, drilled on a line ½ inch (1.52 cm) from the crest communicating with an underlying duct. Two Pitot piers 1 inch (2.54 cm) wide divided the effective width of the weir in to one central bay of 10 inches (0.254 m) and two side bays of 5 inch (0.127m) each . End contraction effects at the piers will be there. In the nose of each pier, at crest level a single hole of 1/16" (1.5 mm) dia led to a second duct connected to the manometers. Four manometers were used for recording upstream and downstream levels. Discharge was measured by a standard V notch situated upstream of the model. In each run the discharge was kept constant while the tail water level was varied by means of a regulating gate situated at the downstream end of the flume. For each setting of the gate upstream and downstream water levels were recorded.

The data for free flow and submerged flow conditions are appended in appendix A·3 and B·3 respectively.

4.3.3 U.S.B.R. had made a lot of studies on ogee shaped weirs in submerged conditions.

Two small dams having different crest shape were subjected to the model tests under submerged condition. These are shown as Dam A and Dam B in figure 17 & 21. The experiments were conducted on two different set up. Dam A was constructed in a flume 1.52 ft.(0.462 m) wide and 24 ft.(7.31 m) long. Dam B was constructed in a flume 1.90 ft. (0.579 m) wide and 30 ft.(0.915 m) long. Adjustable floors were provided both upstream and downstream from the dam in addition to the main floor of the flume. This was done with a view to change the head water and tail water depths required in the model experiments. Regulation of the tail water was done by means of an adjustable hinged gate located at the downstream end of the flume.

During the experiment the tail water level was kept at different levels to create different submergence conditions. The water level at the upstream of the weirs was kept constant for different tail water levels by suitably reducing the discharge entering the flume. For different submergence ratios the quantity of water passing the weir, the upstream and downstream levels were recorded.

Data was taken for one set of values of P for both dams A & B . For Dam A , P was 3.44 ft.(1.05 m) and for Dam B , P = 3.54 ft. (1.08 m). The data are recorded in appendices A.5 and A.6 for free flow conditions. For submerged flow data are available in appendices B.5 and B.6 for both the dams.

CHAPTER-5

ANALYSIS OF DATA

CHAPTER - 5

ANALYSIS OF DATA

5.1.0 GENERAL

The ideas about the effect of submergence on the discharge coefficients or the discharge passing over a weir are not quite explicit and there is a confusion regarding the actual relation of discharge with submergence. This is why specific studies are conducted for particular weirs, spillways to find the actual coefficient under submerged conditions. An engineer in the field finds himself at a loss when he reads that there is no effect of submergence as high as 70 to 80% on the rate of flow(an effect which is more or less true for broad crested weirs) and on the other hand he finds that even a submergence of 20% in case of thin plate weirs is able to lower the rate of flow by 10% and increases with increase in the submergence ratio. No clear cut design criteria are available in the literature where all types of weirs were considered together and analysed to give the effect of submergence on the rate of flow.

~~5.1.1 The actual coefficient of discharge and consequently the discharge over the weir will be a function of weir geometry and flow parameters and the effect of submergence. The determination of discharge may vary from weir to weir, but there seems to be a broad agreement on the effect of submergence on the discharge ratio $\frac{Q}{Q_1}$. It was this reason why the factor Q/Q_1 was chosen for analysis without going into the specific values of coefficient of discharge.~~

~~A knowledge of the effect of submergence on the discharge ratio Q/Q_1 enable the engineer to decipher the exact magnitude of discharge passing over the weir in submerged condition.~~

5.2.0 EVALUATION OF DISCHARGE UNDER FREE FLOW CONDITION

5.2.1 Data of different questers on the various types of weirs is given in appendices A and B. Since in the evaluation of the factor $\frac{Q}{Q_1}$ the denominator will be required for different heads H_1, stage discharge relations are necessary. For this purpose the free flow data from the actual observations was analysed by plotting in double log graph paper with Q_1 as abscissa and ~~Q/Q_1~~ H_1 as ordinate and the relations are charted in figure 22. The equations relating the free discharge Q_1 in cusecs and H_1 the actual head over the crest including velocity of approach in feet are given below:

<u>Sharp Crested Weirs:</u>

1. Francis data $Q_1 = 3.35\ H_1^{1.39}$
2. Fteley and stearns data $Q_1 = 3.46\ H_1^{1.51}$

<u>V.Notch:</u>

3. Anderson and Dahl data $Q_1 = 1.61\ H_1^{1.5}$

Q_1 and H_1 in cumec & 'm' respectively.

<u>Thick Weirs:</u> (WEIRS NOT SHARP CRESTED)

4. R.A.Smith data (Broad crested weir)= $Q_1 = 3.72\ H_1^{1.56}$
5. E.S.Crump data (Triangular weir) $Q_1 = 4.02\ H_1^{1.745}$
6. U.S.B.R. Dam A (Ogee) $Q_1 = 4.11\ H_1^{1.665}$
7. U.S.B.R. Dam B (Ogee) $Q_1 = 3.72\ H1^{1.72}$

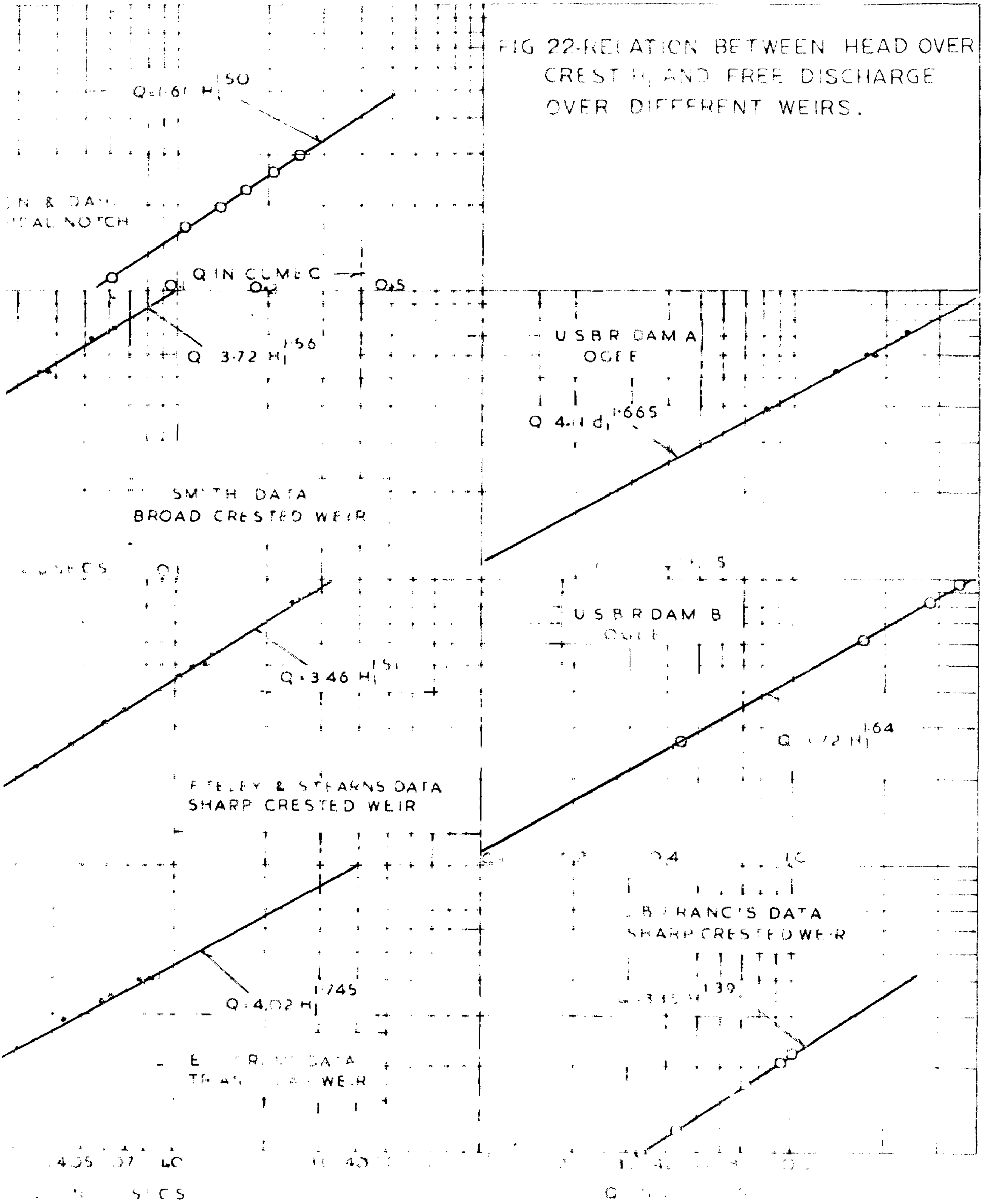


FIG 22-RELATION BETWEEN HEAD OVER CREST H_1 AND FREE DISCHARGE OVER DIFFERENT WEIRS.

5.3.0 EFFECT OF SUBMERGENCE ON DISCHARGE

5.3.1 F.T.Mavis[5] has given a relation corelating discharge ratio and the submergence factor 'S'. According to him it is possible to have one relation defining the effect of submergence which he expressed by equation:

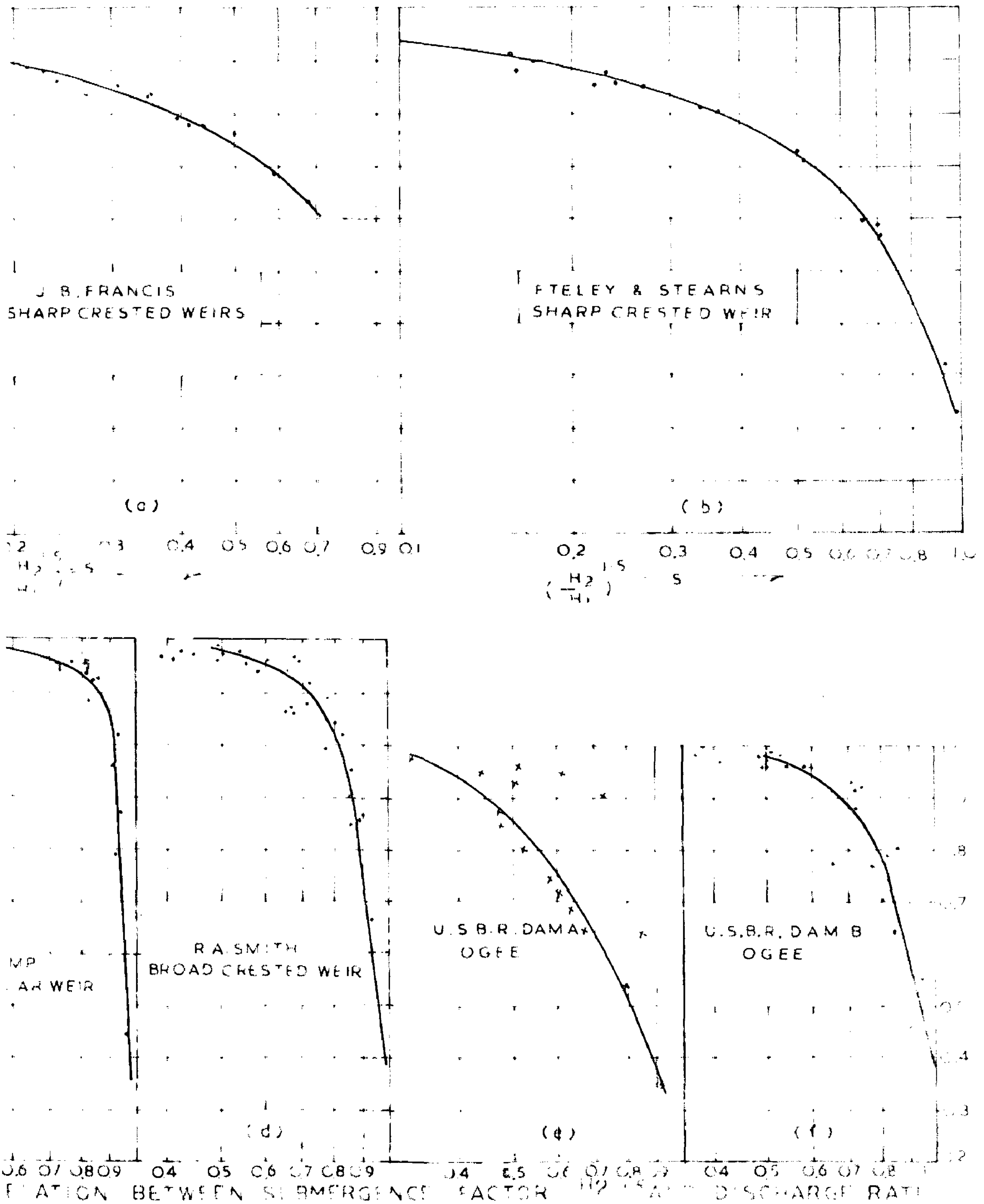
$$\frac{Q}{Q_1} = 1.00 - \left(0.45\,S + \frac{0.40}{2(10-10S)} \right)$$

With a view to study the effect of submergence factor and Q/Q_1 the relations were plotted and are exhibited in fig.23. A study of the figures will reveal the following:

1. There is a marked difference in the curvature of curves for thin plate weirs, (sharp crested weirs) and thick weirs like ogee and broad crested. In the case of sharp crested weirs the submergence effect is visible from the very outset but in the broad crested and ogee weirs practically no reduction in the discharge takes place up to a submergence factor of 0·5

2. The data of triangular and U.S.B.R. Dam A fall in between the tendencies of the sharp crested and broad crested weirs .

It will be worthwhile to note that the dam A section was in between the thin and thick geometry of the thin and thick weirs and therefore also showed results in between. The triangular weir through similar to the broad crested profile showed different results than the broad crested weir. The possible reason is in the effect that where as in the case of all the weirs the approach depth was not less than the head over the crest, in case of triangular weir it was ~~less~~ 0·25 ft in comparison with H_1 ranging from 0·79 ft to 2·1 ft. Thus the effect of approach depth might have affected the discharge



RELATION BETWEEN SUBMERGENCE FACTOR $\frac{H_2}{H_1}^{1.5}$ AND DISCHARGE RAT in triangular weirs. It was on this reason the data of triangular weir was not studied further.

5.3.2 The submergence factor takes into account the areas upstream and downstream of the weir. For simplicity pf corelation and calculations further studies were done with submergence ratio only $\underline{H2}$ = S_1.* The results of

for rect. weirs, Mavis' $S = \left(\frac{H_2}{H_1}\right)^{1.5}$

into these two broad categories for further corelation and statistical relations.

5.4.0 STATISTICAL RELATION BETWEEN S_1 and Q/Q_1

5.4.1 The two limiting conditions for any equation are that when submergence ratio is zero Q/Q_1 should be unity and when submergence ratio is one Q/Q_1 should be zero. With this aim in view the relation was determined between Q/Q_1 and S_1.

As would be clear from chapter 3, the two important factor governing the flow over submerged weirs are the Froude's number and the submergence ratio. R.A.Smith[15] and Anderson & Dahl[9] have ~~tried~~ attempted to give some corelation with Froude's number. But in the final equation they proposed only ratio of heads i.e. $\frac{h_2}{H_1}$ This approach ~~was not followed in this case~~ ~~because~~ The Froude's number for submerged weirs will range in very low limits from 0.4 to 0.7 approximately ~~and necessity~~

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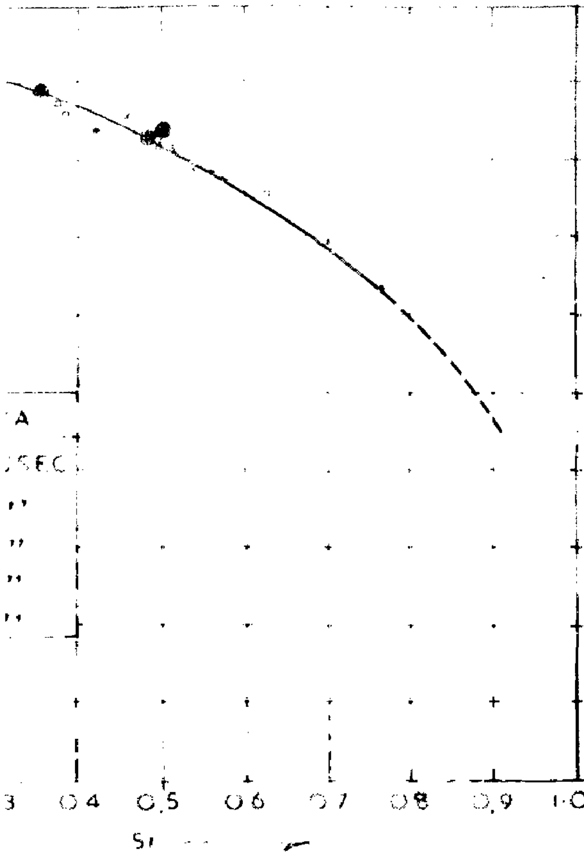
* **Maries has used the parameter 'S' for finding out relation for submerged weirs where as others have used the parameter S_1. The later parameter has been used by the author for reasons of simplicity.**



LATION OF Q/Q1 AND S1
CRESTED WEIR

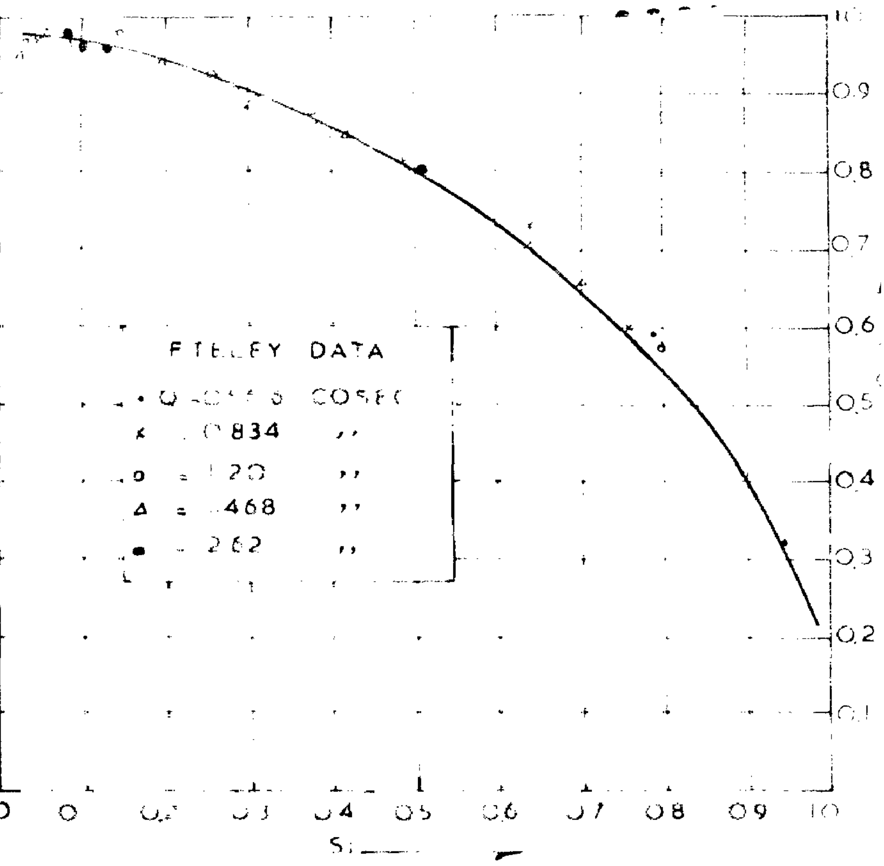
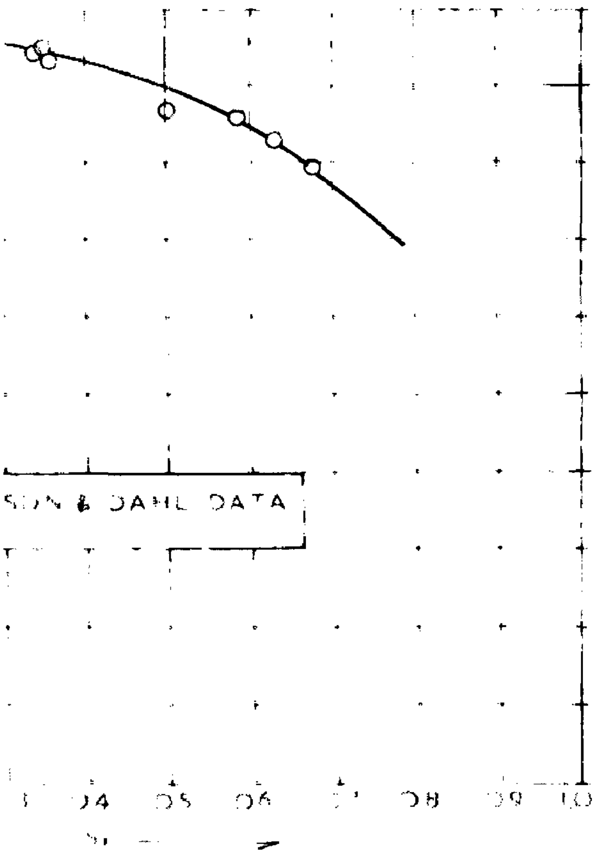


FIG.24(b) CORRELATION OF Q/Q1 AND S1 SQUARE
CRESTED WEIR



RELATION OF Q/Q1 AND S1
ZOIDAL [illegible]

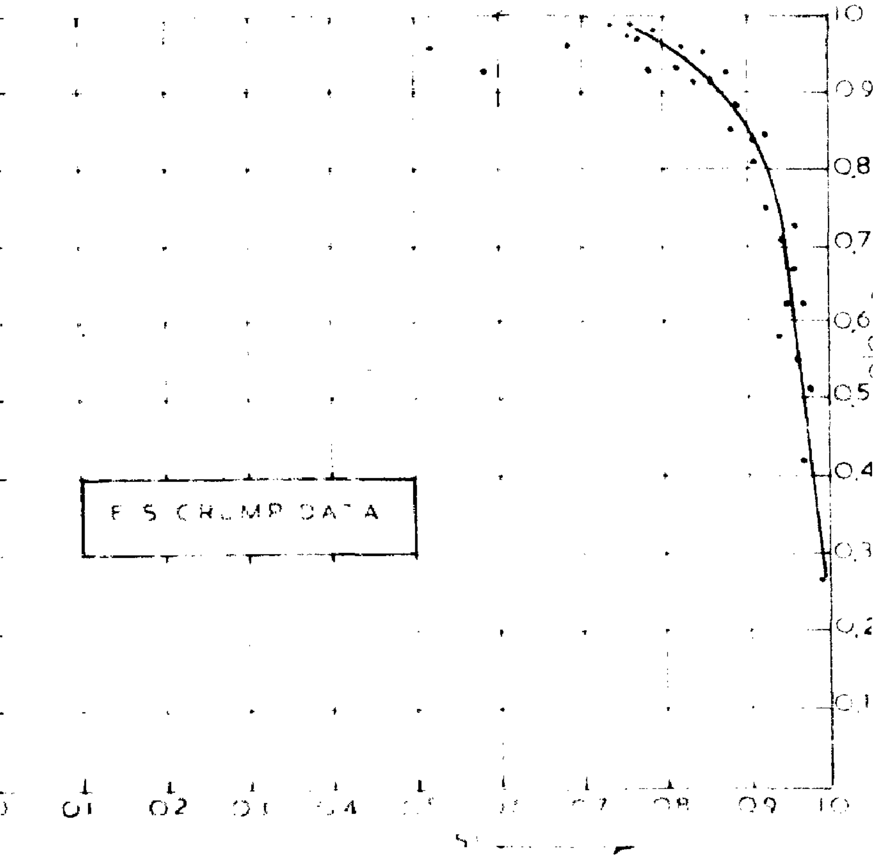
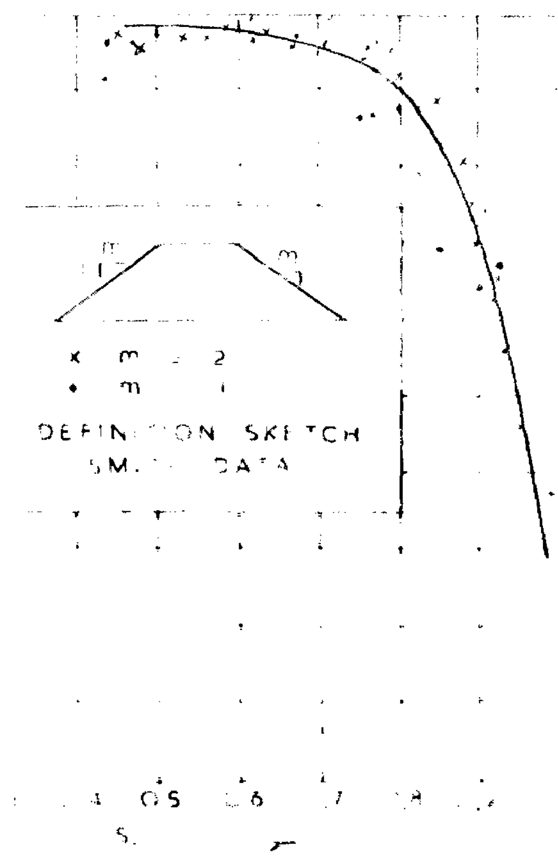


FIG 24(d) CORRELATION OF Q/Q1 AND S1
TRIANGULAR WEIR



[illegible] CRESTED WEIR

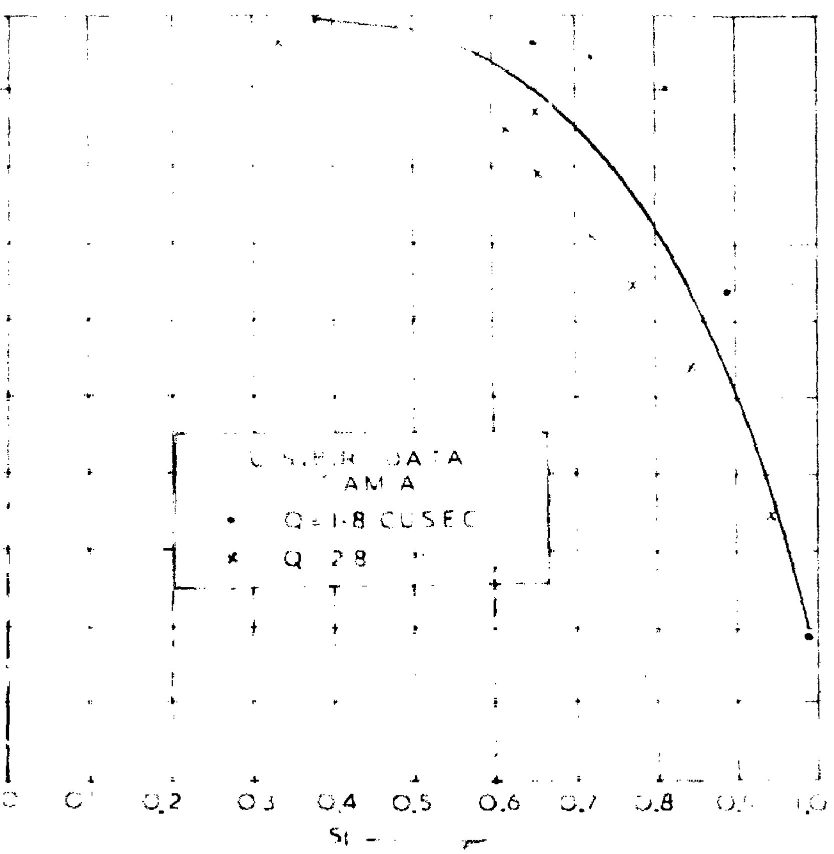
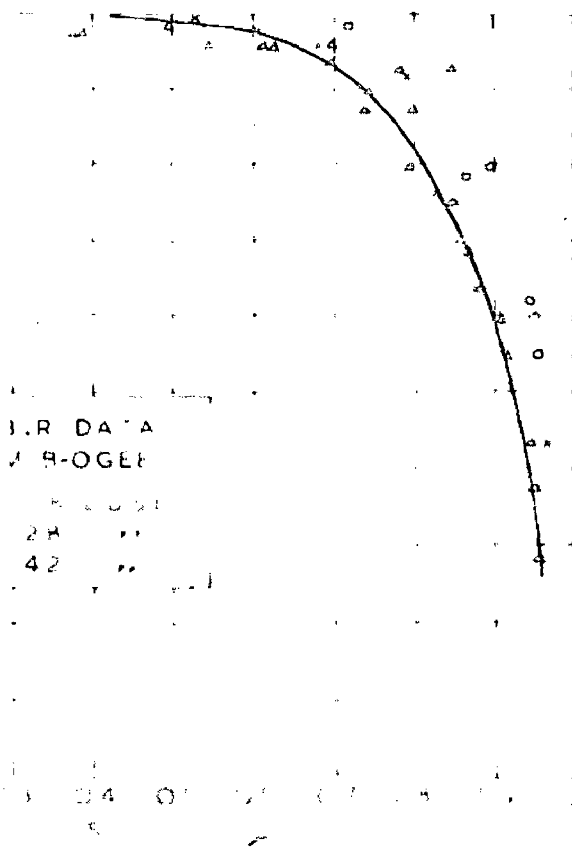
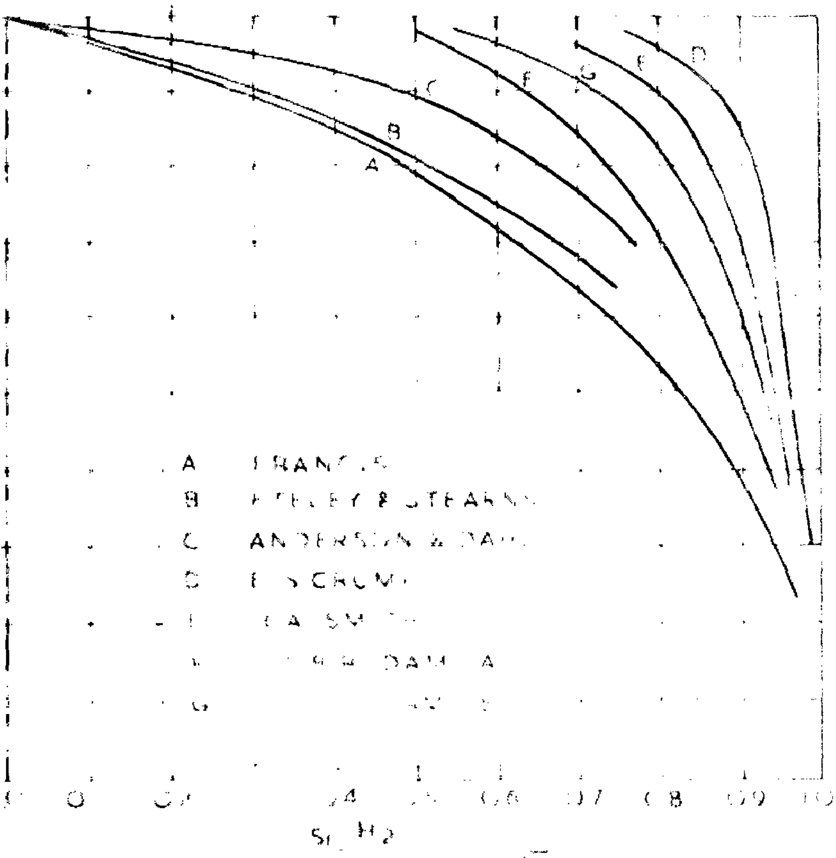


FIG 24 (C) CORRELATION OF Q/Q1 VS [illegible]
OGEE CREST



[illegible] OGEE CREST



[illegible]

~~of defining the Froude's number with respect to the upstream channel-or-flow over the crest or flow downstream. All the more the velocity profile from upstream of the weir to downstream of the weir the change is quite fast and a slight variation in the determination of velocity at any particular point or the location of the point can have a large effect on the actual determination of theeffect of submergence on discharge factor. It was for this reason a simple corelation with the submergence ratio was considered desirable.~~

5.5.0 U.P.I.R.I. has given curves for different types of submergence in weirs for Ganga river at Narora[16], Sarda type falls[18], Sarda canal regulator[17]. In all these relations over simplification have been attempted by giving a straight line variation between Q/Q_1 & S_1 (when points are plotted on a log log graph paper). Actual plotting showed that a straight line fit for points on double log paper was a rough approximation when the submergence ratio was in the range 0.3 to 0.2 .

5.6.0 RELATION FOR SUBMERGED SHARP CRESTED WEIRS

5.6.1 A study of the Francis & Fletey & Stearn's data for submerged sharp crested weirs as shown in figure 26 will show that the points lie on a smooth curve from 3 percent submergence onwards. Points for submergence from zero to 3 percent were not available because in this range the discharge ratio wavered arround unity. All the more the fluctuations in the readings was perhaps such as no definite observations other than free flow could be observed. A statistical relation as given in appendix C was tried to

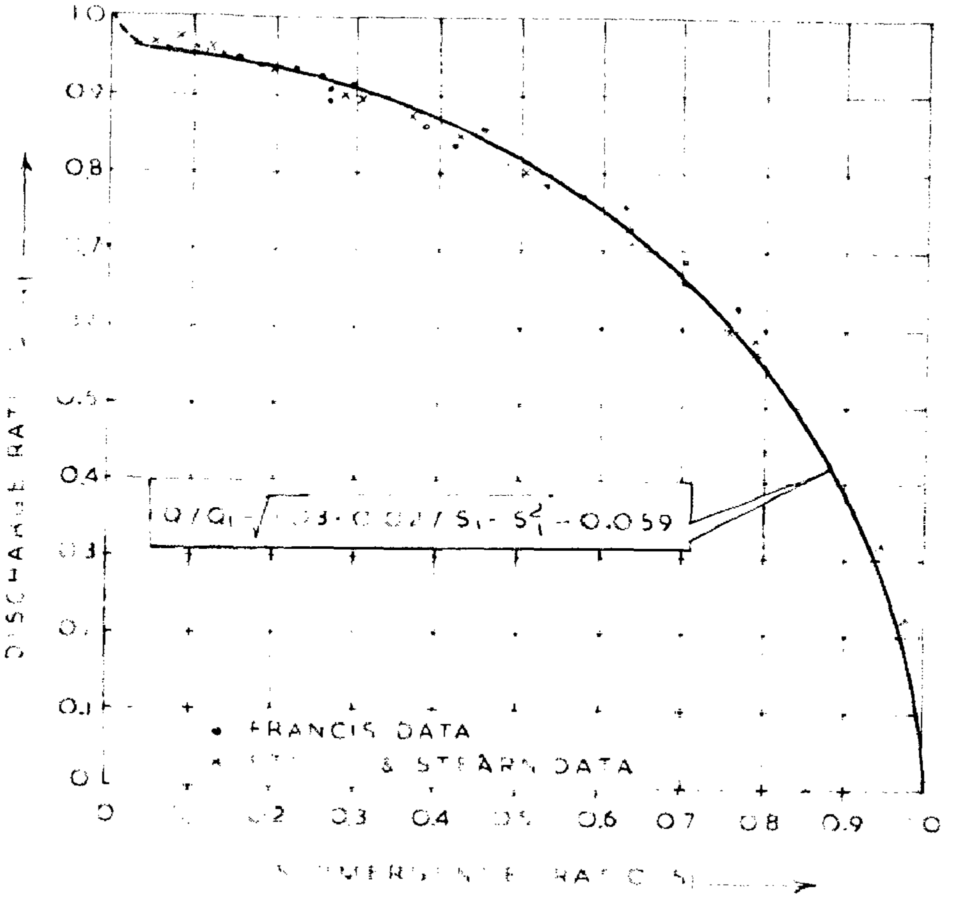
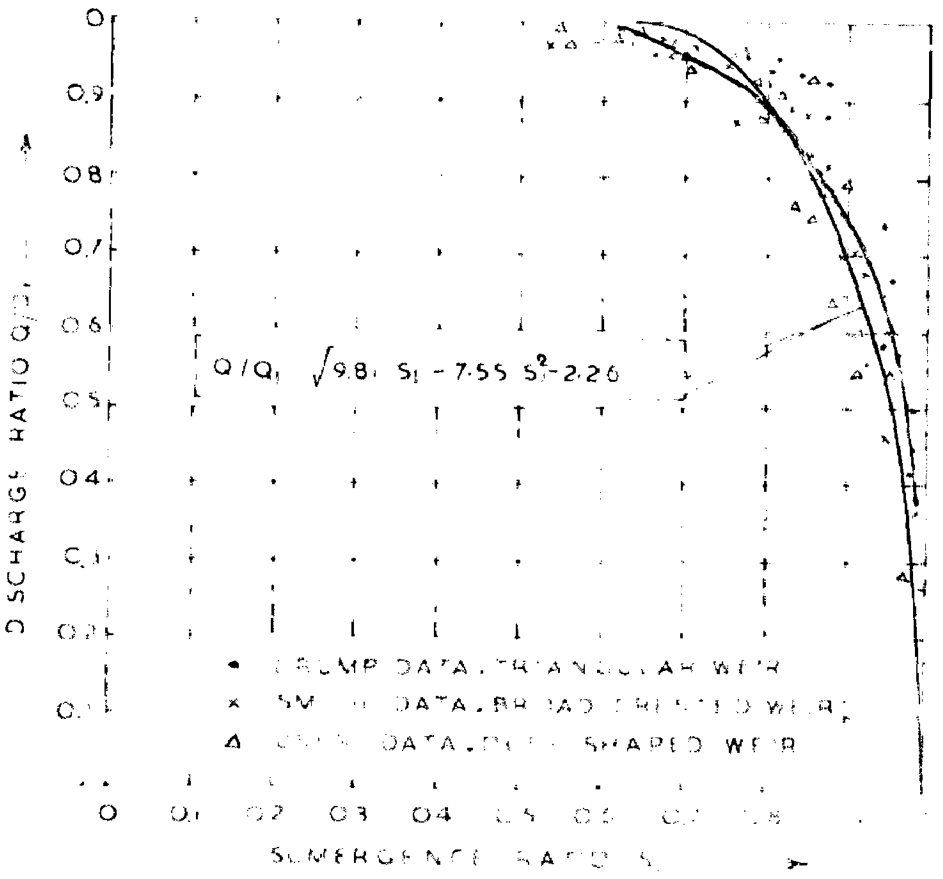


FIG. 26 CORRELATION OF SUBMERGENCE RATIO S_1 & DISCHARGE RATIO Q/Q_1 SHARP CRESTED WEIRS



SMITH CURVE

CRUMP DATA PLOTTED BUT NOT TAKEN IN ANALYSIS

FIG. 27 CORRELATION OF SUBMERGENCE RATIO S_1 AND DISCHARGE RATIO Q/Q_1 THICK WEIRS cover the points from 3 percent to 100 percent submergence. The relation works out to be:

$$\frac{Q}{Q_1} = \sqrt{1.03 - 0.027\,S_1 - S_1^2} - 0.059$$

The curve has been extended to join the discharge ratio one, at the top corner ordinate by a dotted line. Even a straight line variation of $\frac{Q}{Q_1} = 1$ at $S_1 = 0$ to $\frac{Q}{Q_1} = 0.966$ at $S_1 = 0.03$ would be a quite accurate assumption. The equation so derived can be used for sharp crested weirs.

The equation given by Villemoute[8] is superposed for comparision (Fig. 28). It is seen that the two curves agree fairly well. Hence the equation suggested by the author can also be used for sharp crested weirs between $S_1 = 0.03$ to $S_1 = 1.00$. In figure 29 discharge ratio is plotted against

---

* The equation found by the author is based on the data of sharp crested rectangular weir of J.B.Francis[7] and Fbtely and Stearns[6]. The relations given by Villemonte and Mavies cover all types of Sharp crested thin plate weirs. A plot of the authors relation with that of Mavies and Villemonte in Fig. 28 & 29 shown a maximum variation of 6% of discharge which may be due to the selection of particular data for analysis. Villemonte and Mavis relating may be used for all types of sharp crested weirs and author's relation may be taken only as a guide in the case of sharp crested rectangular weirs only.

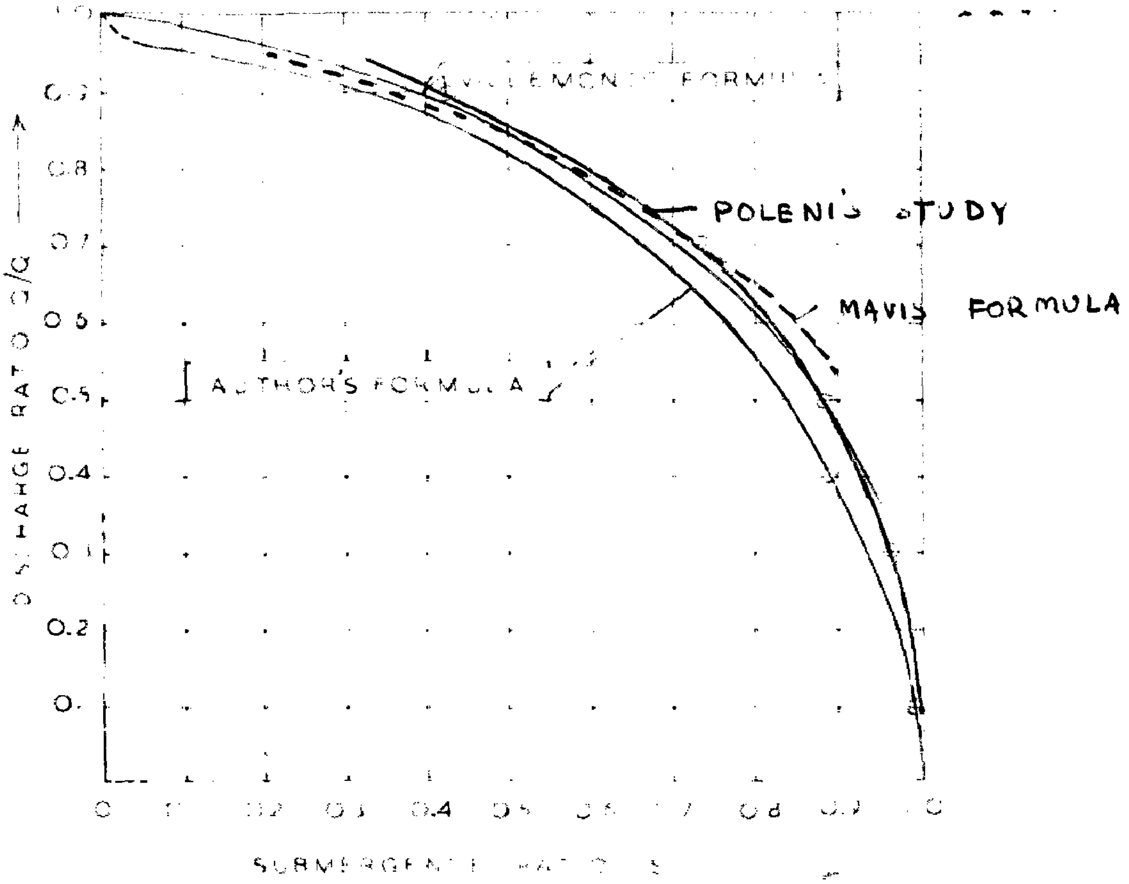


FIG 28 VILLEMONTE'S & AUTHOR'S FORMULA COMPARI

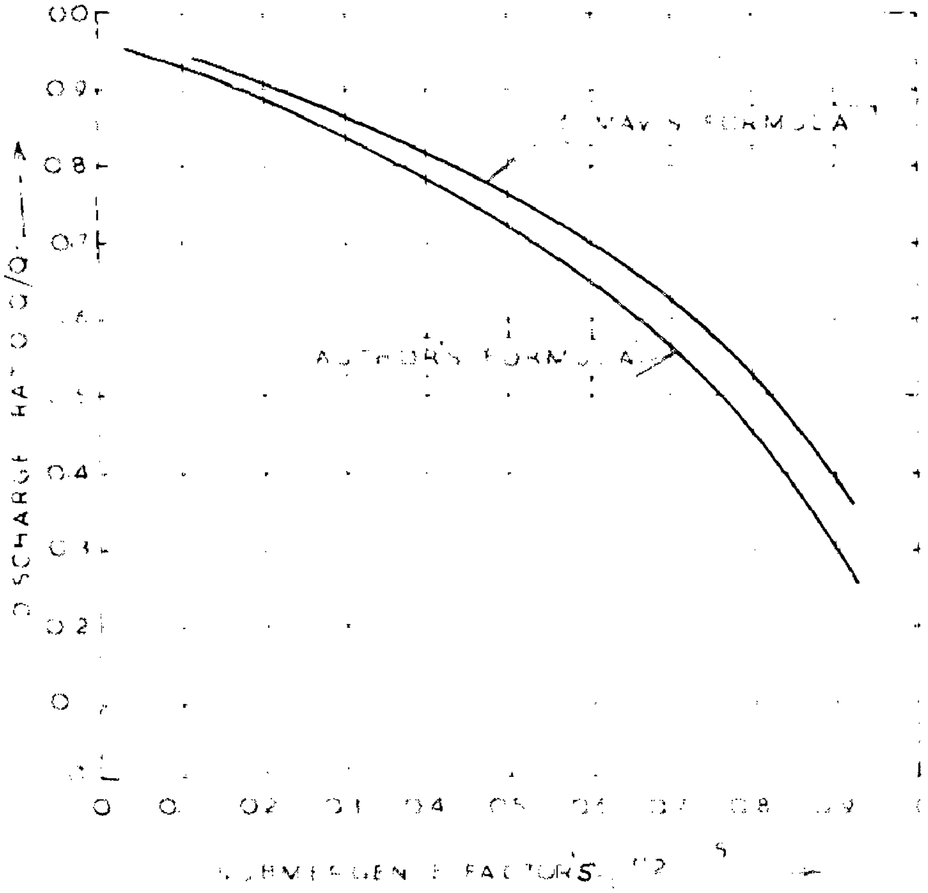


FIG 29 COMPARISION OF MAVIS & AUTHOR'S FORMULA

Crump, Smith and U.S.B.R. Dam B data (Fig. 24 and 25) show that though the submergence starts affecting the flow at about 45 - 55% submergence the effect is negligible upto a 65 percent range of submergence and there is a sharp reduction in rate of flow after a submergence ratio of 0.65. The relation derived on this limiting value of S_1=0.65 gives corelation of Q/Q_1 and S_1.

$$\frac{Q}{Q_1} = \sqrt{9.81\ S_1 - 7.55\ S_1^2 - 2.26}$$

t may be mentioned that no empirical or statistical
n has been offered by other investigators except
th[15]. The results from his relation (eqn. given in
4.4.2.) are superposed in Fig.27 for comparision. It
seen that the author's relation is simpler and has
the same accuracy as that of Smith.

The U.S.B.R. also has given a set of curves which
be used both for finding the d/s apron effect and the
mergence for ogee weirs. Those curves (Fig. No.19)
be used by the designing.

CHAPTER-6

CONCLUSION

CHAPTER-6

CONCLUSIONS

6.0.0 Submerged flow on weirs is a common occurence in the field. The usual criteria of finding the quantity of water is to consider the flow in two parts with a free weir flow over the downstream tail water level and the orifice flow between crest elevation and the tail water elevation. To derive an equation for submerged flow suitable coefficients have to be assumed for the free weir flow and orifice flow. The determination of these coefficients is a subject which is condusive to controversies and correct determination of them is an experimental procedure. All the more the coefficients so determined will vary from weir to weir and flow conditions.

6.1.0. J.R.Villemonte & F.T.Mavis were perhaps the first hydraulicians who attempted to find out universal equations for submerged weirs. Their work however is confined to thin plate weir e.g. Sharp crested weir. On the other hand most of the hydraulic Structures in the field fall in to the category of broad crested weirs. Data collected in the dissertation from various investigators show that the effect of submergence on thin plate weir like sharp crested or V notch is very much different than from the broad crested or ogee weirs. Where as in the former case the submergence effect is obvious from the initial stage of submergence in the case of latter a 65% submergence has practically no effect. With this finding, the data collected has been analysed and stastistical relations have been derived between the discharge ratio Q/Q_1 and the submergence ratio S_1

6.2.0. For the case of Sharp crested weirs the following relation holds good.

$$Q/Q_1 = \sqrt{1.03 - 0.0270\,S_1 - S_1^2} - 0.059$$

The equation found by the author is based on the data of sharp crested rectangular weirs of J.B.Francis[7] and Fbtely and Stearns[6]. The relations given by Villemonte and Mavies cover all types of Sharp crested thin plate weirs. A plot of the author's relation with that of Mavies and Villemonte in Fig. 28 & 29 shown a maximum variation of 6% of discharge which may be due to the selection of particular data for analysis. Villemonte and Mavies relation may be used for all types of sharp crested weirs and author's relation may be taken only as a guide in the case of sharp crested rectangular weirs only.

6.4.0. The above two relations can be adopted by field

The U.S.B.R. also has given a set to curves which can be used both for finding the d/s apron effect and the submergence for ogee weirs. Those curves (Fig, No.19) may be used by the designing.

cope Of Future Work:

There is a lot of scope to do future work in this ubject. The co-relation of the submergence and the ischarge ratios may be done for a wide range of discharges ver different widths of the flume. From the data analysed, hough various discharge rates are allowed over the weirs o definite trend, as to whether the rate of flow has any earing on the corelation of Q/Q_1 and S_1, could be established he aspect of scale effect also could not be studied. For ery low range of discharges the viscosity and surface ension effects may be Predominent and these must also be onsidered. Further studies in these regard are needed.

More studies may also be made to analyse the effect f the height of the weir. From a very low value of P/H_1 o high values the effect of this non-dimentional parameter n the discharge ratio may be established.

The geometry of the weir is another very important actor. Although some studies have been made on ogee type rests very few studies have been made on broad crested eirs. Different shapes of broad crested weirs and other eirs over a wide range of discharges may be subjected to odel testing up to almost 100 percent submergence. This may be done in the same manner as that of James. G.Woodburn n the non modular range of flow.

APPENDICES

APPENDIX - A

FREE FLOW OVER DIFFERENT WEIRS

In appendix A, seven tables are presented bearing numbers A-1, A-2, A-3, A-4, A-5, A-6 & A-7. The head discharge relationship under free flow condition has been derived for the types of weirs. The effect of velocity of approach has also been taken into account while deriving the expression for free flow discharge. General explanation for the appendices are as follows:-

A - 1

The data has been taken from J.B.Francis[7]' experiments on sharp crested weirs. Column 3, Column 4, and Column5 have been extracted from Francis data. The negative values in col. 4 show the distance by which the tail water is below the crest. Other columns are tabulated by the author to determine the head over crest. While calculating the velocity head, the energy correction factor is taken as unity so that $h_a = V^2/2g$. This is true for all other data collected.

A - 2

Data was taken from A.Fteley's and F.P.Stearns[6] experiments on sharp crested weirs. Column 3 and column 4 which give H_1 and Q_1 respectively were extracted from their data. Other columns are tabulated as before.

A - 3

The experimental results were taken from the work of E.S.Crump[13] on triangular weir. Column 2, column 4 and column 5 were relevant to the author's study. Column 4 gives tail water depths over the crest which means that the flow is not free. However it may be seen that for any constant discharge the upstream water depth H_1' does not change appreciably for different values of H_2'. This shows that the submergence created by the tail water levels given by column 4 has no effect on discharge. As no other free flow data is furnished these are taken to represent the free flow discharge.

A - 4

Data was from R.A.Smith's[15] work on broad crested weirs. Smith measured directly the head over crest instead of water depths.

A - 5 & A - 6

These are taken from USBR experiments on Dam A and Dam B respectively. No free flow data has been given as such. For the purpose of author's study discharge data for 25% submergence and less has been taken to represent free flow data. This assumption is fairly correct as it may be seen in the text that submergence of 50% and above affect the rate of discharge.

A - 7

The data was extracted from V. Mandrup Anderson & N. Dahl's[9] experiments on submerged trapezoidal notch. The units in this case is in metric units. In all the six readings, the tail water level was below the crest. Average discharge per meter length has been calculated taking the mean width of the notch calculated in column 10.

EXPERIMENTAL DATA RECORDED BY J.B. FRANCIS[7]

Sl. No.	No. of the experiment	H_1' ft.	H_2' ft.	Q' cusecs	Q cusecs	Depth of approach $P + H_1'$ ft.	$V=\frac{Q}{P+H_1'}$ ft./sec.	$h_a=V^2/2g$ ft.	$(H_1' + h_a) = H_1$ ft.
1	2	3	4	5	6	7	8	9	10
1.	1	1.175	-.448	94.88	4.27	3.175•	1.345	0.028	1.203
2.	3	1.182	-.476	95.00	4.28	3.182•	1.345	0.028	1.210
3.	13	1.185	-.318	95.12	4.29	3.185•	1.345	0.028	1.213
4.	36	1.674	-.553	159.97	7.20	3.674•	1.960	0.06	1.734
5.	48	1.679	-.143	160.55	7.23	3.679•	1.965	0.06	1.739
6.	51	1.675	-.738	160.26	7.21	3.675	1.960	0.06	1.735
7.	54	1.968	-.743	203.94	9.17	3.968	2.31	0.083	2.051
8.	60	1.965	-.127	203.79	9.16	3.965	2.31	0.083	2.048

Contd...../

1	2	3	4	5	6	7	8	9	10
9.	63	1.962	-.049	203.79	9.16	3.962	2.31	.083	2.045
10.	77	2.114	-	227.06	10.22	4.114	2.47	.095	2.209
11.	83	2.119	-.483	228.35	10.30	4.119	2.47	.095	2.214
12.	84	2.120	-.521	229.19	10.33	4.120	2.50	.095	2.215
13.	88	1.000	-.543	73.93	3. 34	3.000	1.11	.0188	1.0188
14.	99	0.994	-.724	73.26	3.28	2.994	1.095	.0188	1.0128
15.	101	0.993	-.715	73.26	3.28	2.993	1.095	.0188	1.0127

£

APPENDIX – A-2

FREE FLOW OVER SHARP CRESTED WEIR

EXPERIMENTAL DATA RECORDED BY A.FTELEY & F.P.STEARNS[6]

Sl. No.	No. of the experiment.	H_1' ft.	Q' cusecs	Q cusecs	Depth of approach $P + H_1'$ ft.	$V = \frac{Q}{P+H_1'}$ ft./sec.	$h_a = V^2/2g$ ft.	$(H_1' + h_a) = H_1$ ft.
1	2	3	4	5	6	7	8	9
1.	1	0.8198	12.7500	2.5500	3.9898	.655	.0070	.8270
2.	3	0.8118	12.4660	2.4730	3.9818	.62	.006	.8180
3.	4	0.6761	9.4300	1.8860	3.8461	.49	.0040	.680
4.	5	0.6713	9.3220	1.8640	3.8413	.485	.0040	.6750
5.	6	0.5203	6.3420	1.2680	3.6903	.344	.0020	.5220
6.	7	0.4310	7660 5.[illegible]0	1.1530	3.6510	.316	.0020	.4830
7.	8	0.4761	5.5470	1.1090	3.6461	.305	.0020	.4780
8.	9	0.4569	5.1990	1.0400	3.6269	.296	.001	.4570
9.	10	0.3890	4.0940	0.8190	3.5590	.230	.001	.390
10.	11	0.3424	3.3916	0.6780	3.5124	.193	-	.343

Contd...../

Table Contd.......

1	2	3	4	5	6	7	8	9
11.	13	0.3114	2.9355	0.5871	3.4814	.169	-	.312
12.	14	0.2598	2.2415	0.4483	3.4298	.131	-	.260
13.	16	0.2293	1.8764	0.3753	3.3993	.110	-	.230
14.	18	0.2182	1.7211	0.3442	3.3882	.102	-	.218
15.	20	0.1650	1.1705	0.2341	3.3350	.07	-	.165
16.	22	0.1444	0.9469	0.1894	3.2844	.0575	-	.144
17.	24	0.1225	0.7526	0.1505	3.2625	.0460	-	.122
18.	26	0.1125	0.6303	0.1261	3.2525	.039	-	.112
19.	28	0.1008	0.5877	0.1175	3.2408	.036	-	.100
20.	30	0.0746	0.3652	0.073	3.2146	.023	-	.075

FREE FLOW OVER TRIANGULAR WEIR

EXPERIMENTAL DATA RECORDED BY E.S. CRUMP[13]

Sl. No.	Sl.No.of the Expt.	H'_1 ft.	H'_2 ft.	Q' cusecs	Q cusecs	Depth of approach $P+H'_1$ ft.	$V=\frac{Q}{P+H'_1}$ ft./sec.	$h_a=V^2/2g$ ft.	$H_1=(H'_1+h_a)$ ft.
1	2	3	4	5	6	7	8	9	10
1.	Table 1-10	.216	.126	.5769	.315	.466	.675	.0071	.2231
2.	Table 2-9	.262	.077	.7929	.433	.512	.847	.0112	.2732
3.	Table 3-18	.324	-.104	1.1720	.640	.574	1.115	.0193	.3433
4.	Table 3-19	.324	.037	1.1720	.640	.574	1.115	.0193	.3433
5.	Table 3-20	.324	.077	1.1720	.640	.574	1.115	.0193	.3433
6.	Table 3-21	.324	.002	1.1720	.640	.574	1.115	.0193	.3433
7.	Table 3-22	.324	.148	1.1720	.640	.574	1.115	.0193	.3433
8.	Table 3-23	.324	.173	1.1720	.640	.574	1.115	.0193	.3433
9.	Table 4-15	.375	.284	1.4371	.784	.625	1.255	.0244 .0[illegible]	.3994

Contd...../-

1	2	3	4	5	6	7	8	9	10
10.	Table 4-16	.373	.273	1.4371	.784	.623	1.255	.0244	.3992
11.	Table 4-17	.373	.261	1.4371	.784	.623	1.255	.0244	.3992
12.	Table 4-18	.372	.245	1.4371	.784	.622	1.255	.0244	.3991
13.	Table 4-19	.372	.228	1.4371	.784	.622	1.255	.0244	.3991
14.	Table 4-20	.370	.205	1.4371	.784	.620	1.265	.0248	.3948
15.	Table 4-21	.370	.180	1.4371	.784	.620	1.265	.0248	.3948
16.	Table 6-1	.367	.154	1.510	.825	.617	1.335	.0277	.3947
17.	Table 6-2	.366	.222	1.510	.825	.616	1.335	.0277	.3947
18.	Table 6-3	.366	.243	1.510	.825	.616	1.335	.0277	.3946

APPENDIX- A-4

FREE FLOW OVER BROAD CRESTED WEIR
EXPERIMENTAL DATA RECORDED BY R.A.SMITH[15]

Sl. No.	Sl.No. of the Expt.	Q cusecs	$H_1' = (H_1' + h_a)$ ft.
With Side Slopes 1:1			
1.	1	.0183	.034
2.	3	.0369	.053
3.	12	.0610	.074
4.	19	.0840	.089
5.	26	.1006	.100
6.	40	.1411	.126
7.	52	.1832	.148
With Side Slopes 2:1			
8.	61	.0218	.039
9.	63	.0349	.053
10.	67	.0527	.069
11.	70	.0838	.090
12.	73	.1014	.101
13.	86	.1410	.082
14.	95	.1009	.101
15.	99	.0804	.087

FREE FLOW OVER OGEE SHAPED WEIR

EXPERIMENTS CONDUCTED BY U.S.B.R.[11]

DAM-A

Sl. No. 1	H_1' ft. 2	Q cusecs 3	Depth of approach $P+H_1'$ ft. 4	$V=\frac{Q}{P+H_1'}$ ft./sec. 5	$h_a = V^2/2g$ ft. 6	$H_1 = H_1' + V^2/2g$ ft. 7
1.	.598	1.865	4.038	.46	.003	.601
2.	.600	1.865	4.040	.46	.003	.603
3.	.599	1.826	4.039	.45	.003	.602
4.	.599	1.793	4.039	.445	.003	.602
5.	.600	1.751	4.040	.434	.003	.603
6.	.521	1.416	3.961	.356	.002	.523
7.	.713	2.308	4.153	.49	.004	.717
8.	.714	2.382	4.154	.573	.005	.719
9.	.382	.826	3.822	.216	.001	.383

FREE FLOW OVER TRAPEZOIDAL NOTCH

EXPERIMENTAL DATA RECORDED BY ANDERSON & DAHL[9]

Sl. No.	Test No.	H_1' cm.	Q' cumec	b+a H_1' in 'm'	$F=H_1'(b+aH_1')$ m^2	$V=\frac{Q'}{F}$ m/sec.	$h_a = V^2/2g$ m	H_1 m	$b'+a'H_1'$ m	Q' cumec.
1	2	3	4	5	6	7	8	9	10	11
1.	278	26.100	.0489	0.99	.258	.1895	0.00183	.26102	.231	.212
2.	279	29.960	.0650	1.05	.315	.2065	0.00217	.29962	.25	.260
3.	283	22.620	.0365	0.94	.2125	.1715	0.00150	.22622	.213	.172
4.	287	19.540	.0273	0.893	.1745	.1565	0.00125	.19541	.197	.1385
5.	291	16.600	.0198	0.849	.141	.1405	0.00100	.16601	.183	.108
6.	293	11.340	.0096	0.77	0087	.1100	0.0006	.11341	.1566	.0612

FREE FLOW OVER OGEE SHAPED WEIR

EXPERIMENTS CONDUCTED BY U.S.B.R.[11]

DAM-B

Sl. No.	H'_1 ft.	Q cusecs	Depth of approach $P + H'_1$ ft.	$V = \frac{Q}{P+H'_1}$ ft./sec.	$ha = V^2/2g$ ft.	$H_1 = H'_1 + V^2/2g$ ft.
1	2	3	4	5	6	7
1.	.617	1.746	4.057	0.43	0.003	.620
2.	.617	1.713	4.057	0.42	0.003	.620
3.	1.054	3.991	4.494	0.9	0.012	1.066
4.	1.047	4.027	4.487	0.9	0.012	1.059
5.	1.056	4.181	4.496	0.94	0.014	1.070
6.	1.056	4.290	4.496	0.965	0.015	1.071
7.	0.819	2.809	4.259	0.66	0.007	0.826

FREE FLOW OVER TRAPEZOIDAL NOTCH

EXPERIMENTAL DATA RECORDED BY ANDERSON & DAHL[9]

Sl. No.	Test No.	H_1' cm.	Q' cumec	$b+a\,H_1'$ in 'm'	$F=H_1'(b+aH_1')$ m^2	$V=\frac{Q'}{F}$ m/sec.	$h_a = V^2/2g$ m	H_1 m	$b'+a'H_1'$ m	Q' cumec.
1	2	3	4	5	6	7	8	9	10	11
1.	278	26.100	.0489	0.99	.258	.1895	0.00183	.26102	.231	.212
2.	279	29.960	.0650	1.05	.315	.2065	0.00217	.29962	.25	.260
3.	283	22.620	.0365	0.94	.2125	.1715	0.00150	.22622	.213	.172
4.	287	19.540	.0273	0.893	.1745	.1565	0.00125	.19541	.197	.1385
5.	291	16.600	.0198	0.849	.141	.1405	0.00100	.16601	.183	.108
6.	293	11.340	.0096	0.77	0087	.1100	0.0006	.11341	.1566	.0612

APPENDIX - B

SUBMERGED FLOW OVER WEIRS

In this appendix seven tables are also presented having numbers B-1, B-2, B-3, B-4, B-5, B-6 & B-7 dealing with submerged flow over the different types of weirs. The data collected from various investigators for their works were for the head water (H_1') and tail water (H_2') depths and the discharge (Q) in submerged condition. All other columns are tabulated by the author. The velocity head is calculated taking energy correction factor as unity. It is assumed that the velocity head is same for both the upstream and downstream of the weir. In the actual case it may be slightly different depending on the upstream and downstream water depths of flow. However the velocity head is so small that this difference is negligible. The submergence ratio $S_1 = \frac{H_2}{H_1}$ and the discharge ratio $\frac{Q}{Q_1}$ are calculated. Q_1 is the free flow discharge for the corresponding head H_1 and is calculated from the relation established for different types of weirs in para 5.2.1 from the experimental results appended in Appendix A. Another parameter ($\frac{H_2}{H_1}$)$^{1.5}$ has also been calculated. This parameter represents the submergence factor 'S' as defined by F.T.Mavis. According to him:-

$$S = \frac{a_2\sqrt{H_2}}{a_1\sqrt{H_1}}$$

Since in the present study discharge per unit length is calculated, the ratio $\frac{a_2}{a_1}$ is same as $\frac{H_2}{H_1}$

Hence $$S = \frac{H_2}{H_1}\sqrt{\frac{H_2}{H_1}} = \left(\frac{H_2}{H_1}\right)^{1.5}$$

The various tables furnished in this appendix are as follows:-

B - 1	Sharp crested weir	Data from J.B.Francis
B - 2	Sharp crested weir	Data from A.Fteley & F.P.Stearns
B - 3	Triangular weir	Data from E.S.Crump
B - 4	Broad crested weir	Data from R.A.Smith
B - 5 & B - 6	Ogee weir	Data from U.S.B.R.
B - 7	Trapezoidal notch	Data from Anderson & Dahl

APPENDIX- B-1

Sl. No.	H_2+h_a ft.	$S_1=H_2/H_1$	$(H_2/H_1)^{1.5}$	$Q_1=3.35H_1^{1.39}$ cusecs	Q/Q_1
1	1	12	13	14	15
1.	071	.07	.02	3.48	.956
2.	.155	.152	.06	3.51	.950
3.	.282	.267	.138	3.72	.890
4.	466	.421	.272	3.96	.835
5.	674	.575	.44	4.285	.771
6.	652	.560	.42	4.25	.775
7.	.030	.765	.68	5.20	.630
8.	050	.04	.01	4.44	.966
9.	162	.134	.05	4.50	.954
10.	276	.224	.106	4.59	.935
11.	254	.207	.096	4.55	.943
12.	124	.1023	.043	4.50	.954
13.	117	.097	.03	4.50	.954
14.	.234	.181	.078	4.60	.934
15.	.336	.268	.140	4.72	.910
16.	.504	.387	.24	4.98	.860
17.	.884	.625	.50	5.62	.76

Contd........./

Table B

1		12	13	14	15
18.	62	.70	.59	6.20	.684
19.	35	.049	.01	7.45	.953
20.	32	.048	.01	7.54	.953
21.	35	.078	.02	7.54	.977
22.	4	.295	.161	7.84	.917
23.	21	.291	.158	7.96	.904
24.	38	.299	.164	7.97	.896
25.	47	.456	.308	8.36	.855
26.	50	.532	.393	9.11	.788
27.	12	.055	.015	9.65	.95
28.	62	.065	.017	9.65	.95
29.	10	.102	.032	9.7	.936
30.	65	.124	.043	9.7	.936
31.	6	.197	.088	9.8	.934
32.	08	.288	.156	10.08	.907
33.	08	.372	.228	10.42	.876
34.	10	.374	.229	10.40	.88
35.	28	.499	.355	11.13	.818
36.	45	.505	.362	11.20	.814
37.	87	.355	.213	11.40	.888
38.	896	.500	.365	12.15	.835
39.	890	.498	.350	12.20	.830

Table B-1 Contd.........

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
18.	25	1.491	1.039	94.15	4.24	3.491	1.212	.023	1.514	1.062	.70	.59	6.20	.684
19.	40	1.670	0.025	159.68	7.19	3.670	1.96	.060	1.730	0.085	.049	.01	7.45	.953
20.	41	1.670	0.022	159.68	7.19	3.670	1.96	.060	1.730	0.082	.048	.01	7.54	.953
21.	42	1.670	0.075	159.68	7.19	3.670	1.96	.060	1.730	0.135	.078	.02	7.54	.977
22.	43	1.720	0.466	159.25	7.18	3.720	1.94	.058	1.778	.524	.295	.161	7.84	.917
23.	44	1.740	0.465	159.25	7.18	3.740	1.92	.056	1.796	0.521	.291	.158	7.96	.904
24.	45	1.743	0.483	158.97	7.15	3.743	1.91	.055	1.798	0.538	.299	.164	7.97	.896
25.	46	1.804	0.792	159.83	7.15	3.804	1.88	.055	1.859	0.847	.456	.308	8.36	.855
26.	47	1.917	0.996	159.25	7.18	3.917	1.835	.054	1.971	1.050	.532	.393	9.11	.788
27.	64	1.965	0.029	203.63	9.15	3.965	2.31	.083	2.048	0.112	.055	.015	9.65	.95
28.	65	1.965	0.099	203.63	9.15	3.965	2.31	.083	2.048	0.132	.065	.017	9.65	.95
29.	16	1.976	0.128	203.47	9.145	3.976	2.3	.082	2.058	0.210	.102	.032	9.7	.936
30.	67	1.976	0.173	203.47	9.145	3.976	2.3	.082	2.058	0.255	.124	.043	9.7	.936
31.	68	1.994	0.327	203.16	9.14	3.994	2.29	.081	2.075	.408	.197	.088	9.8	.934
32.	69	2.034	0.528	203.01	9.14	4.034	2.27	.080	2.114	0.608	.288	.156	10.08	.907
33.	70	2.092	0.730	203.32	9.145	4.092	2.235	.078	2.170	0.808	.372	.228	10.42	.876
34.	71	2.090	0.732	203.47	9.145	4.090	2.235	.078	2.168	0.810	.374	.229	10.40	.88
35.	72	2.188	1.054	202.54	9.11	4.188	2.18	.074	2.262	1.128	.499	.355	11.13	.818
36.	73	2.190	1.071	202.54	9.11	4.190	2.18	.074	2.264	1.145	.505	.362	11.20	.814
37.	80	2.212	0.727	225.29	10.14	4.212	2.4	.090	2.302	.817	.355	.213	11.40	.888
38.	81	2.319	1.111	226.17	10.1	4.319	2.34	.085	2.404	1.196	.500	.365	12.15	.835
39.	82	2.318	1.102	228.35	10.3	4.318	2.38	.088	2.406	1.190	.498	.350	12.20	.830

APPENDIX - B-2

SUBMERGED FLOW OVER SHARP CRESTED WEIR

DETERMINATION OF SUBMERGENCE AND DISCHARGE RATIOS

EXPERIMENTAL DATA RECORDED BY FTELEY & STEARNS[6]

Sl. No.	Experiment No.	H_1' ft.	H_2' ft.	Q cusec	$P + H_1'$ ft.	$V = \frac{Q}{P+H_1'}$ ft/sec.	$h_a = V^2/2g$ ft.	$H_1 = H_1' + h_a$ ft.	$H_2 = H_2' + h_a$ ft.	$H_2/H_1 = S$	$(H_2/H_1)^{1.6}$	$Q_1 = 3.46\ H_1^{1.61}$ cusecs.	Q/Q_1
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1.	9	.3251	.0956	.5616	3.4951	.161	Negligible	.3251	.0956	.294	.16	0.64	.878
2.	19	.4282	.3373	.5617	3.5982	.1565	"	.4282	.3373	.788	.707	0.96	.585
3.	21	.6246	.5894	.5617	3.7946	.148	"	.6246	.5894	.945	.93	1.77	.318
4.	22	.8149	.7947	.5617	3.9848	.141	"	.8149	.7947	.975	.965	2.50	.225
5.	3	.3960	.0165	.8336	3.5660	.234	.001	.397	.0175	.0441	.01	.86	.969
6.	8	.4157	.1185	.8342	3.5857	.233	.001	.4167	.1195	.287	.156	0.92	.907
7.	14	.4504	.2195	.8339	3.6204	.230	.001	.4514	.2205	.488	.34	1.03	.81
8.	15	.4857	.3078	.8336	3.6557	.228	.001	.4867	.3088	.635	.512	1.15	.725
9.	18	.5454	.4186	.8339	3.7154	.224	.001	.5464	.4196	.757	.663	1.40	.596
10.	6	.5080	.0735	1.2035	3.6780	.326	.0016	.5096	.0751	.148	.057	1.23	.973
11.	12	.5462	.2095	1.2017	3.7162	.324	.0016	.5478	.2111	.386	.24	1.40	.855
12.	20	.7193	.5731	1.1993	3.8893	.309	.0015	.7208	.5748	.797	.710	2.1	.570
13.	2	.5777	.0098	1.4652	3.7477	.391	.0024	.5801	.0122	.021	.006	1.55	.946
14.	5	.5743	.0463	1.4682	3.7443	.391	.0024	.5767	.0485	.084	.025	1.53	.959
15.	7	.5922	.1122	1.4663	3.7622	.390	.0024	.5946	.1146	.193	.085	1.57	.934

Contd....../

Table B-2 Contd......

1	2	3	4	5	6	7	8	9	10	11	12	13	14
16.	10	.6098	.1859	1.4671	3.7798	.388	.0023	.6121	.1882	.308	.172	1.64	.893
17.	13	.6328	.2614	1.4641	3.8028	.386	.0023	.6351	.2637	.415	.27	1.73	.847
18.	17	.7425	.5186	1.4633	3.9125	.375	.0022	.7447	.5208	.70	.59	2.2	.656
19.	4	.6625	.0330	1.8103	3.8325	.472	.0035	.6660	.0365	.055	.01	1.86	.973
20.	11	.7165	.2665	1.8078	3.8865	.466	.0033	.7198	.2698	.375	.23	2.07	.874
21.	16	.8123	.5157	1.8058	3.9823	.454	.0031	.8154	.5188	.636	.525	2.56	.707
22.Expt.by Francis	1	.8532	.0200	2.6211	4.0232	.651	.0067	.8599	.0267	.031	.008	2.72	.965
23.	2	.8485	.0650	2.6211	4.0185	.653	.0070	.8555	.0720	.0841	.02	2.70	.973
24.	3	.8522	.0850	2.6211	4.0222	.651	.0067	.8589	.0917	.107	.036	2.715	.968
25.	4	.8571	.1050	2.6211	4.0271	.651	.0067	.8638	.1117	.129	.046	2.74	.958
26.	5	.8820	.2200	2.6211	4.0520	.647	.0064	.8884	.2264	.255	.13	2.85	.922
27.	6	.9706	.4900	2.6211	4.1406	.633	.0063	.9769	.4963	.508	.366	3.28	.801

SUBMERGED FLOW OVER TRIANGULAR WEIR

DETERMINATION OF SUBMERGENCE AND DISCHARGE RATIOS

EXPERIMENTAL DATA RECORDED BY E.S.CRUMP[13]

Sl. No.	Experiment No.	H'_1 ft.	H'_2 ft.	Q' cusecs	Q cusecs	$P+H'_1$ ft.	$V=Q/P+H'_1$ ft./sec.	$h_a=V^2/2g$ ft.	$H_1=H'_1+h_a$ ft.	$H_2=H'_2+h_a$ ft.	$S_1=H_2/H_1$	$(H_2/H_1)^{\overset{1.5}{n}}$	$Q_1=4.02H_1^{1.745}$	Q/Q_1
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1.	Table. 1-1	0.77	.766	0.5769	.315	1.02	.309	.00148	.771	.767	.995	.99	2.295	.138
2.	1-2	.75	.748	0.5769	.315	1.00	.315	.0015	.751	.749	.995	.99	2.32	.136
3.	1-3	.71	.708	0.5769	.315	0.96	.328	.00167	.712	.710	.995	.99	2.12	.149
4.	1-4	.494	.4915	0.5769	.315	.744	.424	.0028	.497	.494	.995	.99	1.175	.268
5.	1-5	.28	.2665	0.5769	.315	0.53	.595	.0055	.286	.272	.952	.94	.47	.67
6.	1-6	.305	.286	0.5769	.315	.555	.568	.0050	.310	.291	.938	.92	.539	.584
7.	1-7	.255	.236	0.5769	.315	.505	.625	.0061	.261	.242	.928	.91	.405	.777
8.	1-8	.224	.194	0.5769	.315	.474	.665	.0069	.231	.201	.871	.825	.331	.924
9.	1-9	.208	.157	0.5769	.315	.458	.687	.0074	.215	.164	.764	.675	.298	1.00
10.	1-10	.222	.127	0.5769	.315	.472	.658	.0067	.229	.134	.585	.45	.330	.924
11.	2-1	.788	.786	0.7928	.433	1.038	.417	.0027	.791	.789	.997	.992	2.52	.172
12.	2-2	.7585	.754	0.7928	.433	1.0085	.433	.0030	.761	.757	.995	.99	2.30	.189
13.	2-3	.723	.719	0.7928	.433	.973	.445	.0031	.726	.722	.995	.99	2.19	.198
14.	2-4	.481	.474	0.7928	.433	.731	.592	.0055	.487	.480	.986	.975	1.13	.384
15.	2-5	.3605	.3445	0.7928	.433	.6105	.709	.0075	.368	.352	.958	.94	.714	.607

Contd......./-

Table B-3 Contd........

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
16.	2-6	.311	.286	0.7928	.433	.561	.772	.009	.320	.295	.923	.91	.569	.762
17.	2-7	.284	.256	0.7928	.433	.534	.811	.010	.294	.266	.87	.825	.490	.884
18.	2-8	.269	.2145	0.7928	.433	.519	.834	.010	.279	.224	.804	.73	.460	.941
19.	2-9	.262	.075	0.7928	.433	.512	.846	.011	.273	.086	.315	.177	.450	.963
20.	3-1	.615	.606	1.1720	.640	.865	.740	.008	.623	.614	.985	.975	1.70	.377
21.	3-2	.591	.581	1.1720	.640	.841	.762	.009	.600	.590	.985	.975	1.60	.40
22.	3-3	.568	.558	1.1720	.640	.818	.784	.0095	.578	.568	.983	.973	1.51	.424
23.	3-4	.547	.536	1.1720	.640	.797	.806	.010	.557	.546	.981	.971	1.416	.452
24.	3-5	.524	.512	1.1720	.640	.774	.890	.011	.535	.523	.979	.97	1.280	.500
25.	3-6	.499	.487	1.1720	.640	.749	.858	.012	.511	.499	.978	.97	1.230	.520
26.	3-7	.472	.453	1.1720	.640	.722	.892	.013	.487	.466	.962	.95	1.130	.567
27.	3-8	.447	.424	1.1720	.640	.747	.860	.012	.459	.436	.950	.94	1.03	.621
28.	3-9	.423	.397	1.1720	.640	.673	.957	.014	.437	.411	.940	.92	.95	.675
29.	3-10	.398	.372	1.1720	.640	.648	.989	.015	.413	.387	.940	.92	.867	.739
30.	3-11	.374	.346	1.1720	.640	.624	1.025	.016	.390	.362	.930	.92	.790	.810
31.	3-12	.367	.335	1.1720	.640	.617	1.038	.016	.383	.351	.915	.895	.763	.839
32.	3-13	.356	.323	1.1720	.640	.606	1.055	.017	.373	.340	.911	.89	.73	.877
33.	3-14	.343	.304	1.1720	.640	.593	1.08	.018	.361	.322	.881	.84	.694	.922
34.	3-15	.339	.293	1.1720	.640	.489	1.088	.019	.358	.312	.873	.82	.685	.933
35.	3-16	.338	.282	1.1720	.640	.588	1.088	.019	.357	.301	.842	.78	.685	.933

Table B-3 Contd......

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
36.	3-17	.3325	.267	1.1720	.640	.5825	1.10	0.019	.351	.288	.815	.745	.670	.956
37.	3-18	.324	-.104	1.1720	.640	.0574	1.12	0.019	.343	-	-	-	.637	1.00
38.	3-19	.323	.0375	1.1720	.640	.573	1.12	0.019	.342	.056	.164	.066	.637	1.00
39.	3-20	.324	.0766	1.1720	.640	.574	1.12	0.019	.343	.096	.281	.150	.637	1.00
40.	3-21	.323	.123	1.1702	.640	.573	1.12	0.019	.342	.142	.414	.268	.637	1.00
41.	3-22	.324	.148	1.1720	.640	.574	1.12	0.019	.343	.167	.486	.34	.637	1.00
42.	3-23	.323	.173	1.1720	.640	.573	1.12	0.019	.342	.192	.560	.420	.637	1.00
43.	3-24	.324	.202	1.1720	.640	.574	1.12	0.019	.343	.221	.644	.52	.637	1.00
44.	3-25	.324	.230	1.1720	.640	.574	1.12	0.019	.343	.249	.726	.625	.637	1.00
45.	3-26	.323	.2625	1.1720	.640	.573	1.12	0.019	.342	.281	.823	.76	.637	1.00
46.	3-27	.366	.250	1.1720	.640	.616	1.04	0.016	.382	.866	.959	.95	.755	.847
47.	4-1	.747	.738	1.4371	.784	.997	0.786	0.099	.756	.747	.987	.98	2.34	.335
48.	4-2	.712	.700	1.4371	.784	.962	0.814	0.009	.721	.709	.985	.98	2.17	.361
49.	4-3	.677	.666	1.4371	.784	.927	0.845	0.010	.687	.676	.985	.98	2.00	.398
50.	4-4	.647	.632	1.4371	.784	.897	0.873	0.011	.658	.643	.978	.96	1.862	.421
51.	4-5	.604	.583	1.4371	.784	.854	0.981	0.012	.616	.595	.965	.95	1.675	.468
52.	4-6	.557	.537	1.4371	.784	.807	0.971	0.015	.571	.551	.965	.95	1.52	.516
53.	4-7	.523	.499	1.4371	.784	.773	1.013	0.016	.539	.515	.956	.95	1.345	.582
54.	4-8	.482	.455	1.4371	.784	.732	1.07	0.017	.499	.472	.946	.94	1.183	.663
55.	4-9	.457	.426	1.4371	.784	.707	1.11	0.019	.476	.445	.935	.92	1.094	.716

Contd..........

Table B-3 Contd......

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
56.	4-10	.433	.399	1.4371	.784	.683	1.145	0.020	.453	.419	.926	.905	1.01	.777
57.	4-11	.403	.370	1.4371	.784	.653	1.20	0.022	.425	.392	.923	.90	.909	.862
58.	4-12	.389	.339	1.4371	.784	.639	1.225	0.023	.412	.362	.879	.74	.854	.918
59.	4-13	.384	.326	1.4371	.784	.634	1.235	0.023	.407	.349	.857	.80	.848	.924
60.	4-14	.380	.314	1.4371	.784	.630	1.24	0.024	.404	.338	.835	.772	.837	.936
61.	4-15	.375	.284	1.4371	.784	.625	1.253	0.024	.399	.308	.773	.68	.819	.958
62.	4-16	.373	.273	1.4371	.784	.623	1.26	0.025	.398	.298	.749	.66	.815	.962
63.	4-17	.373	.261	1.4371	.784	.623	1.26	0.025	.398	.286	.719	.62	.815	.962
64.	4-18	.372	.245	1.4371	.784	.622	1.26	0.025	.397	.270	.680	.565	.815	.962
65.	4-19	.372	.228	1.4371	.784	.622	1.26	0.025	.397	.253	.638	.515	.815	.962
66.	4-20	.372	.205	1.4371	.784	.622	1.26	0.025	.397	.230	.580	.445	.815	.962
67.	4-21	.366	.179	1.4371	.784	.616	1.27	0.025	.391	.204	.522	.380	.785	.962
68.	6-1	.367	.154	1.510	.825	.617	1.27	0.025	.392	.179	.456	.308	.785	.962
69.	6-2	.366	.222	1.510	.825	.616	1.27	0.025	.391	.247	.631	.505	.785	.962
70.	6-3	.366	.243	1.510	.825	.616	1.27	0.025	.391	.268	.686	.572	.785	.962
71.	6-4	.366	.305	1.510	.825	.616	1.27	0.025	.391	.330	.845	.785	.785	.962
72.	6-5	.366	.365	1.510	.825	.616	1.27	0.025	.391	.390	.999	.99	.785	.962
73.	6-6	.434	.403	1.510	.825	.684	1.15	0.020	.454	.423	.932	.91	1.01	.877

SUBMERGED FLOW OVER BROAD CRESTED WEIR

DETERMINATION OF SUBMERGENCE AND DISCHARGE RATIOS

EXPERIMENTAL DATA RECORDED BY R.A.SMITH[15]

With Side Slopes 1:1

Sl. No.	Run No.	H_1' ft.	H_2' ft.	Q cusecs.	$P + H_1'$ ft.	$V = Q/P+H_1'$ ft./sec.	$h_a = V^2/2g$ ft.	$H_1 = H_1' + h_a$ ft.	$H_2 = H_2' + h_a$ ft.	$S_1 = H_2/H_1$	$(H_2/H_1)^{m1.5}$	$Q_1 = 3.72\ H_1^{1.56}$	Q/Q_1
1.	5	0.053	0.023	0.0358	.478	.075	Negligible	.053	.023	.434	.286	.0391	.915
2.	6	0.053	0.033	0.0356	.478	.075	Negligible	.053	.033	.623	.50	.0391	.912
3.	7	0.059	0.050	0.0354	.484	.073	Negligible	.059	.050	.847	.792	.051	.694
4.	8	0.123	0.121	0.0351	.548	.064	Negligible	.123	.121	.905	.98	.1375	.025
5.	9	0.192	0.191	0.0351	.617	.057	Negligible	.192	.191	.996	.99	.277	.012
6.	10	0.068	0.061	0.0352	.493	.071	Negligible	.068	.061	.897	.86	.0543	.645
7.	11	0.0652	0.00	0.0351	.490	.072	Negligible	.0652	.00	-	-	-	-
8.	14	0.072	0.009	0.0620	.497	.123	Negligible	.072	.009	.125	.04	.0607	1.00
9.	15	0.075	0.057	0.0608	.500	.122	Negligible	.075	.057	.76	.67	.0634	.96
10.	16	0.104	0.096	0.0611	.529	.116	Negligible	.104	.096	.924	.90	.106	.557
11.	17	0.073	0.039	0.0610	.498	.123	Negligible	.073	.039	.534	.395	.0607	1.00
12.	22	0.091	0.061	0.0838	.516	.163	Negligible	.091	.061	.67	.542	.0062	.973
13.	23	0.096	0.077	0.0839	.521	.161	Negligible	.096	.077	.803	.73	.094	.89
14.	24	0.090	0.052	0.0842	.515	.164	Negligible	.090	.052	.578	.445	.0862	.978
15.	25	0.089	0.044	0.0859	.514	.163	Negligible	.089	.044	.494	.35	.086	.974
16.	30	0.109	0.087	0.1002	.534	.188	Negligible	.109	.087	.797	.72	.114	.879
17.	31	0.130	0.118	0.0976	.555	.176	Negligible	.130	.118	.91	.88	.1495	.653
18.	32	0.129	0.118	0.1003	.554	.181	Negligible	.129	.118	.916	.89	.1485	.675
19.	33	0.104	0.078	0.1006	.529	.191	Negligible	.104	.078	.75	.66	.1065	.945
20.	34	0.103	0.070	0.1011	.528	.192	Negligible	.103	.070	.68	.565	.105	.962

Table B-4 Contd.....

1	2	3	4	5	6	7	8	9	10	11	12	13	14
21.	35	0.101	0.062	0.1011	.526	.191	Negligible	.101	.062	.614	.49	.1015	.996
22.	36	0.100	0.051	0.1009	.525	.191	Negligible	.100	.051	.51	.365	.0996	1.00
23.	37	0.100	0.025	0.1009	.525	.191	Negligible	.100	.025	.25	.126	.0996	1.00
24.	38	0.100	0.010	0.1011	.525	.192	Negligible	.100	.010	.10	.03	.0996	1.00
25.	39	0.100	0.000	0.1008	.525	.191	Negligible	.100	.000	-	-	.0996	1.00
26.	42	0.137	0.008	0.1411	.562	.251	0.001	.138	.009	.065	.02	.1645	.859
27.	43	0.161	0.148	0.1411	.586	.241	0.001	.162	.149	.92	.90	.211	.669
28.	44	0.238	[illegible]	[illegible]	.662	.212	0.001	.238	.234	.984	.975	.374	.376
29.	45	0.135	0.100	0.1407	.560	.250	0.001	.136	.101	.743	.65	.1630	.864
30.	46	0.129	0.089	0.1417	.554	.256	0.001	.130	.090	.693	.58	.1505	.942
31.	47	0.126	0.054	0.1405	.551	.255	0.001	.127	.055	.434	.286	.1450	.969
32.	48	0.126	0.019	0.1409	.551	.256	0.001	.127	.020	.158	.062	.1450	.972
33.	49	0.126	0.003	0.1410	.551	.256	0.001	.127	.004	.0314	.006	.1450	.972
		<u>With Side Slopes 2:1</u>											
34.	65	0.052	0.032	0.0337	.477	.071	Negligible	.052	.032	.615	.49	.035	.965
35.	66	0.058	0.048	0.0332	.483	.069	Negligible	.058	.048	.828	.765	.042	.791
36.	68	0.069	0.046	0.0528	.494	.107	Negligible	.069	.046	.666	.55	.055	.96
37.	69	0.072	0.055	0.0529	.497	.106	Negligible	.072	.055	.763	.675	.061	.868
38.	71	0.091	0.028	0.0837	.516	.162	Negligible	.091	.028	.318	.18	.086	.975
39.	72	0.101	0.086	0.0837	.526	.159	Negligible	.101	.086	.851	.80	.101	.83
40.	75	0.099	0.005	0.0989	.524	.189	Negligible	.099	.005	.05	.01	.098	1.00
41.	76	0.099	0.041	0.0987	.524	.189	Negligible	.099	.041	.415	.27	.098	1.00
42.	77	0.119	0.106	0.0989	.544	.182	Negligible	.119	1.06	.89	.85	.131	.755
43.	78	0.099	0.055	0.0990	.524	.190	Negligible	.099	.055	.556	.415	.098	1.00
44.	79	0.126	0.015	0.1407	.551	.255	0.001	.127	.016	.126	.045	.146	.964
45.	80	0.126	0.066	0.1412	.551	.256	0.001	.127	.067	.528	.386	.146	.968

Table B-4 Contd........

1	2	3	4	5	6	7	8	9	10	11	12	13	14
46.	81	0.149	0.134	0.1406	.574	.246	0.001	.150	.135	.90	.86	.187	.753
47.	82	0.141	0.123	0.1410	.566	.249	0.001	.142	.124	.874	.83	.172	.819
48.	83	0.131	0.104	0.1410	.556	.254	0.001	.132	.105	.796	.725	.153	.92
49.	84	0.126	0.089	0.1409	.551	.256	0.001	.127	.090	.71	.608	.146	.965
50.	85	0.126	0.069	0.1411	.551	.257	0.001	.127	.070	.551	.41	.146	.965
51.	87	0.149	0.093	0.1822	.574	.302	0.0015	.150	.0945	.630	.505	.187	.976
52.	88	0.149	0.070	0.1781	.574	.311	0.0015	.150	.0715	.476	.33	.187	.954
53.	89	.149	0.022	0.1822	.574	.308	0.0015	.150	.0235	.157	.06	.187	.975
54.	90	0.132	0.121	0.1008	.557	.181	Negligible	.132	.121	.918	.89	.159	.689
55.	91	0.108	0.091	0.0007	.533	.190	Negligible	.108	.091	.843	.78	.113	.891
56.	92	0.103	0.079	0.1008	.528	.191	Negligible	.103	.079	.767	.68	.104	.969
57.	93	0.102	0.059	0.1009	.527	.191	Negligible	.102	.059	.579	.445	.103	.976
58.	94	0.102	0.046	0.1008	.527	.191	Negligible	.102	.046	.451	.30	.103	.976
59.	96	0.099	0.081	0.0858	.524	.162	Negligible	.099	.081	.818	.75	.098	.875
60.	97	0.090	0.064	0.0843	.516	.164	Negligible	.090	.064	.711	.608	.084	1.00
61.	98	0.087	0.043	0.0803	.512	.157	Negligible	.087	.043	.494	.35	.079	1.00
62.	100	0.100	0.091	0.0631	.525	.120	Negligible	.100	.091	.91	.88	.101	.625
63.	101	0.077	0.060	0.0622	.502	.124	Negligible	.077	.060	.78	.69	.065	.958
64.	102	0.073	0.028	0.0620	.498	.125	Negligible	.073	.028	.384	.24	.060	1.00
65.	103	0.095	0.090	0.0421	.520	.081	Negligible	.095	.090	.945	.93	.091	.464
66.	104	0.073	0.065	0.0120	.498	.085	Negligible	.073	.065	.891	.85	.060	.70
67.	105	0.053	0.011	0.0395	.478	.083	Negligible	.053	.011	.208	.096	.038	1.00
68.	106	0.053	0.001	0.0395	.478	.083	Negligible	.053	.001	.02	.006	.038	1.00

APPENDIX- B-5

SUBMERGED FLOW OVER OGEE SHAPED WEIR

DETERMINATION OF SUBMERGENCE AND DISCHARGE RATIOS

EXPERIMENTS CONDUCTED BY U.S.B.R.[11]

DAM-A

Sl. No.	H'_1 ft.	h'_d/H'_1	h_d' ft.	H'_2 ft.	Q cusecs	$P + H'_1$ ft.	$V = \frac{Q}{P + H'_1}$ ft/sec.	$h_a = V^2/2g$ ft.	$H_1 = H'_1 + h_a$ ft.	$H_2 = H'_2 + h_a$ ft.	$H_2/H_1 = S$	$(H_2/H_1)^{\overset{1.5}{n}}$	$Q_1 = 4.11\ H_1^{1.665}$	Q/Q_1
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1.	.599	.763	.458	.141	1.831	4.039	.453	.0033	.602	.144	.239	.118	1.80	1.00
2.	.596	.753	.449	.147	1.819	4.036	.450	.0033	.599	.150	.25	.126	1.80	1.00
3.	.594	.630	.375	.219	1.792	4.034	.444	.0031	.597	.222	.372	.228	1.76	1.00
4.	.596	.522	.312	.284	1.766	4.036	.436	.0030	.599	2.87	.478	.33	1.80	.982
5.	.596	.379	.216	.380	1.734	4.036	.4205	.0029	.599	.383	.64	.515	1.80	.965
6.	.597	.283	.169	.428	1.694	4.037	.419	.0028	.600	.431	.718	.616	1.80	.943
7.	.601	.196	.118	.483	1.622	4.041	.401	.0025	.603	.485	.805	.73	1.81	.903
8.	.591	.115	.068	.523	1.105	4.031	.274	.001	.592	.524	.885	.84	1.73	.638
9.	.592	.012	.0071	.585	.327	4.032	.08	-	.592	.585	.988	.98	1.73	.189
10.	.812	.668	.542	.270	2.891	4.252	.679	.0068	.819	.277	.336	.195	3.01	.963
11.	.815	.550	.449	.366	2.885	4.255	.678	.0068	.822	.373	.454	.305	3.02	.961
12.	.813	.430	.350	.463	2.852	4.253	.670	.0069	.820	.470	.572	.44	3.01	.95
13.	.809	.372	.301	.508	2.792	4.249	.656	.0068	.816	.515	.630	.505	3.00	.93

Table Contd......./

Table B-5 Contd........

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
14.	.813	.408	.330	.483	2.622	4.253	.617	.0060	.819	.489	.597	.47	3.01	.874
15.	.807	.401	.324	.483	2.530	4.247	.596	.0056	.813	.489	.602	.471	2.95	.851
16.	.809	.359	.290	.519	2.369	4.249	.556	.0049	.814	.524	.644	.523	2.95	.797
17.	.811	.315	.255	.556	2.210	4.251	.520	.0042	.815	.560	.688	.58	2.95	.744
18.	.813	.289	.235	.578	2.120	4.253	.497	.0039	.817	.582	.713	.605	2.97	.713
19.	.815	.275	.224	.591	2.053	4.255	.481	.0036	.819	.595	.727	.63	3.01	.69
20.	.811	.237	.192	.619	1.930	4.251	.453	.0032	.814	.622	.764	.675	2.95	.65
21.	.808	.158	.128	.680	1.610	4.248	.378	.0022	.810	.682	.841	.792	2.95	.541
22.	.808	.066	.053	.755	1.040	4.248	.245	.0010	.809	.756	.935	.92	2.95	.35

SUBMERGED FLOW OVER OGEE SHAPED WEIR

DETERMINATION OF SUBMERGENCE AND DISCHARGE RATIOS

EXPERIMENTS CONDUCTED BY U.S.B.R.[11]

DAM B

Sl. No.	H_1' ft.	h_d'/H_1'	h_d' ft.	H_2' ft.	Q cusecs	$P+H_1'$ ft.	$V=Q/P+H_1'$ ft./sec.	$h_a = V^2/2g$ ft.	$H_1=H_1'+h_a$ ft.	$H_2=H_2'+h_a$ ft.	$H_2/H_1=S_1$	$(H_2/H_1)^{1.5}$	$Q_1=3.72\,H_1^{1.72}$ cusec	Q/Q_1
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1.	.617	.548	.338	.279	1.728	4.057	.425	.0028	.620	.282	.455	.308	1.72	1.00
2.	.619	.359	.222	.397	1.708	4.059	.42	.0028	.622	.400	.645	.521	.174	.982
3.	.640	.131	.084	.556	1.410	4.080	.34	.0018	.642	.558	.87	.82	1.80	.785
4.	.620	.048	.030	.590	0.955	4.060	.235	.0010	.621	.591	.954	.94	1.73	.552
5.	.617	.666	.411	.206	1.746	4.057	.43	.0028	.620	.209	.337	.196	1.72	1.00
6.	.617	.497	.306	.311	1.739	4.057	.428	.0028	.620	.314	.506	.361	1.72	1.00
7.	.616	.376	.226	.390	1.713	4.056	.422	.0028	.619	.393	.634	.51	1.725	.988
8.	.617	.282	.174	.443	1.697	4.057	.417	.0027	.620	.446	.72	.62	1.72	.985
9.	.617	.102	.063	.554	1.376	4.057	.34	.0018	.619	.556	.896	.86	1.72	.80
10.	.615	.049	.030	.585	1.063	4.055	.26	.0010	.616	.586	.945	.94	1.70	.625
11.	.819	.729	.597	.222	2.809	4.359	.645	.0066	.826	.229	.278	.148	2.76	1.00
12.	.820	.555	.455	.365	2.789	4.360	.64	.0065	.826	.371	.45	.302	2.76	1.00
13.	.820	.469	.385	.435	2.748	4.360	.63	.0064	.826	.441	.536	.396	2.76	.995
14.	.820	.394	.323	.497	2.702	4.360	.62	.0063	.826	.503	.61	.48	2.76	.978
15.	.819	.314	.258	.561	2.640	4.359	.605	.0060	.825	.567	.688	.58	2.75	.960
16.	.817	.203	.166	.651	2.496	4.357	.573	.0053	.822	.656	.796	.72	2.73	.914
17.	.816	.166	.136	.680	2.078	4.356	.475	.0035	.820	.683	.83	.765	2.71	.765
18.	.815	.137	.112	.703	1.898	4.355	.435	.0030	.818	.706	.857	.80	2.70	.702
19.	.812	.109	.097	.715	1.718	4.352	.394	.0024	.814	.717	.878	.84	2.68	.640
20.	.809	.029	.023	.786	1.171	4.349	.268	.0010 0.0010	.810 0.810	.787 0.787	.963 0.963	.955	2.67	.439

Table B-6 Contd.....

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
21.	1.042	.640	.668	.374	4.109	4.582	.89	.0124	1.054	.386	.365	.222	4.10	1.00
22.	1.041	.495	.516	.525	4.032	4.581	.878	.0120	1.053	.537	.508	.365	4.10	.981
23.	1.041	3386	.402	.639	3.934	4.581	.854	.0113	1.052	.650	.615	.49	4.10	.958
24.	1.051	.221	.232	.819	3.883	4.591	.845	.0112	1.062	.830	.785	.70	4.20	.926
25.	1.038	.250	.260	.778	3.703	4.578	.804	.0100	1.048	.788	.745	.65	4.08	.906
26.	1.050	.132	.139	.911	2.820	4.590	.614	.0059	1.056	.917	.869	.82	4.12	.685
27.	1.034	.089	.092	.942	2.178	4.574	.474	.0036	1.038	.946	.913	.88	3.96	.550
28.	1.043	.065	.068	.975	1.790	4.583	.39	.0024	1.045	.977	.942	.92	4.05	.442
29.	1.033	.050	.052	.981	1.434	4.573	.32	.0017	1.035	.983	.947	.925	3.90	.38
30.	1.037	.049	.050	.987	1.114	4.577	.24	.0010	1.038	.988	.952	.935	3.96	.289
31.	1.058	.616	.651	.407	4.183	4.598	.91	.0130	1.071	.420	.392	.245	4.30	.972
32.	1.050	.301	.316	.734	4.037	4.590	.878	.0120 [illegible]	1.062	.746	.703	.59	4.20	.96
33.	1.051	.150	.158	.393	3.911	4.591	.85	.0113	1.062	.904	.85	.80	4.20	.93
34.	1.054	.258	.273	.781	5.231	4.594	.70	.0077	1.062	.789	.742	.645	4.20	.77
35.	1.048	.084	.088	.960	2.455	4.588	.534	.0045	1.052	.965	.908	.88	4.10	.60
36.	1.057	.629	.664	.393	4.152	4.597	.905	.0128	1.070	.406	.38	.235	4.30	.966
37.	1.056	.454	.479	.577	4.114	4.596	.895	.0126	1.069	.590	.552	.41	4.30	.957
38.	1.055	.297	.314	.741	3.993	4.595	.868	.0119	1.067	.753	.706	.592	4.26	.937
39.	1.051	.207	.217	.834	3.683	4.591	.80	.0100	1.061	.844	.793	.71	4.20	.875
40.	1.053	.147	.155	.898	3.161	4.593	.686	.0074	1.060	.905	.85	.715	4.18	.752
41.	1.050	.044	.046	1.006	2.517	4.590	.545	.0047	1.055	1.011	.95	.94	4.12	.61
42.	1.057	.790	.835	.222	4.275	4.597	.93	.0136	1.071	.236	.220	.104	4.30	.995
43.	1.057	.563	.595	.462	4.235	4.597	.92	.0132	1.070	.475	.443	.296	4.30	.985
44.	1.057	.457	.482	.575	4.157	4.597	.905	.0128	1.070	.588	.548	.41	4.30	.968
45.	1.056	.371	.398	.658	4.095	4.596	.89	.0124	1.068	.670	.630	.505	4.28	.956
46.	1.052	.209	.220	.832	3.365	4.592	.732	.0084	1.060	.840	.791	.71	4.20	.801

APPENDIX- B-7

SUBMERGED FLOW OVER TRAPEZOIDAL NOTCH
DETERMINATION OF SUBMERGENCE AND DISCHARGE RATIOS
EXPERIMENTAL DATA RECORDED BY ANDERSON & DAHL[9]

Sl. No.	Test No.	H_1' Cm.	h_d' cm	H_2' cm.	Q' cumec.	b +a H_1' 'm'	$F=H_1'(b+aH_1')$ m^2	$V=Q'/F$ m/sec.	$h_a=V^2/2g$ m	H_1 m	H_2 m	H_2/H_1 $= S_1$	$(H_2/H_1)^n$	$b'+a'H_1$ m^2	$Q_1 = 1.61H_1$ cumec/sec.	Q_1' cumec	Q'/Q_1'
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1.	275	28.35	10.51	17.84	.0488	1.025	0.29	.1685	.0015	.2850	.1799	.63	.505	.2425	.242	.0586	.831
2.	276	26.65	17.38	9.17	.0488	0.998	.265	.1840	.00175	.2672	.0934	.349	.208	.2336	.22	.0514	.95
3.	277	26.19	21.45	4.74	.0489	.992	.260	.1880	.00183	.2637	.0492	.187	.082	.2318	.218	.0504	.973
4.	280	25.23	8 .10	17.13	.0366	.979	.247	.148	.0011	.2534	.1724	.68	.565	.2267	.203	.0461	.793
5.	281	22.62	14.73	7.89	.0352	.94	.2125	.166	.0014	.2276	.0803	.354	.21	.2138	.175	.0374	.943
6.	282	22.57	19.05	3.52	.0363	.936	.211	.168	.0014	.2271	.0366	.162	.064	.2136	.1745	.0372	.975
7.	284	20.79	9.41	11.38	.0273	.912	.189	.145	.0011	.2090	.1149	.55	.42	.2045	.154	.0315	.866
8.	285	19.99	12.88	7.11	.0273	.900	.180	.152	.0012	.2011	.0722	.342	.202	.2006	.145	.291	.939
9.	286	19.60	16.97	2.63	.0273	.894	.175	.156	.0012	.1972	.0275	.140	.05	.1986	.140	.0278	.983
10.	288	17.66	7.40	10.26	.0193	.865	.153	.126	.0007	.1773	.1033	.584	.45	.1886	.120	.0226	.856
11.	289	16.97	10.88	6.09	.0197	.854	.145	.136	.0009	.1706	.0618	.362	.218	.1853	.114	.0212	.93
12.	290	16.64	14,95	1.69	.0198	.849	.141	.140	.00095	.1674	.0179	.107	.03	.1837	.110	.0202	.978
13.	292	12.58	4.27	8.31	.00957	.788	.098	.098	.0006	.1264	.0837	.66	.54	.1632	.073	.012	.796

APPENDIX - C

CURVE FITTING

From the analysis of data it was observed that in general two types of relations can be established for submerged flow over weirs.

1. For sharp crested weirs
2. For thick weirs e.g. ogee, Broad crested etc.

For obtaining mathematical relationship from the experimental data suitable curves are fitted for the above two cases.

1. Sharp Crested Weirs:

It is seen that the points lie on a circle for most of the range of submergence.

The general equation of a circle is :

$x^2 + y^2 + 2gx + 2fy + c = 0$ in a, (x, y) cordinate axes.

One of the boundary condition is that when $S_1 = 1.00$ the discharge ratio $Q/Q_1 = 0$ i.e. when $x = 1$, $y = 0$. Several trial circles were tried and the best circle passing through most of the points from 3% to 100% submergence was selected. For $S_1 = 0$ to $S_1 = 0.03$ a straight line was suggested.

The following values were taken to derive the general formula for the circle:

When $S_1 = 0.1$ $Q/Q_1 = 0.95$

$S_1 = 0.6$ $Q/Q_1 = 0.75$

$S_1 = 1.00$ $Q/Q_1 = 0$

Putting these values in the general equation the values of constants g , f & c are obtained. The values are:

$g = 0.01350$

$f = 0.0589$

$c = -1.0270$

The final equation derived in terms of $\frac{Q}{Q_1}$ and S_1 is as follows:-

$$Q/Q_1 = \sqrt{1.03 - 0.0270\,S_1 - S_1^2} - 0.0589$$

2. Thick Weirs

Values of Q/Q_1 and S_1 was plotted in double log graph paper. It was observed that straight line relation is not possible for the entire range of submergence. The author tried to define the relationship by drawing two straight lines.

A polynomial relationship was assumed and the general equation was derived. However it was found that the polynomial does not fit properly with experimental points.

Finally a quadrant of an ellipse was assumed and it was found that this fits best in comparision with the others.

It is clear that the discharge factor starts reducing only after a submergence ratio of about 50% . Different Ellipses were drawn taking this limiting point at which $\frac{Q}{Q_1}$ starts reducing. ~~different~~. It was found that the curve with $S_1 = 0.65$ is the best fit.

The boundary conditions are as follows:

when $S_1 = 1.00$ $Q/Q_1 = 0$

$S_1 = 0.65$ $Q/Q_1 = 1.0$

The semi major axes = 1.00

Semi minor axes = 1 - 0.65 = 0.35

The centre of the Ellipse has got the following co-ordinates

$x = 0.65$

$y = 0.00$

The equation of the above ellipse is as follows:

$$\frac{(x - 0.65)^2}{(0.35)^2} + \frac{(y - \text{zero})}{(1)^2} = 1$$

After solving and putting in terms of $\frac{Q}{Q_1}$ & S_1 the equation is as follows:

$$\frac{Q}{Q_1} = \sqrt{9.81 S_1 - 7.55 S_1^2 - 2.26}$$

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