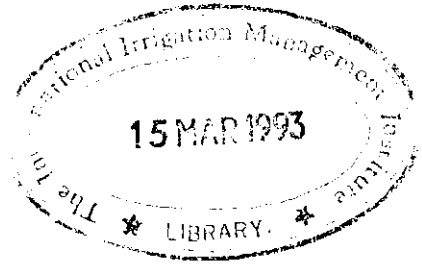
Flow Measurements at Drop Structures for Irrigation System Management in Sri Lanka



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Executive Summary

ONE OF THE objectives of the Irrigation Systems Management Project (ISMP) funded by USAID is to develop cost-effective methodologies for improving irrigation system operation and management through better water distribution and control. Flow measurements at salient points of an irrigation system form an important activity for better water control and flow regulation. As part of the ongoing research activity of ISMP undertaken by the International Irrigation Management Institute (IIMI), a subcontract was given to Lanka Hydraulic Institute (LHI) to carry out both laboratory and field measurement of water flow to calibrate flow-regulating drop structures in the four systems of Polonnaruwa District. A similar exercise of field flow measurements was carried out in the above schemes by a team of Technical Assistants as part of their training program under the direction of Prof. G. Skogerboe of Utah State University, U.S.A. Data were also collected from the Kirindi Oya Project in southern Sri Lanka. This wealth of carefully collected field data was analyzed and it has led to meaningful results. We have used all these data to suggest a flow measurement methodology which can be used to:

i. Carry out a systematic recalibration program of drop structures generally encountered in this country;

ii. Provide necessary precautions to be adhered to, before using calibration curves of drop structures for flow computations;

iii. Update periodically these calibration curves, in deteriorating systems and/or dislocation of gauge posts;

iv. Construct calibration curves for other drop structures of similar geometry;

v. Use the methodology as a training tool at the Sri Lanka Irrigation Training Institute at Galgamuwa.

Two methods are suggested in this publication which provide simplified and cost-effective procedures for calibrating drop structures and/or updating drop structure calibration curves. The suggested procedures do not require altering of irrigation flow for calibration purposes. By just measuring one single discharge and the corresponding head at the upstream gauge post, the methods suggested make it possible to update the already existing calibration curve and also facilitate obtaining a similar curve for drop structures also of similar geometry. The convenience of the proposed methods outweighs the sacrifice in rigor and precision; in any event, the resulting degree of accuracy is sufficient for most practical purposes.

A procedure is also outlined for using the methodology for training and demonstration in the field.

Acknowledgements

THIS STUDY IS based on an analysis of data collected through both laboratory and field tests conducted by the Lanka Hydraulic Institute (LHI), the Irrigation Department under the Irrigation Systems Management Project (ISMP) in Giritale and Parakrama Samudra schemes, and IIMI researchers in the Kirindi Oya Project. Grateful acknowledgement is made to the above organizations, and particularly to the following personnel for having allowed us to use their data: Mr. S. Sabanathan, Senior Research Engineer and Mr. N. Karunakaran, Additional Chief Research Engineer from the Lanka Hydraulic Institute, Prof. G. Skogerboe of Utah State, University, USA, who was a training consultant to ISMP, and Mr. B. R. Ariyaratne, IIMI Research Officer at Kirindi Oya.

One of the research objectives of ISMP funded by the United States Agency for International Development (USAID) is to develop cost-effective methodologies for improving irrigation system operation and management. When the idea of bringing out a publication for wider dissemination among the practicing professionals was put forward to the members of the Research Advisory Committee (RAC) of ISMP, they readily agreed and requested the professionals of the Irrigation Department to give their comments to be incorporated in the publication. Grateful acknowledgement is made to them for reviewing the initial draft of this publication. We thank Mr. Linton Wijesuriya, Senior Deputy Director for Rehabilitation and Modernization, for his active assistance in facilitating the study and review of earlier drafts.

Several of our IIMI and non-IIMI colleagues have reviewed the initial draft of this publication and have offered valuable comments. We express our thanks to all of them and particularly to the following: Dr. Hilmy Sally, IIMI, Burkino Faso, Dr. Senen Miranda, IIMI, Pakistan, Dr. Fred Valera, IIMI, Nepal, Prof. G. Skogerboe, Utah State University, Mr. Nihal Fernando, IMPSA Secretariat, and Mr. H.A. Karunasena, Research Associate, IIMI.

Our thanks are also due to Ms. Yvonne Fernando for her excellent typing and for the quick preparation of a number of revised versions of this draft.

We wish to pay special thanks to Mr. G.T. Jayawardena, the ISMP Director, who has been very supportive of IIMI's work, which he rightly insists should be directly useful to the project; and to Mr. Dan Jenkins, ISMP Officer, USAID, who took a strong interest in this study and critically reviewed various LHI draft reports and draft of this study.

Finally, we thank the United States Agency for International Development (USAID), Sri Lanka, which has supported this work under a Cooperative Agreement with IIMI (Agreement Number 383-0080-A-PG-7040-00).

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CHAPTER 1

Introduction

INCREASED DEMAND ON available water resources and ever-increasing development costs dictate that water be used economically with minimum wastage. Experience shows that economic use can be achieved only when water is measured. Water flow measurement in conveyance and distribution systems of irrigation projects, enables the maintenance of planned delivery schedules, the determination of the volume of water delivered, the singling out of anomalies and the detection of excessive conveyance, distribution and application losses.

Flow measurement is the cornerstone for improving hydraulic performance of an irrigation system. Modern management methods such as computer-based irrigation scheduling require that the flow of water at various points in an irrigation project be monitored systematically and at regular intervals. Flow monitoring and feedback through flow measurement form important processes for effective and efficient control, regulation and utilization of canal water.

Irrigation managers would like to know two types of information about water flow for efficient operation of a canal system. The first relates to the question of how much water a canal carries at a given point and at a given point of time. This information would be useful to determine the adequacy and equity of water supply, channel or constriction losses, etc. The second, addresses the question of level of water to be maintained in a canal section to pass a certain discharge. There are many methods by which this information can be obtained; however, the most widely used method in gravity irrigation systems is through the use of flow calibration curves obtained at control sections and/or measuring structures and, through the measurement of head causing flow at these structures.

Flow calibration curves or simply calibration curves, are graphical functional relationships between the rate of flow (discharge) and the head causing flow. This functional relationship is obtained by measuring various discharges and the corresponding heads causing the flow at the control/ measuring structures.

The discharge is usually measured using a current meter or calibrated measuring devices such as notches, weirs or flumes. The head causing flow both upstream and downstream of the structure at which measurement is made, is recorded in a gauge well or at a gauge post fixed in the flow direction. To prepare reliable calibration curves, careful field measurements are needed which are tedious, time-consuming and costly. Also, the calibrated curves must be updated periodically to account for changed system conditions.

In many large irrigation systems, to control and effectively monitor the water distribution, a large number of structures needs to be calibrated, and the calibration curves must be updated periodically to account for changed flow conditions and system configurations. Any method which simplifies the calibration procedure would be cost-effective; moreover, it should be easily accepted and adopted by field personnel engaged in flow measurement and monitoring. The method suggested in this manual provides a simplified procedure for calibrating and/or updating drop structure curves. The convenience of the proposed method outweighs the lack of rigor and precision, and in any event, the resulting degree of accuracy appears sufficient for most operational purposes.

The data used for developing this procedure were collected from both laboratory and field tests conducted by the Lanka Hydraulic Institute Ltd. (LHI), and the Irrigation Department under the Irrigation Systems Management Project in Giritale and Parakrama Samudra schemes in Polonnaruwa District. For details of the procedure adopted in collecting the data, please refer to the Final Report submitted by LHI to IIMI in June 1990. Additional data were also collected from the Kirindi Oya Project in southern Sri Lanka to test and verify the suggested methodology.

CHAPTER 2

Drop Structures

A NUMBER OF irrigation projects in Sri Lanka have installed drop structures along the main and branch canals and distributary channels due to undulating topography of the command areas. A variety of types of drop structures have been used in these irrigation systems. They differ widely in their shape, inlet and outlet conditions, hydraulic performance, and unit cost for comparable size and drop height. In many schemes, they are distributed throughout the command area with offtake outlets, generally located just upstream of these drop structures. Hydraulic measuring structures such as broad crested weirs, notches, flumes, etc., were found to be not well-received at times by the farmer beneficiaries; the farmers perceive these flow measuring structures as an obstruction and hindrance to the free flow of water in conveyance and distributary channels. Whenever we visited irrigation systems, we observed that there were more damaged measuring structures than other structures. Therefore, testing applicability of existing control structures such as drops for flow measurements could be a step in the right direction; moreover, it is cost-effective to use an existing control structure for flow measurement rather than to build one which is not only costly, but is not quite accepted by the farming community. In addition, flow measurements made at drop structures can be used for a variety of purposes such as computation of conveyance loss and offtake discharge, etc. The commonly adopted type of drop structures are Flumed, Ogee, Sheladia, Cascade and Rectangular Inclined. Even within a particular type, there are great variations with respect to the throat width, inlet and outlet conditions, energy dissipating arrangements and entrance conditions.

A brief description of the different types of drop structures used in Polonnaruwa Scheme is presented below. These are also the types commonly used in most of the Sri Lankan irrigation projects.

FLUMED DROPS

Flumed drop structures (like Parshall Flume) are widely used in many irrigation systems. For example, a total of 16 drop structures with throat widths of 1.25 *m* (5 *ft*), 1.83 *m* (6 *ft*) and 2.13 *m* (7 *ft*) are in use in Giritale Right Bank Main Canal over a distance of about 9 *km*. All of them are Flume type with slight differences in shape. When the dimensions of each structure were expressed as ratios of the respective throat widths, it was found that they could be classified into three distinct types. These are designated as type *A*, type *B* and type *C* in Figure 1 and are referred to as large drops because of the high discharge carrying capacity from 0.71 m^3/sec (25 *cusecs*) to 4.96 m^3/sec (175 *cusecs*).

OGEE DROPS

The rectangular inclined drop structure has an Ogee type sill section to link the upstream and downstream canal beds (Figure 2). On the upstream side it has a straight headwall. A large number of drop structures of this type are in use at Giritale and Parakrama Samudra schemes.

It was found possible to group these structures into three distinct lots with the following basic parameters:

i. $\frac{B}{BW} = 1.0$ and $\frac{X}{B} = 0.10$

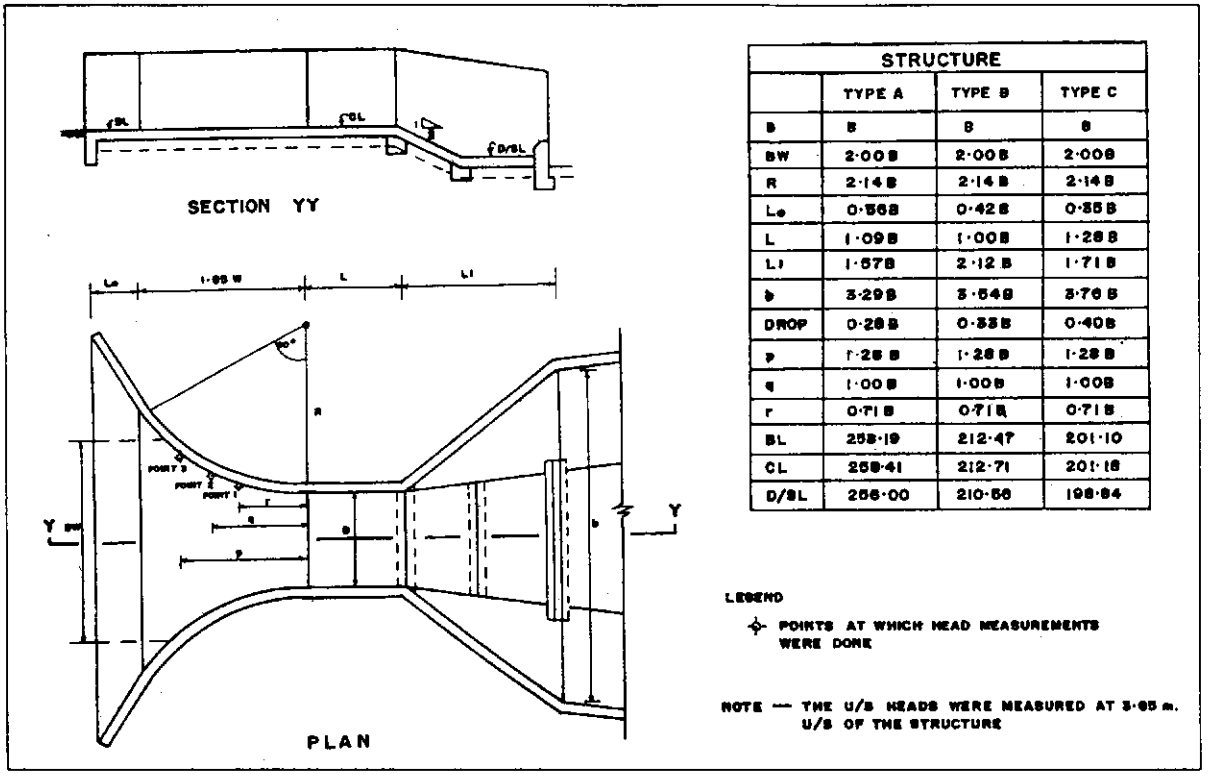
ii. $\frac{B}{BW} = 0.9$ and $\frac{X}{B} = 0.11$

iii. $\frac{B}{BW} = 0.9$ and $\frac{X}{B} = 0.125$

where B = throat width of the drop structure,
BW = bed width of the canal, upstream of the structure,
X = the height of the sill of the drop structure, above the bed level of the canal.

The maximum designed carrying capacity is about 0.85 m3/sec (30 cusecs).

Figure 1. Giritale large drop (Flumed drop).



STRUCTURE			
	TYPE A	TYPE B	TYPE C
B	B	B	B
BW	2·00B	2·00B	2·00B
R	2·14B	2·14B	2·14B
Le	0·56B	0·42B	0·35B
L	1·09B	1·00B	1·28B
L1	1·57B	2·12B	1·71B
b	3·29B	3·54B	3·76B
DROP	0·28B	0·33B	0·40B
p	1·28B	1·28B	1·28B
q	1·00B	1·00B	1·00B
r	0·71B	0·71B	0·71B
BL	258·19	212·47	201·10
CL	258·41	212·71	201·18
D/SL	256·00	210·56	198·84

SHELADIA DROPS

This type of drop structure was designed by Sheladia Associates, the consultants to the Irrigation Systems Management Project (ISMP) in Polonnaruwa District under the USAID-funded project. It is a simple design with a vertical drop and a straight head wall (Figure 3). Designs are available for drops ranging from 25 *cm* (10 *inches*) to 125 *cm* (4 *ft*). The maximum designed capacity is 1.13 m^3/sec (40 *cusecs*). Design details of this drop structure are given in Figure 3. Some of the old damaged drop structures in the Polonnaruwa schemes are being replaced with this new type.

CASCADE DROPS

This type of drop structure with alternate vertical drops and horizontal steps is in use for drops of 30 *cm* (1 *foot*), 60 *cm* (2 *ft*), 90 *cm* (3 *ft*) and 120 *cm* (4 *ft*) and throat widths ranging from 45 *cm* (1.5 *ft*) to 83 *cm* (2.75 *ft*). The maximum design discharge through this type of structure is about 0.85 m^3/sec (30 *cusecs*). Details of this drop are presented in Figure 4.

RECTANGULAR INCLINED DROPS

These large rectangular inclined drop structures are found along D-1 East Main Canal of the Parakrama Samudra Scheme. The largest of these has a throat width of 366 *cm* (12 *ft*). The maximum carrying capacity is about 7.08 m^3/sec (250 *cusecs*). The large drops in Parakrama Samudra Scheme are grouped into three different types - Type I, Type II and Type III as shown in Figure 5.

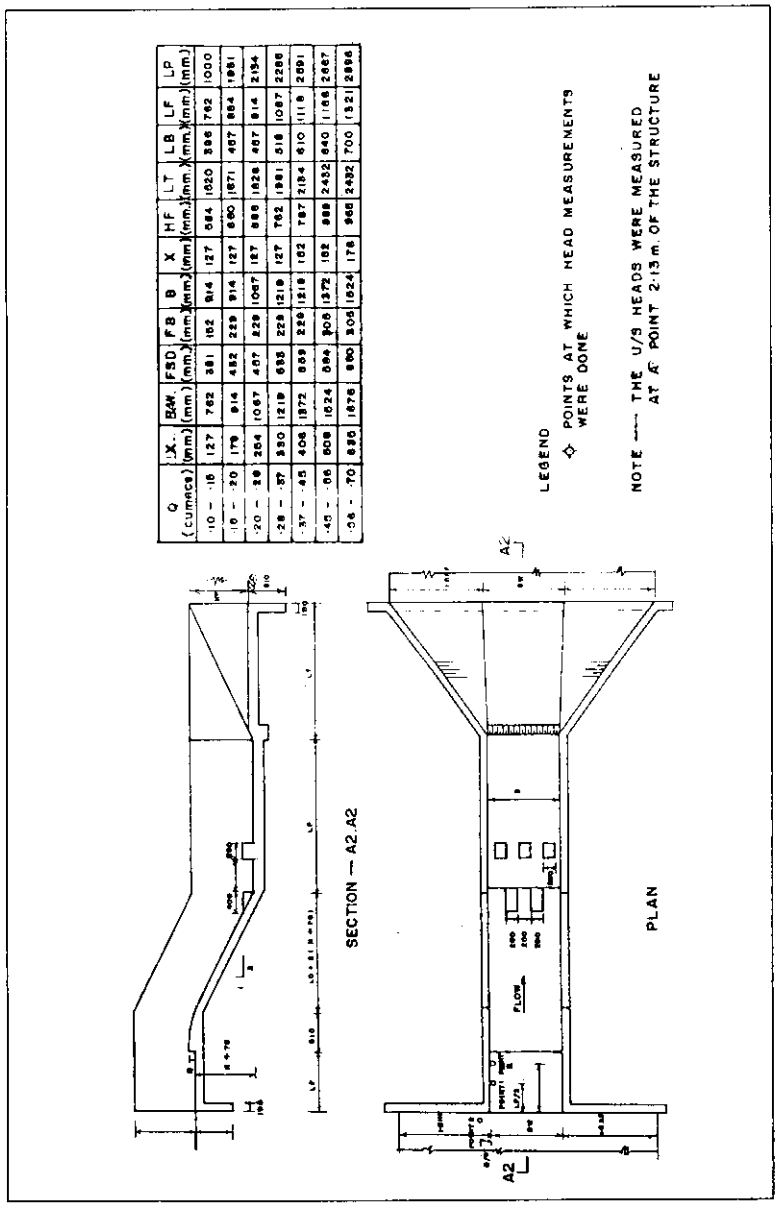
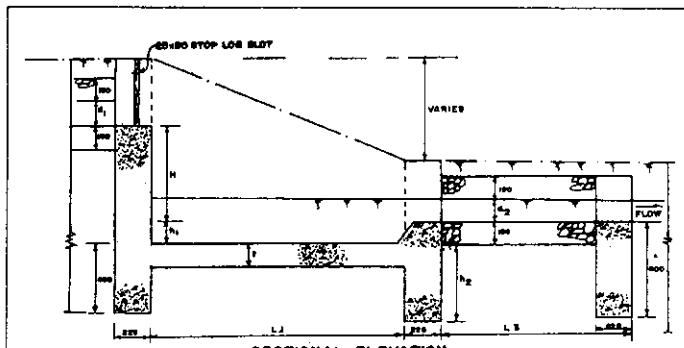


Figure 2. Ogee drop.

Figure 3. Sheladia drop.



H mm	h1 mm	h2 mm	L1 mm	L2 mm	L3 mm	t mm
150	150	450	1200	1200	1200	150
225	150	450	1200	1200	1200	150
300	225	450	1200	1200	1500	150
500	225	600	1750	1500	1500	150
750	300	650	2000	1500	2000	200
1000	400	600	2250	1600	2000	200
1250	400	650	2500	1500	2000	225

LEGEND

◇ POINTS AT WHICH HEAD MEASUREMENTS WERE DONE

NOTE — THE U/S HEADS WERE MEASURED AT A POINT 2500 mm. U/S OF THE STRUCTURE

DROPS AS FLOW MEASURING STRUCTURES

One of the great advantages of using drop structures for flow measurements is that there is no downstream submergence effect and therefore, it is sufficient to measure the upstream head alone for discharge computation.

An ideal place to operate head measurements is at the throat of a drop structure; however, due to the extreme curvilinear nature of the critical flow condition at the throat section, and also due to the practical difficulty of measuring the depth accurately at the constricted section of the throat, it is not convenient to measure the head at the throat. Therefore, head measurements are made upstream of the throat and especially in the transition zone (if one exists) where, due to the accelerated nature of flow, very little sediment deposition takes place and the transition or head walls provide a convenient location to fix or mark a gauge post. Gauge wells are not normally found in these structures since these were not originally designed for flow measurement.

The head causing flow is computed with reference to the sill level of the throat, and therefore while fixing the gauge post, the zero of the scale is placed to coincide with the sill level at the throat section. The calibration curve is very sensitive to the location at which the upstream head is measured and therefore, location of the upstream head measurement must be selected carefully and should be free of any disturbance. In addition, the calibration curve of a drop will change with the change of location of head measurement. Therefore, to make use of the same calibration curve, the location of the upstream head measurement, once selected, should not be changed.

During field visits, it has been observed that in single banked canals, very often the gauge markings in the upstream wing walls are smeared with mud and are illegible. To overcome this difficulty, plastic scales with permanent markings can be fixed on the walls, facilitating cleaning and washing of the scales before measurement. Sometimes, the wall over which the gauge post is marked or fixed is not vertical; the verticality of the wall is to be checked before fixing the scale. Many a time the gauge post was found displaced and/or missing. When a new gauge post is fixed, there is a need to check and update the calibration curve. If B is the width of transition, a distance beyond $2B$ from the drop is a convenient location to fix a gauge post.

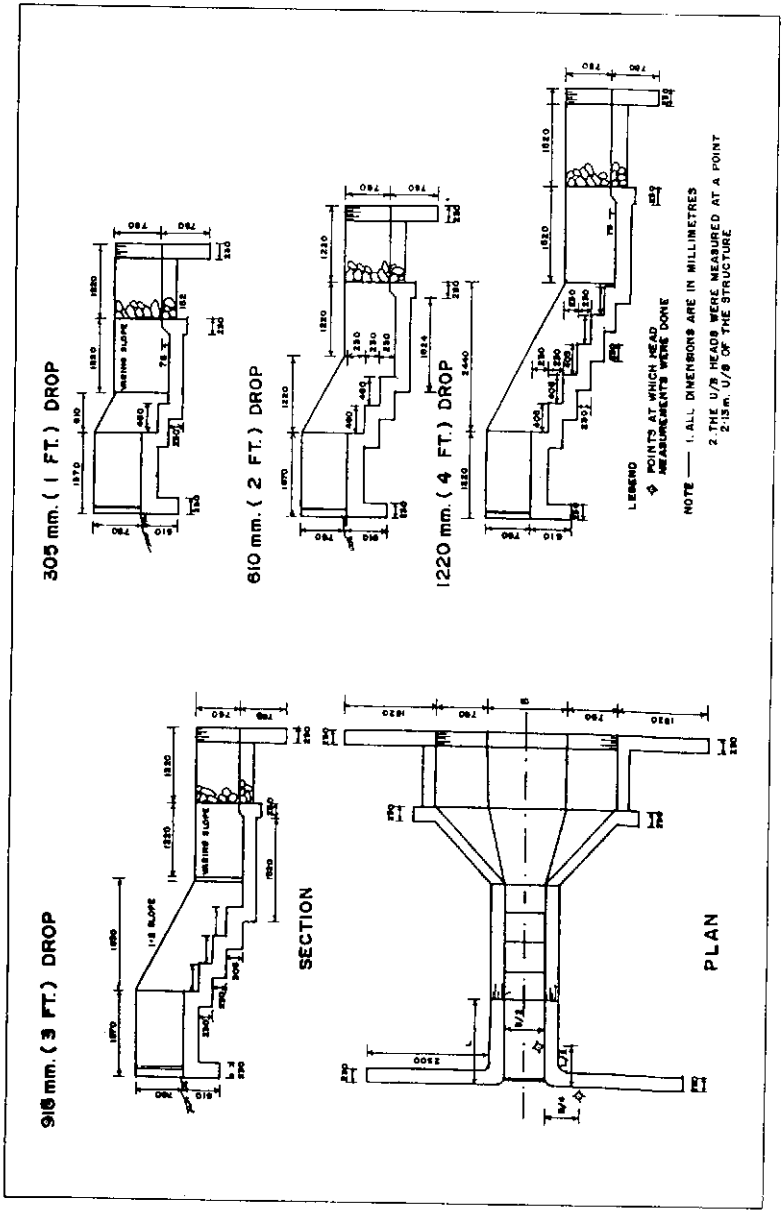


Figure 4. Cascade drop.

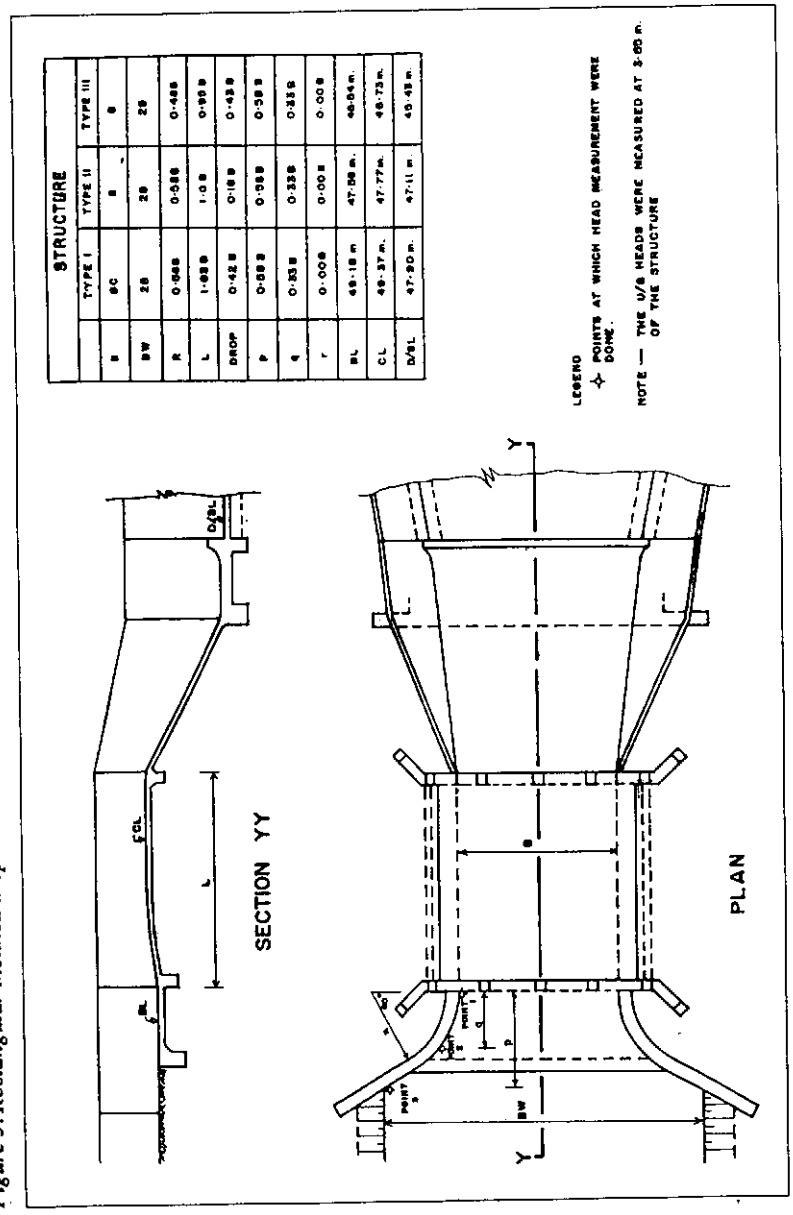


Figure 5. Rectangular inclined drop.

CHAPTER 3

Laboratory and Field Data

DATA COLLECTION

ALL THE DROP structures listed in Chapter 2 were model-tested in the laboratory, using a flume 80 *cm* wide and 12 *m* long. Details of the dimensions of the drop structures chosen for the study and the linear scale employed are given in Table 1.

Each model drop structure was tested for seven discharges within the prototype discharge ranges listed below:

Type of drop structure	Discharge range	
	From	To
1. Giritale — large drops	0.71 m^3/sec (25 *cusecs*)	4.96 m^3/sec (175 *cusecs*)
2. Rectangular inclined drops	0.20 m^3/sec (7 *cusecs*)	0.85 m^3/sec (30 *cusecs*)
3. Sheladia type drops	0.10 m^3/sec (3.5 *cusecs*)	1.13 m^3/sec (40 *cusecs*)
4. Cascade type drops	0.20 m^3/sec (7 *cusecs*)	0.85 m^3/sec (30 *cusecs*)
5. Large drops — PSS	0.55 m^3/sec (20 *cusecs*)	4.27 m^3/sec (150 *cusecs*)

The flow measurements were made on a 90 degree 'V' notch. The water level measurements were monitored at selected points along the upstream using wing wall/head wall of the drop structures for calibration.

Table 1. Drop structures chosen for model study using model scales.

Type of drop structure	Model scale (linear)	Throat width (prototype)	Remarks
Giritale large			
Type *B*	1:12	2.13*m* (7 *ft*)	Refer Figure 1 for details
Type *C*	1:12	2.13*m* (7 *ft*)	Dimensionally, Type *A* was a combination of Types *B* and *C*.
Rectangular			
inclined	1:6.6	1.68*m* (5.5 *ft*)	$\frac{B}{BW} = 1.0; \frac{x}{BW} = 0.1$
	1:6.6	1.68*m* (5.5 *ft*)	$\frac{B}{BW} = 0.9; \frac{x}{B} = 0.11$
	1:6.6	1.68*m* (5.5 *ft*)	$\frac{B}{BW} = 0.9; \frac{x}{B} = 0.125$ where B = throat width, BW = bed width, x = sill height above bed level
Sheladia	1:8	1*m* (3.28 *ft*)	Bed width = 2*m* (6.56 *ft*) Drops = 1,250*mm* (4 *ft*) and 500*mm* (1.6 *ft*)
Cascade	1:6.6	0.84*m* (2.75 *ft*)	Bed width = 1.68*m* (5.5 *ft*)
PSS large Types *A*, *B* and *C*	1:12	3.66*m* (12 *ft*)	Refer Figure 5 for information on the distinguishing features of Types *A*, *B* and *C* which are the same as Types I, II, and III, respectively.

Table 2. Comparison of Q-H relationships obtained for the drop structures, using model scales and field measurements (dimensionally compatible equation is used).

Type of drop structures	Q versus H (using model scales)	Q versus H (field measurements)	Remarks
Giritale large			
Type *A*	$Q = 1.706B.H^{1.5}$	$Q = 1.651B.H^{1.5}$	$B = 2.16m$
Type *B*		$Q = 1.712B.H^{1.5}$	$B = 1.83m$
Type *C*		$Q = 1.775B.H^{1.5}$	$B = 1.524m$
Rectangular inclined	$Q = 1.767B.H^{1.5}$	$Q = 1.688B.H^{1.5}$	$B = 1.25m$
Sheladia	$Q = 1.672B.H^{1.5}$	$Q = 1.790B.H^{1.5}$	$B = 1.25m$
Cascade	$Q = 1.417B.H^{1.5}$	$Q = 1.381B.H^{1.5}$	$B = 0.96m$
PSS large			
Type *A*	$Q = 1.65B.H^{1.5}$	$Q = 1.59B.H^{1.5}$	$B = 3.65m$
Type *B*			$B = 3.72m$
			$B = 3.15m$

Field calibration of drop structures given in Table 2 was carried out during the *maha* (wet season) 1989/90 and *yala* (dry season) 1990. For this purpose, nine locations with well-defined flow sections were selected, and velocity measurements were made using:

i. OTT propeller type current meter, with a blade diameter of 4 *cm* for depths of flow less than 50 *cm*; and

ii. KHALSICO propeller type current meter, with a blade diameter of 10 *cm*, for depths of flow greater than 50 *cm*.

The velocity measurements and discharge computations were made as per the method outlined in Annexure 1.

ANALYSIS

Both laboratory and field measured discharge data, and the corresponding heads for different types of drop structures were plotted separately on log-log paper with head along the X-axis and discharge along the Y-axis (Figures 6–11).

From these plots, the following observations were made:

i. For all types of drop structures, the plot of both laboratory and field data can be fitted with a linear line on a log-log paper;

ii. Two types of equations were fitted for the straight lines; the first one which is dimensionally compatible is of the form $Q = CBH^{3/2}$ .....(1)

where Q = discharge (m^3/sec).

B = throat width (m).

H = upstream head measured with respect to the sill level of drop structures at the throat (m).

C = constant of proportionality which depends on a number of factors, the most important of which are: location of upstream head measurement, geometry of drop structure, roughness condition of the bed and sides, and entrance flow conditions.

The second equation is of the form $Q = KH^n$ where K and n are parameters determined by the method of least squares.

The numerical forms of both the above equations for all the tested drops are almost similar; a typical set of equations obtained for Giritale large drops is given below:

$Q = 1.7258\, BH^{1.536}$
$Q = 1.7064\, BH^{1.5}$

Figure 6a. Field calibration: Rectangular inclined drop (B = 1.25;C = 1.688).

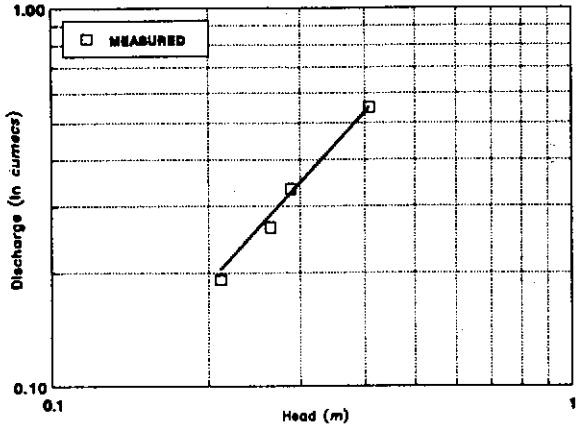


Figure 6b. Field calibration: Cascade drop (B = 1.25 ; C = 1.19).

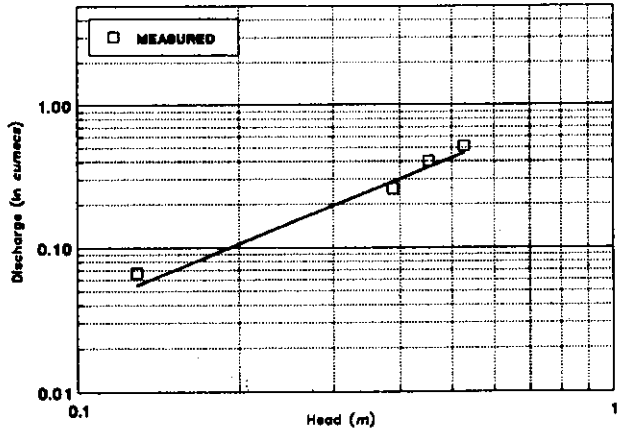


Figure 7a. Field calibration: Giritale large drop - type C (B = 1.524m ; C = 1.776).

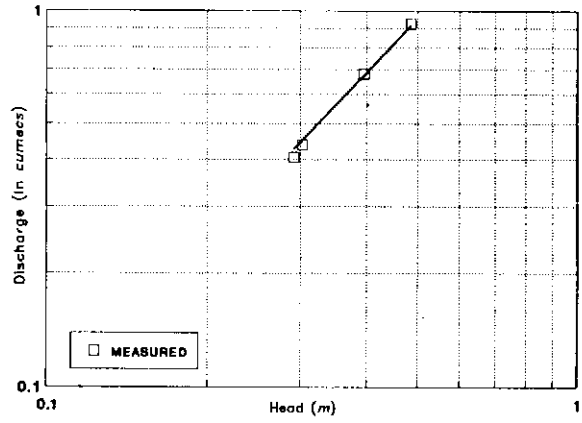


Figure 7b. Field calibration: Sheladia drop (B = 1.25 ; C = 1.790).

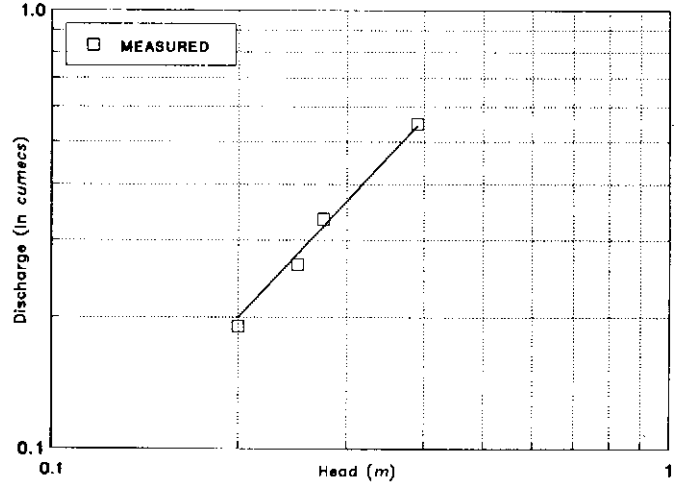


Figure 8a. Field calibration: Giritale large drop - type A (B = 2.16 ; C = 1.651).

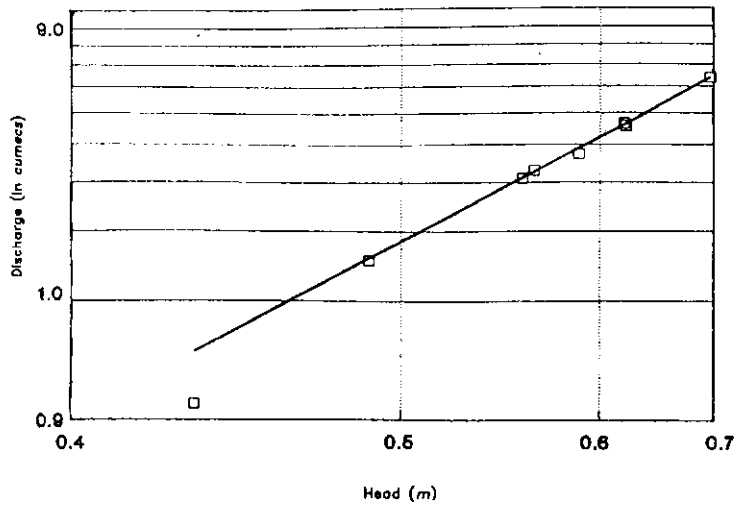


Figure 8b. Field calibration: Giritale large drop - type B (B = 1.83 ; C = 1.712).

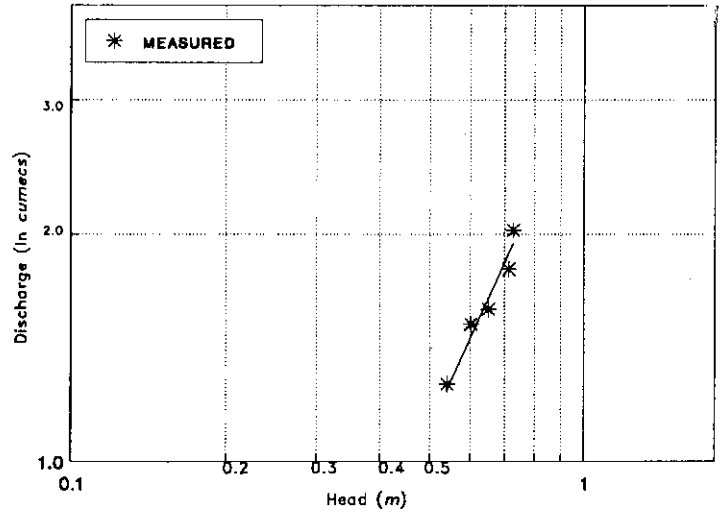
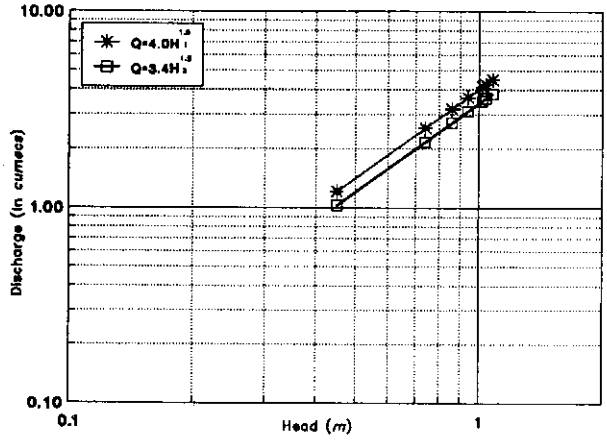


Figure 9. Giritale large drop - Flume type: Model tests (Throat width = 2.16 m).



Note: H measurements at two independent points U/S of the throat.

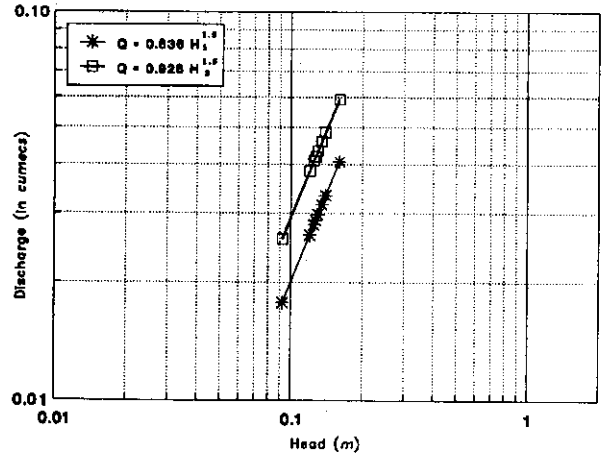
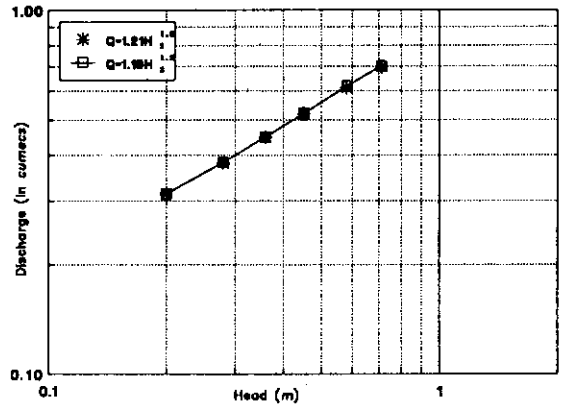
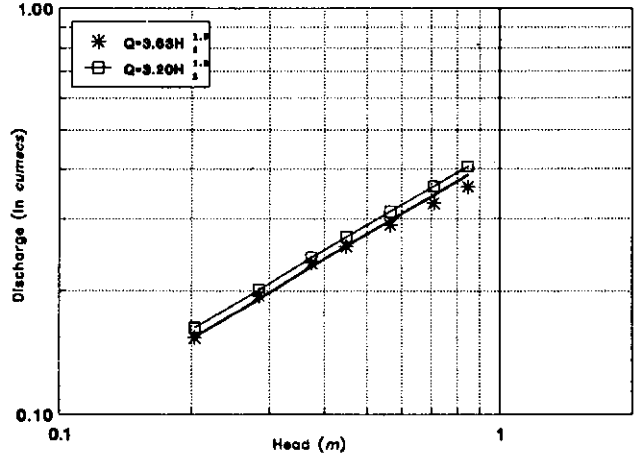


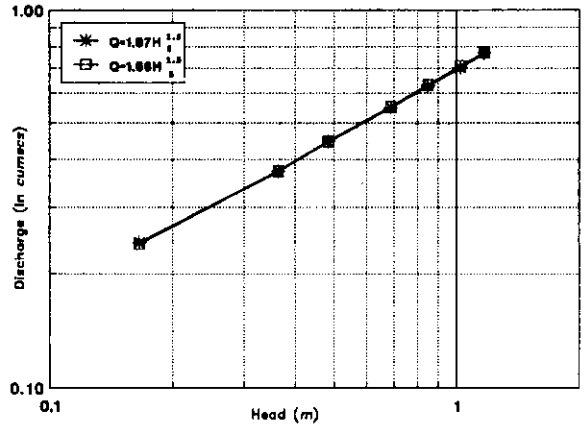
Figure 10. Cascade drop: Model tests.





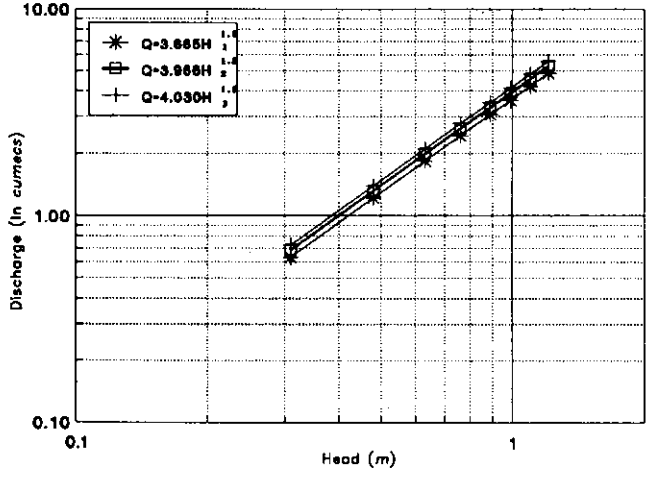
Note: H_1 and H_2 refer to different points of upstream head measurements (refer to Figure 4).

Figure 11a. Giritale large drop - type B: Model test (B=23).



Note: H_1 and H_2 in both figures refer to different points of upstream head measurements.

Figure 11b. Giritale large drop - type C: Model results.



A sensitivity analysis of variation between these two equations indicates that for normal operating heads, these two equations do not differ by more than 5 percent. Therefore, it is decided to use a dimensionally compatible equation (equation 1) for all further analyses.

iii. The value of C in equation (1) differs for each type of drop structure.

iv. For a particular type of drop structure, the same relationship given in equation (1) holds good for heads measured at different upstream locations but with a differing value of C (Figure 12).

v. Laboratory experiments conducted with different roughnesses, entrance conditions, and non-verticality of rectangular throat walls indicate that equation (1) can still be used but with differing values of C. The variation of C in these cases were within $\pm$ 10 percent from that of the original condition (Figures 13 and 14).

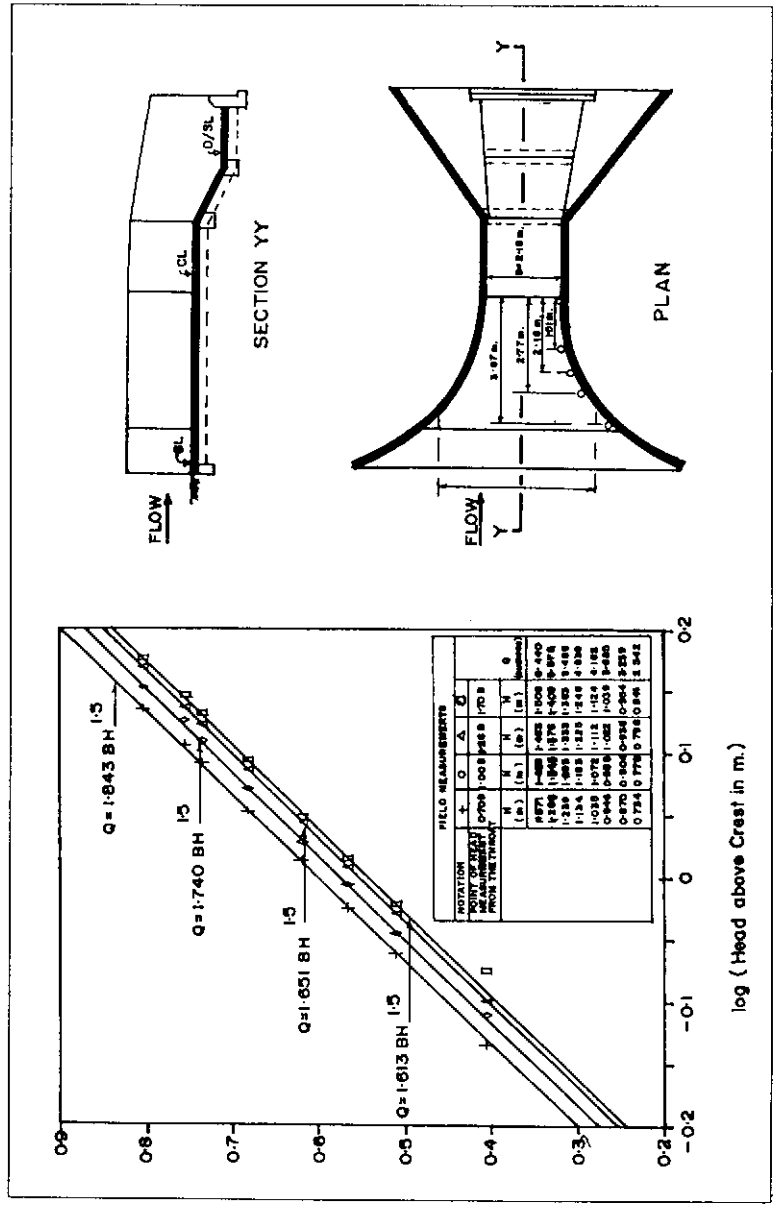


Figure 12. Q-H relationships for different points of head measurements: Giritale large drop - type A (B = 2.16).

Figure 13a. Q-H relationships for different entry conditions: Giritale large drop — type C.

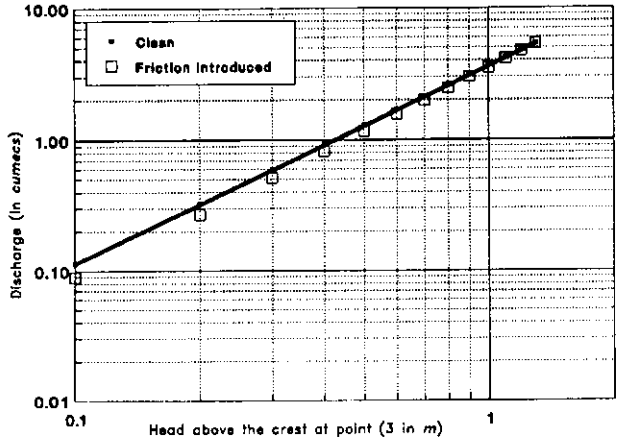


Figure 13b. Q-H relationships for different entry conditions: Rectangular inclined drop.

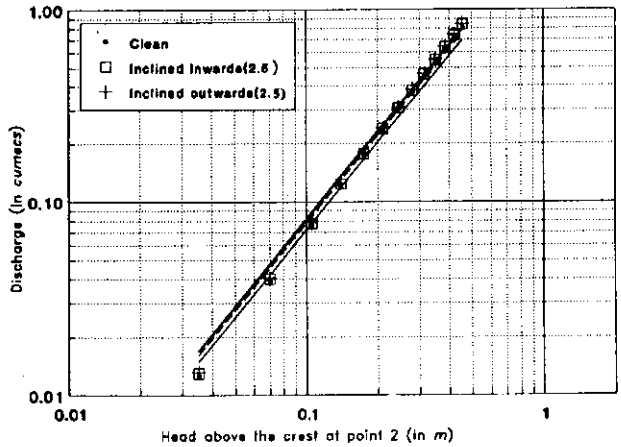


Figure 14a. Q-H relationship for different entry conditions: Rectangular inclined drop (effect of U/S siltation).

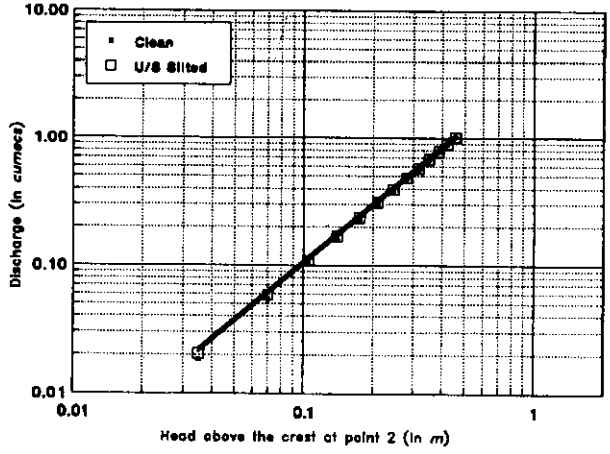
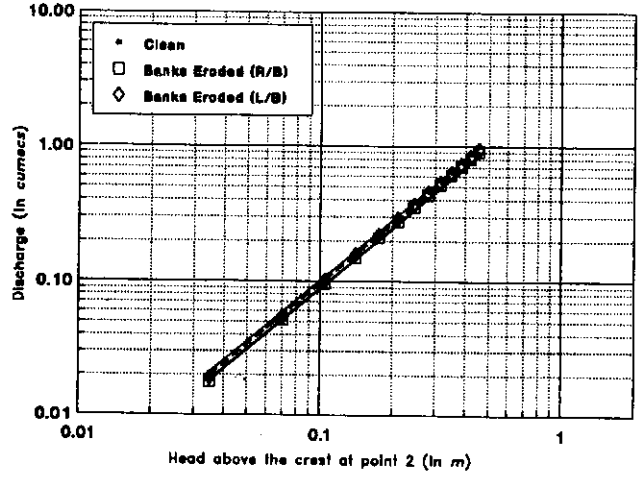


Figure 14b. Q-H relationship for different entry conditions: Rectangular inclined drop (effect of U/S erosion).



CHAPTER 4

Discussion of Results

SINGLE AND MULTIPLE POINT CALIBRATIONS

FOR DROP STRUCTURES that are commonly used in irrigation systems in Sri Lanka we have shown that the head-discharge relationship can be expressed by equation 1. This equation is now made use of in calibrating and/or updating the calibration curves of ungated drop structures under free flow (unsubmerged) conditions which have a horizontal or flat sill or crest with vertical walls at the sill. Two procedures can be adopted for calibration. They are designated as single point and multiple point calibrations.

The multiple point calibration is a conventional method wherein flow discharges are measured for a series of upstream heads; then the measured discharge, Q, is plotted against head, H, on a log-log paper; the plotted points are fitted with a straight line having a slope of 3/2. The best fit value C can then be computed.

On the other hand, since we know that equation 1 with a known exponent of 3/2 has only one unknown, that is C, it is sufficient to have one discharge measurement with the corresponding upstream head to compute the value of C. The value of C so computed by one set of data would provide equally reliable results if the single set of discharge and head measurements are made with utmost care and precision. This method of computing C based on a single set of measurement is called single point calibration.

INTERPRETATION OF RESULTS

Let us discuss the advantages and disadvantages of using the single point calibration for establishing and/or updating a calibration curve of a drop structure against the multiple point calibration. In many irrigation schemes,

altering the discharge for calibrating a structure during a crop growing season is difficult. Farmers resent intervention as it disrupts their supply. It is a time-consuming and costly operation. It also introduces a state of unsteady condition in the channel which takes a long time to attenuate to a steady state. On the other hand, one can question the accuracy of C computed by single point calibration. To test the variability of C (computed with a single set of measurement) against the best fit C value, the value of C was computed for each set of field measurements carried out by the LHI in the Polonnaruwa Scheme. The computed values of C together with the maximum positive and negative percent deviations of C from the best fit value are presented in Table 3. It can be seen from Table 3 that the maximum positive and negative percent deviations are within 10 percent for all the sets of data tested, except for one which represents a flow with a low head. From this it is concluded that if one can allow deviation up to a maximum of ± 10 percent from the best fit value of C, then the single point calibration can be used to calibrate and/or update a calibration curve.

In view of the large number of drop structures to be calibrated and updated, it is suggested that the single point calibration method be tried to minimize cost and management effort; however, if such a method is used, utmost care and precaution must be taken to obtain a single set of discharge and head measurements preferably under repeated trials at or near the full supply discharge.

Table 3. Variation of C by the single point calibration against best fit value.

Serial No.	H (m)	$H^{3/2}$	Q (m3/s)	Q/unit width	C by single point calibration	Maximum positive and negative percent variation of C with respect to "best fit value"
Giritale large drop - Type A. B = 2.16. Best fit value of C = 1.651.						
1	1.488	1.815	6.625	3.067	1.690	+ 2.36
2	1.263	1.420	4.995	2.313	1.629	
3	1.258	1.411	5.065	2.345	1.662	
4	1.149	1.232	4.233	1.960	1.591	- 3.63
5	1.049	1.074	3.852	1.783	1.660	
6	1.024	1.036	3.661	1.695	1.636	
7	0.749	0.648	2.272	1.052	1.623	
8	0.519	0.374	0.986	0.457	1.222	-25.98*

(Table 3. Continued)

Serial No.	H (m)	$H^{3/2}$	Q (m3/s)	Q/unit width	C by single point calibration	Maximum positive and negative percent variation of C with respect to "best fit value"
Giritale large drop - Type B. B = 1.83. Best fit value of C = 1.712.						
1	0.728	0.621	2.020	1.104	1.778	
2	0.713	0.602	1.798	0.983	1.633	–4.61
3	0.651	0.525	1.591	0.869	1.655	
4	0.601	0.466	1.519	0.830	1.781	+4.03
5	0.541	0.398	1.266	0.692	1.738	
Field calibration - Giritale large - drop - Type C. B = 1.524 m. C = 1.775.						
1	0.485	0.338	0.925	0.607	1.796	+1.18
2	0.396	0.249	0.680	0.446	1.791	
3	0.303	0.167	0.439	0.288	1.725	
4	0.292	0.158	0.406	0.260	1.646	–7.26
Field calibration - Sheladia B = 1.25. C = 1.790.						
1	0.200	0.089	0.191	0.1528	1.709	
2	0.250	0.125	0.264	0.2112	1.690	–5.58
3	0.275	0.144	0.334	0.2672	1.855	+3.63
4	0.390	0.243	0.549	0.4392	1.807	
Field calibration - Rectangular inclined drops B = 1.25. C = 1.688.						
1	0.211	0.0968	0.191	0.1528	1.579	
2	0.263	0.1349	0.264	0.2112	1.566	–7.22
3	0.288	0.1547	0.334	0.2672	1.727	+2.31
4	0.408	0.2607	0.549	0.4392	1.685	
Field calibration - Cascade type drops B = 0.96. C = 1.381.						
1	0.528	0.3836	0.512	0.5333	1.390	
2	0.453	0.3049	0.401	0.4177	1.370	
3	0.388	0.1965	0.257	0.2677	1.362	–1.30
4	0.129	0.0463	0.067	0.0698	1.507	+9.12

Notes: H = Head
Q = Discharge
m = meter
*low head

The following are some important precautions to be adhered to while making discharge and head measurements:

i. The head and discharge measurements must be carried out under steady state conditions of flow.

ii. The upstream head measurement must be effected in a nonturbulent, disturbance-free and preferably accelerating zone (if one exists).

iii. The head is to be measured with respect to the sill level of the throat section. Therefore, zero error, if any, between the gauge and the sill is to be accounted for.

iv. The verticality of the gauge post, or in the case of markings on the wall, verticality of the wall, must be ensured.

v. The discharge measurement is to be carried out using a current meter and following the general procedure suggested in Annexure 1 for using current meter measurements.

vi. It would be preferable that the single discharge measurement be close to the design discharge for normal operations.

vii. Discharge measurement should be carried out at the downstream or upstream of the structure depending on the site conditions. In case of upstream and downstream discharge measurements, an average of the two discharges can be taken for computation of C.

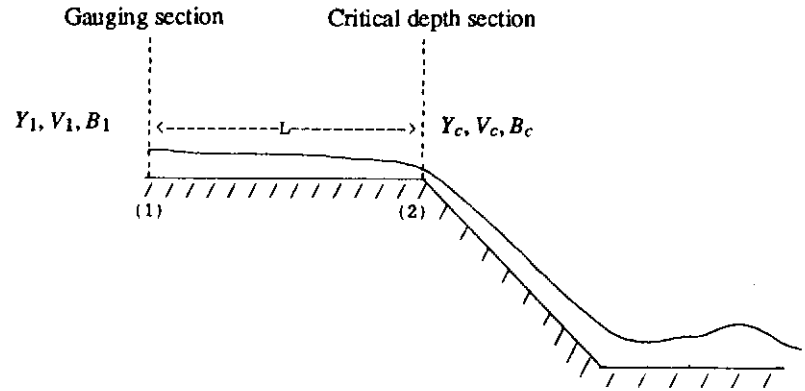
THEORETICALLY GENERATED CALIBRATION CURVES

In almost all types of drop structures, critical depth is established at the throat section. Using the concept of critical energy, it is possible to generate a theoretical calibration curve, and check its accuracy with a few sets of field

measurements. A procedure to obtain a theoretically generated calibration curve and update it with a set of field data is described below. This method is different from the previous one and does not make the assumption that drop structure calibration curves follow equation 1.

Let us consider a typical longitudinal section of a drop structure as shown in Figure 15 for calibration. Section (1) refers to the point where the gauge post is fixed while section (2) is the throat section where critical depth occurs. Let B_1, Y_1, and V_1 be the breadth, depth and velocity of flow at section (1) while the corresponding values at critical flow section are respectively B_c, Y_c and V_c, respectively. Let the distance between the two sections be L and the Manning's Roughness Factor be n.

Figure 15. Longitudinal section of a drop structure.



At critical depth section, we know $Y_c^3 = \frac{q^2}{g}$, where q is discharge/unit width,

or $Y_c^3 = [Q/B_c]^2/g$

or $Y_c = [Q/B_c]^{2/3}/g^{1/3}$, where Q is the discharge through the drop.

Also, $\frac{Y_c}{2} = \frac{V_c^2}{2g}$; g = acceleration due to gravity.

$\therefore$ Total energy with respect to sill level, $E = Y_c + \frac{V_c^2}{2g}$

$\therefore E = \frac{3}{2} Y_c$ (for rectangular section)

or $E = \frac{3}{2} [Q/B_c]^{2/3}/g^{1/3}$ (2)

Equating the total energy at section (1) and (2)

$Y_1 + \frac{V_1^2}{2g} = Y_c + \frac{V_c^2}{2g}$; + losses between section (1) and (2)

$Y_1 + [Q/B_1Y_1]^2/2_g = E + h_L$ (3)

where h_L = head loss between sections (1) and (2)

h_L can be computed as follows:

$$V_1 = \frac{Q}{B_1Y_1}\ ;\ V_c = \frac{Q}{B_cY_c}\ ;\ V_{av} = \frac{V_1+V_c}{2}$$

$A_{av} = \frac{A_1+A_c}{2}$ where A_1 and A_c are flow areas at section (1) and (2),

$P_{av} = P_1 + P_c$, where P_1 and P_c are wetted perimeters at section (1) and (2),

$R_{av} = \frac{A_{av,}}{P_{av}}$ where R_{av} = average hydraulic radius.

$$V_{av} = \frac{1}{n} R_{av}^{2/3} S^{1/2} = \frac{1}{n} R_{av}^{2/3} \left[\frac{hL}{L}\right]^{1/2}$$

$$\therefore h_L = \frac{L_n^2 V_{av}^2}{R_{av}^{4/3}} \quad (4)$$

Having known h_L, the right-hand side of equation (3) is known. By trial and error, Y_1 can be computed for a given or assumed value of Q.

PROCEDURE TO GENERATE THEORETICAL CALIBRATION CURVES

i. First assume a certain discharge Q.

ii. For the assumed discharge, compute critical energy, E, using equation (2).

iii. Assume h_L is negligible to start with and solve equation (3) by trial and error to determine Y_1.

iv. Having determined Y_1, compute V_1, V_{av} and R_{av}. Assuming a representative value for Manning's n, obtain h_L using equation (4).

v. With the computed h_L, again solve equation (3) by trial and error to obtain Y_1.

vi. The above procedure can be repeated till the h_L computed between two consecutive trials differs insignificantly (say, less than 0.001).

vii. Repeat the above computations for at least five sets of assumed discharges to generate the corresponding depth of flow at the gauging section.

viii. Knowing depths of flow and the corresponding discharges, the theoretically generated calibration curve can be obtained by plotting depth, H (m) along the X-axis and discharge Q (m^3/sec) along the Y-axis.

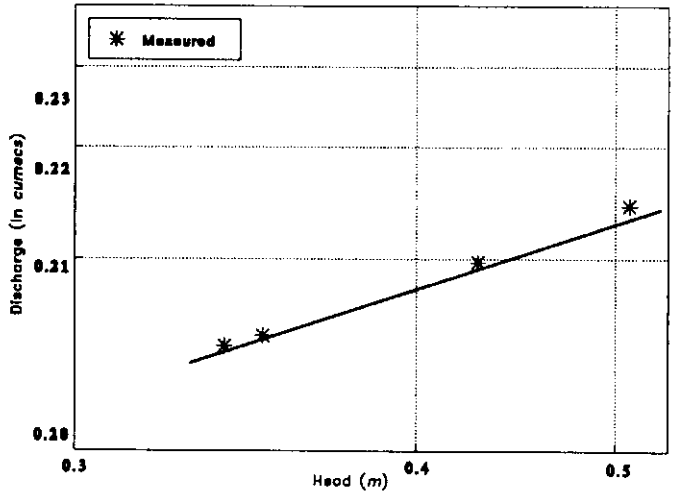
The theoretically generated calibration curve will be field-tested as follows:

A typical field measurement of discharge and the corresponding head at the gauging section at or near the full supply level of canal will be made and the point plotted on the theoretical calibration curve. If the point lies on the theoretical calibration curve, then the assumed Manning's Roughness Coefficient is correct; if not, with the measured head and discharge, compute the correct value of Manning's n using equation (4).

x. With the recomputed value of Manning's n, generate a new calibration curve by repeating items i. to vii. to obtain the actual calibration curve.

Figure 16 indicates the theoretically generated calibration curve and its field verification for a Giritale large drop having a throat width of 1.524 m. As one can see, the field data match very well with the theoretically generated calibration curve, indicating the soundness of this procedure. Details of calculation are given in Annexure III.

Figure 16. Comparison of measured and computed heads.



The assumptions made in this method are that the critical depth occurs at the throat section, and that the Roughness Coefficient determined for one typical discharge at or near full supply level can be used for all range of discharges. This assumption implies that the flow pattern observed at or near full supply level remains the same for all discharge conditions.

In the single point method, it is assumed that the calibration curve can be represented by an equation of the form $Q = CBH^{3/2}$ for all range of discharges and that the value of C can be determined by a single discharge measurement at or near full supply level.

We are not in a position to say which one of the suggested methods is preferred. We believe that both methods need extensive field-testing before arriving at a conclusion as to which one of the methods has to be recommended for field application. In the initial stages, both methods can be tried.

CHAPTER 5

Suggested Procedure

TO CALIBRATE A DROP STRUCTURE

i. THE PRACTICE OF placing obstructions to head up the flow upstream of a drop structure is quite common among the farmers. The presence of stop logs or stones near the throat of a drop structure will alter its discharge characteristics. Therefore, the drop structure should be initially checked for obstructions, and if any, these should be removed before attempting to measure the discharge.

ii. Make sure that the post used for head measurements or wall markings has its zero coincide with the sill elevation of the throat; if not, apply the necessary correction to the heads measured.

iii. Verify the verticality of post or the wall in which marking is made using a plumb bob; if not, apply the necessary correction for verticality.

iv. Install two temporary gauge posts, one at the upstream and the other at the downstream reach of the drop structure and maintain a constant level of flow to keep the discharge constant during the time of flow measurement.

v. Choose a cross-section at the downstream or at the upstream where uniformity of flow prevails to carry out discharge measurements. Measure the depth of flow at different points across the section and obtain the flow cross-section.

vi. Choose a minimum of 10 verticals at equal intervals of 10 percent of top width across the flow section and determine point velocities using a current meter along these verticals. A current meter will record the

number of revolutions from which the velocity of flow can be determined with a calibration chart. For details, refer to Annexure 1.

vii. Use the two point method, i.e., measure velocities at two-tenths and eight-tenths depth below the water surface. The mean of these two velocities gives the average velocity along that vertical. In shallow water less than 30 *cm* near the banks, make a single velocity measurement at six-tenths depth from the water surface which gives the mean velocity at that vertical.

viii. Compute the discharge across the flow section as indicated in Table 4 of Annexure 1. Repeat the discharge measurement at least twice for the same upstream head to get a more precise and average value. The discharge measurements should not differ by more than 5 percent. If it differs, repeat the measurement.

ix. Record periodically the height of flow at the gauge post positioned at the drop structure. If there is a slight variation (less than 5 *mm*) use the average head.

x. If one uses the single point calibration, then locate a point on log-log paper with the measured head plotted along the X-axis against discharge (Y-axis), and through this point draw a line with a slope of 1.5 (1 horizontal to 1.5 vertical). This line represents the calibration curve.

xi. In the case of the theoretically generated calibration curve, plot the measured discharge Q, against the head H, on the theoretically generated curve. If the plotted point lies on the theoretical curve, then that gives the actual calibration curve; if the point lies above or below the theoretical curve, then compute the correct Manning's Roughness Coefficient n and then generate the actual calibration curve as suggested under the section on theoretically generated calibration curve.

TO UPDATE A CALIBRATION CURVE

i. Follow the same procedure (items 1 to 9 as described under previous section).

ii. In the case of single point calibration, plot the measured head against discharge in the existing calibration curve. If the plotted point lies on the existing line, then the same curve can be used; if the point does not lie on the existing line, then draw a line parallel to the existing line through the plotted point. This new line will give the updated calibration curve.

iii. In the case of the theoretically generated curve, the procedure is the same as outlined under item xi. above.

CONCLUDING REMARKS

Two methods are suggested for obtaining a new or updated calibration curve for drop structures commonly used in Sri Lanka. The suggested procedures are based on limited data obtained from the drop structures of Giritale and Parakrama Samudra schemes in Polonnaruwa District and from the laboratory model tests conducted on the above structures at LHI, Colombo. The log-log plot of a linear relationship between discharge and head appears to be sound, especially for operation depths close to the design depths at which irrigation canals are normally operated. Similarly, the theoretically generated calibration curve matches very well with field data. However, additional verification of these procedures with more field data from different irrigation schemes would be necessary to apply these procedures more confidently.

A procedure to use the manual for training and demonstration purposes at the field is outlined in Annexure II.

Annexure I

A Brief Note on Current Meter Measurements

A CURRENT METER accurately determines the velocity in a channel. It is a small instrument containing a revolving wheel or vane that is turned by the movement of water. The two most commonly used types of meters are the Price current meter and the Hoff current meter.

The Price current meter contains an impeller which consists of six conical-shaped cups mounted on a vertical axis. When the meter is immersed in moving water, the impeller revolves, and the time for a given number of revolutions is determined by the operator.

The completion of every revolution or every fifth revolution is indicated by an electrical sounding device connected to earphones which the operator wears.

The Hoff meter contains a rubber impeller mounted on a horizontal axis. The chief advantage of using this type of meter is that it is less affected by eddies or turbulence.

Current meters are either mounted on a rod, or suspended from the end of a cable above a heavy weight. Rod mountings are generally used in measuring shallow streams that can be waded. For deep streams or for measurements from a bridge or cableway some distance above the water surface, the cable suspension should be used. Before being used in the field, current meters are rated or calibrated to determine the relation between the speed of rotation of the impeller and the velocity of the water. From this rating a graph or table is prepared showing the velocity for a given number of revolutions in a given time interval.

METHOD OF MEASUREMENT

Several measuring points are laid off across the channel at right angles to the direction of flow. These are generally spaced on equal distances apart, neither more than the

mean depth of the channel nor more than 10 percent of its width, making a total of not less than 10 measurements. The depth and mean velocity of the stream are then determined at each measuring point as shown in Table 4.

Four methods are generally advocated for determining the mean velocity with a current meter: multiple-point, two-point, single-point, and vertical-integration.

The multiple-point, being the most accurate, is the method by which the accuracy of other methods is generally checked. At each measuring point the velocity is determined at several closely spaced points from the bottom of the channel to the water surface. If these are equally spaced, the mean velocity in the vertical approximates the average of the measured velocities. Or else, one could use double integration to compute the discharge across the flow section. Software packages that will accomplish this are available. This method is not generally preferred in irrigation practice because it is time-consuming.

In the two-point method, the velocity is determined at 0.2 and 0.8 of the depth from the free water surface. The average of these two measurements approximates the mean velocity for ordinary conditions.

In the single-point method, the velocity is determined at a point 0.6 of the stream depth below the water surface. This method is generally employed at depths less than 30 *cm* (1 *foot*) which are insufficient for the two-point method.

In the vertical-integration method the meter is lowered and raised at a uniform rate in each of the selected verticals in the measuring section. Due to the possibility of introducing errors, this method is not generally used in routine stream gauging.

A typical method of current meter data recording and computation is given in Table 4.

Table 4. Current meter recording and computation of discharge.

Gauging of : D1 distributary of tract 2, RBMC, Kirindi Oya
Date : 19.12.1990
Meter No. :
Gauging done by : B.R. Ariyaratne

Distance from initial point (*cm*)	Depth of water (*cm*)	Depth of observation	Time in seconds	Resolution	Point velocity (*cm/sec*)	Mean velocity in depth (*m/sec*)	Mean in section (*cm/sec*)	Area (*cm*)	Mean depth (*cm*)	Width (*cm*)	Discharge (Q m^3/*sec*)
120	0.0	0	0	0	0	0					
							8.7	750	12.5	60	.006525
180	25.0	15.0	56	12	17.4	17.4					
							20.8	2280	38.0	60	.047420
240	51.0	10.0	56	18	25.2	24.2					
		40.0	56	16	23.1						
							26.3	3060	51.0	60	.080476
300	51.0	10.0	57	22	30.3	28.5					
		40.0	58	20	26.7						
							25.2	2790	46.5	60	.070308
360	42.0	8.5	64	18	22.2	21.9					
		34.0	69	16	21.6						
							20.4	2160	36	60	.044064
420	30.0	18.0	61	14	18.9	18.9					
							17.1	1530	25.5	60	.026163
480	21.0	13.0	62	11	15.3	15.3					
							7.65	945	10.5	90	.007229
570	0	-	0	0	0	0					
											Total flow: 0.282185 m^3/*sec*

Remarks: For each section between two measuring points, the area is taken as the product of the average depth and width; the mean velocity as the average of the mean velocities in the two vertical sections; and the flow as the product of the area and mean velocity. The total flow is the sum of the flows between.

Annexure II

The Method and Procedure for Training and Demonstration

IN THE CLASSROOM the participants can be taught the procedure of calibrating drop structures using a current meter. In the field, the trainer may demonstrate the use of a current meter in flow measurement. After that, individual groups should be assigned a practice session in the use of the current meter on the real canal system for them to become familiar with the use of the equipment and to learn how to apply the field data to calibrate drop structures. Each group of about four participants is assigned to calibrate one or more drop structures by using a current meter. The trainees are requested to take all the precautions indicated in the manual while calibrating drop structures. Discharge measurements can be made both on the upstream and downstream sections of the structure to verify the accuracy of measurement. For calibration purposes, an average of the upstream and downstream discharges is taken. It is important that the trainees check the data carefully after each set of measurement to make sure that the infonnation collected is valid and that no obvious mistakes have been made. Each group should make all discharge computations at the field site as the data are collected. During each set of discharge measurement, the elevations of the water level at the upstream and downstream of the drop structure are kept constant to maintain a steady state of flow.

The trainees must be asked to get at least five sets of discharge measurements and the corresponding upstream heads. Using all the five sets of data, a calibration curve of the form $Q = CBH^{3/2}$ will be developed. Then it will be demonstrated to the trainees how to use individual sets of data to obtain a calibration curve; they will also be demonstrated the likely errors caused by using a single discharge measurement for calibration, and the need to measure and obtain field data in the most accurate manner, especially when single measurement is used to update or calibrate drop structures.

Annexure III

Data Set 1

Given: $Q = 0.925\ m^3/sec \quad B = 1.524m \quad B_1 = 2.7m$

Step 1: $Q/B = .6070 \quad [Q/B]^2 = 0.3684;\ Y_c^3 = [Q/B]^2 \times \frac{1}{g} = 0.03755$

$Y_c = 0.3348 \quad E = \frac{3}{2} Y_c = 0.50229;\ \frac{V_c^2}{2g} = 0.1674$

Step 2: $Y_1 + [Q/B_1 Y_1]^2 \times \frac{1}{2g} = 0.50229 = E$

$Y_1 + [0.925/2.7\ Y_1]^2 \times \frac{1}{19.62} = 0.50229$

Step 3: Trial and Error

Try: Let $Y_1 = 0.485$

$0.485 + [0.925/2.7 \times 0.485]^2 \times \frac{1}{19.62} = 0.4850 + 0.0254 = 0.5104 > 0.50229$

Try: 0.480

$0.480 + [0.925/2.7 \times 0.480]^2 \times \frac{1}{19.62} = 0.4800 + 0.0260 = 0.5060 > 0.50229$

Try: 0.477

$0.477 + [0.925/2.7 \times 0.477]^2 \times \frac{1}{19.62} = 0.4770 + 0.0263 = 0.5033 > 0.50229$

Take $Y_1 = 0.476\ m$

Step 4 : $V_1 = \frac{0.925}{2.7 \times 0.476} = 0.721$

$V_c = 0.1674$ x $19.62 = 1.812$

$V_{av} = 1.266\ m/s$

Step 5 : $A_1 = 2.7$ x $0.476 = 1.2825$

$A_c = 1.524$ x $0.3358 = 0.5102$

$A_{av} = 0.896\ m^2$

Step 6 : $P_1 = 2.7 + 0.95 = 3.65$

$P_c = 1.524 + 6.7 = 2.20$

$P_{av} = 2.93\ m$

$R_{av} = \frac{A_{av}}{P_{av}} = \frac{0.896}{2.93} = 0.305m$

Step 7 : $h_L = 1.95 \times [.02 \times 1.266]^2 / [0.305]^{4/3} = .0061$

Step 8 : Revised $E = E + h_L = .50229 + .00608 = 0.50837$

Step 9 : $0.50837 = Y_1 + [\ 0.925/2.7 \times Y_1\ ]^2 \times \frac{1}{19.62}$

Step 10: By Trial and Error

Try : 0.482

$0.482 + [0.925/2.7 \times 0.482]^2 \times \frac{1}{19.62}$

$= 0.483 + 0.257 = 0.5077;\ 0.5077 < 0.5077$

Try : $0.485 + [0.925/2.7 \times 0.485]^2 \times \frac{1}{19.62} = 0.5104;\ 0.5104 > 0.50837$

Step 11 : $Y_1 = 0.483$

Try : $0.483 + [0.925/2.7 \times 0.483]^2 \times \frac{1}{19.62} = 0.50865;\ 0.50865 > 0.50837$

Take Y_1 : 0.483 corresponding to $Q = 0.925$

Data Set 2

$Q = 0.680$ $B =$ 1.524

$Q/B = 0.4462;\ [Q/B]^2 \times \frac{1}{g} = 0.02029$ $Y_c =$ 0.2727

$\frac{V_c^2}{2g} =$ 0.1363

$E =$ 0.4090

$Y_1\ [\ 0.680/2.7 \times Y_1\ ]^2 \times \frac{1}{19.62} = 0.4090$

Try : $Y_1 = 0.396$

$0.396 + [0.680/2.7 \times 0.396]^2 \times \frac{1}{19.62} = 0.4166;\ 0.4166 > 0.4090$

Try : $Y = 0.385$

$0.385 + [0.680/2.7 \times 0.385]^2 \times \frac{1}{19.62} = 0.40681;\ 0.40681 < 0.4090$

Take : $Y = 0.387$

$$V_1 = \frac{0.680}{.7 \times 0.387} = 0.650 = 0.650 \qquad V_c = \sqrt{0.1363 \times 19.62}$$

$V_c = 1.635$

$V_{av} = 1.143$

$A_1 = 2.7 \times 0.385 = 1.0395$

$A_c = 1.524 \times 0.385 = 0.5867$

$A_{av} = 0.81312$

$P_1 = 2.7 + 0.77 = 3.47$

$P_c = 1.524 + 0.77 = 2.29$

$P_{av} = 2.88$

$$R_{av} = \frac{A_{av}}{P_{av}} = \frac{0.813112}{2.88} = 0.2823$$

$h_L = 1.95 \times [.02 \times 1.143]^2 / [0.2823]^{4/3} = 0.0055$

$E = 0.4090 + 0.0055 = 0.4145$

Try $Y_1 = 0.394$

$$0.394 + [0.680/2.7 \times 0.394]^2 \times \frac{1}{19.62} = 0.4148$$

$0.4148 \approx 0.4145$

Take $Y_1 = 0.394$ corresponding to $Q = 0.680$

Data Set 3

$Q = 0.439$ $B = 1.524$

$Q/B = 0.2880$ $[Q/B]^2 = 0.08297$; $[Q/B]^2 \times \frac{1}{g} = 0.0084$

$$Y_c = 0.20374 \frac{V_c^2}{2g} = 0.10187$$

$E = 0.3056$

$$Y_1 + [\, Q/B_1 Y_1 \,]^2 \times \frac{1}{2g} = 0.3056$$

$$Y_1 + [\, 0.439/2.7 Y_1 \,]^2 \times \frac{1}{2g} = 0.3056$$

Try $Y_1 =$ .303

$$.303 + [0.439/2.7 \times 0.303]^2 \times \frac{1}{19.62} = 0.31767 \; ; \; 0.3176 > 0.3056$$

Try $Y_1 =$ 0.298

$$.298 + [0.439/2.7 \times 0.298]^2 \times \frac{1}{19.62} = 0.31317 \; ; \; 0.3132 > 0.3056$$

Try $Y_1 =$ 0.290

$$0.290 + [0.439/2.7 \times 0.290]^2 \times \frac{1}{19.62} = 0.30602 \; ; \; 0.30602 > 0.3056$$

Take $Y_1 = 0.290$

$$V_1 = \frac{0.439}{2.7 x 0.290} = 0.560 \; ; \; V_c = 1.4137$$

$V_{av} = 0.98687$

$A_1 = 2.7 \times 0.290 = 0.783$

$A_c = 1.524 \times 0.290 = 0.44196$

$A_{av} = 0.61248$

$P_1 = 2.7 + 0.58 = 3.28$

$P_c = 1.524 + 0.58 = 2.10$

$P_{av} = 5.38/2 = 2.69$

$$R_{av} = \frac{A_{av}}{P_{av}} = \frac{0.61248}{2.69} = 0.22768$$

$h_L = 1.95\,[.02 \times 0.98687]^2 / [0.22768]^{4/3} = 0.00546$

$E = .3056 + 0.00546 = 0.31106$

Try $Y =$ 0.295

$$0.295 + [0.439/2.7 \times 0.295]^2 \times \frac{1}{19.62} = 0.31159$$

$0.31159 \approx 0.31106$

Take $Y_1 = 0.295$ corresponding to $Q = 0.439$

Data Set 4

$Q = 0.406$ $B = 1.524$

$Q/B = 0.2664$ $[Q/B]^2 = 0.007097$ $[Q/B]^2 \times \frac{1}{g} = 0.00723$

$Y_c = 0.1934$ $E = 0.2901$ $\frac{V_c^2}{2g} = 0.0967$

$$Y_1 + [0.406/2.7 \times Y_1]^2 \times \frac{1}{19.62} = 0.2901$$

Try = 0.275

$$0.275 + [0.406/0.275 \times 2.7]^2 \times \frac{1}{19.62} = 0.29023$$

$0.29023 \approx 0.2901$

Y_1 = 0.275

V_1 = 0.5468 ; $V_c = 1.377$; $V_{av} = 0.9262$

A_1 = $2.7 \times 0.275 = 0.7425$

A_c = $1.524 \times 0.275 = 0.4191$

A_{av} = 0.5808

P_1 = $2.7 + 0.55 = 3.25$

P_c = $1.524 + 50.38 = 1.90$

P_{av} = 2.575

$$R_{av} = \frac{A_{av}}{P_{av}} = \frac{0.5808}{2.575} = 0.2255$$

$$h_L = 1.95 \times [.02 \times 0.9262]^2/[0.2255]^{4/3} = 0.004875$$

$$E = 0.2901 + 0.004875 = 0.294975$$

Try Y_1 $= 0.280$

$$= 0.280 + [0.406/2.7 \times 0.280]^2 \times \frac{1}{19.62} = 0.2947$$

$$0.2947 \approx 0.2949$$

Take Y_1 $= 0.280$ corresponding to Q = 0.406