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EFFECT OF TOP SLOPE ON SUBMERGENCY
RATIOS FOR STANDING WAVE WEIRS
BY

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I. ABSTRACT:

It has been always the main objective of many of the Egyptian investigators to find the best ways to restore enough irrigation water to be well regulated and distributed with the right amount and the best HOW and WHEN to be applied all over the agricultural lands. Standing wave weirs became, since long time ago, one of the very widely traditional irrigation work used to regulate, control and measure flow rates through irrigation distribution system.

The main objective of this research study was to find out the effect of changing the top slope of the standing wave weirs on the ratio of submersion for the sake of practicing higher flow rates influenced by higher values of submergency ratios, experiencing minimum heading up, resulting the least head loss through the system.

A well proportional scaled dynamically similar standing wave weir model was constructed across an artifically made controllable trapeziodal canal. Under different flow conditions and with the variation of seven top slopes, (3: 1, 4 : 1, 5 : 1, 10 : 1, 15 : 1, 20 : 1 and Zero : 1), relationship

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between upstream and down-stream water heads were drawn to reach the modular ranges applying different flow rates. Also, distinct Correlations between submergency ratio and the upstream water heads versus the seven top slopes individually, as a third variable, were achieved where an inverse relationship was found.

Moreover, illustrations regarding the effect of the top slope on both the stability of flow over the weir and on the submergency ratios and thus on the weir flow rates were demonstrated. Definite conclusions were drawn; anong them the suitability of the top slope 5:1 for relatively smaller discharges while the top slope 20:1 were found to be sutiable for the higher ones. Also, several recommendations were emphasised concerning the limitations for using some of these top slopes for practical purposes. That in addition to other recommendations for future studies relevant to the same course of study.

II. INTRODUCTION:

There ha s been always great concern towards Egyptian irrigation distribution system in order to Convey the right amount of water to the users in a scientific controlable manner. Among the various irrigation works serving this objective and plays a vitally important role in that regard is the standing wave weir, which has been traditionally one of the most useful devices to measure, controle and regulate irrigation water. Due to the flatness of the country, mostly, and due to the fact that it is always favourable to pass big discharges under small heads with minimum head loss, standing wave weirs proved to fulfill such requirements.

Moreover, standing wave weirs are considered practical device to divert the flow and for discharge measurement because of its simplicity, easy to be construoted plus the fact that they are less likely to undergo alterations over long periods of operation. Consequently, they need the least effort for maintenance especially in case the water is clear. This is in addition to other advantages regarding the hydraulic stability of the earthen canals besides irrigation management and the economical points of view.

It has to be noticed that standing wave weirs a re also used to adjust the "synoptic diagrams" and sometimes work as subsidiary weirs to share the differences in water heads upstream and dewnstream head regulators and barrages and balancing out excess energy leaving out downstream such irrigation works. Meanwhile, broad crested weirs a re broadly used too which can be considered a special kind of standing wave weirs in which the top slope is zero, i.e. - flat horizontal.

When the discharge passes over the weir is relatively small, the proportional dimensions of the weir become effective; and for higher discharges these dimensions influence the flow behaviour quantitatively as well as qualitatively to great extent especially at higher submergency ratios. Keeping the overall dimensions of the standing wave weir in a well proportional fashion according to specified standard rules, the top slope becomes the most effective variable that would characterises the flow in that sense.

III. THE OBJECTIVES:

The main objective of the present study is to investigate the effect of changing the top slope of a standing weir on the ratio of subersion and so on the flow rate. The output of this research study is practically fruitful, when looking for the best top slope that fits the best operational requirements, offering the mest favourable stable flow condition, undergoing the maximum discharge, possessing minimum heading up and furnishing the least head loss through the system under higher values of submergency ratios.

One way to accomplish the objectives of such a study is to construct a proportional scaled physical model dynamically similar with an actual prototype experiencing wide variation of discharges and excersising higher values of submergency ratios.

IV. LITERATURE REVIEW:

Back literature concerning this subject are mostly covered in the countries that were obliged, and still, using extensive numbers of standing wave weirs, like: India, Pakistan and Egypt, etc. (Refs. 1,11,12 & 16, say).

An attempt to describe the flow over broad crested weirs was carried out by Belanger (Ref. 7) where the head loss due to friction and turbulence over the weir were neglected leaving:

$$h_1 + \frac{V_1^2}{2g} = h_c + \frac{V_c^2}{2g} = H \qquad \text{......(1)}$$

in which: h_c = the critical water depth = $\frac{2}{3}$ H .

(N.B.: h_c is located somewhere over the crest. H= total head.). Solving for the critical velocity V_c, one can get:

$$V_c = \sqrt{\frac{2}{3} g H} \qquad \text{......(2)}$$

So, the discharge passes over a broad crested weir of width b, without side contraction be expressed as follows:

$$Q = V.A = b(\frac{2}{3} H) \sqrt{\frac{2}{3} g H}$$

or:

$$Q = b. \sqrt{g} \quad (\frac{2}{3} H)^{3/2} \qquad \text{......(3)}$$

If the above basic equation is simplified, neglecting the velocity of approach and introducting the coefficient of discharge C_q to stand for that effect in addition to viscosity, surface tension and head loss effects on the crest, it turns out to be:

$$Q = C_q \cdot b \cdot \sqrt{2g} \cdot \left(\frac{2}{3} h_1\right)^{3/2} \qquad \ldots\ldots(4)$$

The value of C_q can be determined numerioally from experiments, where:

$$C_q = f\left(\frac{h_1}{P}, \frac{h_1}{L}, \mathrm{IR}, W\right).$$

in which:

- P = vertical upstream height of weir,
- L = horizontal total length of weir,
- h_1 = height of water U.S. over the crest,
- IR = Reynolds number based on the characteristic undisturbed uniform approaching flow, and
- W = Weber number, which can be neglected as h_1 is not too relatively small.

On should take into consideration that somewhere along the crest, a critical water depth is located at a seotion, depending upon the crest breadth L.

Rao and Muralidhar (1963) made an investigation over a wide spread ranges of (h/P) and (h/L) summing up their findings into identified regions shown in Fig.(1) in the following manner:

I) <u>Long Crested</u>: $C_q = 1.65\ (h/L)^{0.022}$, for:

$$\left(0.02 < \frac{h}{L} \leqslant 0.10\right) \qquad \ldots\ldots(5)$$

II) <u>Properly crested</u>: $C_q = 0.0825\ (h/L) + 1.551$, for:

$$\left(0.10 \leqslant \frac{h}{L} \leqslant 0.35\right) \qquad \ldots\ldots(6)$$

III) $C_q = 0.352\ (h/L) + 1.4465$, for:

$$\left(0.45 \leqslant \frac{h}{L} \leqslant 1.50\right) \qquad \ldots\ldots(7)$$

For the transitional zone: $\left(0.35 \leqslant \frac{h}{L} \leqslant 0.45\right)$, both formulas (6) and (7) are applicable.

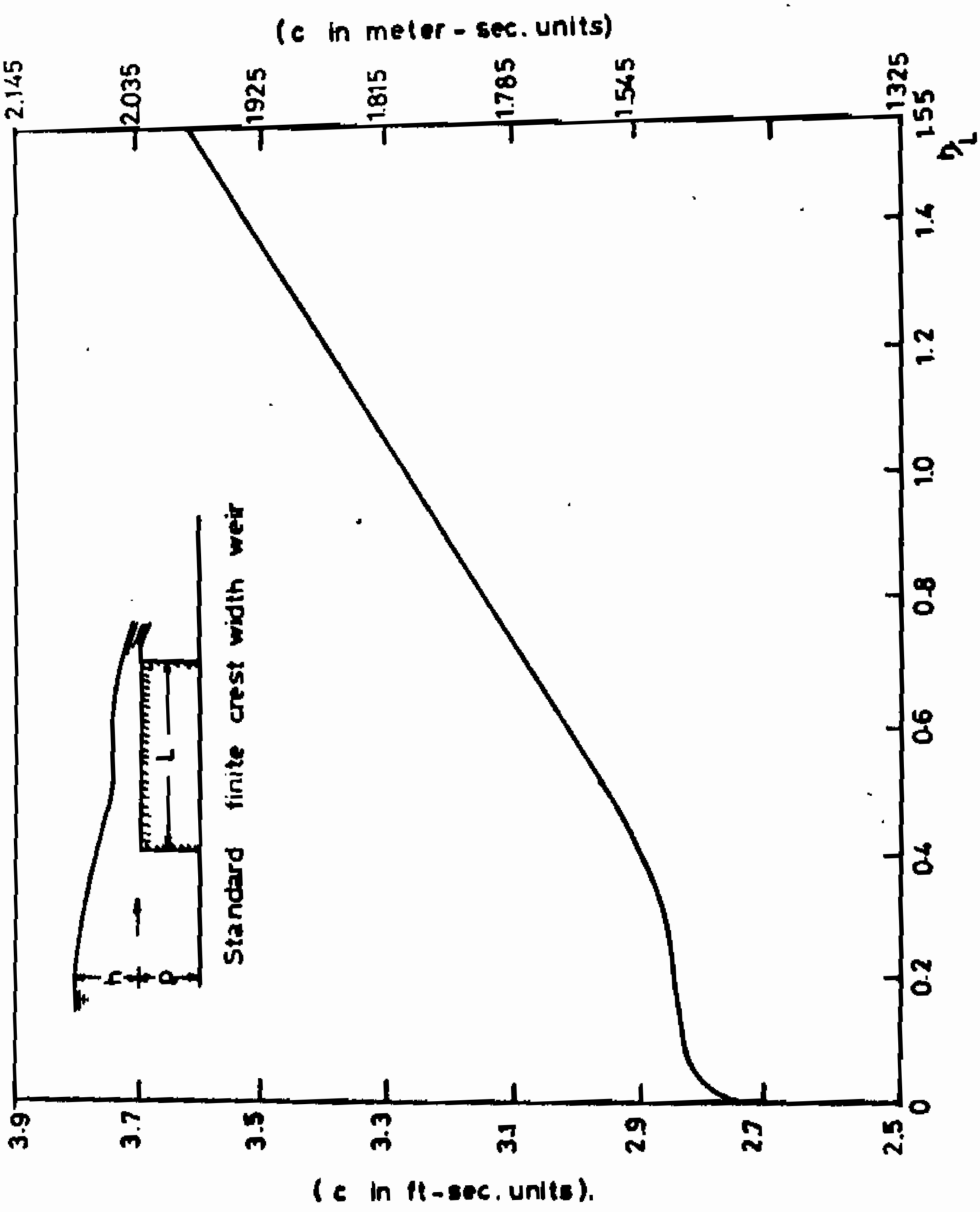


Fig.(1)_Standerd finite crest width weir

The above formulas are all in metric units and their domaine showed an accuracy of 2 percent, under the same specified conditions and circomstances.

Singer (Ref. 14) showed a graphical representation for determining a value for the discharge coefficient C_q shown in Fig.(2) and for rounded upstream face of Fig.(3). While Fig.(4) showed the most common submerged weirs or the so called drowned weirs, where:

$$h_1 + \frac{V_1^2}{2g} = h_2 + \frac{V_2^2}{2g} + \Delta \qquad \text{......(8)}$$

in which: Δ is the total head loss through the weir.

Meanwhile, Fig.(5) shows a graphical representation for the calibration curve of a triangular profile weirs (with a broken top surface). A simplified formula for the discharge could be:

$$Q = C_q . \; B \sqrt{2g} . \; H^{3/2}, \qquad \text{......(9)}$$

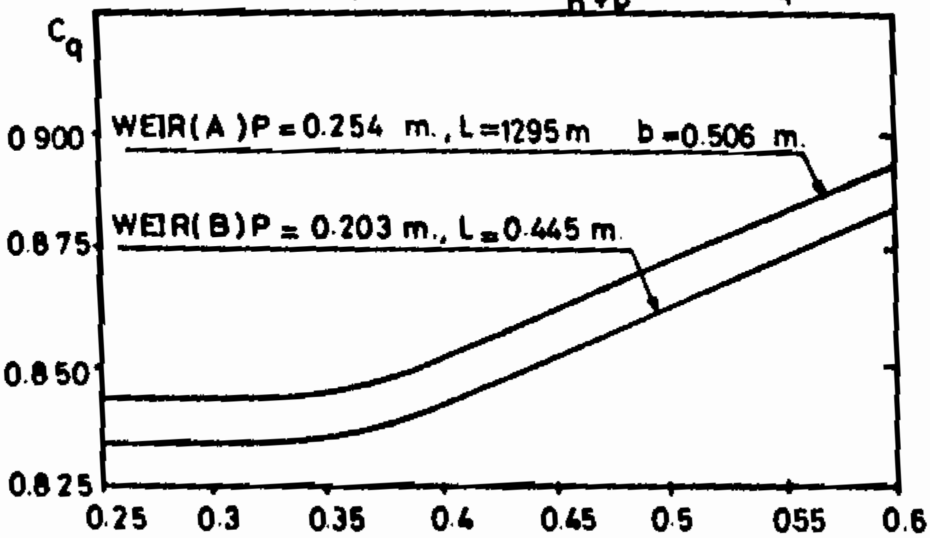
in which the total head $H = h + \frac{V^2}{2g}$.

It is noticed that Fig.(5) shows that the coefficient of discharge C_q is almost constant within the modular range, where the upstream water level is independent of the downstream water levels.

Woodburn and Webb (Ref. 15) carried out their tests changing the top slope of the weirs from: 0.0150 to 0.0260 to 0.0040 and sometimes preceding by a horizontal part. They tried to allocate the critical water depth under different flow conditions for different flow rates. They found that the critical depths always lie near the weir crest which was evaluated as for a rectangular section as follows:

$$Y_{cr} = \left(\frac{Q^2/B^2}{g}\right)^{1/3} \qquad \text{......(10)}$$

Relationship between $\frac{h}{h+p}$ and C_q



$$C_q = \frac{Q}{2/3\sqrt{g}\cdot b\cdot H^{1.5}}$$

$$H = h + \frac{v^2}{2g}$$

Fig(2)

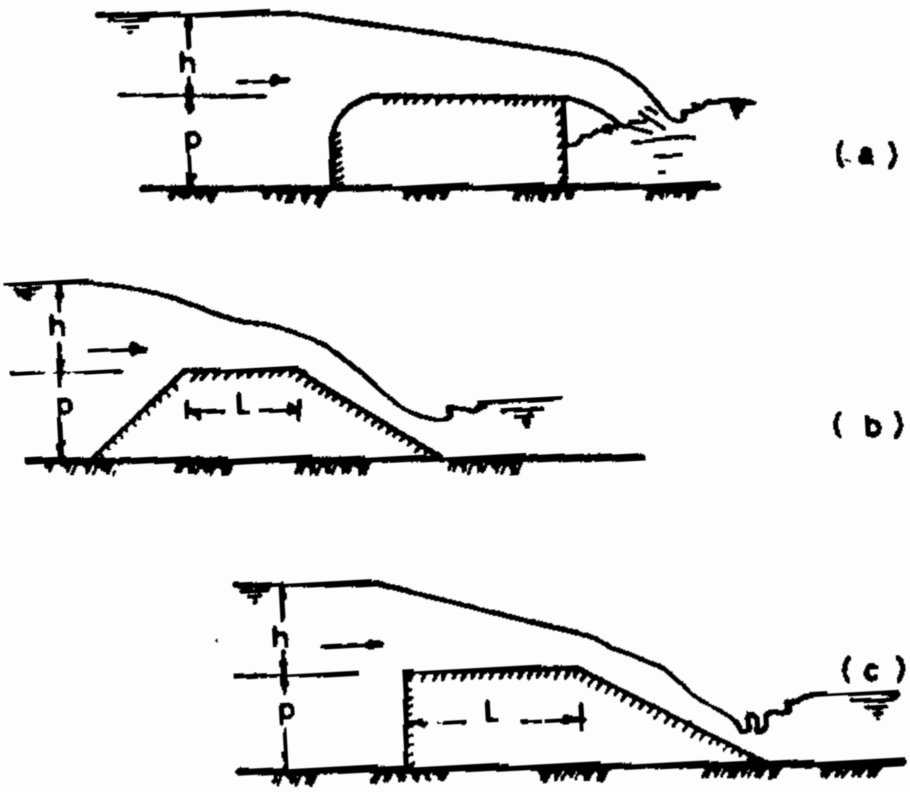


Fig (3)- Rounded upstream face weir

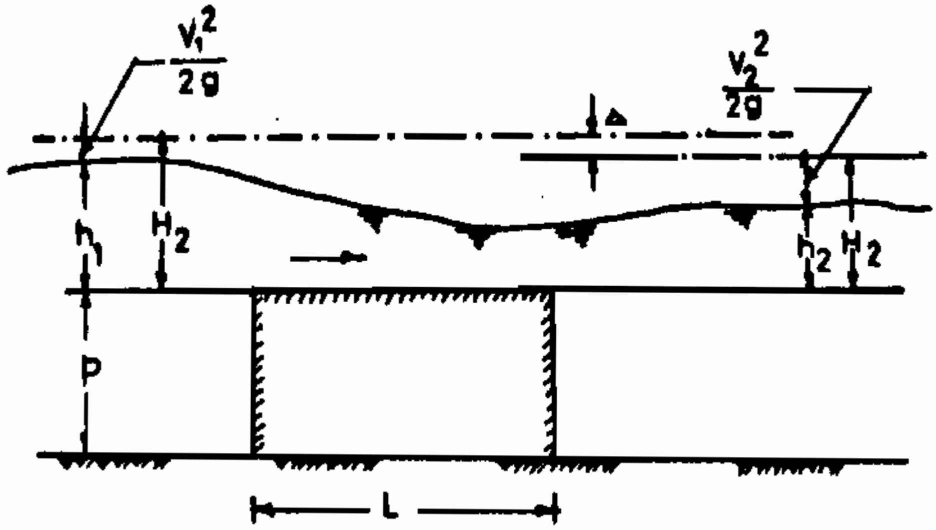


Fig (4)-
Drowned Weir

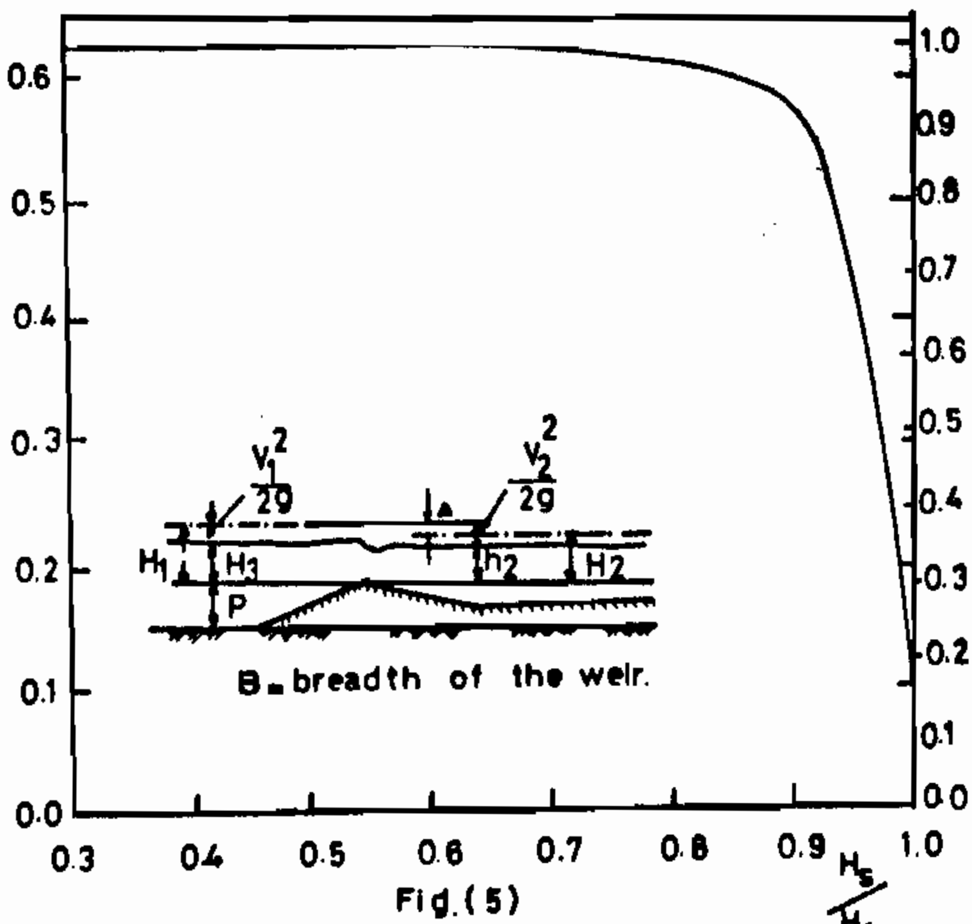


Fig.(5)
Calibration curve of a triangular weir.

Konsoh (Ref. 9) discoussed the above argument and tried to adjust the findings by carrying out several similar experiments on a rectangular tiltable flume using a sufficient water supply system, and checking the critical slope against both the location and the evaluation of the critical water depth.

Hersohel (1938) carried out experiments on submerged suppressed rectangular weir and applied the following equation for the discharge:

$$Q = 3.33\ L.n.H)^{3/2}, \qquad ......(11)$$

in which the discharge Q_1 in second feet, and n was a correction factor depending on the amount of submergence. That was simuilar to Stevens' findings (1938) and close to what was accomplished by Francis later on, furnishing tables for C_q in terms of the ratio of the head downstream to that upstream.

Rao and Shokla (Ref. 12) carried out system of experiments on weirs of finite crest width B provided with sharp upstream corner and summing up their analysis with the following expressions for the discharge coefficient C_d as follows:

$$C_d = 0.611 + 0.08\ (h/p),$$

for sharp crested weir, $(h/b > 1.60)$ ......(12)

$$C_d = 0.578 + 0.061\ (h/P), \text{ for: } (h/B = 1.60) \qquad ......(13)$$

$$C_d = 0.527 + 0.049\ (h/P), \text{ for: } (h/B = 1.00) \qquad ......(14)$$

$$C_d = 0.482 + 0.020\ (h/P), \text{ for: } (h/B = 0.08) \qquad ......(15)$$

The discharge equation used was similar to that of equa tion (9), where:

$$Q = \frac{2}{3}\ C_d .\ \sqrt{2g}.L.h\ \frac{3}{2} \qquad ......(16)$$

Moreover, for broad-crested weirs streamlined upstream corner, an expression for C_d similar to that given by Ippen (1950) was expressed as follows:

$$C_d = Q/(3.09\ L.H_o^{3/2}) \qquad \text{......(17)}$$

Also in terms of the "brink depth", Y_b, which was suggested by Rouse (1949), where,

$$Y_b = 0.715\ Y_c \text{ (the free flow case):}$$

$$Q = K.L.\sqrt{g}.\ Y_b^{3/2} \qquad \text{......(18)}$$

after introducting the coefficient K as a now parameter plotted versus (h/P) and (h/B) as a third parameter plotted in Fig.(6). The value of K was estimated by Rouse as 1.65 for smooth horizontal weirs with stream lined upstream corner for free overfall case.

Furthermore, Fig. (7) illustrates few examples of dimensionless water surfac profiles, a part of an investigation carried out by Raouse (1971) under various flow conditions.

V. EXPERIMENTAL PROGRAMME:

5.1- Experimental Devices and Equipment

A scaled proportional model for a standing wave weir was constructed across an especially made trapeziodal masonry channel, which was lined with cement mortar (Fig. "8"). The standing wave weir shown in Fig. (9) was situated within the test section, where the undisturbed flow is well established and the approaching flow is uniform. The general arrangement for the experimental set up is shown in Figs.(8) & (9). The water was supplied by a constant head tank which was provided with an overflow system and where the water was orginally pumped from an underground reservoir which served this recirculating flow system. The channel was provided with an

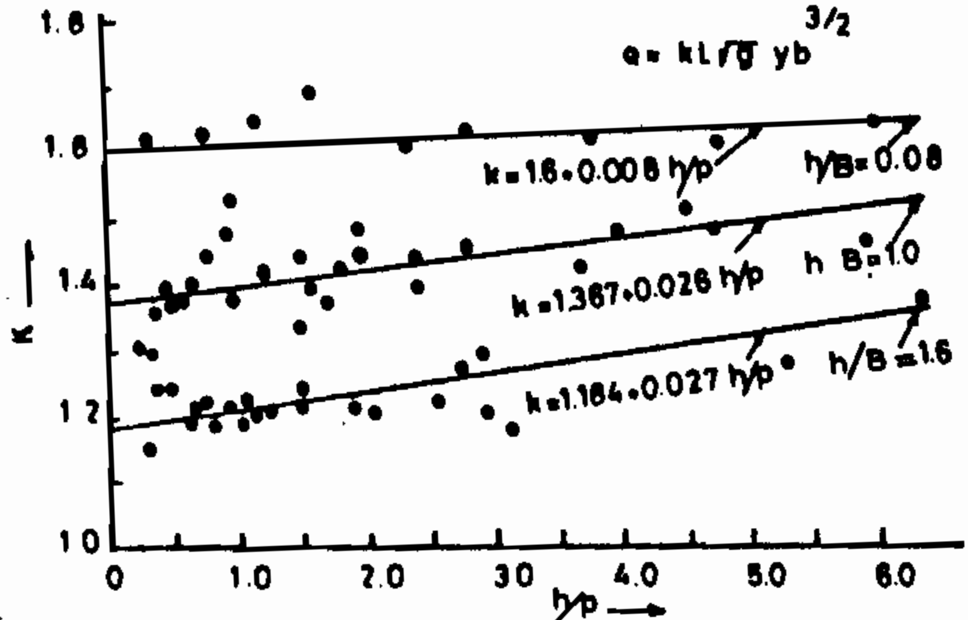


Fig (6) - Values of k in $q = kL\sqrt{g}\, y_b^{3/2}$ for weirs of finite crest width with sharp upstream corner.

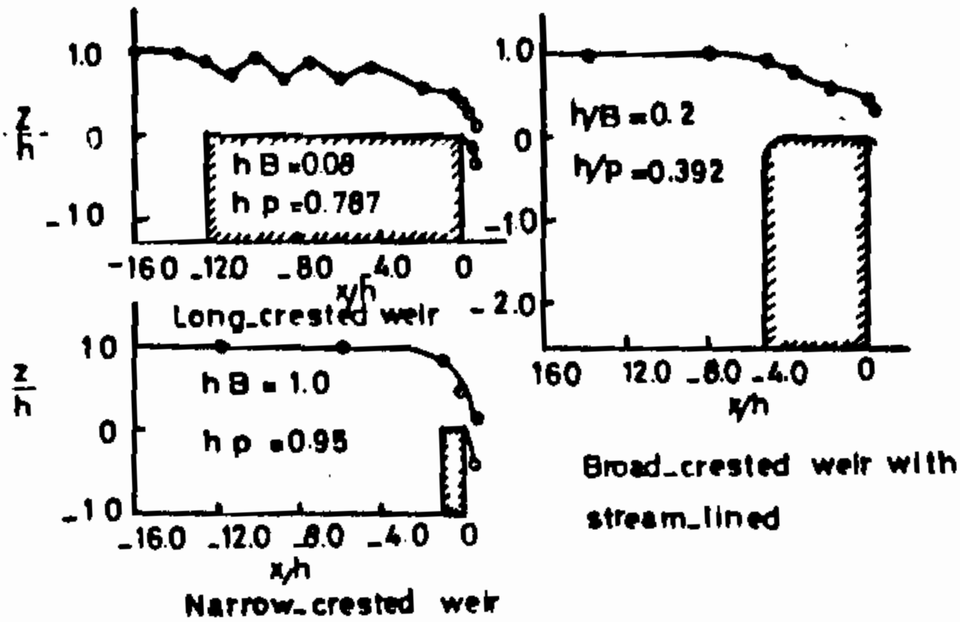
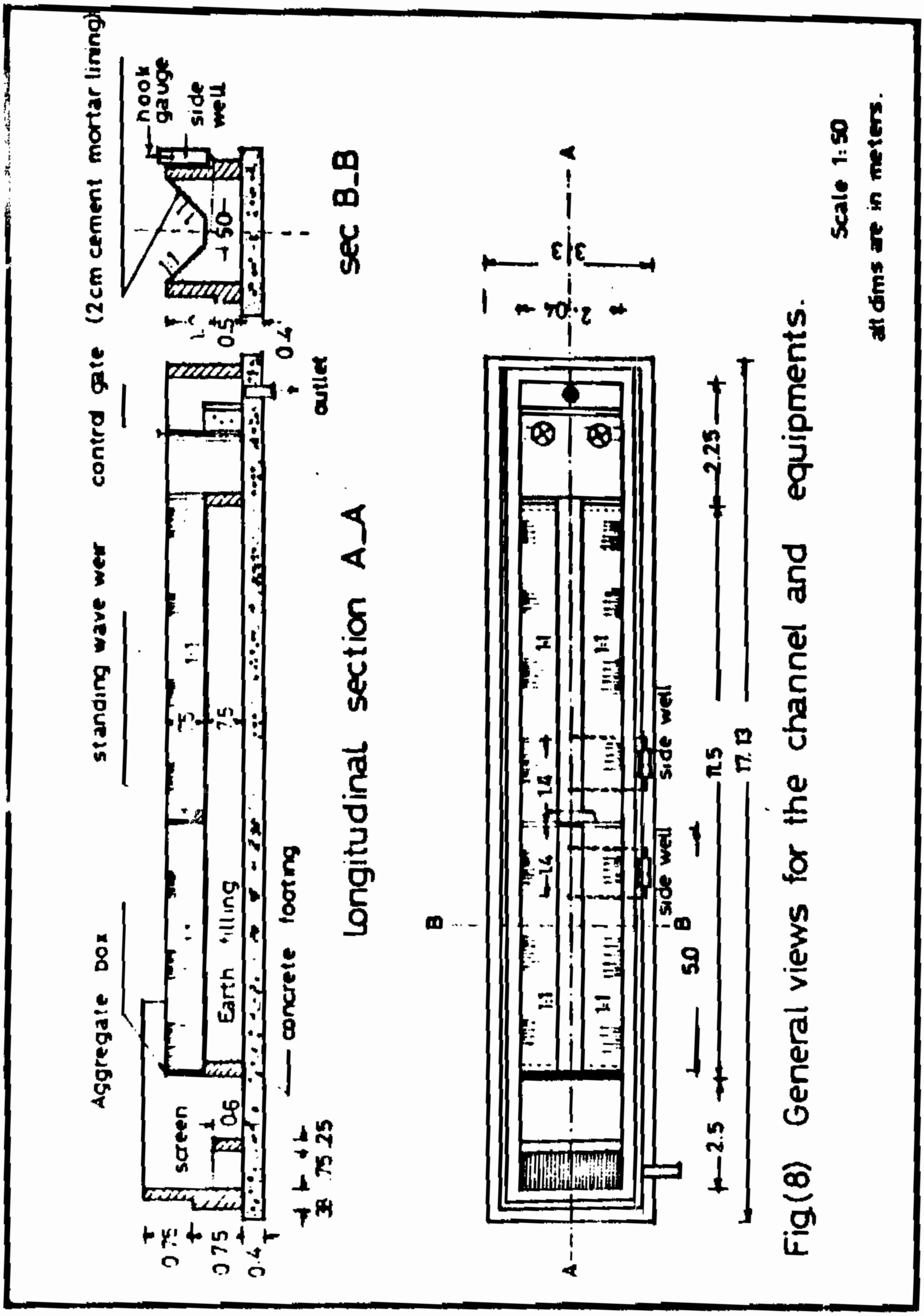


Fig.(7). Typical Water-Surface Profiles.



Fig(8) General views for the channel and equipments.

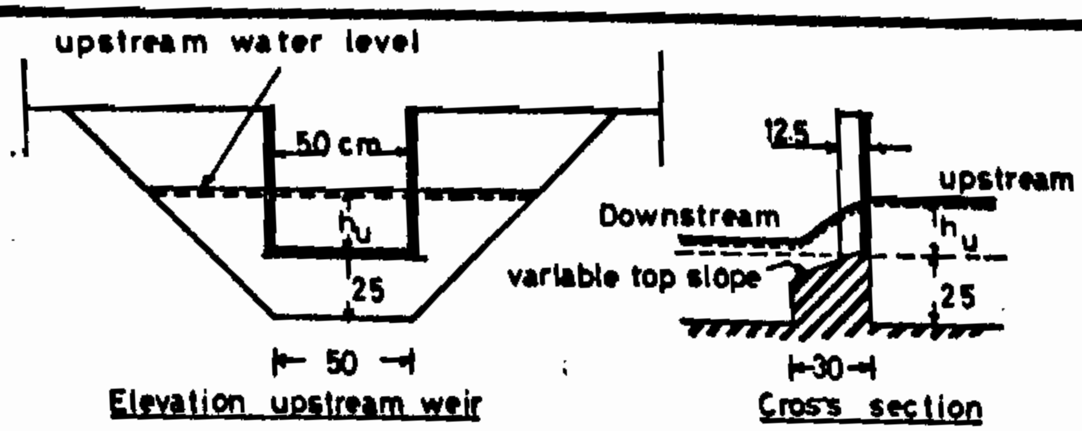


Fig.(9)_ Details of the weir

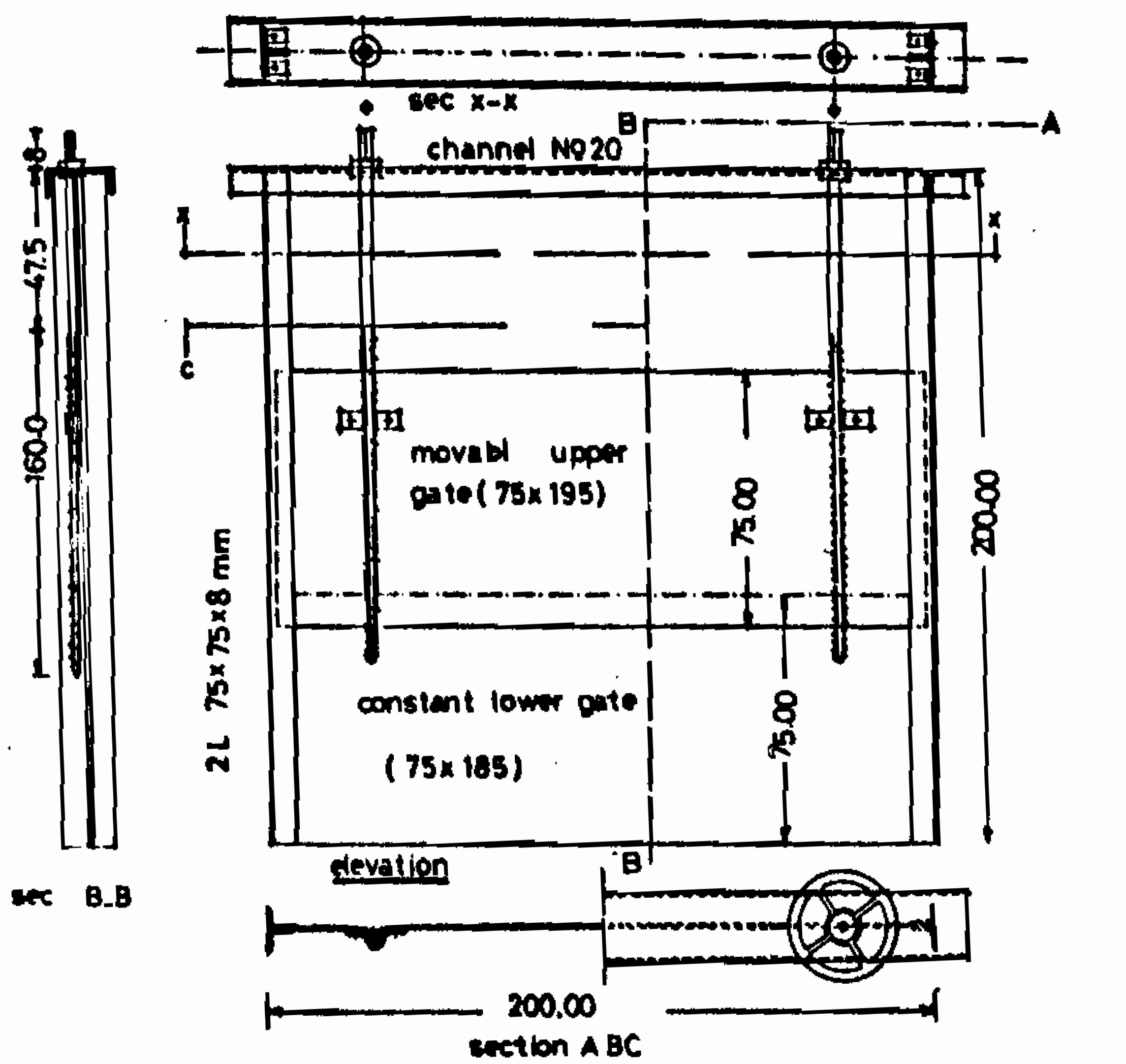


Fig.(10) Downstream control gate.

aggregate box at the intake upstream to assure uniformity of the incoming flow and minimizing any sort of large-size turbulence and unfavourable waves effect. It was also provided with a movable adjustable vertical steel sluice tail gate provided with a sump at its ends to regulate the flow and controle the flow characteristics downstream the weir. Fig. (10) shows drawings and dimensions of this downstream control gate.

The slope of the top surface of the standing wave weir was varied according to the following slopes:
3 : 1, 4 : 1, 5 : 1, 10 : 1, 15 : 1, 20 : 1 & Zero : 1 (i.e.- horizontal) in the direction of flow (Fig. 9)). The weirs are made out of masonry and covered with cement mortar and its overall shape was shaped out correctly and smoothed out uniformly using a standared sand paper.

Two side-wells each of 105 cm. height, 25 cm. depth and 50 cm. width with clear glass faces and partioned with frosted glass provided with lights and reflectors. They were located upstream and downstream the weir to adjust, check and record the water depths. The hook gauge of each of the side-wells was attached to a scale rod provided with a knob in conjnction with a vernierindex readable for an accuracy of $\frac{1}{10}$ mm. The Zero readings for both upstream and downstream side-wells were adjusted after they were exactly checked by different means.

5.2- Experimental Procedure:

The seven selected slopes taken for the top surface of the standing wave weir, which were named in item (5.1), made seven main series of tests where each slope established one individual series fixing all other variables under variable flow rates to obtain different submergency ratios within their possible corresponding modular ranges. That was partly accomplished by the controllable tail gate.

5.3- Experimental Results

Figures (11, 12 & 13) show three illustrative examples for three of the previously mentioned series and covered the top slopes 5:1, 20:1 & Zero : 1, where the downstream water depth was correlated to the upstream water depth. These curves were used to match the modular point at which independence of upstream water level due to changing the downstream water levels was achieved. The samething happened for other slopes.

Meanwhile, Fig.(14) illustrates the relationship between the upstream water level and the submergency ratios at the same different slopes like 5:1, 20:1 and Zero : 1.

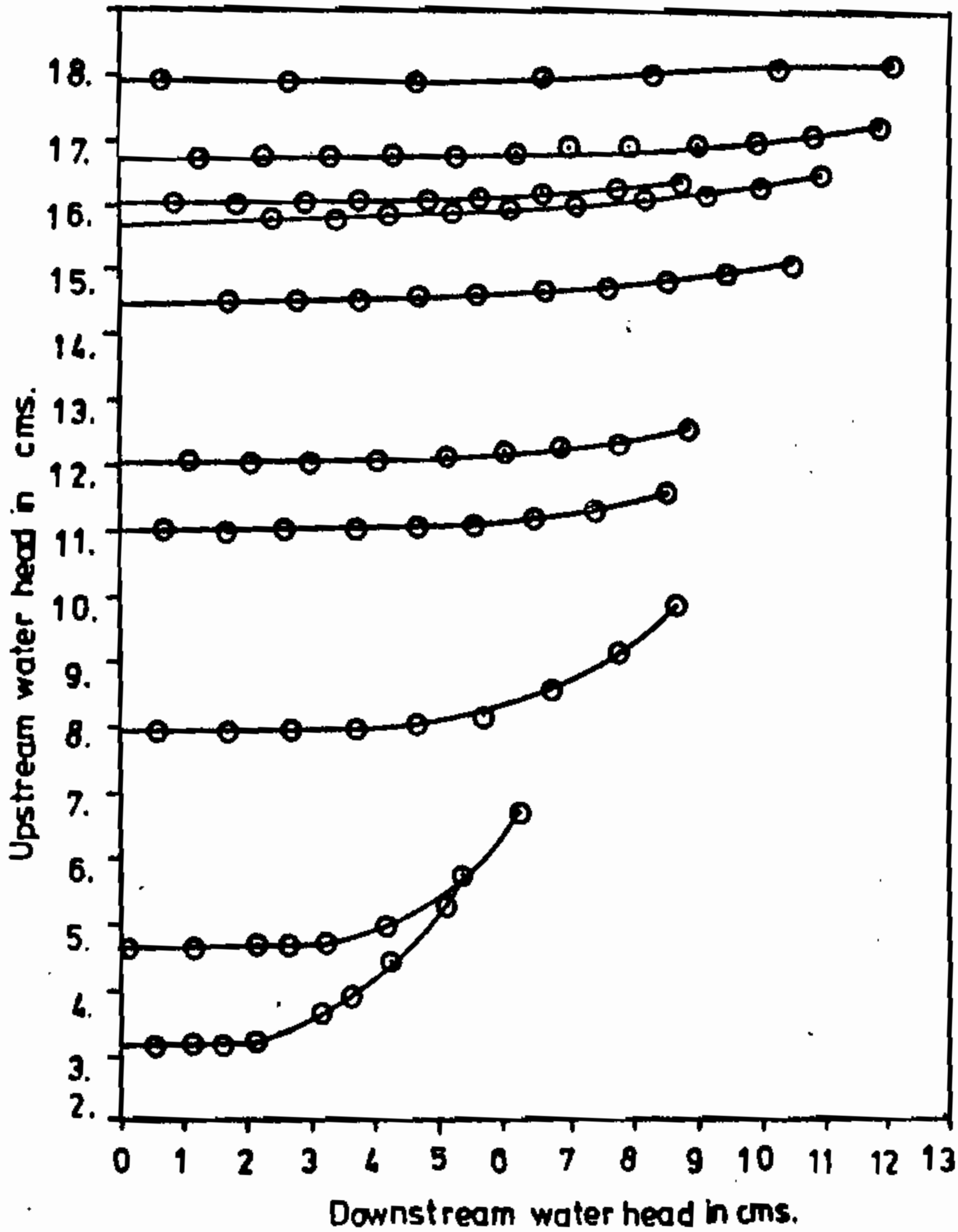
Furthermore, Fig.(15) demonstrates the whole situation where the ratio of submersion over the indicated standing wave weir was plotted against the upstream water head at the different named seven top slopes.

VI. CONCLUSIONS AND RECOMMENDATIONS

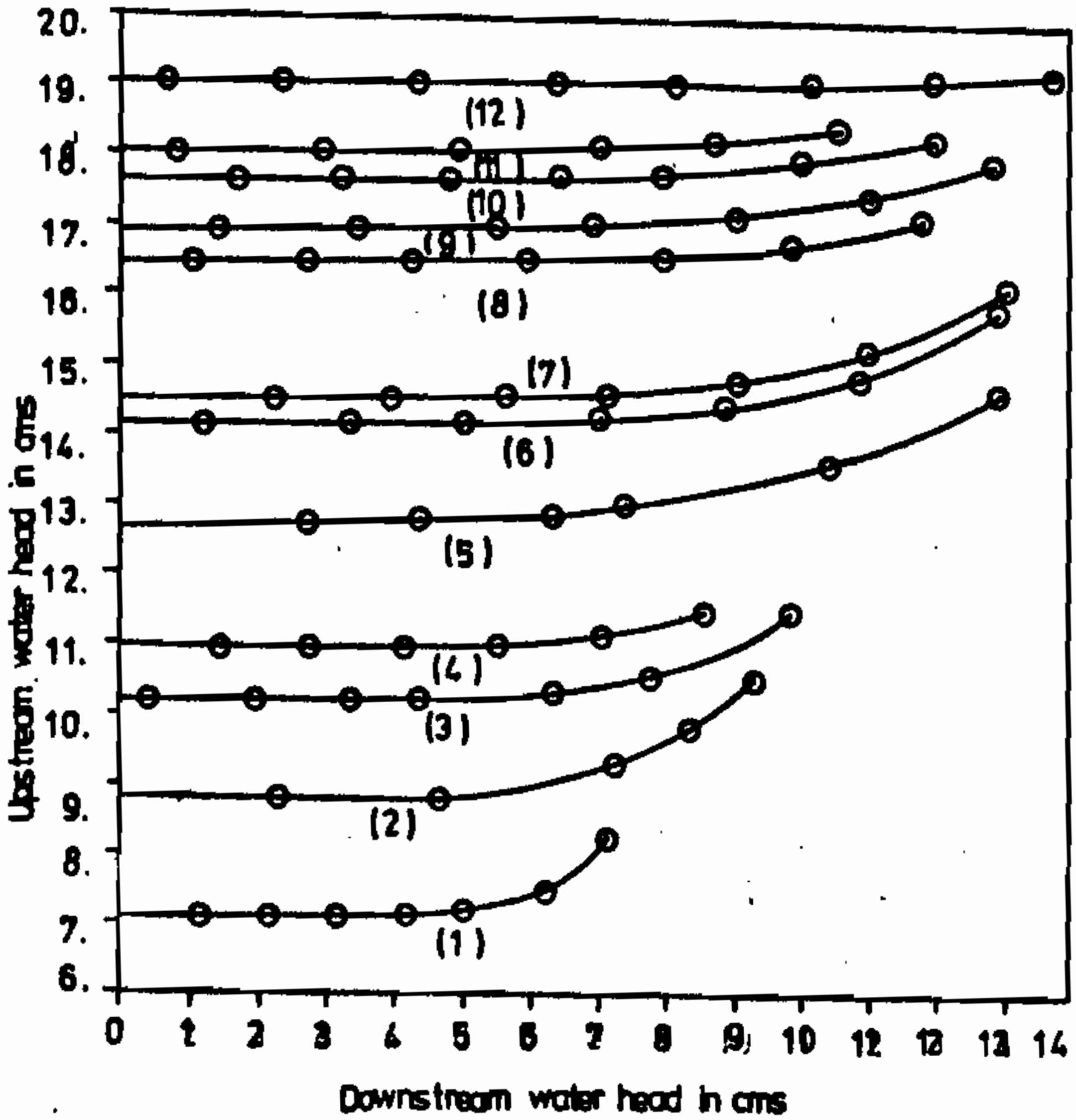
From the previously discussed results, the following conclusions and recommendations can be drawn.

1) There is a definite effect of changing the top slope of a standing wave weir on the submergency ratio and thus on both the flow behaviour and the flow rate.
2) In general, the submersion ratio is inversely proportional to the upstream water heads for any of the indicated top slopes (3:1, 4:1, 5:1, 10:1, 15:1, 20:1 & Zero :1).

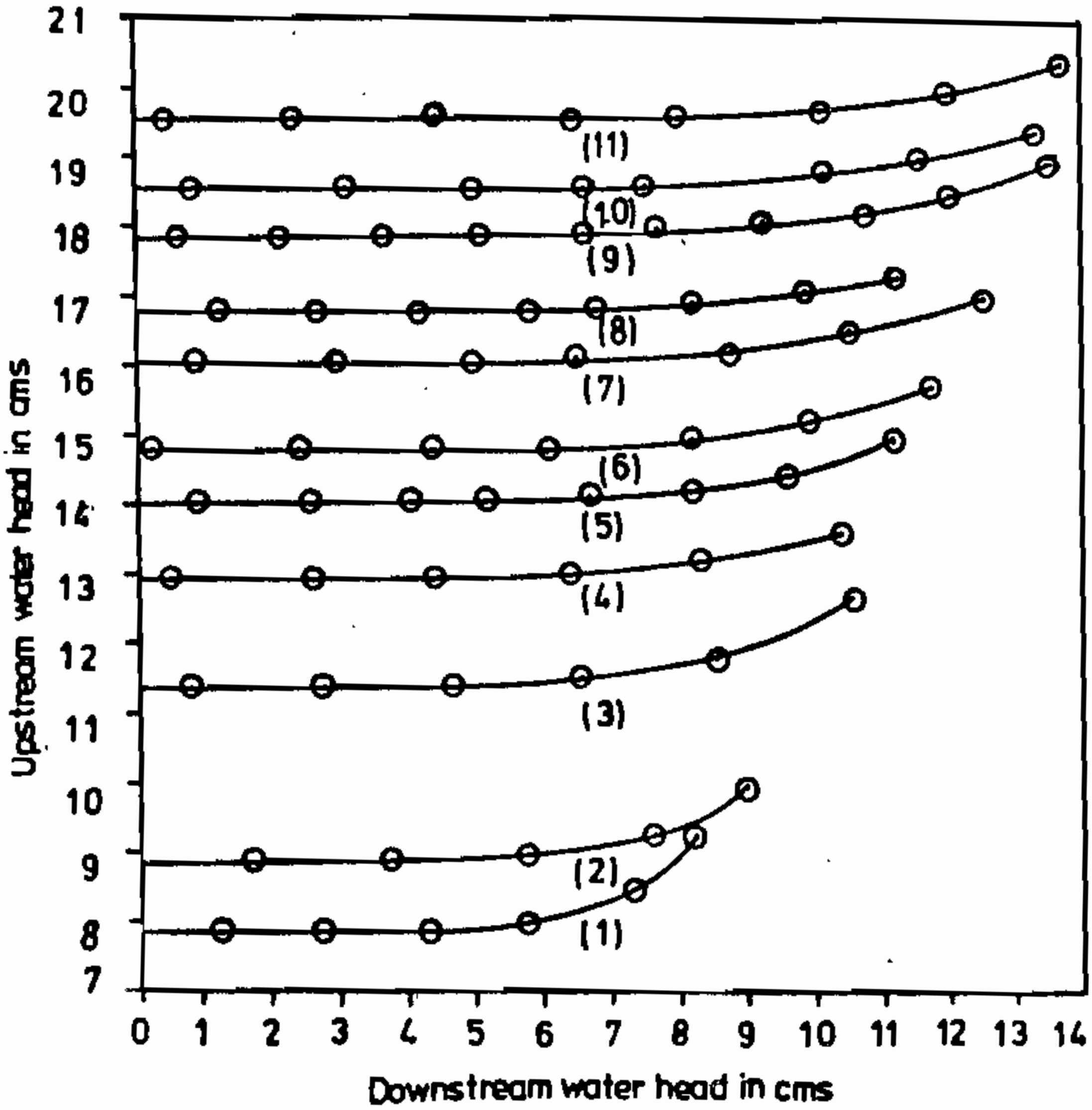
Such relationship is nonlinear and becomes more sensitive for smaller values of submersion ratio and upstream water heads and consequently at relatively lower values of flow rates. That defines the region where the weir action becomes effective, Moreover, the plotted curves describing these same relationships showed sort of independent trend as these variables



Fig(11) The relationship between the upstream and downstream water heads for top slope equals (5:1).



Fig(12) -The relationship between the upstream and downstream water head for top slope equals (20: 1).



Fig(13)- The relationship between the upstream and downstream water head for top slope equals zero.

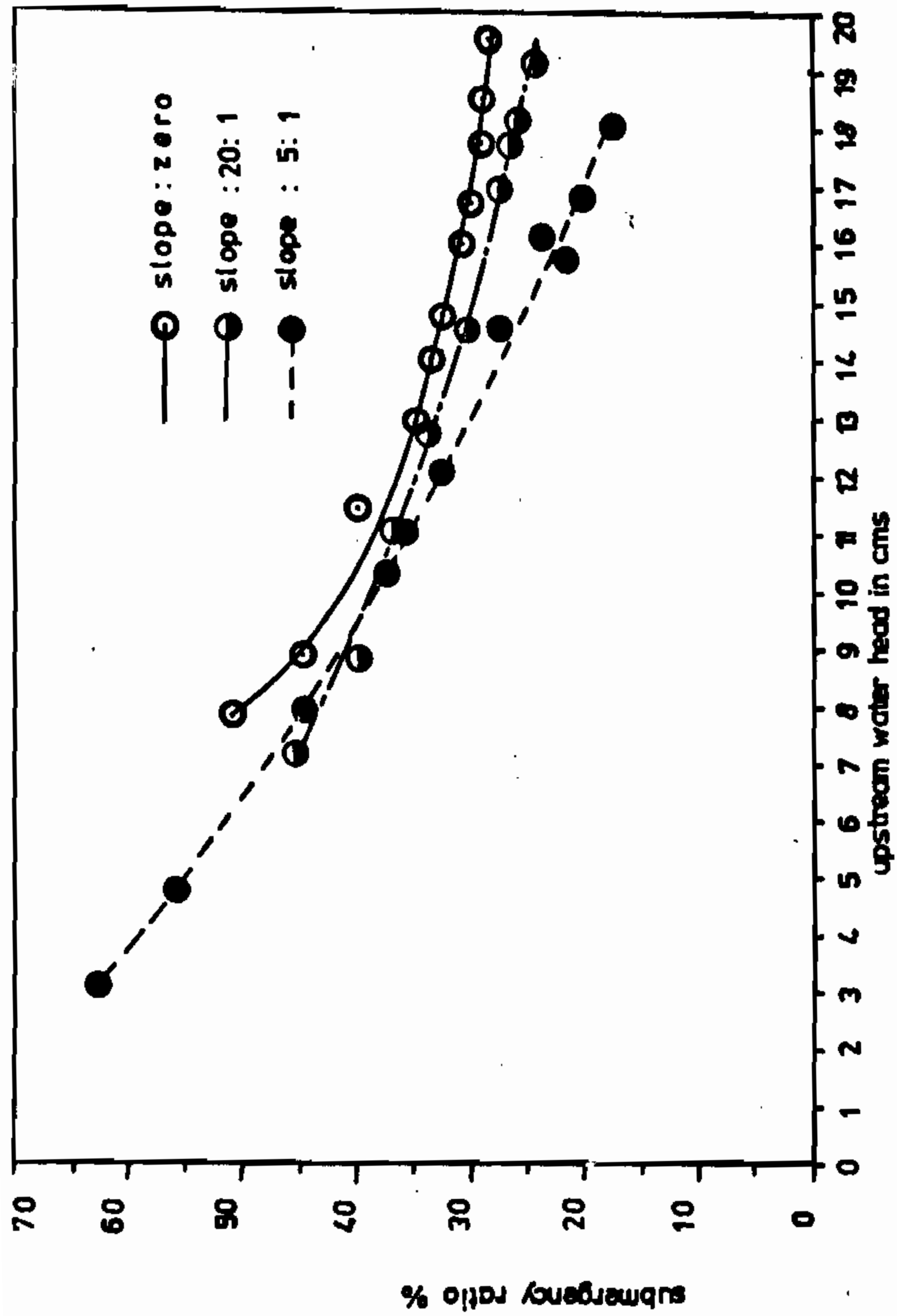
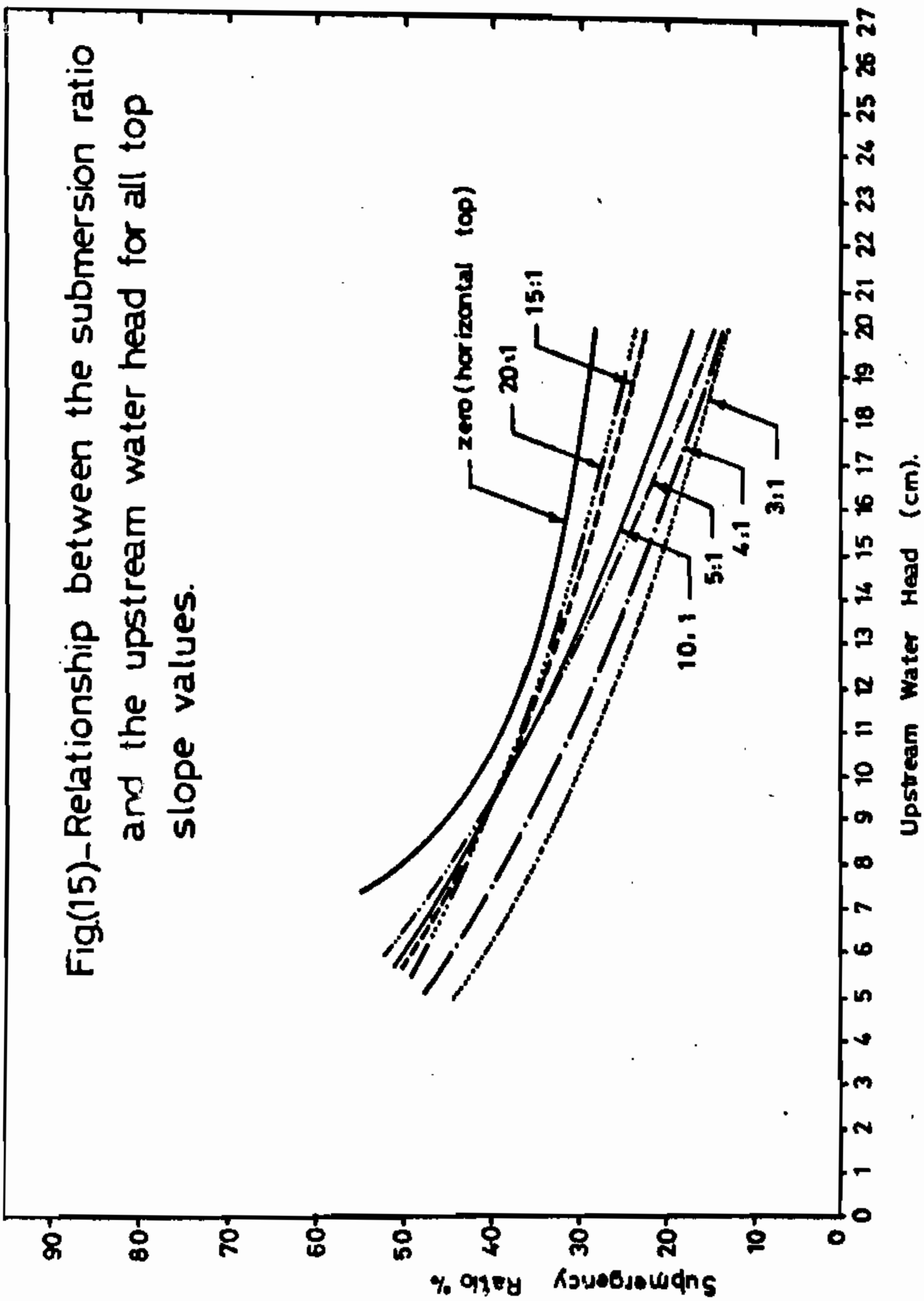


Fig (14) - The relationship between the submergency ratio and the upstream water head for different top slopes.



Fig(15)-Relationship between the submersion ratio and the upstream water head for all top slope values.

(mainly the upstream water depth and ratio of submersion) were increased.

3) For moderate and relatively higher values of upstream water heads, while established weir flows were really achived, standing wave weirs with horizontal top slope, (such weirs were acting then like broad crested weirs), experienced the highest values of corresponding submergency ratio followed by the 20:1 slope followed by the 15:1 then 10:1 to 5:1 to 4:1 till the 3:1 slope which showed the least accordingly.

The above domonstrates the significante role of the weir action and how this influence was reflected on other effective variables and its variational characteristic according to assigned variation of the top slope in a relative manner.

4) It was found out from the experiments undertaken in this research and during watching the flow characteristics, that the top slope 5:1 led a considerable stable flow conditions, relatively, when experienced small discharges up to a ratio of $\frac{12}{25}$ for the upstream water head to the height of the weir.

Meanwhile, the top slope 20:1 showed the best results for the relatively higher discharge values.

5) The present research study showed that it is advised not to construct a standing wave weir with a top slope steeper than 5:1.

6) It is recommended to extend this study to include rough surfaces tanding wave weirs and achieving higher values of submergency ratios.

7) It would be more beneficially for practical purposes if dynamically similar models of this sort can cover other step-wise multiple broken top slopes type, which can be

adjusted to fulfill better flow conditions that fit higher discharges under minimum heading up and the least head loss amongst all other cases. Such studies can be extended to include curved top surfaces too.

8)It is recommended to establish deterministic formulas that relate the rate of discharge with the submersion ratio along with the other variables involved, influenced by definite criterion, achieving standard set of design equations or fixed relationships which fully descriping the flow behaviour under the effect of such weir action. That would enable the designer to judge for the best condition always.

BILIOGRAPHY

1. Butcher, A.D., 1922: "Submerged weirs and Standing Wave Weirs", Delta Barrages Research center, Ministry of Irrigation, Egypt.
2. Chow, V.T., 1959: "Open Channel Hydraulics" McGraw-Hill Book Co., Inc., N.Y., U.S.A.
3. Clyde, C.G., G.V. Skogerboe and M.L. Hyatt, 1966: "Submerged Trapeziodal Meaeuring Flumes", A.S.C.E. Transactions, Vol. 9, 1966, N.Y. U.S.A.
4. Elmadani, M., 1957: "The self Adjusting weir", The Third Congress on Irrigation and Drainage, pp. 19-28, 1957, San Francisco, U.S.A.
5. Grover, N.C. and A.W. Harrington, 1966: "Stream Flow: Measurements, Records and their Uses", Dover publications Inc., N.Y., U.S.A.
6. Hanna, F.W. and R.C. Kennedy, 1938: "The design of Dams", Amercian Soc. of Civil Enge. pub., N.Y., U.S.A.
7. Institute of Hydraulics, Padova Univ., 1970: "Lecture on Hydrometry", The Fifth International Post-Graduate Course in Hydrology, Jan. 1970, Italy.

8. Israelsen, O.W. and V.E. Hansen, 1962: "Irrigation Principles and Practices", Third edition, John Wiley & sons, Inc., N.Y., U.S.A.

9. Konsoh, M.M., 1968: "Notes on Hydraulics, Fluid Mechanics and Water Power", parts II & III, pp. 71-77, Irrigation & Hydr. Dept., Alexandria Univ., Egypt.

10. King, H.W. and E.F. Brater, 1963: "Handbook of Hydraulics", McGraw-Hill Book Co., Inc., N.Y., U.S.A.

11. Leliavsky, S., 1950: "Design textbooks in Civil Engineering", Vol., II, pp. 111-116, Chapman and Hall Ltd., London, England.

12. Rao, S.S. and M.K. Shokla, 1971: "Characteristics of Flow Over Weirs of Finite Crest Width", Journal of Hydr. Div., A.S.C.E., Vol. 97, No. HY.11, pp. 1807-1816, Nov. 1971, N.Y. U.S.A.

13. Rouse, H., 1949: "Engineering Hydraulics", John Wiley & Sons, Inc., N.Y., U.S.A.

14. Singer, J., 1964: "Square-Edged Broad-Crested Weir as a Flow Measurement Device", Water & Water Engineering Journal, N.Y., U.S.A.

15. Woodburn, J.G. and A. Webb, 1932" (Experiments on Broad Crested Weirs)", Trans. A.S.C.E., Vol. 69, pp. 387, 1932, N.Y., U.S.A.

16. Zaky, Hassan, 1957: "Control and Distribution of Irrigation Water by Means of Hydraulic Structures in Egypt", Third Congress on Irrigation and Drainage, pp. 1-18, 1957, San Francisco, U.S.A.