EFFECT OF CONVERGENCE ON THE
DISCHARGE OF A VENTURI-FLUME

BY

Dr. Abdel-Hadi Abul-Fetouh*, Dr. Talaat M. Owais**
and Eng. Mahmoud H. Ahmed***

I. INTRODUCTION:

There are many ways to measure the discharge in open channels, either directly or indirectly. Volumetric, electric and electro - magnetic methods are just few examples. Direct measurement means the determination of the volume rate or weight of the fluid that passes through a section within a given time interval. One of the common measuring device is what was called, "Venturi-Flume" in which the discharge may be computed by using a calibrated formula in terms of the difference in heads between the entrance and its throat, possessing minimum head loss.

On the contrary, the use of weirs in measuring stream flow involves a considerable loss of head, since a definite drop in the surface of the water as it passes over the weir is necessary, even if the weir is submerged. A similar situation takes place if a sluice gate is used. In measurements obtained by using free sharp-crested weirs, there follows most of the time the existance of a free fall over the weir along with free entrance of air under the nappe. In all cases of positive bed slopes, both weirs and sluice gates create retarted flow upstream forming backwater curves causing siltation proceesing. On the other hand such const-

* Prof. of Hydraulics, Faculty of Engineering, Ellazikia University, Syria.

** Assoc. Prof. of Hydraulics & Irrigation, Civil Eng. Dept., Faculty of Eng., El-Mansoure, Univ., Egypt.

*** Asst. Lecturer, Civil Eng. Dept., Faculty of Eng., Assiut Univ., Egypt.

Mansoura Bulletin June 1977.

ructions create accelerated flow downstream accompained by large eddy sizes with high turbulence intensity causing local scouring processing.

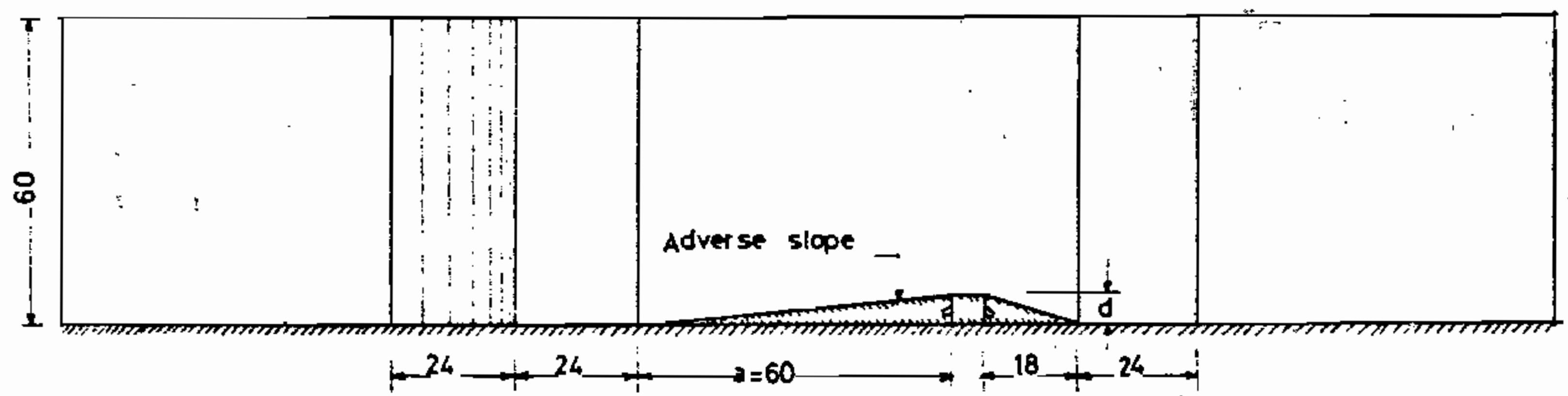
In many situations, such as in feeding canals and laterals of large irrigation system, pressure head must be conserved. For this reason, the use of weirs is frequently impracticable, especially for relatively small branch and distributary canals possessing low values of conveyance factor.

The venturi flume, which is based on the well-known Venturi -principle, is essentially a structure made out of steel, wood or rienforced concrete which provides, within a short distance, a limited constricted portion converging the flow towards its throat which then diverging outwards towards its exit. The discharge passed through can be estimated by applying calibrated formula after rating the flume in the laboratory according to similarity theories and in terms of flume dimensions and the difference in heads for both free and submerged cases. In general, a significant advantage of the veturi-flume is that its ability to function with a relatively small net head lose.

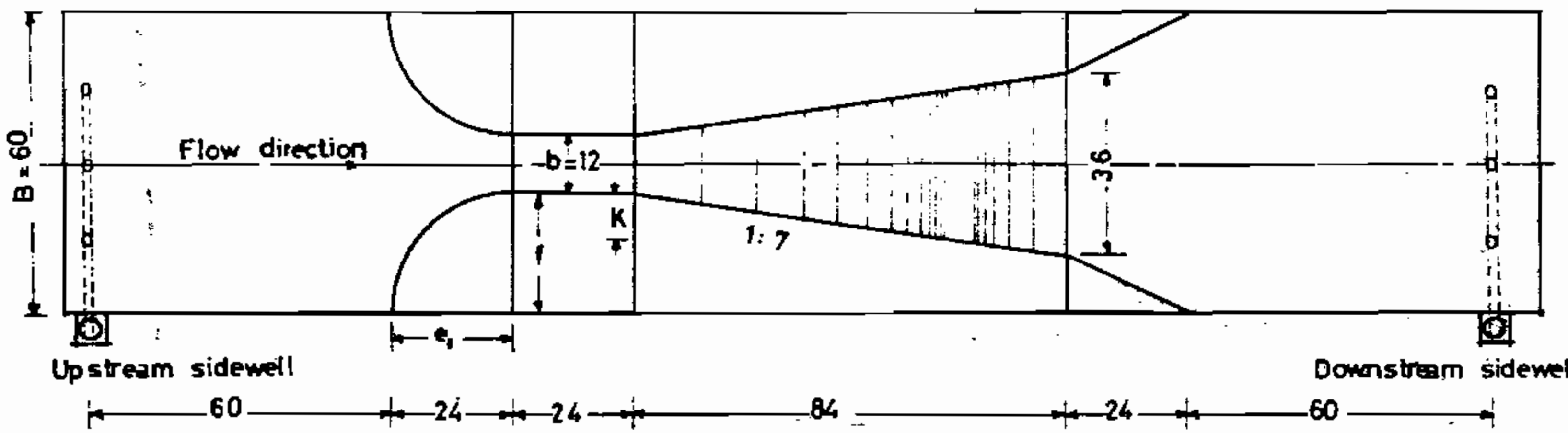
The present study demonstrates a new type of a venturi flume provided with an adverse slope through its diverging part, in which it is believed that the loss of head is reduced to minimum. This can be accomplished by giving the flume floor various artificial adverse slopes within the diverging part beyond the throat, to enable selecting the most workable, thereby permitting the control section to adjust itself automatically to the condition of minimum loss of energy transfer (refer to Fig. (1)).

The classical equation for the discharge through a venturi flume may be expressed in the following form:

$$Q = C.b. \sqrt{2g} \; . \; H_1^n , \qquad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$



Sectional elevation



All dimensions in centimeters

Fig(1) The Venturi Flume with Adverse Slope.

i which: "C" is the coefficient of discharge; and "n" is the power e ponent of head "H_1." Both "C" and "n" vary dimensionally with t e proportional ratios of the flume like: (b/B), (K/a), (d/a),... e c. Accordingly it is obvious to see that both "C" and "n" of t e suggested flume would vary with its proportional dimensions l ke: (a, b, d, ..., B), etc. presented in Fig. (1).

I . THE OBJECTIVES:

The main objective of this study is to investigate the beha- v our of the flow characteristics through a special venturi flume t king different values of its adverse slope, and finding out the c rresponding value of the discharge coefficient "C", and the power e ponent "n". This besides the intention of carrying out a rather c mplete investigation covering a wide modular ranges, yielding f nally to the best proportional dimensions that function accord- i g to a simple rating practical formula hoping to possess minimum h ad loss.

I I. LITERATURE REVIEW

In many countries where water resources plays a most vital r le and in places where irrigation activities are of great concern l ke in Egypt, many investigators recommend using the standing wave f ume and throated flumes as they yield little loss of head which p oved to be more economically and fit synoptio diagrams in flat c untries and so workable for canals of relatively small and mode- r te discharges. For bigger discharges, standing wave weirs and s uice gate regulators may be practically recommended.

The Fifth International Post-Graduate Course in Hydrology (Ref. " ") recommended a general basic equation for the discharge through a throated flume as follows:

$$Q = \emptyset_1 . C_q . \sqrt{2g} \, . \, H_1^{1.5} \qquad \dots\dots\dots\dots\dots \quad (2)$$

in which: ϕ_1: numerical factor depends upon the geometrical conditions of the channel, and

C_q: the discharge coefficient.

Inglis, C.C. (Ref. "5") suggested a flume with some conditions which shown in Fig. (2), and he proposed the following form for the basic equation:

$$Q = (2/3\sqrt{3})\ C_v . C_d . \sqrt{2g}\ .b.\ H_1^{1.5} \qquad \text{.........} (3)$$

To obtain the above coefficients C_v and C_d, Inglis suggested the following formulas:

$$C_d = \left(\frac{b}{b+0.004L}\right)^{1.5} \left(\frac{H_1-0.003L}{H_1}\right)^{1.5}, \qquad \text{.........} (4)$$

and : $\left(\frac{2}{3\sqrt{3}} . \frac{b}{B}\right)^2 . \left(\frac{H_1}{H_1 + P}\right)^2 . C_v^2 - C_v^{1.5} + 1 = 0,$ .... (5)

providing the following limitations:

(a) for the flume with side and bottom contractions:

$b \geqslant 0.091$ m., for: $0.049 \leqslant H_1 \leqslant 1.83$ m, and

$$\frac{b}{B} . \frac{H_1}{H_1 + P} \leqslant 0.70, \text{ for : } \frac{H_1}{b} \leqslant \frac{1}{3} .$$

(b) for the flume with a hump only, i.e., without side contractions:

$$P \leqslant 0.91 \text{ m., when } \frac{H_1}{b} < 0.30$$

The Parshall measuring flume, devised by Ralph L. Parshall (Ref. "9") was an improved modification of the original Venturi - Flume. A sketch for such a flume is shown in Fig. (3). Established tables accompanied with charts for different sizes of the indicated flume were furnished, giving the discharge for each size according to both upstream and dowstream readings for either free or submerged flow cases. He proposed the following formula:

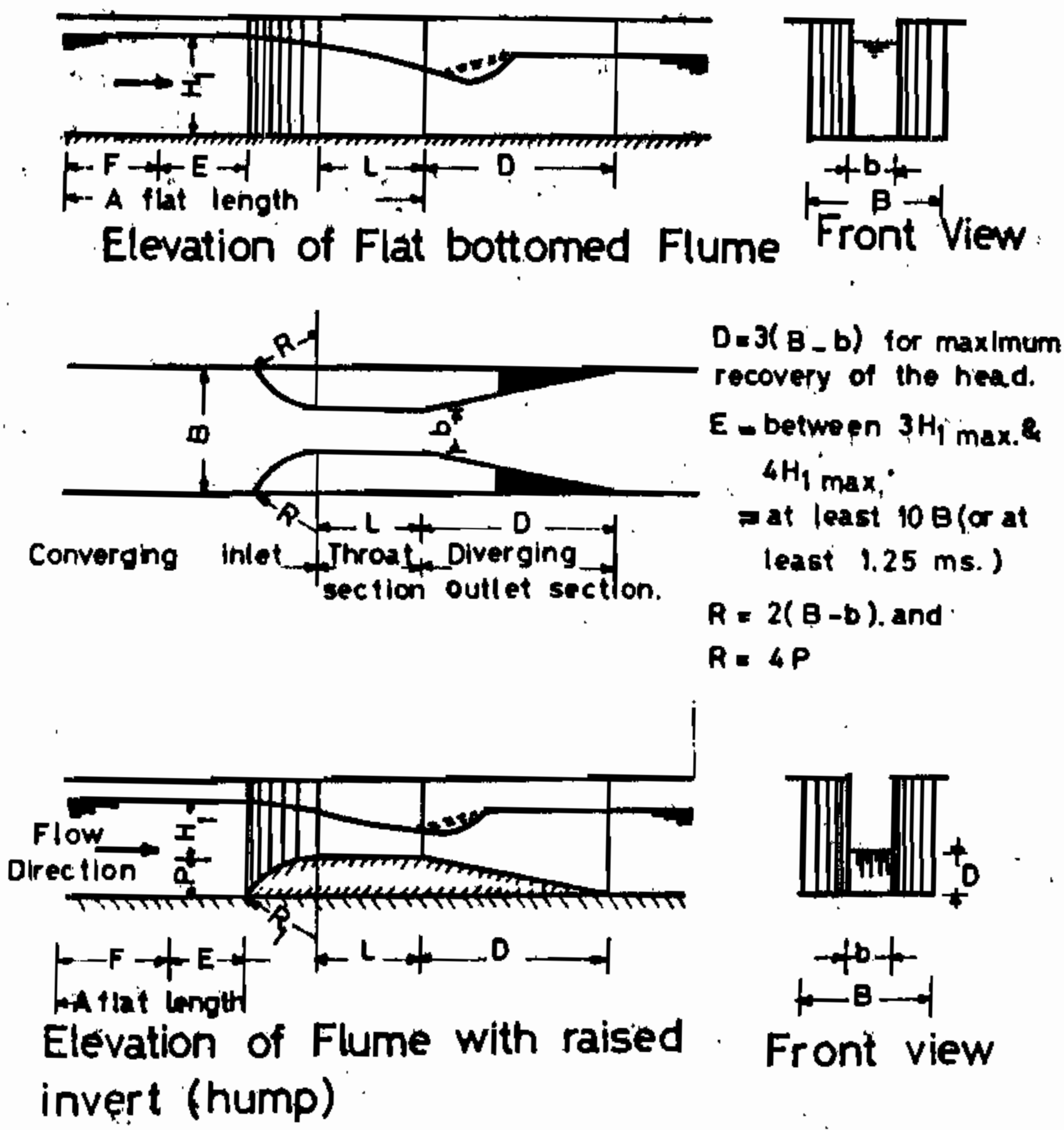


Fig.(2) Standing wave flume proposed by Inglis.

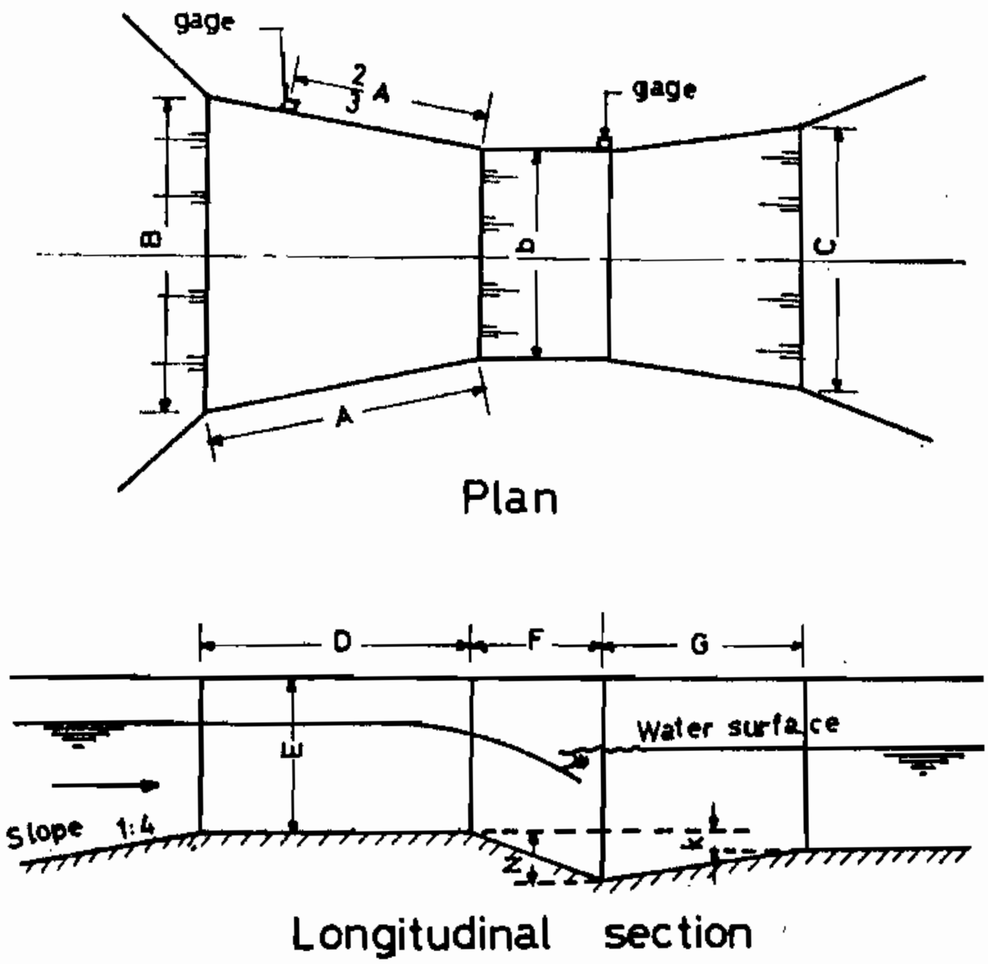


Fig.(3) A sketch for the Parshall flume.

$$Q = 4.b.h^{1.522b^{0.026}} \qquad \text{.......... (6)}$$

in which: h = the upstream water depth at the gaging well.

Skogerboe, walker and Robinson (Ref. "11") made their experiments on 2.50- feet Parshall-flume placed in a rectanglular flume 5 feet by 5 feet and found out that the relationship between the discharge Q (c.f.s), and the difference between the upstream and downstream depths took the following form:

$$Q = \frac{-46.6(H_1 - H_2)^{1.53}}{\left(\log \frac{H_2}{H_1} + 0.004\right)^{1.02}} \qquad \text{........... (7)}$$

Hyatt, Skogerboa and Eggleston (Ref. "6") found that for the six-inches Parshall-flume, the submerged flow discharge equation which fitted the calibrated curve was:

$$Q = \frac{1.66(H_1 - H_2)^{1.58}}{\left[-\left(\log \frac{H_2}{H_1} + 0.0044\right)\right]^{1.08}}; \qquad \text{........... (8)}$$

for one-foot Parshall flume:

$$Q = \frac{3.11\ (H_1 - H_2)^{1.52}}{\left[-\left(\log \frac{H_2}{H_1} + 0.0044\right)\right]^{1.08}}; \qquad \text{........... (9)}$$

for Four-feet Parshall flume:

$$Q = \frac{11.10\ (H_1 - H_2)^{1.57}}{\left[-\left(\log \frac{H_2}{H_1} + 0.0044\right)\right]^{1.185}}; \qquad \text{........... (10)}$$

and for Six -feet parshall flume:

$$Q = \frac{15.89\,(H_1 - H_2)^{1.58}}{\left[-(\log \frac{H_2}{H_1} + 0.0044)\right]^{1.24}} \; ; \qquad \ldots\ldots\ldots\ldots \quad (11)$$

Clyde, Skogerboe and Hyatt (Ref. "2") found that the relationship between the discharge Q, the upstream and the downstream water depths, (H_1 and H_2 respectively), for Parshall flume constructed in submerged trapezoidal measuring flumes was as follows:

$$Q = \frac{13.83\,(H_1 - H_2)^{1.74}}{\left[-(\log \frac{H_2}{H_1} + 0.0044)\right]^{1.32}} \; ; \qquad \ldots\ldots\ldots\ldots \quad (12)$$

Fâthy and Abul-Fetouh (Ref. "3") proposed a new type of venturi flume which is very close to the one shown in Fig.(4), fitted in a rectangular flume of 50 x 50 cms., with an adverse slope of 0.10 as one case, while a flatbottomed flume was the other case. Emperical equations for the discharge were tried out for each case, eventhough necessity for further comprehensive experimental work left out; which was one of the main objectives of the present study.

IV. EXPERIMENTAL WORK

IV.1 Description of Apparatus:

Fig. (5) shows a schematic sketch for the general arrangement of the apparatus used for all experiments carried out for the pressnted study. It consists of a closed system, where the water was circulated by a pump capable of delivering approximately one hundred liters per Sec. out from an underground sump, into a constant low head tank, by means of a 12 inches diameter delivery pipe. The supply tank was provided with a system of skimming weirs for the overflowing process to take place, to ensure constant head condition in all cases. This tank feed a measuring tank below; the later was provided with a constant head

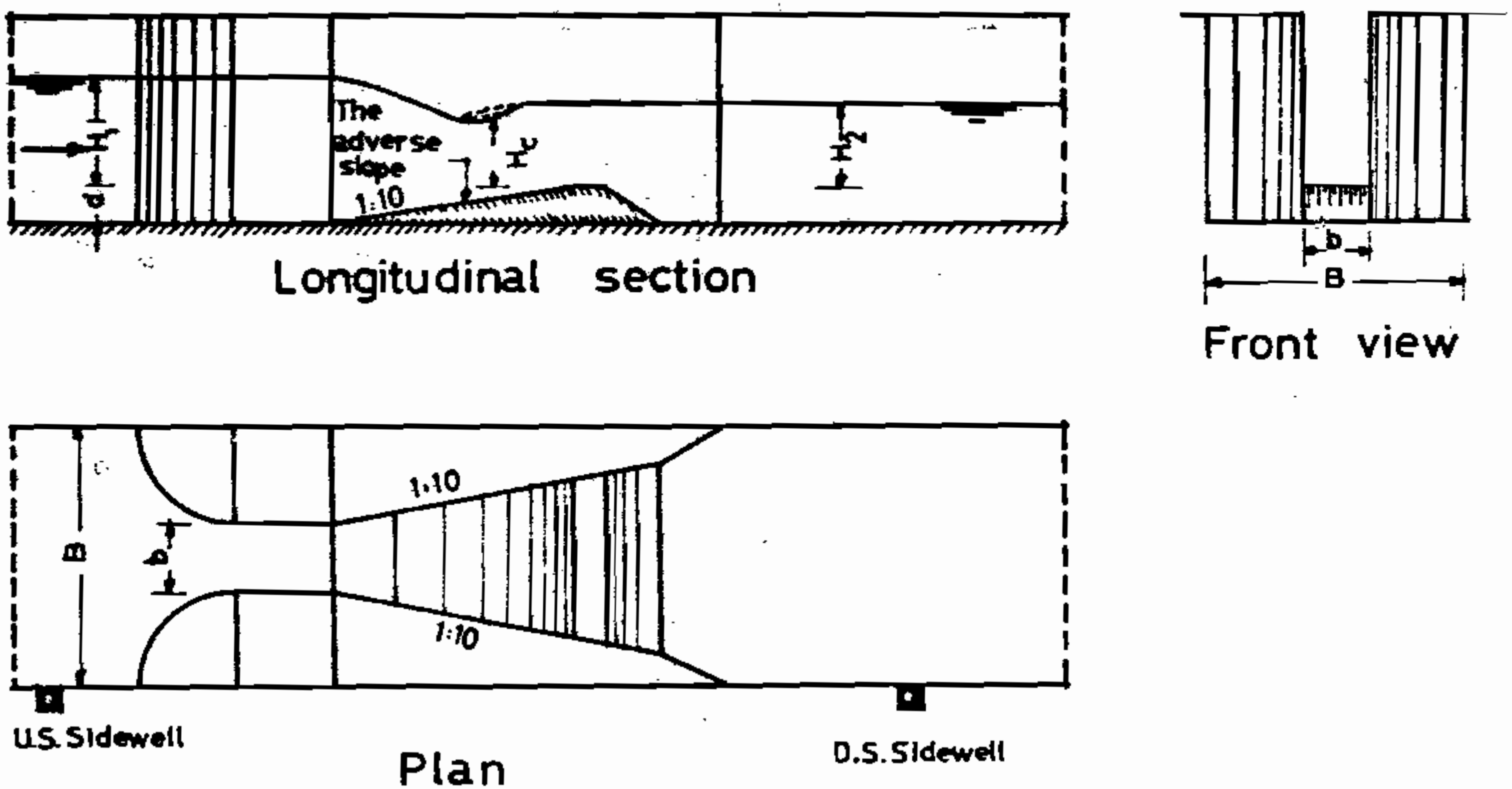


Fig.(4) A sketch for one of type Venturi Flume with Adverse Slope.

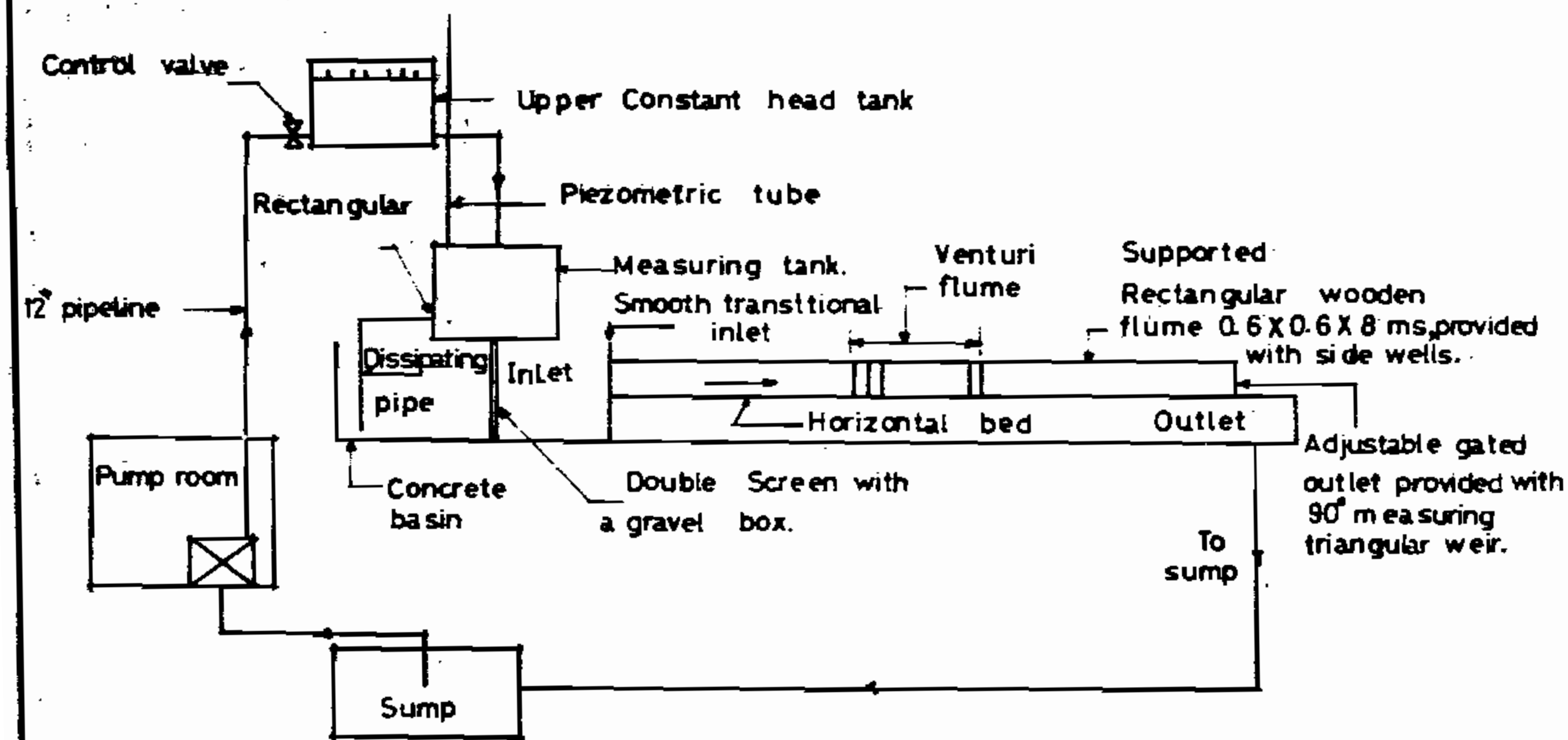


Fig (5) Schematic sketch for the general arrangement of the apparatus used for all experiments.

variable width rectangular orifice. This discharging orifice had a height of 15.0 cms. and a width varied from zero to 15.0 cms., which was controlable by a micrometer with an attached vernier. The water flows from the orifice into a stilling basin through a dessipative device, from which the water was conveyed to a rectangular wooden flume, after passing through a gravel box to secure uniform developed outlet flow established before the test section, and secure its freedom from high turbulence intensity and eddy activities. The dimensions of this rectangular wooden flame was 60.0 cms. wide, 60.0 cms. height and 8.00 meters long. It was stiffined and fixed at different places and laid horizontally. The test section was found to be best located at a distance of 3.84 meters from the inlet, where the proposed venturi-flume was placed and fixed with the main wooden flume, with the arrangement shown in Fig. (1). Nine adverse slopes are made out of fine hard insulated wood, which of the venturi flume.

At the end of the rectangular flume, a movable tail gate was placed to be able to raise the downstream depth and to adjust for different submergence ratios. The outflow from the flume discharged into the same sump from which it was left to be recirculated by the pump again.

IV.2 **Experimental Procedure:**

The following steps show the experimental procedure.

1- The bed of the flume was checked to be horizontal.
2- The constant head, variable width, rectangular orifice was calibrated for different width openings, using a 90° triangular weir.
3- The behaviour of each flume was recorded by running several preplanned experiments on it to determine, at different discharges the upstream heads for free flow conditions. Their, the downstream depth was raised gradually for each experiment using the tail gate to determine the maximum value of the modular range investigating the behaviour of any of the flumes for submerged aonditions. This procedure was carried out for

both the flat-bottomed venturi-flume and for each of the adverse slope ones.

Moreover, necessary measurements were extensively taken for investigating the behaviour of the water surface profile, especially within the vicinity of the venturi flume.

V. EXPERIMENTAL RESULTS AND DISCUSSIONS:

V.1 The Calibration For The Orifice:

The triangular weir was used because the head over the weir would be sufficiently large even at small rates of flow which lessened the percentage error in measuring the head and consequently the corresponding percentage error in discharge. Most literature recommended Cones' equation for a 90° triangular weir (refer to Figs. (6 & 7)). Such an equation could be expressed as follows:

$$Q = 0.0147\ H^{2.4805}, \qquad \ldots\ldots\ldots\ldots\ (13)$$

in which:

H: the water head above the weir sill, cms.,

and Q: the steady discharge over the weir, litres/Sec.

The calibration curve of the orifice was obtained and shown in Fig. (8). It was noticed that the first part of this curve be a straight line on a logarithmic scale, which in turn be expressed as follows:

$$Q = 1.04\ W^{1.68}\ \ldots,\ (W \leqslant 2.75\ \text{cms.})\ \ldots.\ (14)$$

The above equation was found to be valid only for widths less than or equal to 2.75 cms. The second part was found to represent a staight line on a linear scale and its equation could be expressed as follows:

$$Q = 4.60\ W - 6.90\ \ldots,\ (W > 2.75\ \text{cms})\ldots.\ (15)$$

The above equation was valid for widths more than 2.75 cms. The change in the behaviour of the orifice for widths less than

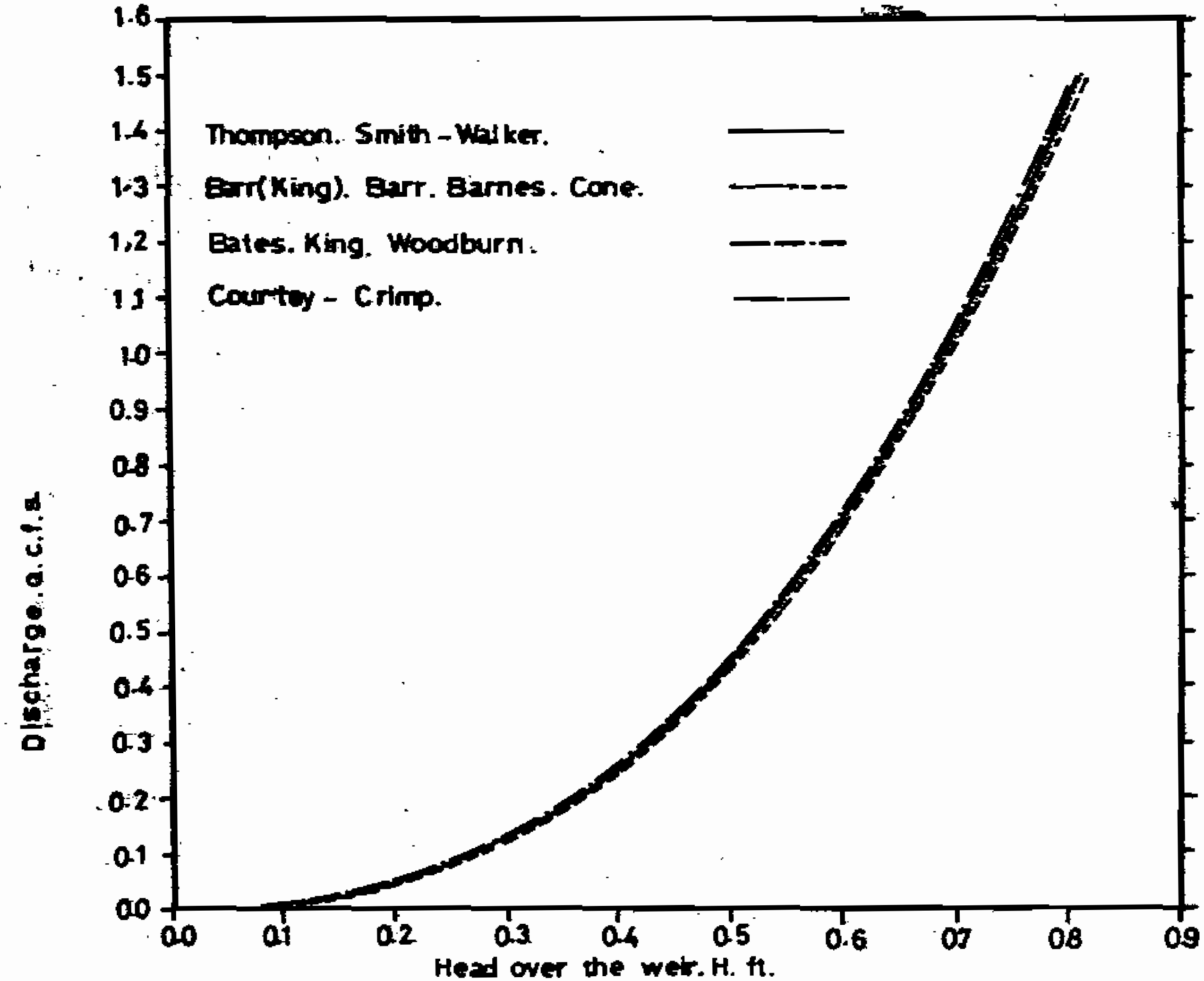


Fig.(6) Solution of the different equations of Triangular weir.

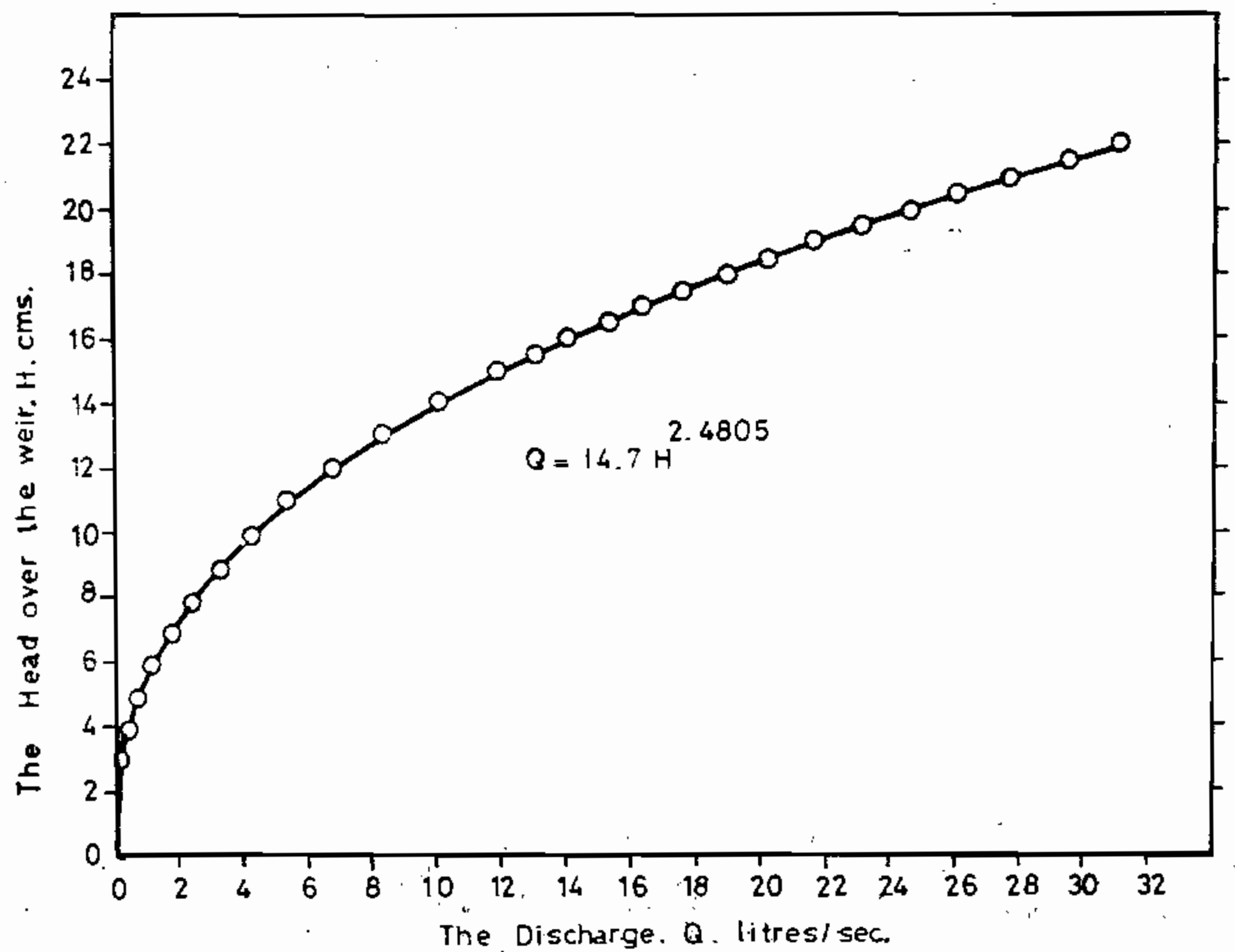


Fig.(7) Solution of Cone's Equation.

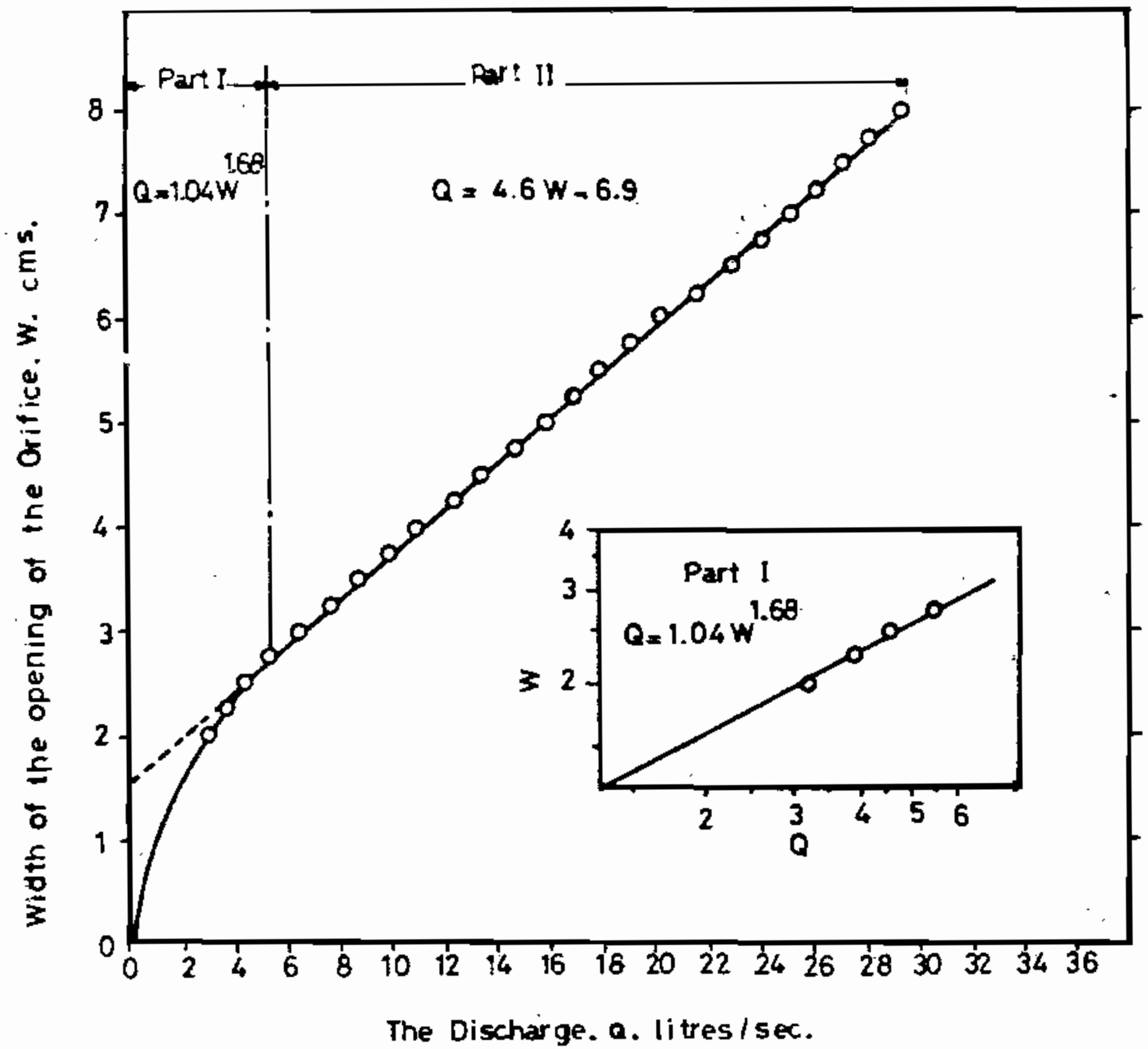


Fig.(8) Calibration Curve for the Constant head variable width rectangular Orifice.

2.75 cms. was probably due to an appreciable effect of side contractions in case of small widths.

V.2 Horizontal Bottomed Venturi-Flume:

In order to derive on expression for the discharge passing through a horizontal bottomed venturi-flume, the following analysis were drawn.

Applying Bernoulli's equation between the section at the throat and upstream the venturi flume:

$$H_1 + \frac{V_1^2}{2g} = H_c + \frac{V_c^2}{2g} = \frac{3}{2} H_c \qquad \ldots\ldots\ldots (16)$$

As the value of V_1 is relatively small, thus the approaching velocity head becomes very small and can be neglected, and consequently:

$$H_c = \frac{2}{2} H_1, \text{ but: } V_c = \sqrt{g\, H_c},$$

and $Q = b.\, H_c.\, V_c.$

So: $Q = \frac{2}{3} b.\, H_1.\, \sqrt{g.}\, \sqrt{\frac{2}{3} H_1}$

or: $Q = 0.387\ b.\, \sqrt{2g.}\, H_1^{1.50} \qquad \ldots\ldots\ldots\ldots (17)$

The above expression is vased on an approaching uniform flow, critical flow conditions at the throat and negligable velocity of approach. However, to secure the validation of such condition and to compensate for the probable losses, a coefficient of discharge C_d was thus introduced. Equation (17) could thus be rewritten in the following fashion:

$$Q = 0.387\ C_d.\ b\ \sqrt{2g.}\ H_1^{1.50} \qquad \ldots\ldots\ldots\ldots (18)$$

That compared w th the emperical expression obtained from the experimental results, according to Fig. (9-a):

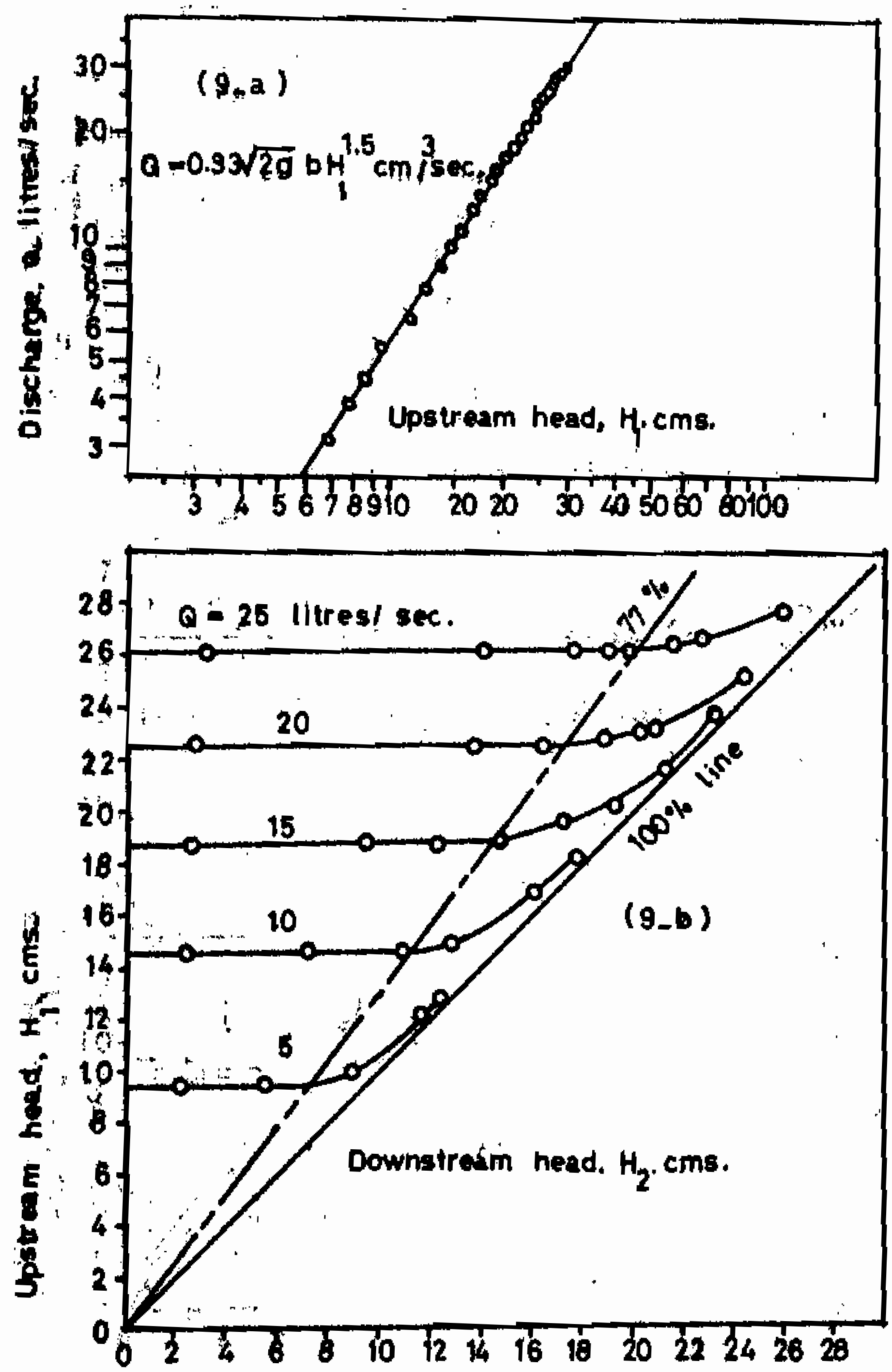


Fig.(9) Characteristic curves for Venturi flume with horizontal bottom.

$$Q = 0.330\ b. \sqrt{2g.}\ H_1^{1.50}, \qquad \text{............} \quad (19)$$

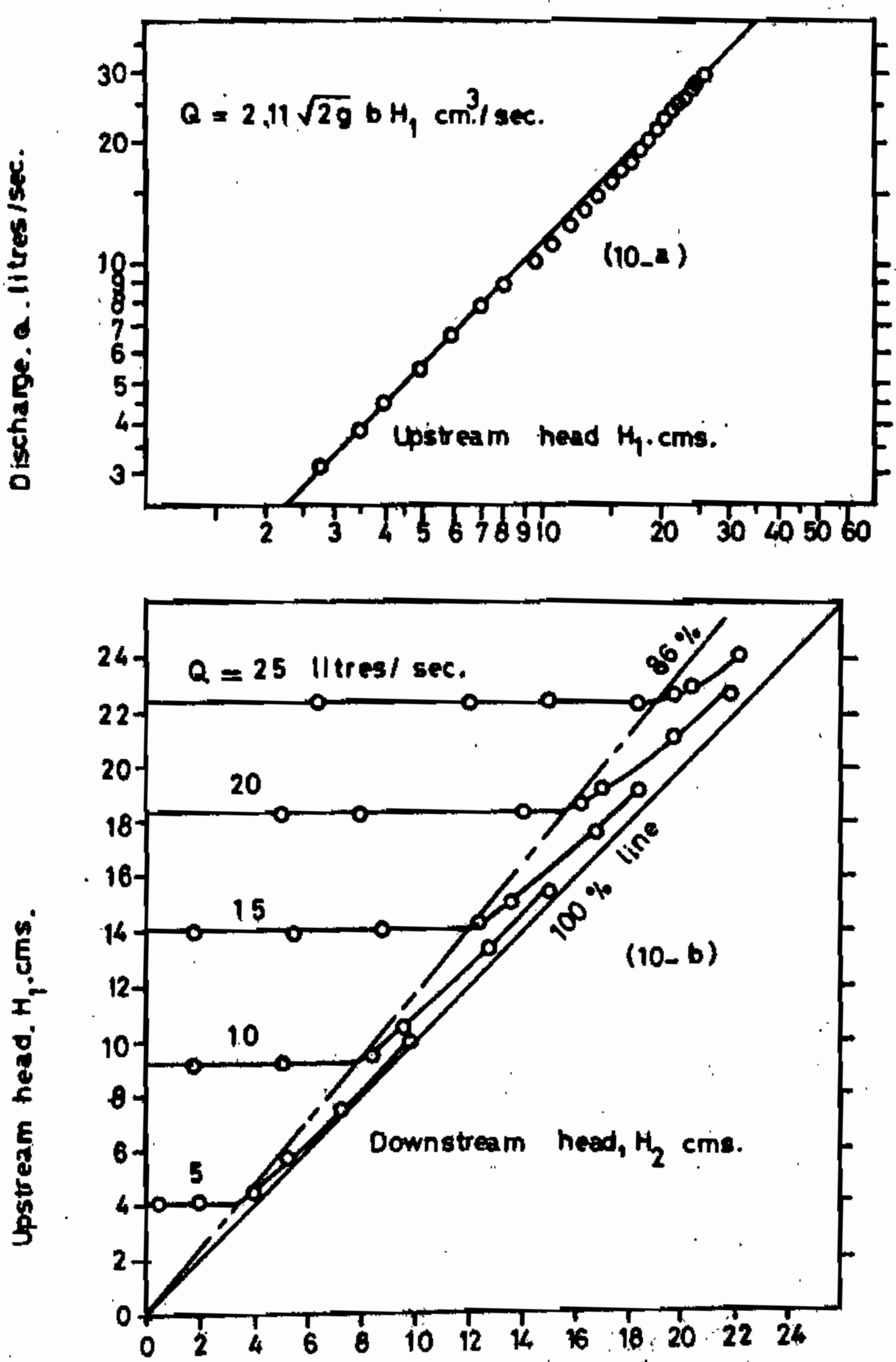
Which made the coefficient of discharge $C_d \simeq$ 0.8527 for such a venturi flume.

The modular range, which is the ratio between the downstream head and the upstream head, when it was just affected by the rise in the downstream head, i.e.showed its dependence, was obtained from Fig. (9-b) and found to be equal to 77.00 percent.

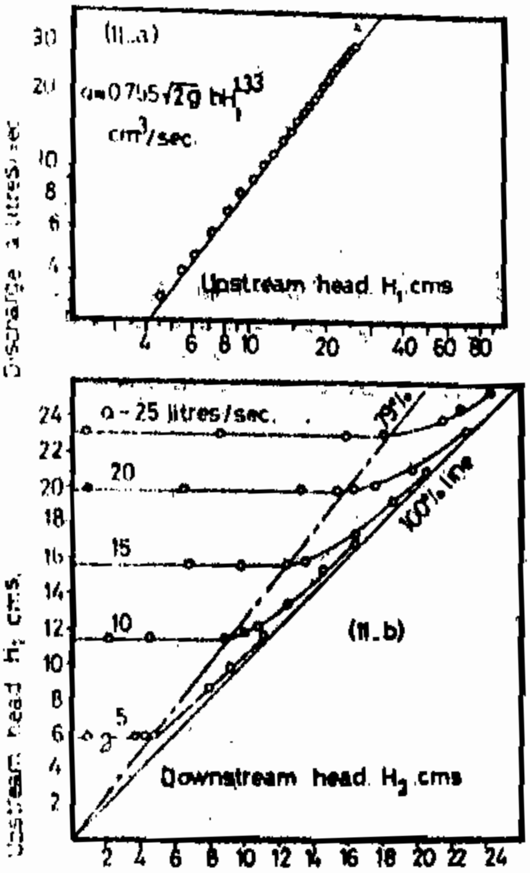
V) 3. Venturi-Flume With Various Adverse Slopes:

The various slopes which were made especially for this study were: 0.000, 0.044, 0.053, 0.062, 0.072, 0.085, 0.100, 0.135, 0.185 and 0.202. For each of these slopes, two relationships were established. Fig. (a) showed the relationship between the discharge and the upstream head within the modular range. While Fig. (b) showed the relationship between the downstream and upstream heads with the modular range for both free and submerged conditions. Fig. (10) showed an example for such relationships for the slope 0.085, which was found later to be the best. For other tried slopes, Figures (11-A) and (11-B) demonstrated some of these relationships examples.

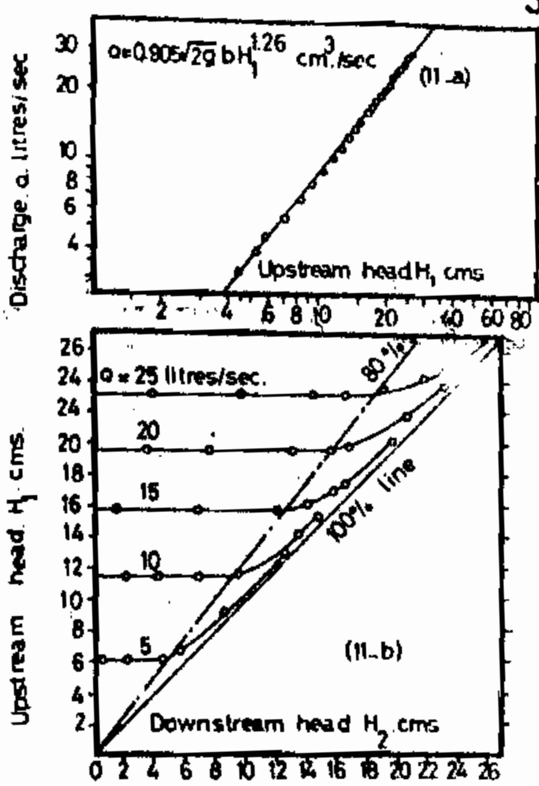
In order to find the best slope which would give the highest value of modular range, a relationship between the slope and the modular range was drawn and shown in Fig. (12), which showed that the slope 0.085 was the best. Also the relationships between the slope and the exponent power of the upstream head in the discharge equation and the slope versus the constant in the discharge equation were projected and plotted on the same Figure (12). All of these curves can be divided into the same notified two parts, which were previously indicated as follows: the first part was for slopes $<$ than 0.085 and the second part was for slopes ranged between 0.085 to 0.202.



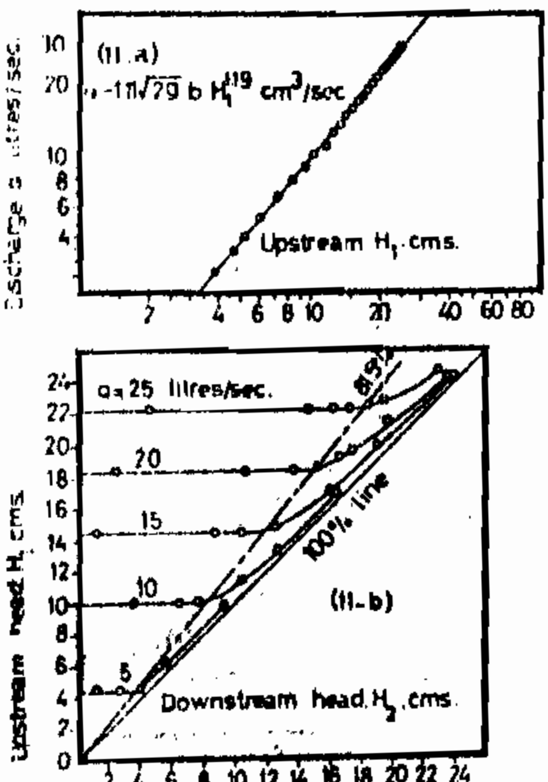
Fig(10) Characteristic curves for Venturi flume with Adverse Slope of 0.085 or (1:11.95).



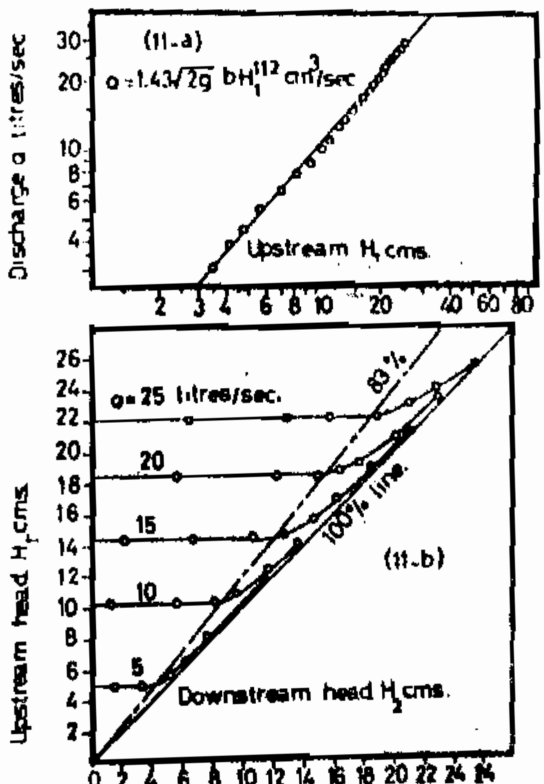
S = 0.044 or (1:22.7).



S = 0.053 or (1:19).



Fig(11-A) Characteristic curves for Venturi flume with Adverse Slope of 0.082 or (1:16.3).



Fig(11-A) Characteristic curves for Venturi flume with Adverse Slope of 0.072 or (1:14).

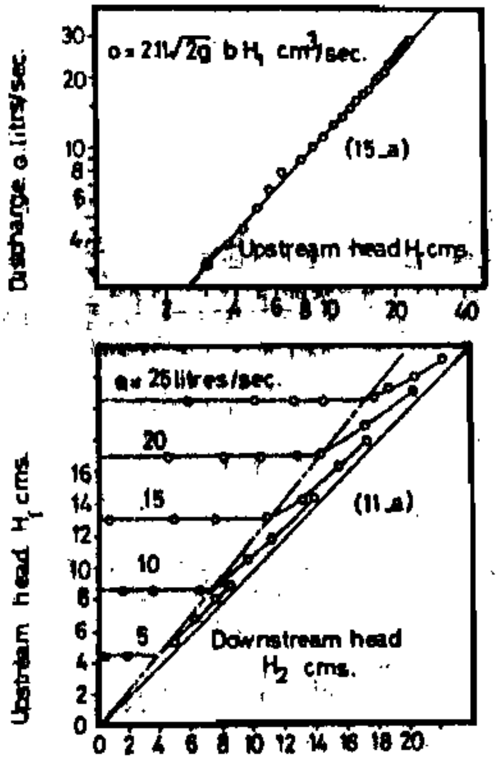


Fig (11-B) Characteristic curves for Venturi flume with Adverse Slope of 0.10 or (1:10).

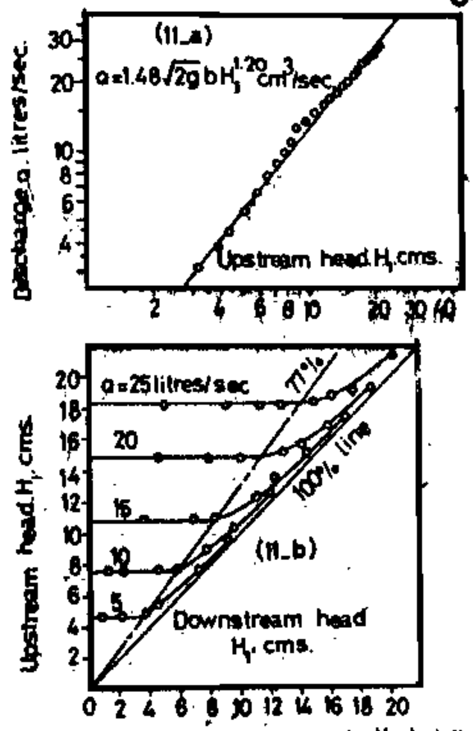
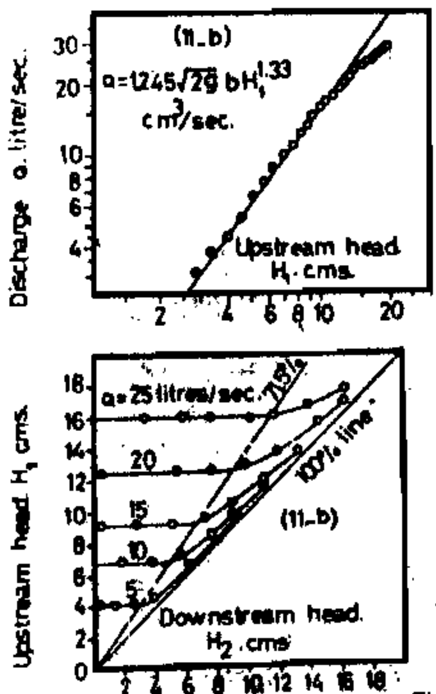
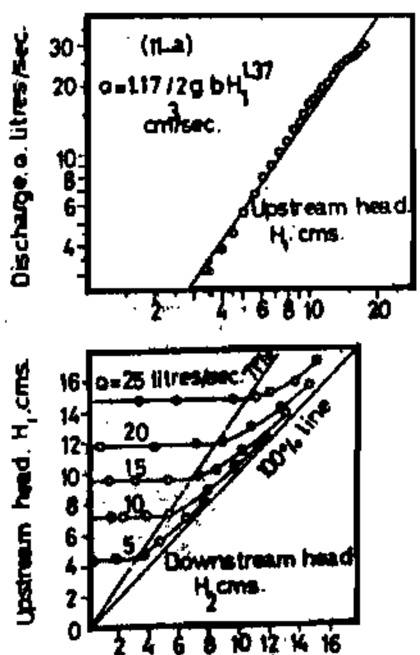


Fig (11-b) Caracteristic curves for Venturi flume with Adverse Slope of 0.135 or (1:7.43)

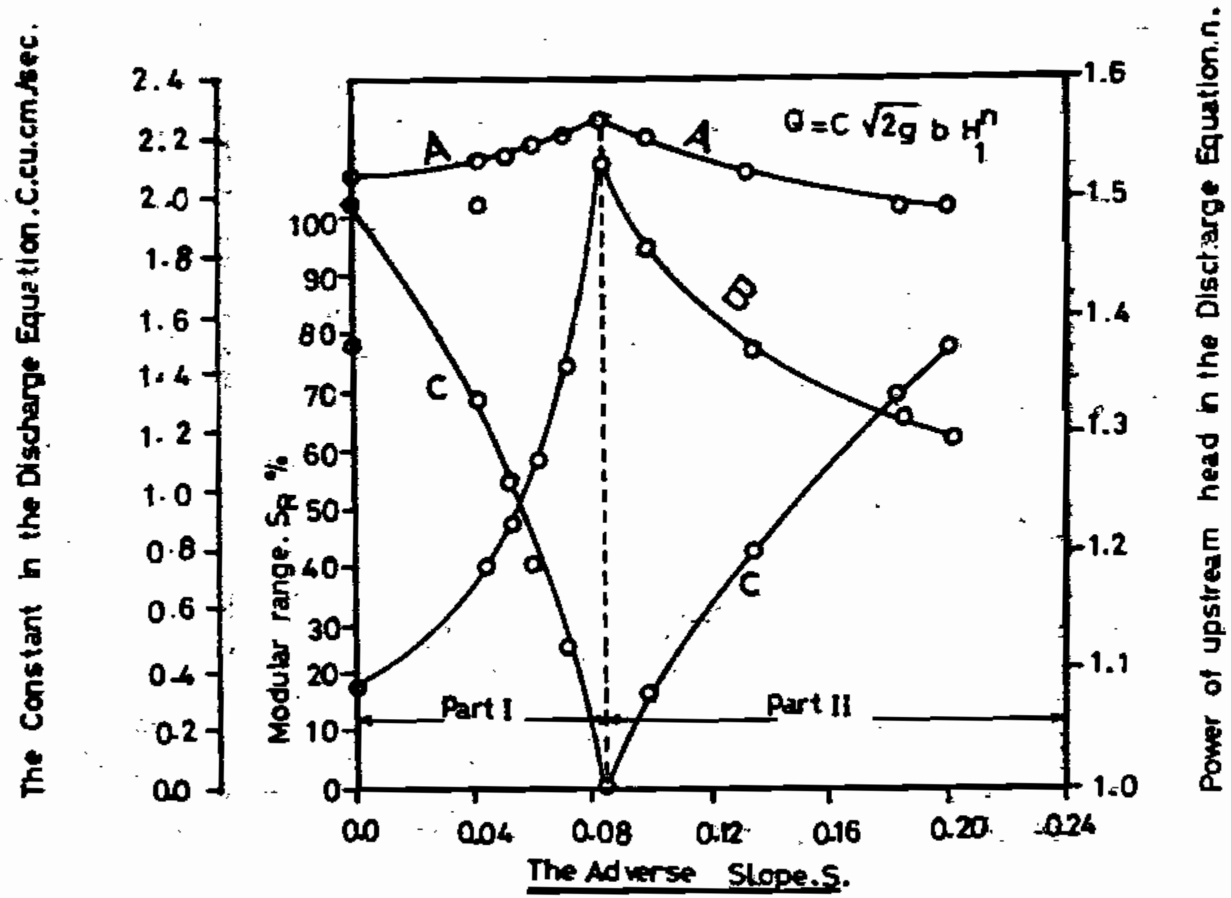


S = 0.185 or (1:5.39)

Fig(11-B) Characteristic curves for venturi flyme with adv-erse slopes of 0.100(1:10) to 0.202 (1:4.97).



S = 0.202 or (1:4.97)



Fig(12_A)

Relationship between the modular Range and the Slope.

Fig(12_B)

Relationship between the Slope and the Constant of the Discharge_Head Equation.

Fig(12_C)

Relationship between the Slope and the Power of the upstream head in the Dischage Equation.

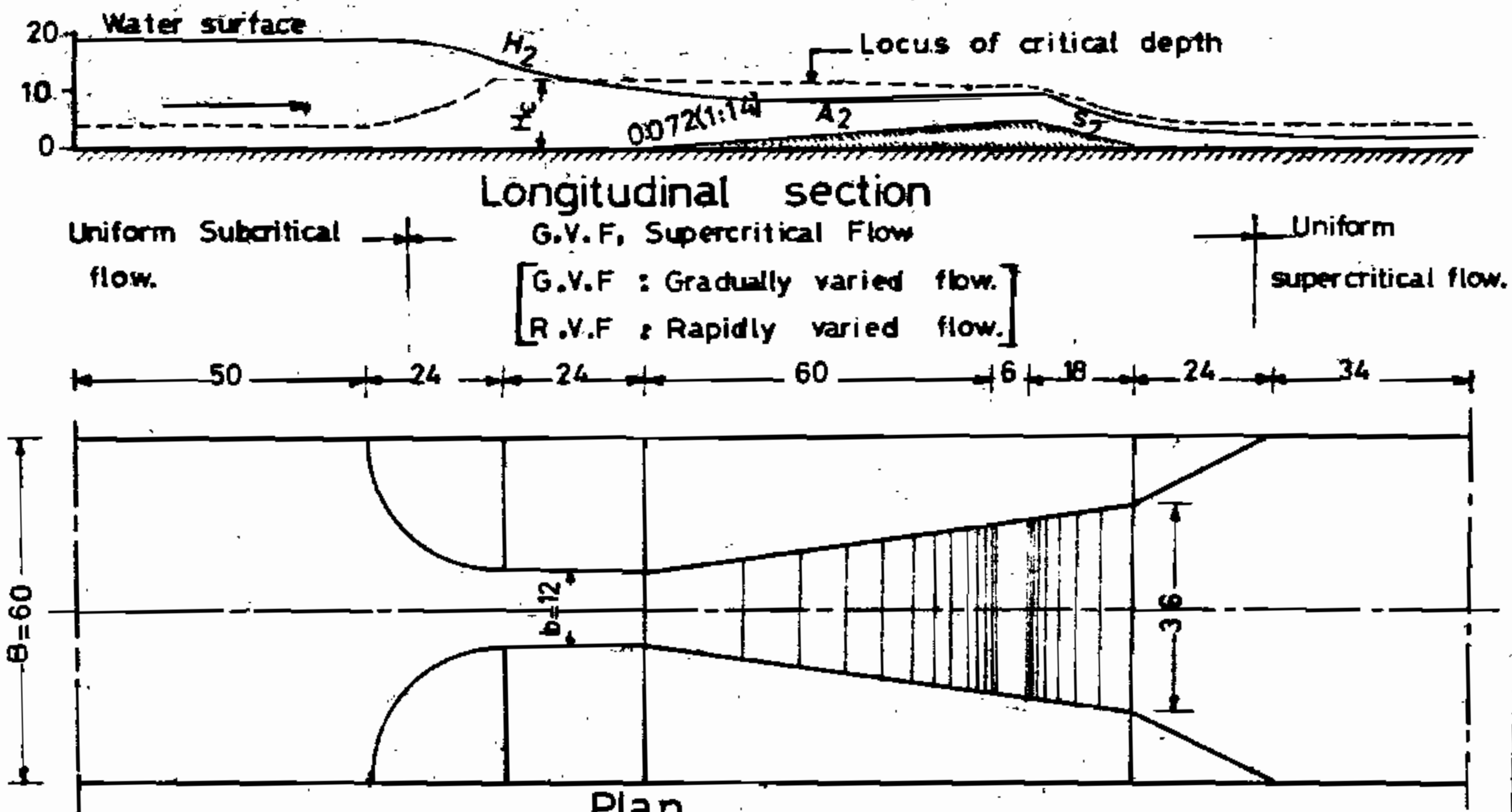
Both the modular range and the constant of the discharge equation were increased in the first part with the increase of the slope, while for the second part it decreased as the slope was increased. In the meantime, the power of the upstream head decreased with the increase of the slope within the first part, while it was increased with the increase of the slope within the second part.

The above description and discussions implied that there existed two phenomena which were demonstrated over the two indicated parts. To find an explanation for this variation, the water surface profile was studied for all slopes.

First, it was found that, for all slopes and considering each part, mentioned previously, no much appreciable change was noticeable concerning the shape of the water surface profile for all slopes. For this reason, two arbitrary water surface profiles were chosen. The first one concerns a chosen slope of part I as shown in Fig. (13). The second one concerned a chosen slope of part II as shown in Fig. (14). These figures were demonstrating the slopes 0.072 and 0.135 for a discharge of 15 litres per second.

Second, investigating the water surface more closely especially within the vicinity of the throat after estimating the critical depth line (C.D.L.). Such plottings are shown in Figures (15) and (16) for slopes 0.072 and 0.135 respectively, just for example.

Generally speaking, it was noticed that, for relatively small adverse slopes (like 0.072), the critical water depth was located just at the beginning toe of the upstream slope and continued as a supercritical flow allover the slope and downstream the crest and never became subcritical. On the contrary, for relatively bigger adverse slopes (like 0.135) the approaching subcritical flow converted into a supercritical flow witbin



Fig(13) Water Surface Profile for relatively smaller adverse slop of 0.072 or (1:14) for a discharge of 15 litres/sec.

All dimenstions in centimeters.

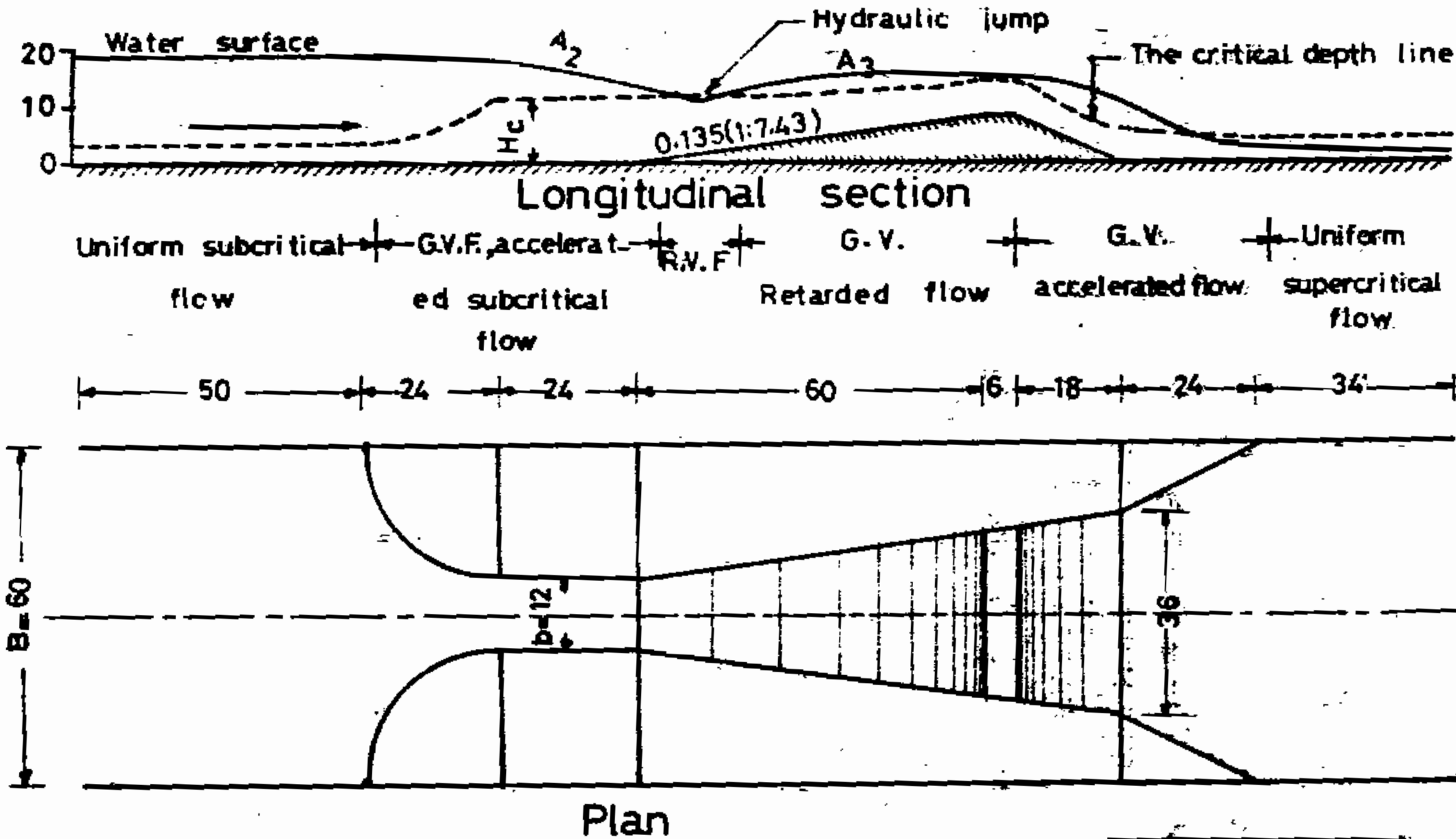
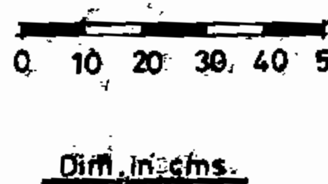


Fig.(14) Water Surface Profile for relatively steeper adverse slope of 0.135 or (1:7.43) for a discharge of 15 litres/sec.



the zono upstream the slope experiencing a dip creating a hydraulic jump over the upstream face of the slope, causing an existance of subcritical flow zone which was soon drawn down downstream the slope where the flow became supercirit-ical again, accelerating downstream to the outlet in a way similar to the other case.

Moreover, it was noticed that all hydraulic jumps which were formed over the approaching part of all adverse slopes located on part II of Figs. (12), (13) and (14) although slight deviation concerning the location of the hydraulic jump was noticed. Meanwhile, all cases of relatively big slopes ($>$ 0.085) the hydraulic jump was formed in the manner previously explained;this besides very slight deviation from linearity was noticed in the Q versus H_1 relationship, especially for higher discharges. A probable correction factor may be introduced here for proper measurement of the approaching flow.

VI. CONCLUSIONS:

Few conclusions could be drawn from the results of the present study as follows:

1) The adverse slope adapted to the bottom of the proposed venturi-flume type modified the originally diverging flow with the result of reducing the energy loss caused by the formed hydraulic jump and implemented with the resulted momentum transfer.
2) For relatively steep adverse slopes, the height of the hump became relatively big and the effective head became slightly greater than that actually measured over the hump. Vice versa was true when relatively milder adverse slopes were used. There were always an intermediate slope which gave the most favourable situation under a wide range of discharges.

3) The general equation of the proposed venturi-flume took always the following general form,

$$Q = C.b \sqrt{2g.} \quad H_1^n.$$

4) For adverse slopes from zero to 0.085, the value of n decreased while the value of C was increasing. For adverse slopes greater than 0.085, the value of n was increasing while C was decreasing. However, for all slopes, the discharge was increasing as H_1 was increased.

A summary of values for corresponding values for both n and C are tabulated below.

The Adverse slope	The power "n"	The constant "C"
Zero	1.50	0.330
0.044	1.33	0.755
0.053	1.26	0.905
0.062	1.19	1.110
0.072	1.12	1.430
0.085	1.00	2.110
0.100	1.08	1.830
0.135	1.20	1.480
0.185	1.33	1.245
0.202	1.37	1.170

5) The modular range, in which the upstream water depth was unaffected by the change in the downstream depth, increased as the adverse slope was increased up to a slope of 0.085, and then decreased for steeper slopes. This is demonstrated below.

The Adverse slope	The modular Range %
Zero	77.0
0.044	79.0
0.053	80.0
0.062	81.5
0.072	83.0
0.085	86.0
0.100	83.0
0.135	77.0
0.185	71.5
0.202	71.0

6) The best adverse slope, which developed the maximum value of modular range, was found to be equal to 0.085(or 1:11.95) with modular range of 86 percent in this research. Consequently, the discharge equation of the proposed venturi-flume type could be expressed as follows:

$$Q = 2.110 \quad b. \sqrt{2g.} \quad H_1 = k. \ b. \ H_1,$$

in which $k = 2.11 \sqrt{2g}$, or

$$Q = K \, H_1, \quad (K = 2.110 \quad b. \sqrt{2g.}).$$

i.e. - for a known throat width, the discharge "Q" through the proposed flume is simply linearly proportional to the upstream water depth "H_1" alone, dessipating minimum head loss.

Such an otuput depends on the general linear proportional dimensions and so the characteristics of the venturiflume and according to the conditions outlined in the presented research.

VII. REFERENCES:

1. Abu-Fetouh, A.H., (1960), "Flow over escape weirs", International commission in Irrigation and Drainage, Fourth Congress, C. 30, Madrid, Spain.

1 4- Mansoura Bulletin, June 1977.

2. Clyde, Calvin G., G.V. Skogerboe, and M.L.Hyatt,(1966), "Submorged Trapezoidal measuring flumes", Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah, U.S.A.

3. Fathy, A. and A.H. Abul-Fetouh, (1952), "A new type of Venturi Flume for the gauging of distributary canals" Irrigation and Hydraulics Laboratory, Civil Engineering Dept. College of Engineering, Alexandria University, Alexandria, Egypt (unpublished).

4. Institute of Hydraulics Padova University, (1970), "Lecture on Hydrometry", the 5th. International Post-Graduate Coruse in Hydrology, Padova, Italy.

5. Inglis, C.C., (1944), "Notes on Standing Wave Flumes and Flume buffle", Annual Rep. of the Indian Water-Ways Experimental Station, Poona, India.

6. Hyatt, M.L., G.V. Skogerboe, and K.O. Eggleston, (1966), "Laboratory investigations of submerged flow in selected Parshall flumes", Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah, U.S.A.

7. Hussien, Mahmoud A., (1974), "Effect of Convergence on the Discharge of a Venturi Flume", M.Sc. Thesis under the supervision of Prof. Dr. A. H. Abul-Fetouh and Dr. Talaat M. Owais, Civil Eng. Dept., Assiut Univ., Assiut, Egypt.

8-a. Owais, Talaat M., (1977), "Irrigation by Portable Equipment", a research study started in 1974 and still under preparation.

8-b. Owais, Talaat M., (1977), "Suggested Modifications for Irrigation System in Fayoum", a research study submitted for publication.

9. Parshall, R.L., (1941), "Measuring Water Irrigation Channels", Division of Irrigation, Soil Conservation Service and in Cooperation with the Colorado Agricultural Experiment Station, U.S.A.

10. Rouse, Hunter, (1949), "Engineering Hydraulics", Proceedings of the Fourth Hydraulics conference, Iowa Institute of Hydraulic Research, U.S.A.

11. Skogerboe, G.V., W.R. Walker and L.R. obinson, (1965), "Design, Operation and Calibration of the Canal A. Submerged Rectangular Measuring Flume", The D.M.A.D. Company, Delta, Utah, U.S.A.