REPORT NO. R-28

PILOT PROJECT FOR
FARMER-MANAGED IRRIGATED AGRICULTURE
UNDER THE LEFT BANK OUTFALL DRAIN,
STAGE I PROJECT

HYDRAULIC CHARACTERISTICS OF PILOT DISTRIBUTARIES IN THE MIRPURKHAS, SANGHAR AND NAWABSHAH DISTRICTS, SINDH, PAKISTAN

By

Bakhshal Lashari
Gaylord V. Skogerboe
Rubina Siddiqui

June 1997

Hyderabad Office
PAKISTAN NATIONAL PROGRAM
INTERNATIONAL IRRIGATION MANAGEMENT INSTITUTE

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ACKNOWLEDGEMENTS

CHAPTER 1. INTRODUCTION

1.1 GENERAL BACKGROUND OF THE PILOT PROJECT

In July 1995, the International Irrigation Management Institute (IIMI) started an action research program in collaboration with the Department of Agricultural Engineering and Water Management of the Government of Sindh to launch three pilot projects on water users organizations. The broad purpose of these pilot projects is two-fold: **(1)** to test the viability of farmers managing parts of the irrigation and drainage systems so that more efficient and equitable allocation of water can be achieved; and (2) to make recommendations on a future extension from the results of the pilot projects.

One of the specific objectives of these pilot projects is to help organize farmers into Water Users Organizations (WUOs) in three selected distributaries/minors canal command areas, one in each of the three LBOD districts: Mirpurkhas, Nawabshah and Sanghar. Another specific objective is to promote the maximum involvement of the water users and the WUOs in the operation and maintenance of distributary/minor canals, without much intervention from the governmental agencies, but with their institutional and technical support, particularly in the early development stages of the pilot projects. Ultimately, the legislative requirements and institutional processes would be identified for effectively organizing and strengthening Water Users Organizations (WUOs) on a wider scale.

The broad concept underlying these pilot projects is that the WUOs would eventually be accountable for the water received at the head of distributary/minor[1] canals, responsible for distribution of water among the member water users associations (WUAs) at the watercourse level according to their own agreed allocation rules, and also responsible for managing groundwater levels in their respective command areas. The WUOs would reach an agreement with their members, as well as with the agencies, for appropriate water charges and operation and maintenance (O&M) costs of irrigation and drainage facilities in their distributary/minor command areas. They will undertake the collection of water/drainage charges, improve water management practices, and other activities related to water, including the maintenance practices for irrigation and drainage facilities.

---

[1] In the Punjab, a minor offtakes from a distributary channel. This definition is also commonly used in Sindh, but in addition, a small distributary is oftentimes called a minor. Of the three pilot sites, two are distributaries and the other one is called minor, but all three pilot sites are distributaries.

1.2   DESCRIPTION OF THE SELECTED DISTRIBUTARIES/MINORS

1.2.1   Bareji Distributary

This distributary is taking off from the East Jamrao Canal at RD.408(which means 408,000feet downstream from the Jamrao Canal Head Regulator). The length of the distributary is 12 km (RDs 39.31) (see Figure 1.1) and the designed discharge is 34.2 cusecs. It is a perennial canal which runs throughout the year except during the canal closure period in January every year. The distributary is irrigating about 14,318acres of land through 24 watercourses. There are six (6)lined watercourses and eighteen (18) unlined watercourses. The head regulator of this distributary is in good working condition and the upstream and downstream gauges were recently installed. The land holding sizes on the distributary is small to medium ranging from 5 acres to 100 acres. There are about 197 land owners. The layout of the distributary is shown in Figure 1.2.

1.2.2   Heran Distributary

The Heran Distributary is taking off from the Nara Canal at RD 129. The total length of the distributary is 10.6 km. The distributary has one minor named Khadwari Minor and the length of this minor is 3.2 km (see Figure 1.1). The total number of watercourses along the distributary channel is 24, from which 23 watercourses are lined and one is unlined. The Khadwari Minor has 7 watercourses, from which 4 watercourses are lined and 3 watercourses are unlined. Thus, the total number of watercourses receiving water from Heran Distributary is 31, while the total culturable command area (CCA) served by Heran Distributary (including Khadwari Minor) is 15,410 acres. The schematic diagram of this distributary is given in Figure 1.3.

1.2.3   Dhoro Naro Minor

Dhoro Naro Minor is taking off from the Gajrah Branch Canal which offtakes from Nusrat Branch Canal of the Rohri Canal (see Figure 1.1) at RD 91.4.The total length of the distributary is 10.39km. The designed discharge is 51.62cusecs which is irrigating an area of about 13,382acres.

The Dhoro Naro Minor is feeding twenty five (25) watercourses. All of the outlets are open flume type. Fifteen (15)watercourses are lined and ten (10) watercourses are unlined. The layout of this minor is shown in Figure 1.4.

2
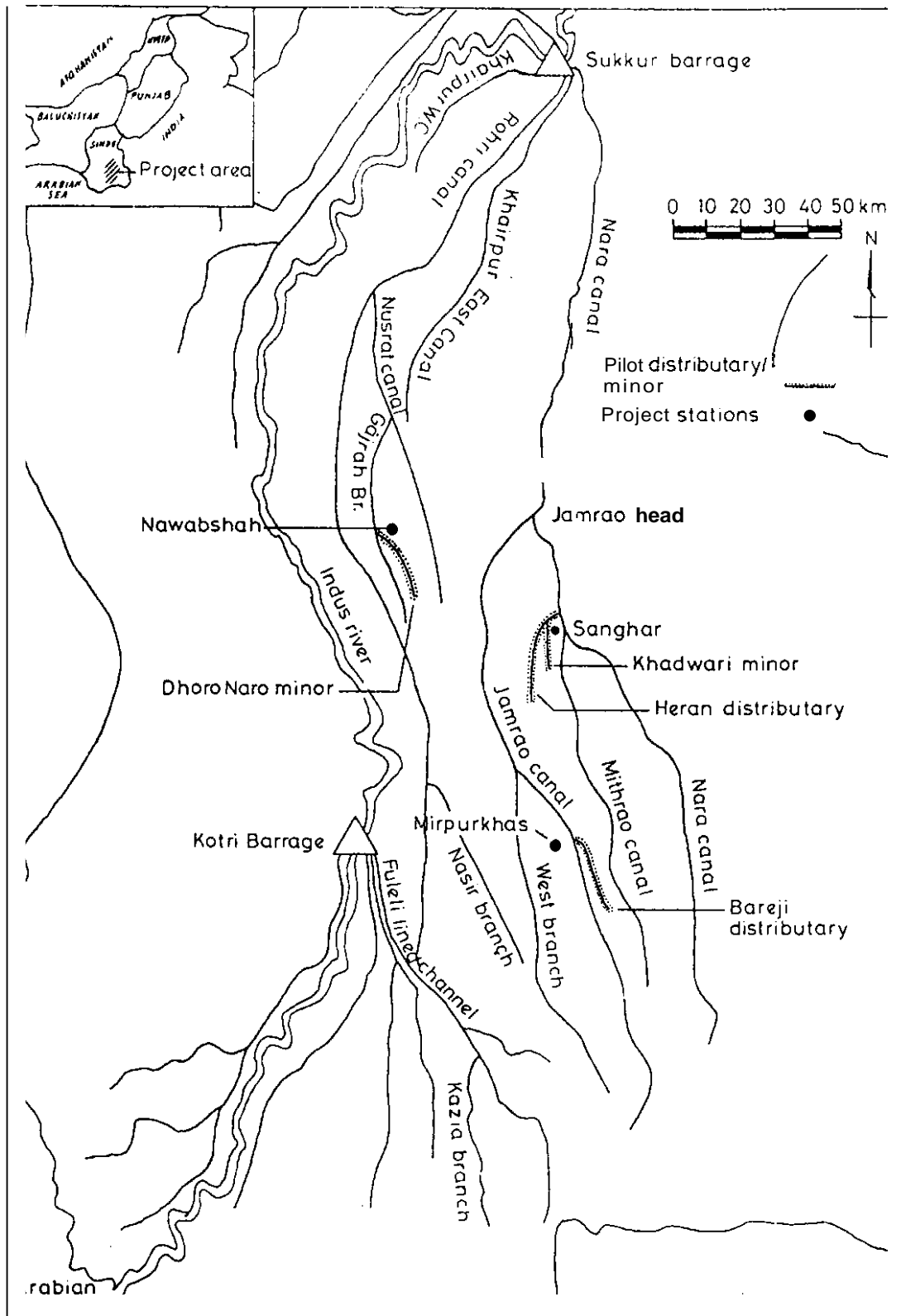
Fig. 1·1. Location **Map** of IIMI Project **Area**

JAMRAO EAST (RD 408-MILES 82)

Head of Bareji

229/1L (0·5 RD)

228/1L (5·7 RD)

228/1AL (11·5 RD)

228/2L (12·1 RD)

228/1AL (12·1 RD)

240/1R (0·5 RD)

240/1 AR (1·75 RD)

239/2R (4·90 RD)

239/2AR (4·90 RD)

240/2R (6·70RD)

240/3R (11·5 RD)

227/1AL (12·1 RD)

227/1L (13·1 RD)

226/1L (14·0 RD)

226/2L (17·3 RD)

N

225/1L (20·0 RD)

239/3R (22·9RD)

238/2R (25·10 RD)

238/1R (25·10·RD)

225/1AL (27·0RD)

237/1R (29·20 RD)

LEGEND

Water course ....
Over head bridge...
Drainage ....

225/2L (30·3 RD)

224/1L (33·5RD)

236/1AR (36 RD)

224/2L (39·31RD)

Fig. 1.2. Schematic Diagram of Bareji Distributary

4

NARA CANAL

1L (2·64 RD)

2R (4·4 RD)

3L (7·0 RD)

5L (7·18 RD)

4R (9·29 RD)

1AL (7·0 RD)   1L (9·0 RD)   3L (17·0 RD)

6T (17·0 RD)
5T (17·0 RD)
4R (17·0 RD)

X-Regulater

6L (10·0 RD)
7L (10·0 RD)

2R (11·0 RD)

ROAD

9AR (11·09 RD)

8L (11·9 RD)

9R (13·07 RD)

8AL (16·7 RD)

10R (17·35 RD)

14L (21·83 RD)

15L (24·99 RD)

17AL (26·72 RD)

17BL (30·951 RD)

ROAD

17AT (32·0 RD

11+12R (18·77 RD)

18AT (32·0 RD)

13R (20·81 RD)

18R (30·951 RD)

16AR (29·89 RD)

16R (26·05)
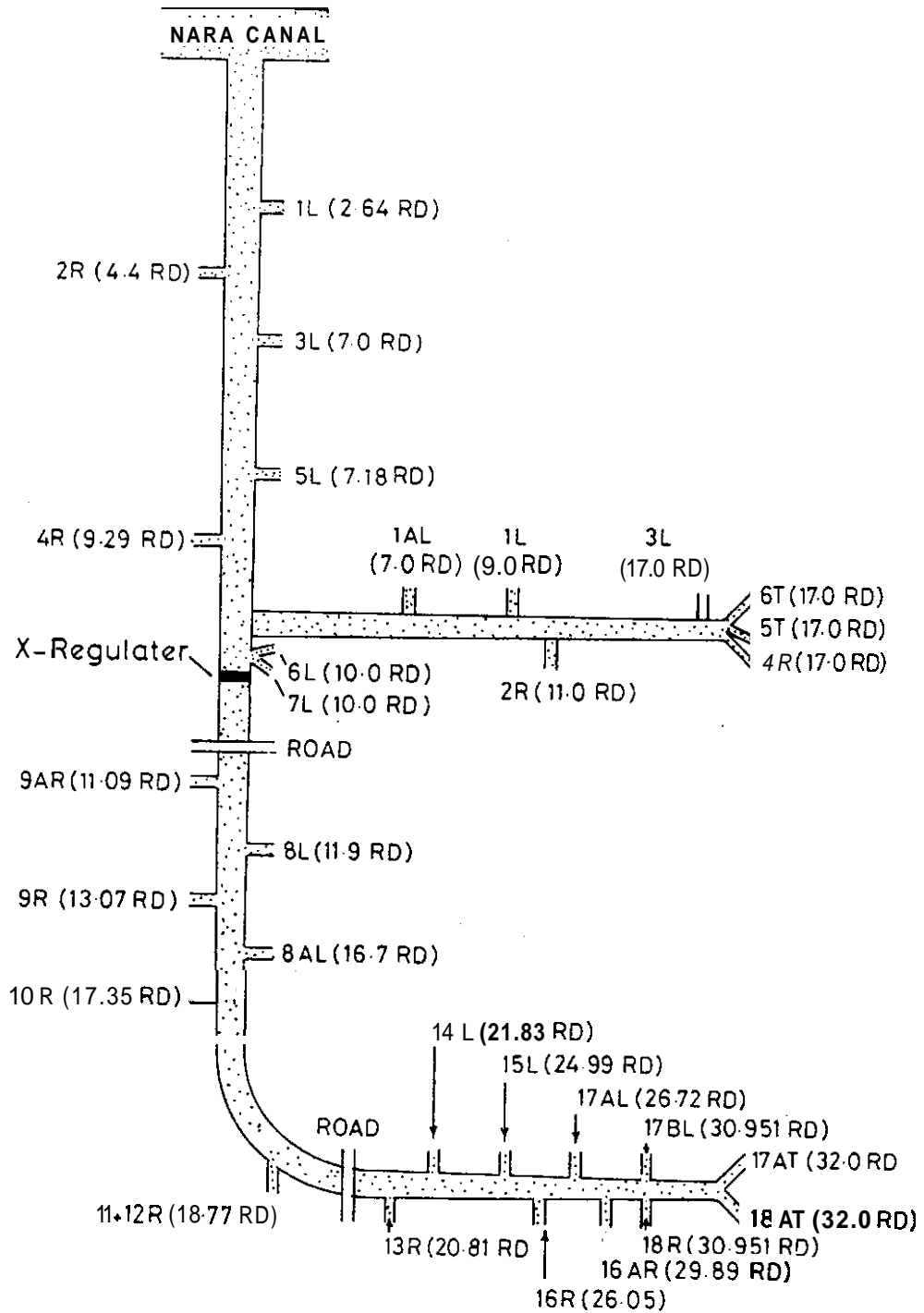
Fig. **1.3.** Schematic Diagram of Heran Distributary

Gajrah Branch Canal

1R (2·0 RD)

1 DL (3·75 RD)

2R (5·7 RD)

1L (6·925 RD)

3R (8·981 RD)

1AL (10·25 RD)
1BL (11·0 RD)
1CL (11·0 RD)
2L (11·0 RD)

4R (12·96 RD)

2AL (12·98 RD)

3L (13·66 RD)

5R (18·801 RD)

4L (14·946)

4BL (15·276 RD)

6R (22·76 RD)

4AL (18·942)

6AR (24·386 RD)

5L (24·17 RD)

7R (29·5 RD)

6L (27·526 RD)

T (32·275 RD)

7L (31·596 RD)
9L (31·795 RD)
10L (32·275 RD)
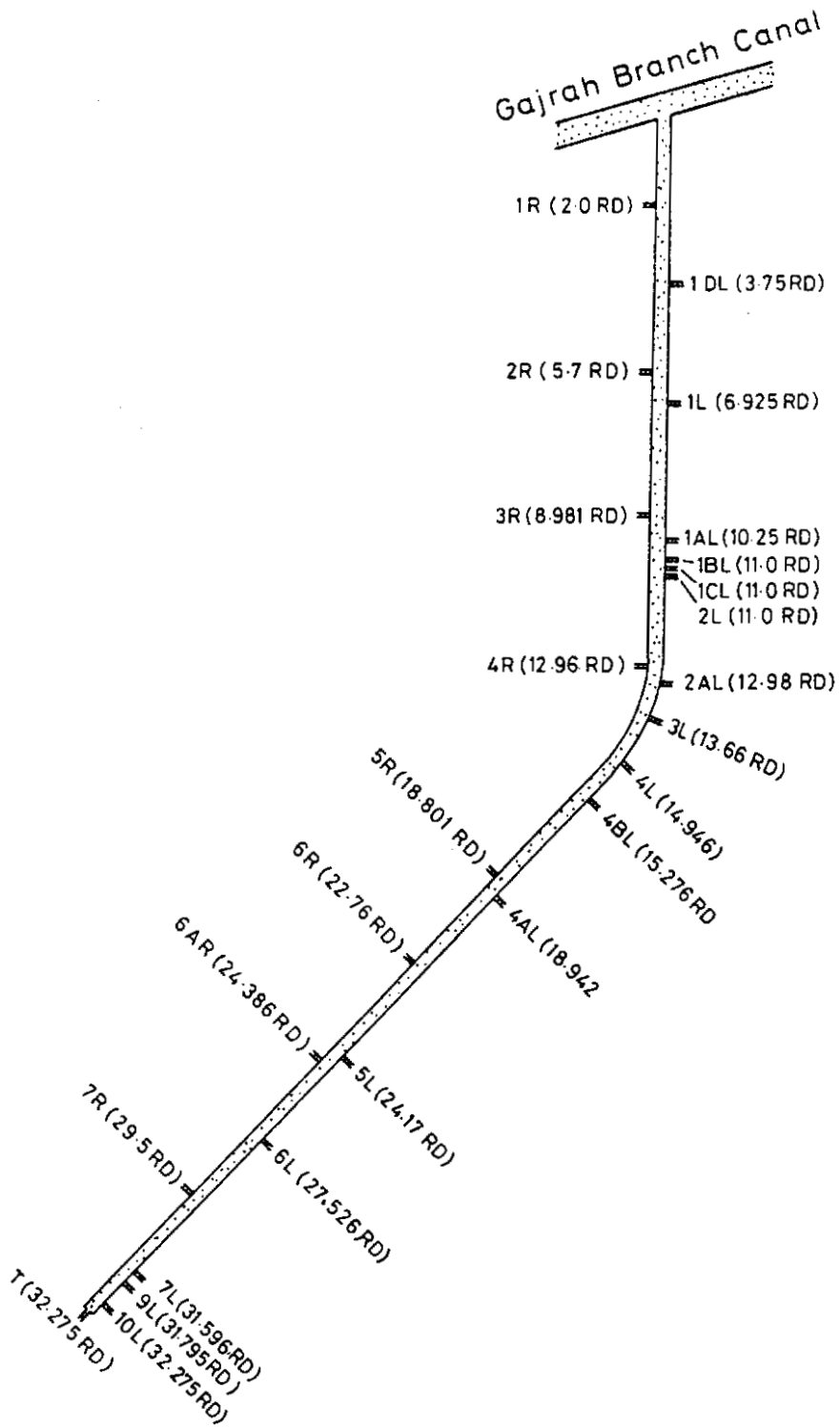
Fig. 1.4. Schematic Diagram of Dhoro Naro Minor

6

The salient features of the selected distributaries/minors are given in Table 1.1

Name of distributary/ minor	Designed discharge (cfs)	Length (Km)	Number of outlets	Type of watercourse		Cultural command area (CCA) (acres)
				Lined	Unlined	
Bareji Distributary (Mirpurkhas)	34.2	12.5	24	06	18	14,318
Dhoro Naro Minor (Nawabshah)	51.6	10.39	25	11	14	13,382
Heran Distributary Channel	58	10.6	24	22	2	12.336
Khadwari minor (Sanghar)	10.62	3.20	7	4	3	3,074
Heran Distributary	*58*	13.8	31	26	5	15,410
TOTALS FOR THREE PILOT SITES	143.8	36.7	80	43	37	43,110

CHAPTER 2. ESTABLISHMENT OF BENCHMARKS

2.1 INTRODUCTION

A benchmark (BM) is referred to as a reference elevation point. With the help of a reference elevation point, water levels can be measured. These benchmarks (BMs), particularly after hydraulic calibratiion of the head regulator for each distributary or minor, as well as outlet structures, will serve as reference points for measuring water levels. The main purpose of these benchmarks is to devlop a simple methodology for observing the flow rate at each structure. The benchmarks have been placed on each headwall so that upstream and downsteam water levels can be measured using a tape, rather than installing staff gauges [1]. The equations for all irrigation control structures are presented in Chapter 5. Using the observed water surface elevation data, the upsteam flow depth, h_u, and downstream flow depth, h_d, can be easily calculated, which can be used in conjunction with a discharge rating table for each hydraulic structure.in order to establish the discharge rate.

2.2 ESTABLISHMENT OF BENCHMARKS

Before calibrating the outlets (moghas), it is necessary to establish permanent benchmarks at the upstream (u/s) and downstream (d/s) sides of the outlet structure so that the upstream (h) and downstream (h) flow depths can be easily measured using a tape, rather than installing staff gauges. The following procedure has been adopted.

*   First of all, the hydraulic flow conditions of an outlet structure (or head regulator) are observed on both the u/s and dls sides of the structure. Tranquil water level locations are sought where there is a minimum of "bounce" in the water level; in other words, locations are prefered where the water surface *is* calm and smooth. But if the structure is damaged at the d/s side, the downstream BM was located a few feet d/s of the structure adjacent to the channel bank.

*   The selected reference elevation points were first rubbed and cleaned by a gauze brush and then a white mark (WM) was painted at each location using a paint brush having oil paint, where the lower side of the WM is the reference elevation.

Upper figure labels:
- d/s
- roof block
- WSL
- u/s
- BM
- check plates
- base plate fixed in crest
- A ... A

Lower figure labels:
- roof block
- top of bank
- 1.5:1
- FSL
- u/s
- BM
- WSL
- d/s
- 0.5:1
- 0.5:1
- 1.5:1
- base plate fixed in crest
- brick pitching
- BL
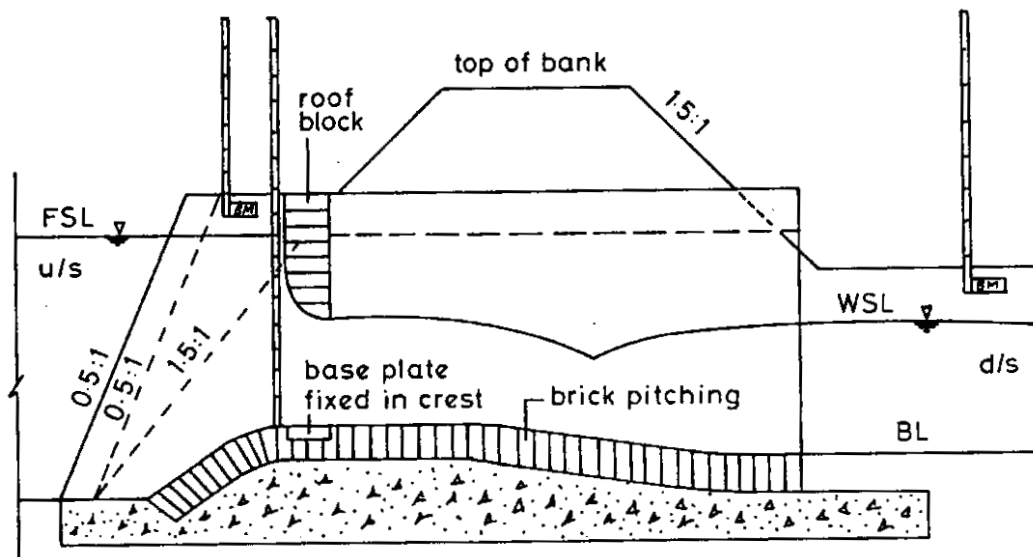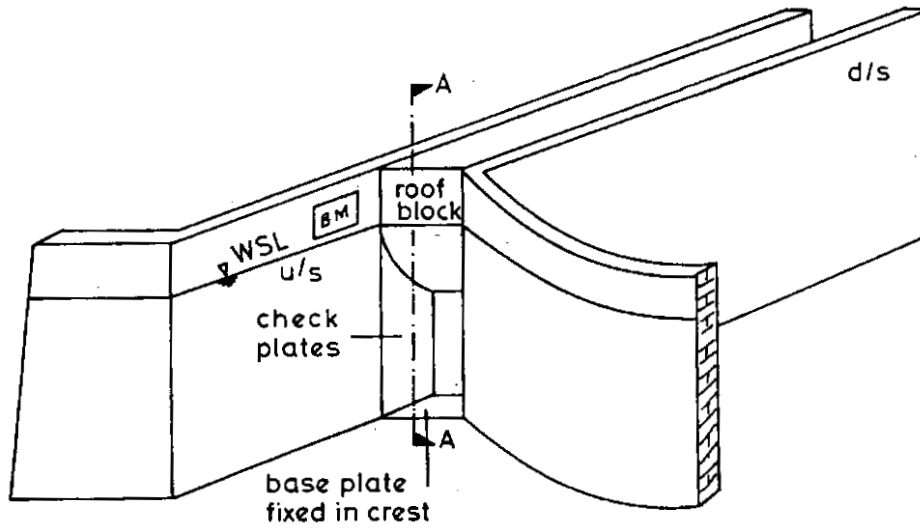
SECTION AT A-A

Fig.2.2 Typical Fixed Orifice Outlet.

Table 2.1. Benchmark elevations established on the headwalls of outlet structures along Bareji Distributary.

Number of outlet	Benchmark Elevation		Number of outlet	Benchmark Elevation	
	U/s	D/s		U/s	D/s
229/1L	3.655	-	226/1L	5.00	3.71
240/1R	4.069	-	226/2L	5.00	2.87
240/1AR	2.215	-	225/1L	5.00	4.613
239/2R	3.742	-	239/3R	5.00	1.97
239/2AR			238/2R	5.00	4.60
228/1L	2.826	-	238/1R	5.00	3.365
240/2R	2.81	-	225/1AL	5.00	5.14
240/3R	5.00	4.62	237/1R	9.585	8.835
228/1AL			225/2L	6.00	3.385
228/2L	5.00	4.752	224/1L	6.00	5.52
227/1AL	5.00	3.64	236/1AR		
227/1L	5.00	4.805	224/2L	DAMAGE	

Note: When the downstream (d/s) benchmark elevation is not listed, this implies that it is not needed because free flow occurs in the outflow structure.

Table 2.2. Benchmark elevations established on the headwalls of outlet structures along the distributary channel of Heran Distributary.

Number of outlet	Benchmark Elevation		Number of outlet	Benchmark Elevation	
	U/s	D/s		U/s	D/s
1L	5.0	4.27	14L	2.48	-
2R	1.83	-	15L	2.61	-
			16R	4.58	4.06
5L	5.0	4.96	17AL	3.45	-
4R	5.0	4.69	16AR	5.0	4.48
6L	3.29	3.35	18R	2.17	2.0
7L	3.81	3.43	17BL	1.81	-
9AR	2.78	2.88	17AT	2.61	2.57
8L	2.6	-	18AT	2.42	2.10
9R	2.85	-			
8AL	2.48	-			
10R	3.32	3.2			
(11+12)R	2.69	-			
13R	2.15	-			

Table 2.3. Benchmark elevations established on the headwalls of outlet structures along Khadwari Minor served by Heran Distributary.

Number of outlet	Benchmark Elevation	
	U/s	D/s
1AL	2.36	-
1L	2.63	-
2R	1.11	
3L	2.19	2.1
4R	2.52	2.32
5T	2.24	2.07
6T	2.97	

Table 2.4. Benchmark elevations established on the headwalls of outlet structures along Dhoro Naro Minor.

Number of outlet	Benchmark Elevation		Number of outlet	Benchmark Elevation	
	U/s	D/s		U/s	D/s
1R	3.98	-	4BL	4.5	3.69
1DL	4.615	3.139	5R	3.95	2.9
2R	4.895	4.795	4AL	2.49	3.425
1L	3.415	3.335	6R	3.897	3.339
3R	4.563	3.173	5L	3.83	3.32
1AL	3.0	5.05	6AR	3.651	2.995
1BL	3.0	4.543	6L	3.3	2.054
1CL	3.673	5.08	7R	2.427	2.837
2L	3.09	3.68	7L	3.42	4.95
4R	2.66	2.65	9L	3.0	4.36
2AL	3.79	5.78	10L	3.05	2.92
3L	2.49	4.77	11L	3.19	3.16
4L	3.65	3.66			

14

2.3   PHYSICAL CONDITION OF OUTLETS

All of the outlets for each pilot distributary or minor was carefully inspected. Almost all of the outlets (moghas) were observed to have been significantly tampered. The crest of most outlets was badly damaged and lowered, while the sides were broken. At some outlets, the local people (water users) had placed stones and sandbags just near to the outlet entrance for raising the water level so that the outlet would receive a greater flow of water. Due to the lowering of the crest level and the placement of stones and sandbags near the entrance, more sediment deposition has occured at the entrance, which has also increased the amount of sediment entering the watercourse. At some outlets, an extra hole has been made to further increase the flow entering the watercourse. Efforts have been made to show the exact situation/condition for the outlets along each distributary. The sketches of some sample outlets from each distributary or minor have been drawn, which represents most of the distributary outlets. These are shown in the following Annexures.

Annex A: Showing the Physical condition of outlets of the Bareji Distributary.

Annex B: Showing the Physical condition of outlets of the Heran Distributary and its Khadwari Minor.

Annex C: Showing the Physical condition of outlets of the Dhoro Naro Minor.

CHAPTER. 3 METHODOLOGY FOR MEASURING DISCHARGE USING A CUTTHROAT FLUME

3.1   DEVELOPMENT OF CUTTHROAT FLUME

The development of the Cutthroat Flume was the eventual outcome of studies in the Delta, Utah area. Because the canal gradients were extremely flat (1 : 8000), a flow measuring flume that would operate with critical depth occurring in the throat section could not be used because the upstream water level would be increased too much for the required carrying capacity of 500 cfs. Thus, a large flume was designed to operate under submerged flow conditions. This was a flume having a flat floor that was placed at the bed elevation of the canal. The walls were vertical. There was an inlet converging section, a throat section with parallel walls, and an outlet diverging section. This structure had to be calibrated in the field.

This situation let to designing another similar type of flume, but with a trapezoidal cross-section, for installation in another canal (Hyatt, 1965). This also resulted in the development of a new method of submerged flow analysis for open channel constrictions (Skogerboe and Hyatt, 1967a).

In 1965, a small research grant was obtained from the U.S. Dept. of the Interior, Office of Water Resources Research for "Design and Calibration of Submerged Open Channel Flow Measurement Structures". Eggleston (1967) reported the hydraulic laboratory results for an experimental flume, much like the geometry of the flume constructed near Delta, Utah, except the throat length was quite long.

Ackers and Harrison (1963) had recommended a maximum convergence of 3:1 for a flume inlet section. Experimental work reported by Eggleston (1967) indicated that this recommendation had merit, and consequently a 3:1 convergence was used in the hydraulic laboratory studies of a flat-bottomed flume.

A report by Skogerboe, Hyatt and Eggleston (1967) showed that flow depths measured in the outlet diverging section of the experimental flume resulted in more accurate submerged flow calibration curves than calibrations employing flow depth measurements in the throat section. The water surface profile changes rapidly in the throat section as compared with the outlet diverging section, where the water surface profile is more nearly horizontal. Consequently, a flow depth in the diverging outlet section of the flat-bottomed flume was selected for measurement.

Since the downstream flow depth was to be measured in the outlet diverging section, there appeared to be no apparent advantage in having a throat section with parallel walls. Consequently, hydraulic testing was initiated with a flat-bottomed flume havingly only an inlet section and an outlet section. The flume performed very well. One distinct hydraulic advantage of reducing the throat length to zero was improved flow conditions in the outlet section. The inlet converging section tended to confine the flow into a jet which traveled along the flume centerline, thus assisting in the prevention of flow separation.

The earlier study of Hyatt (1965) indicated that when the divergence of the flume outlet section was more rapid that 6:1, separation would occur, and a major portion of the flow would adhere to one of the sidewalls. Although numerous divergences and lengths of outlet section were tested, the 6:1 divergence proved most satisfactory as a balance between flow separation and fabrication costs; however, a 9:1 divergence performed better hydraulically in the experimental flume. But, the 6:1 divergence proved to be very satisfactory for a flat-bottomed flume with only an inlet section and outlet section. Without a throat section having parallel walls, the streamlines are curvilinear, which allows the streamlines to diverge more rapidly than parallel streamlines.

The objectives of the research program did not include the development of new flow measuring device. The rectangular fat-bottomed flume that resulted from the hydraulic laboratory research program, which is illustrated in Figure 3.1, was a by-product of this testing program. Since the flume has zero throat length, the flume was given the name "Cutthroat" by the developers (Skogerboe, Hyatt, Anderson and Eggleston, 1967).

All of the hydraulic tests conducted in 1966-67 at the Utah Water Research laboratory, Utah State University were for a flume length of 9 feet. Bennett (1972) conducted hydraulic tests in the Fluid Mechanics Laboratory at Utah State University in 1970 for flume lengths of 3 feet and 4.5 feet. All of the available laboratory data for the Cutthroat flume at this time was analyzed by Skogerboe, Bennett and Walker (1972).

Fiuzat in 1977 (Ren, 1993) conducted hydraulic tests at the Engineering Research Center, Colorado State University on many throat widths for a flume length of 1.5 feet. In 1979 and 1980, Huson and Gardell (no citation, but data reported by Ren, 1993) conducted hydraulic tests at the same facility to verify the discharge rating for a flume length of 9 feet.

Henry (1990) reported additional hydraulic data collected at the Utah Water Research Laboratory, Utah State University for a flume length of 4.5 feet. Ren (1993) analyzed all of the available hydraulic laboratory data for the Cutthroat Flume.

This analysis was further refined by Skogerboe, Ren and Yang (1993), with these results being used herein.

18

PLAN VIEW

SIDE VIEW
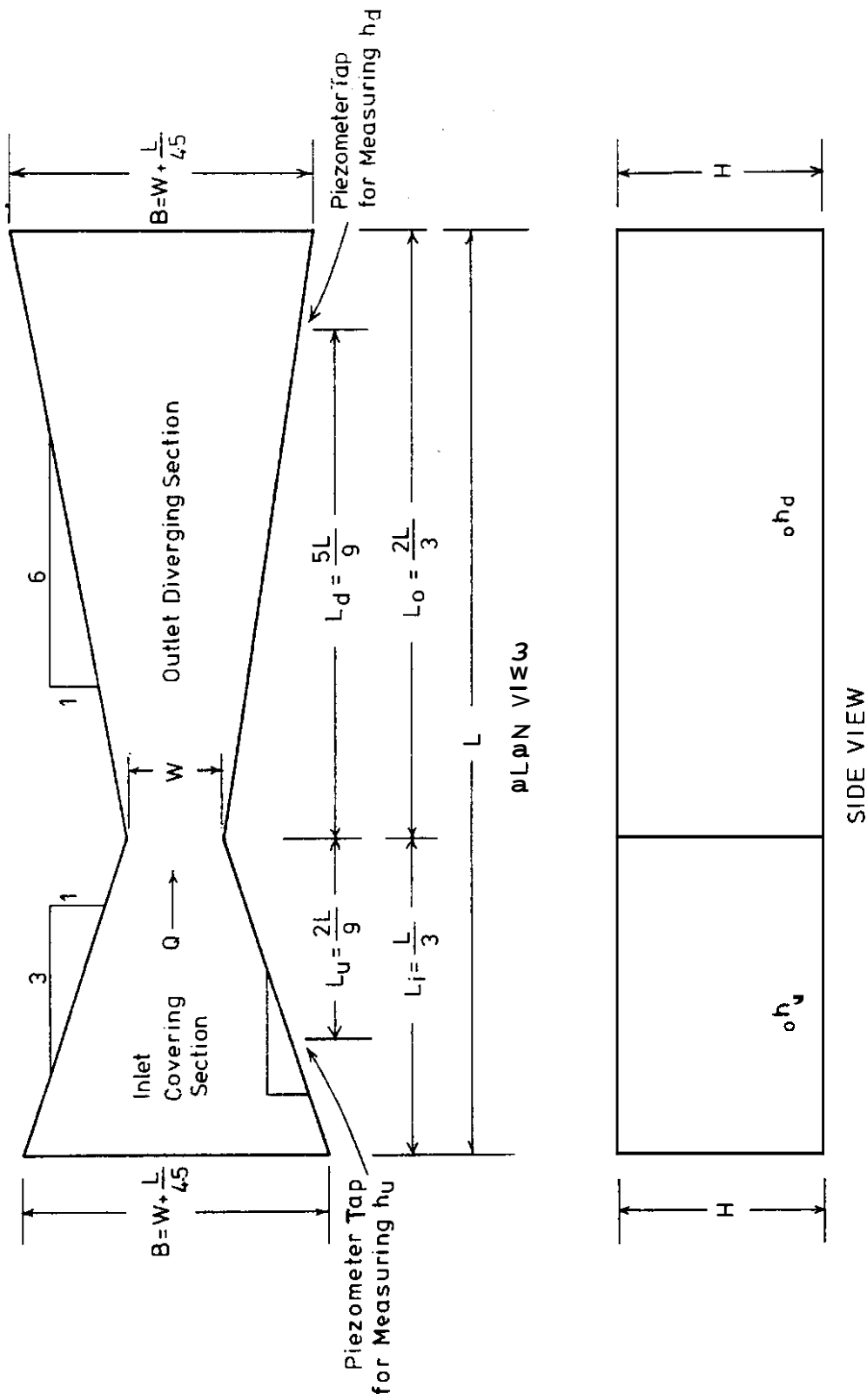
Fig. 3.1. Definition sketch of a Cutthroat Flume

3.2   HYDRAULICS

3.2.1   Flow Conditions

The various flow conditions that can occur in a Cutthroat Flume are illustrated in Figure 3.2 Water surface profile i depicts critical depth flow occurring in a Cutthroat Flume. Critical depth flow, free flow, and modular flow are synonymous terms, although earlier hydraulic literature in the Indian Subcontinent used the term semi-modular, while modular was defined as the discharge rate being independent of flow depths. Critical depth is defined by the value of the Froude Number being equal to unity.

$$F = \frac{V}{\sqrt{gh_c}} = \frac{Q/A}{\sqrt{gh_c}} = 1 \quad \dots \dots \dots \dots \dots \dots \dots \quad (3.1)$$

where,
F = Froude number, dimensionless;
V = velocity of flow, L/T;
g = acceleration due to gravity L/T^2
h_c = critical depth, L;
Q = discharge rate, L^3/T; and
A = cross-sectional area of flow, L^2.

The critical depth is calculated from the following equation:

$$h_c = \sqrt[3]{q^2/g} \quad\quad\quad (3.2)$$

where
q = discharge rate per unit of width, L^2/T.

In a Cutthroat flume, critical depth occurs a very slight distance (a few millimeters) upstream from the throat.

The most important characteristic of critical depth flow is that any changes in flow conditions downstream from the location where critical depth occurs will not change the flow conditions upstream from where critical depth occurs. Thus, a discharge rating can be developed by measuring the flow depth at some location upstream from where critical depth occurs. The exact location for measuring the upstream flow depth will affect the discharge rating, however.

20
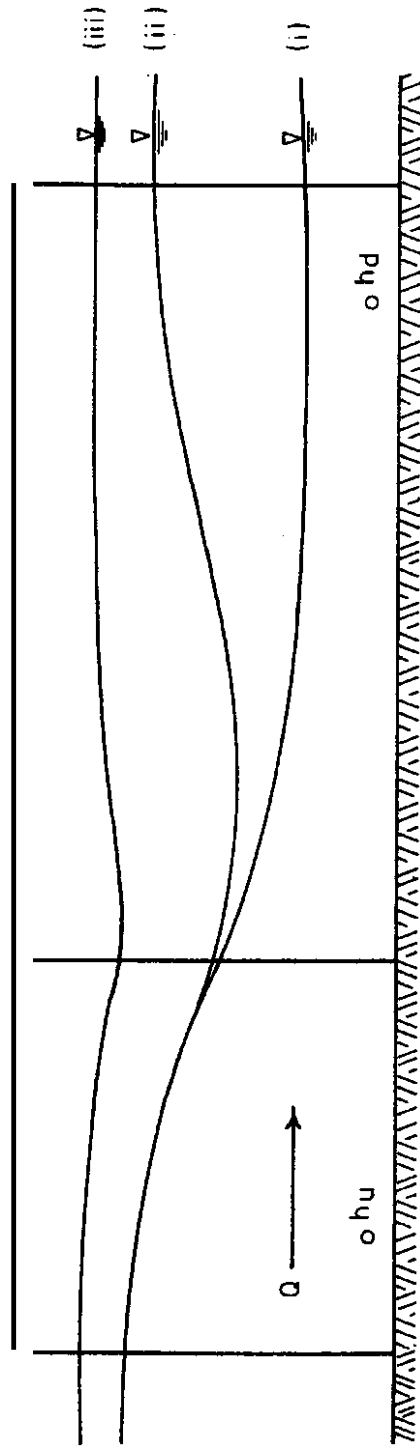
Fig. 3.2. Illustration of flow conditions in a Cutthroat Flume.

21

As the downstream water depth is increased, either due to changes in downstream flow relation (e.g., changes in gate openings), or more sediment deposition on the channel bed, or increased vegetative or aquatic growth in the channel, the water level upstream from the location where critical depth occurs will not increase. Water surface profile ii in Figure 3.2 depicts the maximum level for the downstream water surface without increasing the water level upstream; this is called the transition water surface profile, which is the transition from free flow to submerged flow.

By definition, the submerged flow condition exists when a change in the downstream water surface elevation causes a change in the upstream water surface elevation. However, from energy principles, it can be easily ascertained than the change in upstream water surface level will be less than the change in downstream water surface level.

For the transition water surface profile ii in Figure 3.2, all of the flow depths are grater than the critical depth except at the one location immediately upstream from the throat. Now, as the downstream water level is increased only slightly, the critical depth will not occur anywhere within the Cutthroat Flume and the upstream water surface levels will also increase. Now, the Cutthroat Flume is operating under the submerged flow condition and two flow depths must be measured when developing a discharge rating, with one flow depth being measured upstream from the throat, while the other flow depth is measured downstream from the throat.

3.2.2   Backwater

A Cutthroat Flume placed in an open channel is depicted in Figure 3.3. The dashed line in this figure represents the normal water surface profile in this open channel prior to installing the Cutthroat Flume. The flow through such constrictions is most often in the tranquil range and produces gradually varied flow for a long distance upstream and short distance downstream, although rapidly varied flow occurs at the constriction (Barrett and Skogerboe, 1973). The effect of the constriction on the water surface profile, both upstream and downstream, is conveniently measured with respect to the normal water surface profile, which is the water surface in the absence of the constriction under uniform flow conditions. Upstream of the constriction (throat of the Cutthroat Flume), an $M1$ or M2 backwater profile occurs. The maximum backwater effect, denoted by y^* in Figure 3.3, occurs a relatively short distance upstream. However, the backwater effect may extend for a considerable distance in the upstream direction, particularly for irrigation channels with flat longitudinal gradients. Immediately downstream of the constriction (throat), the flow expansion process begins and continues until the normal water surface has been again established in the channel, which is usually only a few meters downstream from the throat in the Cutthroat Flume.

PLAN VIEW

SECTION AT A-A
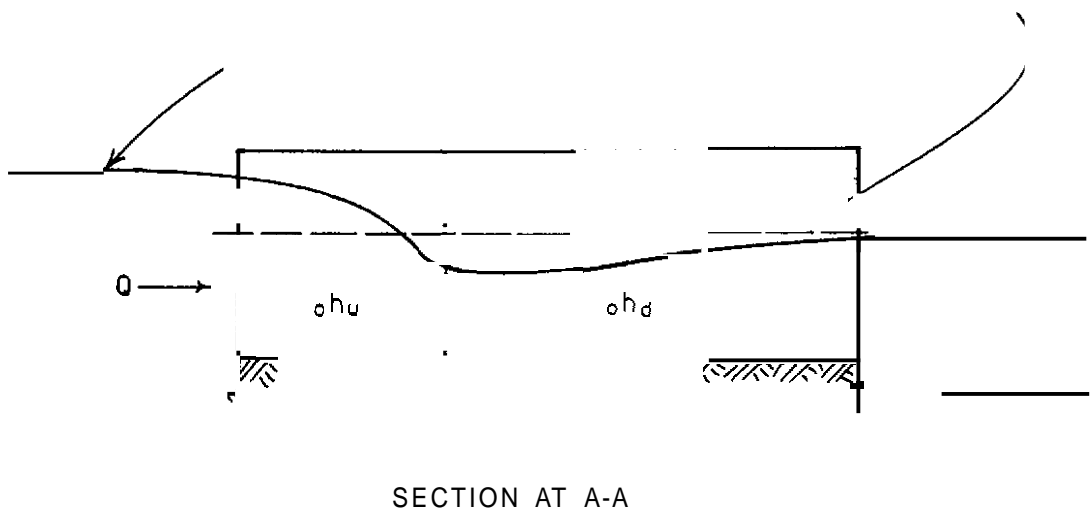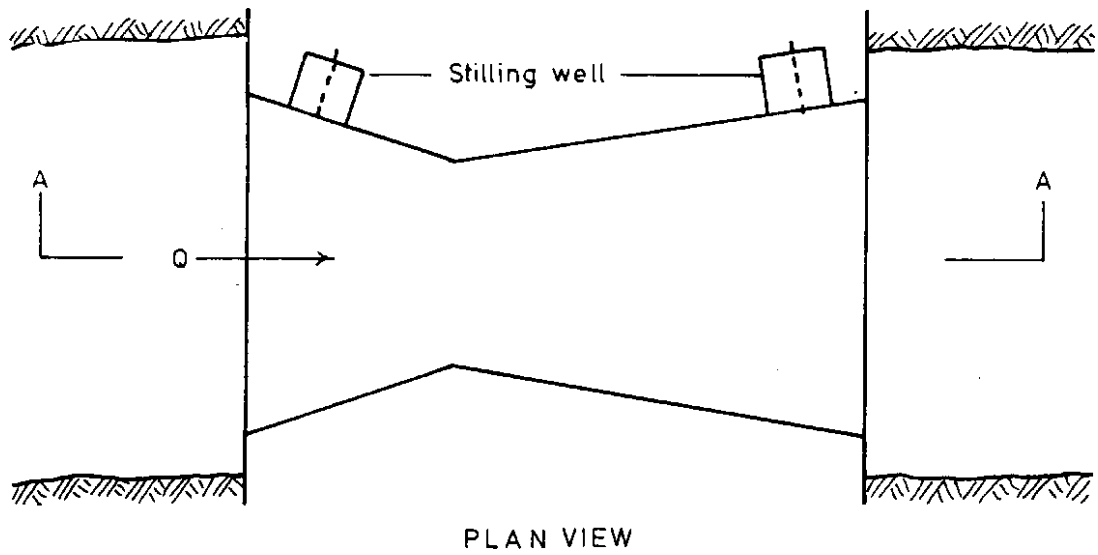
Fig. 3.3.   Illustration of backwater effects resulting from the installation of a Cutthroat Flume.

3.2.3   Free Flow

The general form of the free flow equation for an open channel constriction is:

$$Q_f = C_f h_u{}^{n_f} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots (3.3)$$

Where
  f = subscript denoting free flow;
  u = subscript denoting upstream;
  Q_f = free flow discharge rate, L^3/T;
  C_f = free flow coefficient, L^{3-n_f}/T;
  h_u = upstream flow depth, L; and
  n_f = free flow exponent, dimensionless.

The value of the free flow coefficient, C_f, increases as the throat width, W, of the Cutthroat Flume increases, but the relationship is not quite linear. The value of the free flow exponent, n_f, is primarily dependent upon the geometry of the constriction, with the theoretical value being 3/2 for a rectangular constriction such as the Cutthroat Flume. The theoretical value of n_f is modified by the approach velocity, **so** that n_f increases as the approach velocity increases, but approaches 3/2 as the approach velocity decrease towards zero. The measured values of n_f for the Cutthroat Flume are only slightly greater than 3/2 for the larger sizes (L = 9 feet) having a low approach velocity, but is nearly 2 for the smallest size of Cutthroat Flume having W = 2 inches and L = **18** inches.

The location for measuring the upstream flow depth, h_u, is shown in Figure 3.1. Note that h_u was always measured using a 1/4-inch diameter piezometer tap through the wall of the Cutthroat Flume. The centerline of this piezometer tap was one-half inch above the flume floor. This was also the case for measuring the downstream flow depth, h_d (see Figure 3.1). Each piezometer tap had a pipe or hose connection to a stilling well where the water surface elevation was measured using either a hook gauge or a point gauge during the laboratory calibrations.

The free flow equation is a power function with the upstream flow depth, h_u, being the independent variable and the free flow discharge, Q_f, is the dependent variable. Thus, a plot on logarithmic paper, with h_u potted along the abscissa and **Q,** plotted on the ordinate, will yield a straight-line relationship. If the relationship has a slight curvature, this is indicative that the zero level for measuring the upstream flow depth, h_u, is slightly in error, with greater curvature indicating greater error. The values of the free flow coefficient, C_f, and the free flow exponent, n_f, can be determined graphically or analytically using regression. Graphically, C_f is the value of Q_f for h_u = 1 and n_f is the slope of the line that is determined by using a scale.

To illustrate the procedure for graphically determining the free flow calibration, a 3'*9' Cutthroat flume will be used, where the throat width, W, is 3 feet and the flume length, L, is 9 feet. The free flow data collected in the laboratory is listed in Table **3.1.** This data is plotted in Figure 3.4. Note that the free flow coefficient, C_f, is the value of the free flow discharge, Q_f, when the upstream flow depth, h_u, is equal to 1.0.

$$Q_f = C_f(1.0)^{n_f} = C_f \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (3.4)$$

Table 3.1. Free flow laboratory data for a Cutthroat Flume having W = 3 feet and L = 9 feet.

Q_f (cfs)	h_u (feet)	Q_f (cfs)	h_u(feet)
1.73	0.315	14.80	1.264
2.66	0.417	14.87	1.268
4.90	0.618	15.00	1.270
4.98	0.624	17.12	1.384
6.70	0.755	19.96	1.532
9.97	0.980	20.12	1.528
10.03	0.989	20.31	1.550
11.83	1.086		

Also, the free flow exponent, n_f, is the slope of the line in Figure 3.4, which can be determined by using a scale to measure the vertical and horizontal sides of a triangle as shown in Figure 3.4. If preferred, both C_f and n_f can be determined analytically by using regression on a calculator. However, it is always wise to first plot the data in order to assess whether or not there are some data points that are obviously in error, which would be removed from the data set before proceeding with any analysis. Thus, for the data shown in Figure 3.4, the free flow discharge equation is :

$$Q_f = 10.33 h_u^{1.55} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (3.5)$$
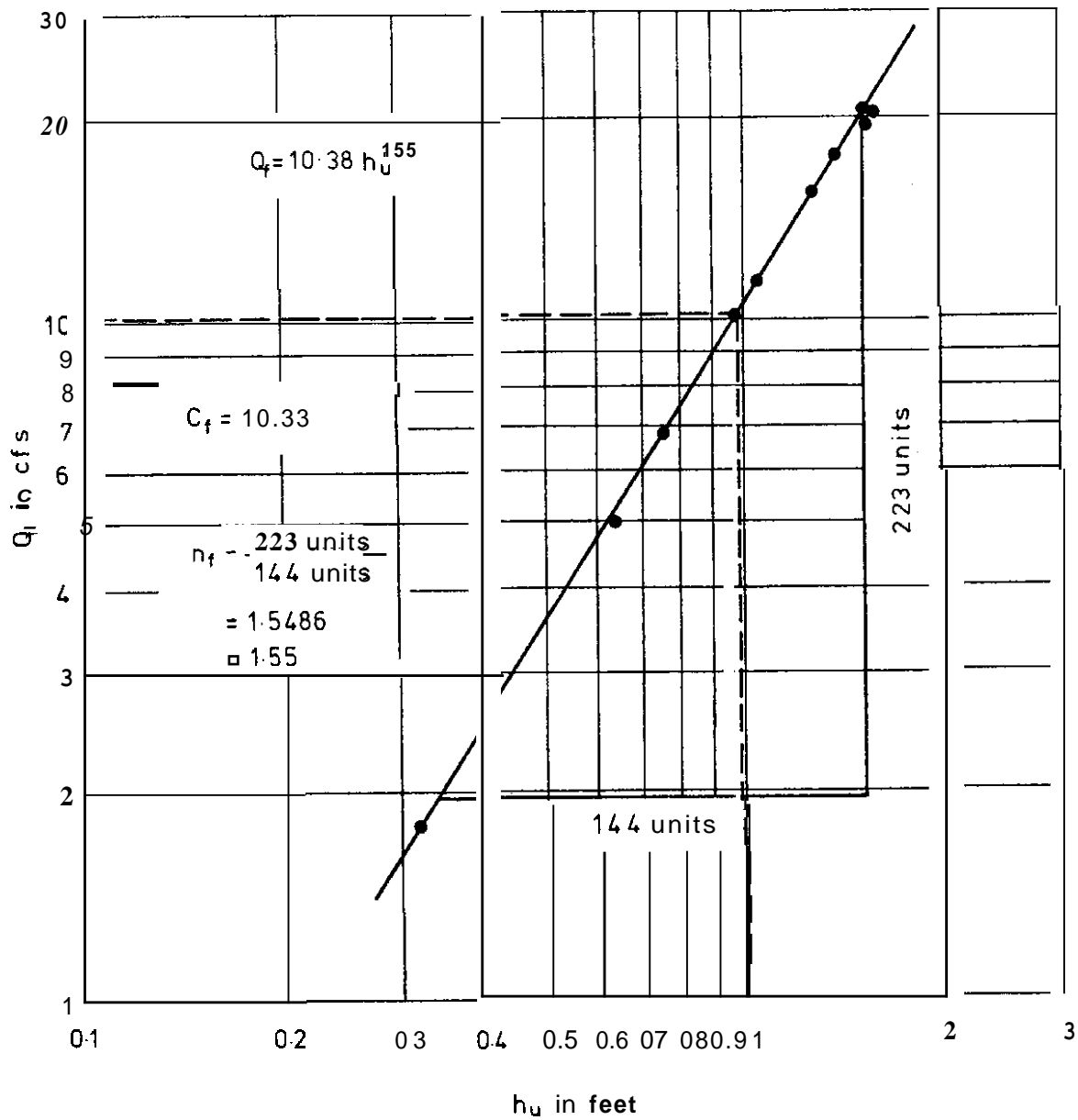
Fig. 3.4. Example of free flow laboratory discharge rating for a Cutthroat Flume having W = 3 feet and L = 9 feet.

26

3.2.4   <u>Submeraed Flow</u>

The general form of the submerged flow discharge equation, as developed be Skogerboe and Hyatt (1967a), **is:**

$$Q_s = \frac{C_s(h_u - h_d)^{n_f}}{(-\log S)^{n_s}} \quad \ldots\ldots\ldots\ldots\ldots \quad (3.6)$$

Where
- s = subscript denoting submerged flow;
- d = subscript denoting downstream;
- Q_s = submerged flow discharge rate, L^3/T;
- C_s = submerged flow coefficient, L^{3-n_f}/T;
- h_d = downstream flow depth, L;
- n = submerged flow exponent, dimensionless: and
- S = submergence; dimensionless.

The submergence **is** defined by:

$$S = h_d/h_u \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.7)$$

Note that the free flow exponent, n_f, is used with the term $h_u - h_d$. Consequently, the value of the free flow exponent, n_f, is obtained from the free flow discharge rating, while the submerged flow coefficient, C_s, and the submerged flow exponent, n_s, must be evaluated using the submerged flow hydraulic data collected in the laboratory. The theoretical variation in n_s is between 1.0 and 1.5 for a rectangular cross - section such as the Cutthroat Flume (Skogerboe and Hyatt. 1967a).

The procedure for developing the submerged flow discharge rating will be illustrated using the 3'*9' Cutthroat Flume. The hydraulic data collected in the laboratory and parameters calculated for each submerged flow data set are listed in Table 3.2. These data have been plotted in Figure 3.5. Note that the straight line has a negative slope $(-n_s)$ and that the submerged flow coefficient, C_s, is the value of $Q_{s(h=1)}$ when $(-\log S)$ is equal to 1.0, which corresponds to $S = 0.10$. For this 3'*9' Cutthroat Flume, the submerged flow discharge equation is:

$$Q_s = \frac{4.52(h_u - h_d)^{1.55}}{(-\log S)^{1.42}} \quad \ldots\ldots\ldots\ldots\ldots \quad (3.8)$$

The principles for developing submerged flow ratings are explained more fully in Chapter **5.**
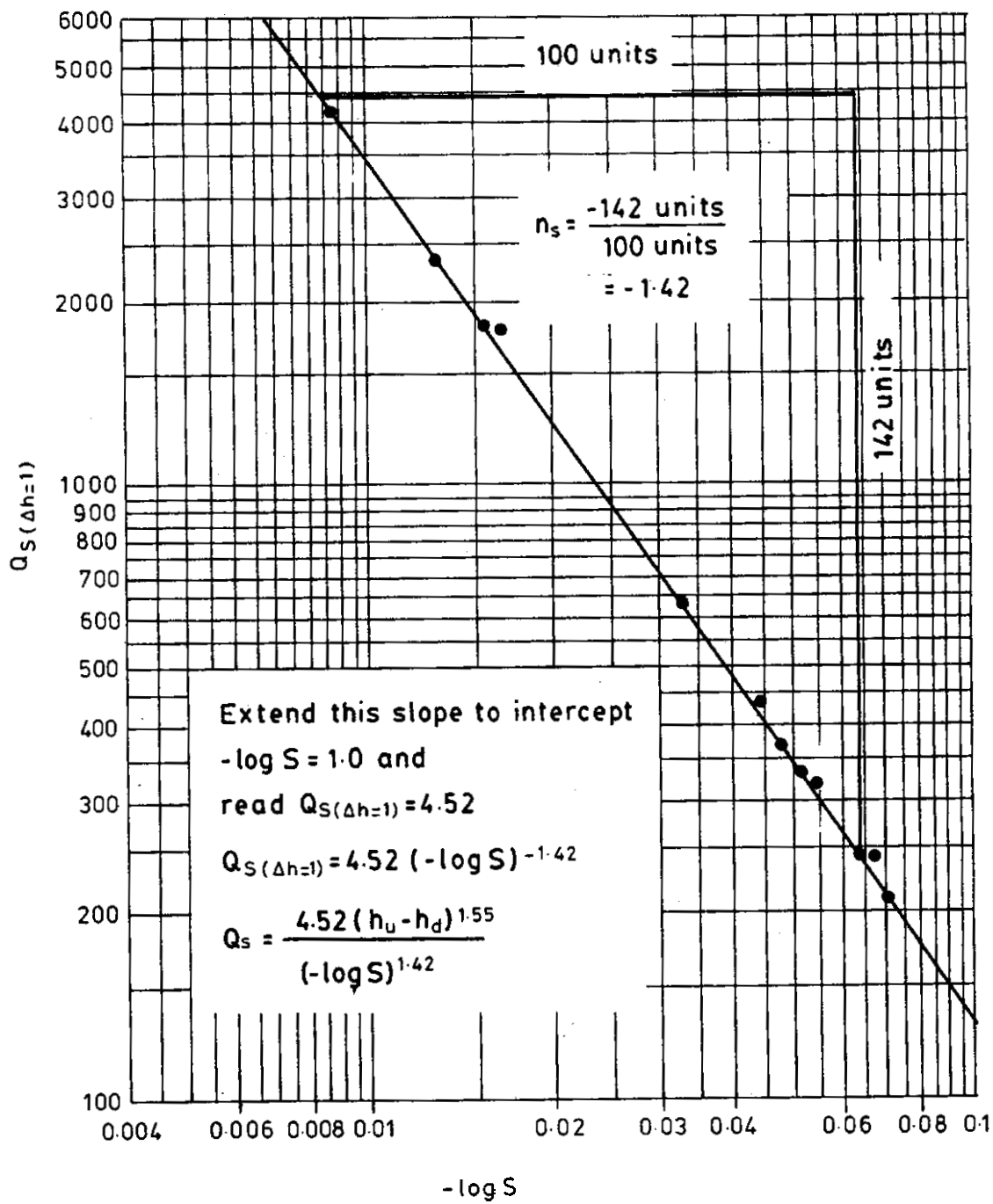
Fig. 3.5. Logarithmic plot for determining the submerged flow coefficient, C_s, and the submerged flow exponent, n_s, for the 3'x9' Cutthroat Flume.

Table 3.2 **Example of free flow laboratory discharge table for the 3'*9' Cutthroat Flume.**

h_u ft.	Q_f cfs	h_u ft.	Q_f cfs	h_u ft.	Q_f cfs	h_u ft.	Q_f cfs
0.32	1.77	1.02	10.65	1.72	23.94	2.42	40.65
0.34	1.94	1.04	10.98	1.74	24.38	2.44	41.17
0.36	2.12	1.06	11.31	1.76	24.81	2.46	41.69
0.38	2.31	1.08	11.64	1.78	25.25	2.48	42.22
0.40	2.50	1.10	11.97	1.80	25.69	2.50	42.75
0.42	2.69	1.12	12.31	1.82	26.13	2.52	43.28
0.44	2.89	1.14	12.66	1.84	26.58	2.54	43.81
0.46	3.10	1.16	13.00	1.86	27.03	2.56	44.35
0.48	3.31	1.18	13.35	1.88	27.48	2.58	44.89
0.50	3.53	1.20	13.70	1.90	27.94	2.60	45.43
0.52	3.75	1.22	14.06	1.92	28.39	2.62	45.97
0.54	3.97	1.24	14.42	1.94	28.85	2.64	46.51
0.56	4.21	1.26	14.78	1.96	29.32	2.66	47.06
0.58	4.44	1.28	15.15	2.98	29.78	2.68	47.61
0.60	4.68	1.30	15.51	2.00	30.25	2.70	48.16
0.62	4.92	1.32	15.89	2.02	30.72	2.72	48.72
0.64	5.17	1.34	16.26	2.04	31.19	2.74	49.27
0.66	5.42	1.36	16.64	2.06	31.67	2.76	49.83
0.68	5.68	1.38	17.02	2.08	32.14	2.78	50.39
0.70	5.94	1.40	17.40	2.10	32.62	2.80	50.96
0.72	6.21	1.42	17.79	2.12	33.11	2.82	51.52
0.74	6.48	1.44	18.18	2.14	33.59	2.84	52.09
0.76	6.75	1.46	18.57	2.16	34.08	2.86	52.66
0.78	7.03	1.48	18.97	2.18	34.57	2.88	53.23
0.80	7.31	1.50	19.37	2.20	35.06	2.90	53.80
0.82	7.59	1.52	19.77	2.22	35.56	2.92	54.38
0.84	7.88	1.54	20.17	2.24	36.06	2.94	54.96
0.86	8.18	1.56	20.58	2.26	36.56	2.96	55.54
0.88	8.47	1.58	20.99	2.28	37.06	2.98	56.12
0.90	8.77	1.60	21.40	2.30	37.56	3.00	56.71
0.92	9.08	1.62	21.82	2.32	38.07		
0.94	9.39	1.64	22.24	2.34	35.58		
0.96	9.70	1.66	22.66	2.36	39.09		
0.98	10.01	1.68	23.08	2.38	36.61		
1.00	10.33	1.70	23.51	2.40	40.13		

3.2.5 Transition Submergence

The validity of the general form of the submerged flow discharge equation (Equation 3.6) was further established by Skogerboe and Hyatt (1967a) when it was shown that the free flow discharge equation (Equation 3.3) was equal to the submerged flow discharge equation at the transition from the free flow condition to the submerged flow condition. Thus, this transition from one flow condition to the other can be evaluated by setting Equation 3.3 equal to Equation 3.6:

$$C_f h_u^{n_f} = \frac{C_s (h_u - h_d)^{n_f}}{(-\log S)^{n_s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.9)$$

Dividing both sides of Equation 3.9 by $h_u^{n_f}$,

$$C_f = \frac{C_s (1 - S)^{n_f}}{(-\log S)^{n_s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.10)$$

By rearranging terms,

$$C_f (-\log S)^{n_s} = C_s (1 - S)^{n_f} \quad \cdots\cdots\cdots\cdots\cdots \quad (3.11)$$

By definition, the value of the submergence in Equation 3.11 is the transition submergence, S_t. Thus,

$$C_f (-\log S_t)^{n_s} = C_s (1 - S_t)^{n_f} \quad \ldots\ldots\ldots\ldots\ldots \quad (3.12)$$

Equation 3.12 must be solved by trial-and-error in order to determine the value of S_t for any particular open channel constriction, including each particular size of Cutthroat Flume. Ren (1993) has shown that the value of S_t is very sensitive to small changes in the coefficients and exponents in Equation 3.12. In fact, of the five discharge parameters (C_f, n_f, C_s, n_s and S_t), the transition submergence is much more sensitive than the other discharge parameters (Ren, 1993).

3.2.6 Representation of Discharge Ratings

Both the free flow and submerged flow discharge ratings can be represented together on logarithmic paper. An example **is** shown in Figure 3.6 for the 3'* 9' Cutthroat flume. The free flow discharge rating (Equation 3.5) is superimposed upon the graphical representation of the submerged flow discharge rating (Equation 3.12) where the submergence, S, is equal to the transition submergence, S_t.

30
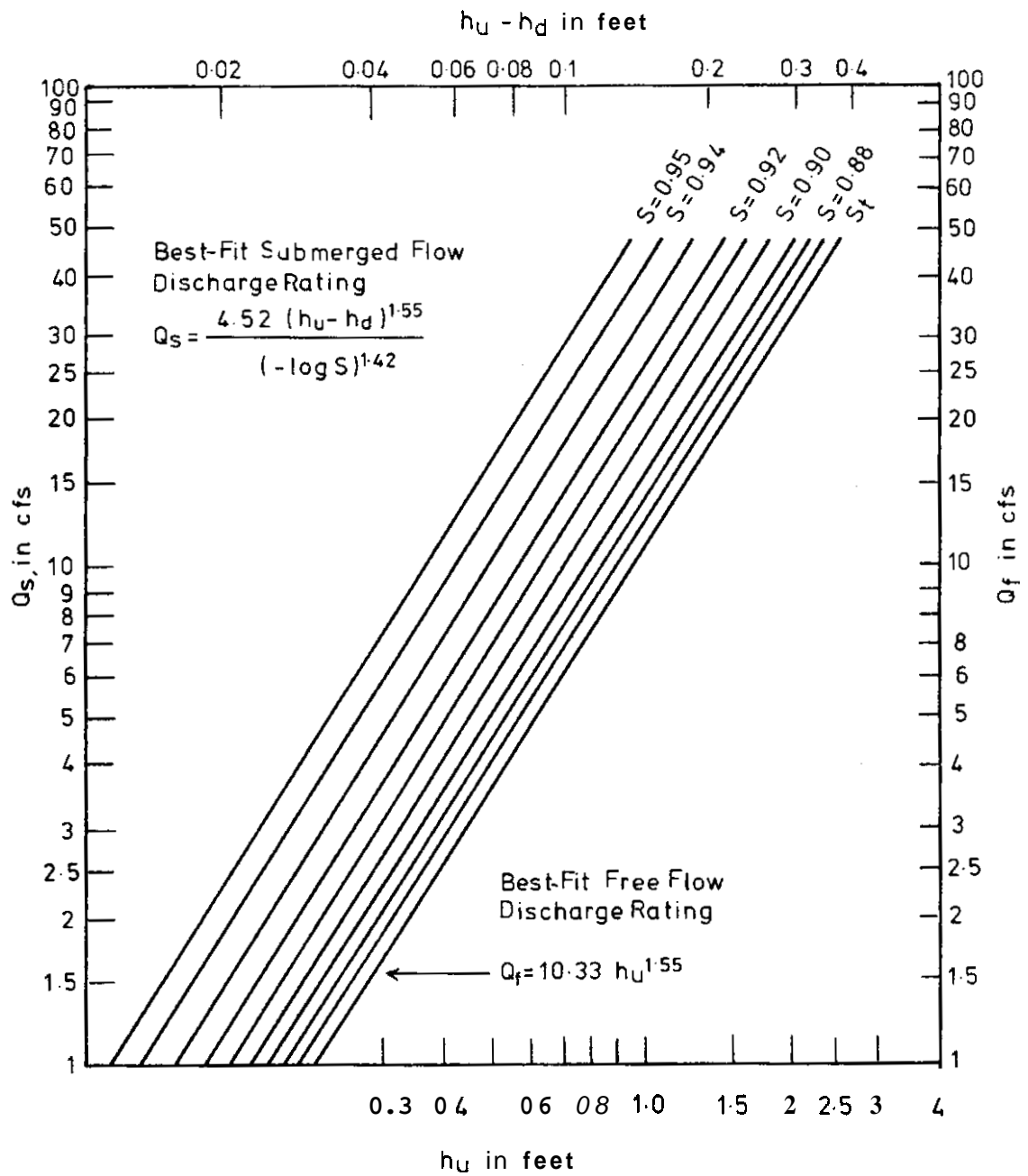
Fig. 3.6. Example logarithmic plot of both the free flow and submerged flow laboratory discharge ratings for the 3'x9' Cutthroat Flume.

The most useful method for representing the free flow discharge equation is in a free flow discharge rating table. Then, an individual can measure the upstream flow depth, h_u, in the open channel constriction and then find the corresponding free flow discharge rate, Q_f, in the rating table. An example of a free flow discharge rating table is shown in Table 3.2 for the 3'* 9 Cutthroat Flume.

Previously, submerged flow discharge rating tables have been published for various sizes of Cutthroat Flume (Skogerboe. Bennett and Walker, 1973). Such tables require the listing of the upstream flow depth, h_u, in the first column, followed by many columns that list incremental values of the difference in upstream and downstream flow depths, $h_u - h_d$. This requires many pages to represent the submerged flow discharge rating for a single size of Cutthroat Flume. Another technique is *to* represent in a submerged flow discharge rating table only a factor that has a unique value for each value of the submergence, S. This can be done by calculating the ratio of the submerged flow discharge rate by the free flow discharge, Q_s/Q_f;

$$\frac{Q_s}{Q_f} = \frac{C_s(h_u - h_d)^{n_f}}{(-\log s)^{m}} \frac{1}{C_f h_u^{n_o}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (3.13)$$

or

$$\frac{Q_s}{Q_f} = \frac{C_s(1 - S)^{n_f}}{C_f(-\log S)^{n_s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (3.14)$$

For each size of Cutthroat Flume, values of the submergence, S, greater than the transition submergence, S_t, are substituted into Equation 3.13 and the submerged flow multiplication factor, Q_s/Q_f, is calculated.

The procedure for using the rating tables to determine the submerged flow discharge rate, Q_s, is to:

1. Use the measured value of the upstream flow depth, h_u, *to* obtain the free flow discharge rate, Q_f, from the free flow rating table (e.g., Table 3.3);

2. Use the measured value of the downstream flow depth, h_d, and divide by the measured upstream flow depth, h_u, to calculate the value of the submergence, $S = h_d/h_u$;

Table 3.3   Free flow discharge ratings for Cutthroat Flumes having a length L = 3.0'

h, ft	W = 4" cfs	W = 8" cfs	W = 12" cfs	W = 16" cfs
0.30	0.153	0.317	0.489	0.668
0.32	0.173	0.357	0.550	0.750
0.34	0.193	0.399	0.614	0.837
0.36	0.214	0.442	0.681	0.927
0.38	0.237	0.488	0.751	1.022
0.40	0.260	0.536	0.824	1.121
0.42	0.285	0.586	0.900	1.223
0.44	0.310	0.638	0.979	1.330
0.46	0.336	0.692	1.061	1.441
0.48	0.364	0.748	1.146	1.555
0.50	0.392	0.806	1.234	1.673
0.52	0.422	0.866	1.325	1.796
0.54	0.452	0.928	1.419	1.922
0.56	0.483	0.991	1.515	2.051
0.58	0.515	1.057	1.615	2.185
0.60	0.548	1.125	1.717	2.322
0.62	0.583	1.194	1.822	2.463
0.64	0.618	1.265	1.930	2.608
0.66	0.654	1.338	2.040	2.756
0.68	0.691	1.413	2.154	2.908
0.70	0.728	1.490	2.270	3.063
0.72	0.767	1.569	2.388	3.222
0.74	0.807	1.649	2.510	3.385
0.76	0.847	1.732	2.634	3.551
0.78	0.889	1.816	2.761	3.721
0.80	0.931	1.902	2.891	3.894
0.82	0.975	1.989	3.023	4.071
0.84	1.019	2.079	3.158	4.251
0.86	1.064	2.170	3.295	4.434
0.88	1.110	2.263	3.435	4.622
0.90	1.157	2.358	3.578	4.812
0.92	1.204	2.454	3.723	5.006
0.94	1.253	2.553	3.871	5.203
0.96	1.302	2.653	4.021	5.404
0.98	1.353	2.754	4.174	5.608
1.00	1.404	2.858	4.330	5.815

3.   Using the calculated value of the submergence obtained in Step 2, obtain the value of the submerged flow multiplication factor, Q_s/Q_f, from the submerged flow rating table (e.g. Table 3.4); and

4.   Multiply the results from steps 1 and 3 to calculate the value of the submerged flow discharge rate, Q_s.

$$Q_s = (Step\,1)(step\,3) = Q_f(Q_s/Q_f) \quad \dots\dots\dots\dots\dots (3.15)$$

3.3   SIZE OF CUTTHROAT FLUME

Laboratory data collected from 1966-90 have been analyzed by Ren (1993) for 15 sizes of Cutthroat Flume fabricated in English units. A total of 842 hydraulic runs were used, of which 375 runs were under free flow conditions and 468 runs of submerged flow conditions. Thus, each size of Cutthroat flume had an average of 25 free flow data and 31 submerged flow data.

Unified free flow (C_f, n_f) and submerged flow (C_s, n_s, S_t) discharge parameters were developed (Skogerboe, Ren and Yang, 1993), first of all in English units, which were then converted to metric units. The discharge parameters for 24 sizes of Cutthroat Flume in English units are listed in Table 3.2.

3.4   INSTALLATION OF CUTTHROAT FLUME

Any flow measuring device must be properly installed to yield adequate results. The first consideration prior to installing a Cutthroat Flume is the location or site of the structure. The flume should be placed in a straight section of channel. If operating conditions require frequent changing of the discharge, the flume may be conveniently located near a point of diversion or regulating gate; however, care should be taken to see that the flume is not located too near a gate because of unstable or surging effects which might result from the gate operation. Also, a Cutthroat Flume should not be located immediately downstream from a constriction (e.g., culvert, gate, bridge, prier, egc.).

After the site has been selected, it is necessary to determine certain design criteria. The maximum quantity of water to be measured, the depth of flow in the channel corresponding to this discharge, and the allowable head loss through the flume must be determined. For design purposes, the head loss may be taken as the difference in water surface elevation between the flume entrance and exit, which is approximately equal to h_u - h_t. The downstream depth of flow will remain essentially the same after installation of the flume as it was prior to installation, but the upstream depth will increase by the head loss. The allowable increase in upstream depth may be limited by the height of the canal banks upstream from the flume. Such a limiting condition dictates the minimum flume size, and may require operation as a submerged flow structure. Economic factors limit the maximum flume size.

34

Table 3.4   Submerged flow multiplication factors for Cutthroat Flumes with L=3.0'

S	W = 4" Q_s/Q_f	W = 8" Q_s/Q_f	W = 12" Q_s/Q_f	W = 18" Q_s/Q_f
0.610	0.998			
0.620	0.997			
0.630	0.995			
0.640	0.992			
0.650	0.989			
0.660	0.986			
0.670	0.982			
0.680	0.978			
0.690	0.973	0.999		
0.700	0.967	0.998		
0.710	0.961	0.997		
0.720	0.955	0.995		
0.730	0.948	0.992		
0.740	0.940	0.989		
0.750	0.931	0.986		
0.760	0.922	0.981		
			0.999	
0.780	0.902	0.971	0.998	
0.790	0.890	0.964	0.996	
			0.994	
0.810	0.865	0.949	0.991	
0.820	0.851	0.940	0.987	

Table 3.4 (Completed)

0.830	0.836	0.931	0.982	0.999
0.840	0.819	0.920	0.977	0.998
0.850	0.802	0.908	0.970	0.996
0.855	0.793	10.901	0.966	0.994
0.860	0.783	0.894	0.963	0.993
0.865	0.773	0.887	0.958	0.991
0.870	0.763	0.880	0.954	0.989
0.875	0.753	0.872	0.949	0.986
0.880	0.742	0.863	0.943	0.983
0.885	0.730	0.855	0.938	0.980
0.890	0.718	0.845	0.932	0.977
0.895	0.706	0.836	0.925	0.973
0.900	0.693	0.825	0.918	0.969
0.905	0.679	0.815	0.911	0.965
0.910	0.665	0.803	0.902	0.960
0.915	0.651	0.791	0.894	0.954
0.925	0.619	0.764	0.874	0.941
0.930	0.602	0.750	0.863	0.934
0.940	0.565	0.717	0.838	0.916
0.945	0.545	0.699	0.824	0.906
0.950	0.524	0.680	0.808	0.895

Table 3.5 Unified Discharge Parameters for Appropriate Sizes of Cutthroat Flume in English Units.

W *L	C_i	n_i	S_i	C_s	n_s	$Q_{f(max)}$ cfs
+2"*1.5'	0.966	1.982	0.553	0.663	1.453	0.24
+4"*1.5'	1.965	1.968	0.651	1.105	1.576	0.50
+6"*1.5'	2.978	1.954	0.734	1.391	1.668	0.77
+8"*1.5'	3.999	1.939	0.798	1.606	1.728	1.04
+4"*3.0'	1.404	1.840	0.580	0.942	1.384	1.40
+8"*3.0'	2.858	1.826	0.674	1.600	1.489	2.86
x12"*3.0'	4.330	1.811	0.754	2.048	1.567	4.33
+16"*3.0'	5.815	1.797	0.815	2.395	1.636	5.82
+6"*4.5'	1.882	1.721	0.614	1.220	1.335	3.78
+12"*4.5'	3.830	1.707	0.708	2.081	1.494	7.65
+18"*4.5'	5.804	1.692	0.788	2.678	1.498	11.53
+24"*4.5'	7.794	1.678	0.849	3.138	1.544·	15.39
x8"*6.0'	2.356	1.657	0.629	1.509	1.303	7.43
x16"*6.0'	4.794	1.643	0.723	2.586	1.391	14.97
x24"*6.0'	7.265	1.628	0.801	3.352	1.454	22.46
x32"*6.0'	9.765	1.614	0.862	3.942	1.497	29.89
x10"*7.5'	2.833	1.613	0.641	1.799	1.281	12.42
x20"*7.5'	5.765	1.599	0.735	3.092	1.365	24.95
x3"0*7.5'	8.735	1.584	0.811	4.026	1.424	37.29
x"40*7.5'	11.731	1.570	0.873	4.724	1.466	49.44
+12"*9.0'	3.340	1.581	0.651	2.104	1.266	18.97
+24"*9.0'	6.796	1.567	0.743	3.629	1.346	38.01
+36"*9.0'	10.327	1.553	0.820	4.741	1.404	56.88
+48"*9.0'	13.831	1.539	0.882	5.558	1.444	75.01

x   Interpolated values

37

A properly installed Cutthroat Flume is aligned straight with the channel and should be level longitudinally and laterally. Flumes tend to settle in time, with the exit usually becoming lower than the entrance.

The most important dimension in constructing a Cutthroat Flume is the width, W. One of the principal advantages of a Cutthroat Flume is that an error in constructing the throat width can be taken into account by writing new free flow and submerged flow discharge ratings by plotting the throat width, W, on the abscissa and each discharge parameter (C_f, n, S_f, C_s, n) on the ordinate using the data in Table 3.5 for the four values of W corresponding to the Flume length, L; then, by interpolation, using the actual measured value of W, read the value for each discharge parameter. If a particular throat width is desired for a concrete Cutthroat Flume, a steel angle could be embedded in the concrete at the throat section.

Experience both in the laboratory and field indicates that a transition structure between the open channel and Cutthroat Flume is not necessary. However, the ratio of upstream flow depth to flume length (h_u/L) should be 0.33, or less, for free flow conditions. For most installations in flat gradient channels, this will insure that approach conditions will satisfy the laboratory conditions under which the ratings were developed. Measurements should be made in the Cutthroat Flume by the use of piezometers connected to stilling wells as illustrated in Figure 3.7. Only fair accuracy is obtained from the use of staff gauges. When used, staff gauges should be set vertically at the specified location for h_u and h_d along the converging and diverging walls. The staff gauge must be carefully referenced to the elevation of the flume floor. Use of stilling wells is strongly recommended, however, for accuracy. Stilling wells have the advantage of providing a calm water surface compared with the fluctuation or "bounce" of the water surface that usually exists within the Cutthroat Flume. Stilling wells are also necessary if continuous recording instruments are to be used. Under submerged flow conditions, two stilling wells placed adjacent to each other as shown in Figure 3.7 are very desirable and facilitate the use of a double head recording instrument for obtaining a continuous record with time of h_u and h

3.4.1   Free Flow

If circumstances allow, it is preferable to have a flow measuring device operate under free flow conditions. The obvious advantage is that only the upstream flow depth need be measured to determine the discharge. Also, the accuracy in determining the discharge rate is better for free flow as compared with submerged flow. The procedure to follow for installing a Cutthroat Flume to operate under free flow conditions is listed below:

38
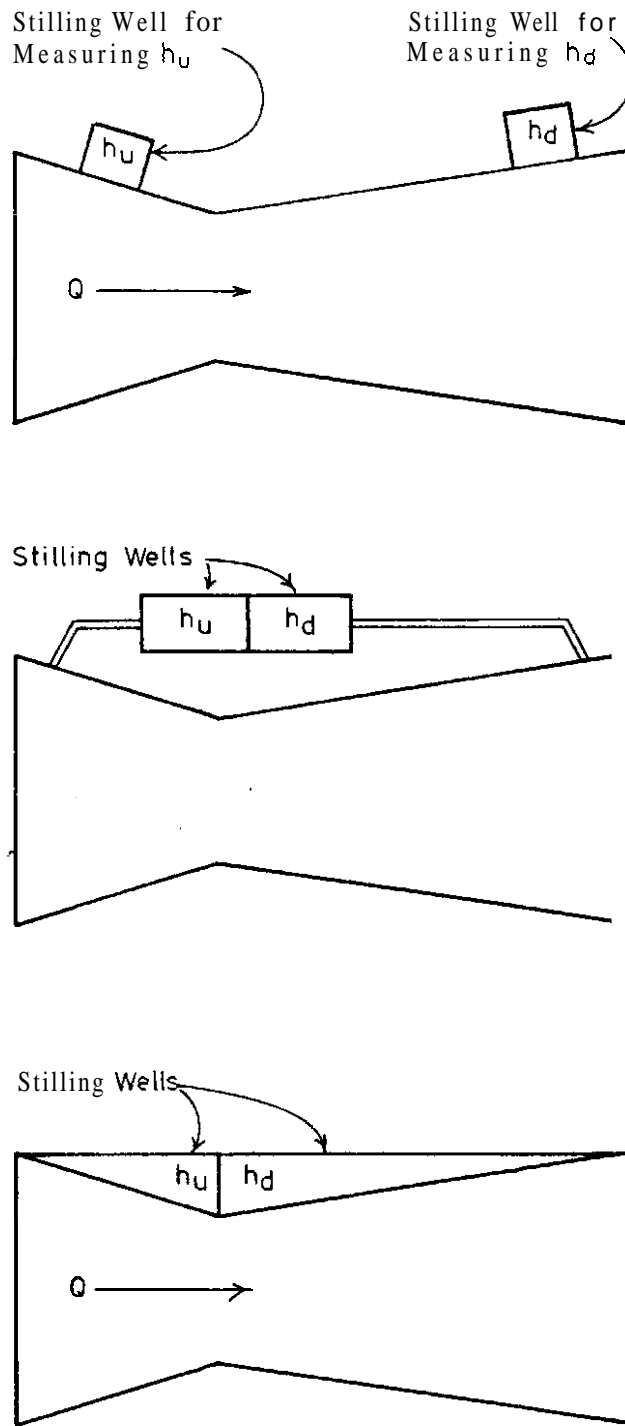
Fig. 3.7. Plan view of Cutthroat Flume showing various methods of constructing stilling wells.

1. Determine the maximum flow rate to be measured.
2. **At** the site selected for installing the flume, locate the high water line on the canal bank and determine the maximum depth of flow.
3. For a selected flume size, use the free flow discharge rating.
4. Place the floor of the flume at an elevation which does not exceed h_u multiplied by the transition submergence (S_t) below the high waterline. Generally, the flume bottom should be placed as high as grade and other conditions permit to insure free flow.

Example : An appropriate size of Cutthroat Flume must be selected for measuring a maximum discharge of 10.5 cfs under free flow conditions. Presently, the maximum flow depth in the channel is 0.95 foot, but there is sufficient freeboard that the water level could be increased another 0.3 foot.

Under these conditions, the maximum downstream flow depth, h_d, would be 0.95 foot and the maximum upstream flow depth, h_u, would be 1.25 feet (0.95 + 030). Thus, the submergence would be :

$$S = 0.95 foot/1.25 feet = 0.760 \quad \cdots\cdots\cdots\cdots\cdots \quad (3.16)$$

A review of Table 3.2 would indicate that a 18in • 4.5ft
Cutthroat Flume might be suitable:

$$Q_f = 5.804(1.25 feet)^{1.692} = 8.47 cfs \quad \cdots\cdots\cdots\cdots\cdots \quad (3.17)$$

So, this flume would be too small.

Therefore, check the free flow discharge capacity for a 24in • 4.5ft Cutthroat Flume:

$$Q_f = 7.794(1.25 feet)^{1.678} = 11.33 cfs \quad \cdots\cdots\cdots\cdots \quad (3.18)$$

Consequently, this would be an appropriate size of Cutthroat Flume
to install.

For a free flow discharge rate of 10.5 cfs:

$$h_u = (10.5/7.94)^{1/1.678} = 1.19 ft \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad \textbf{(3.19)}$$

Since the transition submergence is 0.849:

$$S_t h_u = 0.849(1.19 ft) = 1.01 ft \quad \cdots\cdots\cdots\cdots\cdots \quad \textbf{(3.20)}$$

Consequently, the floor of the 24in * 4.5ft Cutthroat Flume should be set no lower than 1.01 feet below the present maximum water level, which would be lower than the channel bed. However, in order to take advantage of the available freeboard, the flume floor could be placed 1.19 feet below the new maximum water level as shown in Figure 3.8, which would result in h_u = 1.19 feet, h_d = 1.19 - 0.30 = 0.89 foot, and S = 0.89/1.19 = 0.75.

3.4.2   Submerged Flow

The existence of certain conditions, such as insufficient grade or the growth of moss and vegetation, sometimes makes it impossible or impractical to install a flume to operate under free flow conditions. Where such situations exist, a flume may be set in the channel to operate under submerged flow conditions. The principal advantage of submerged flow operation is the smaller head loss which occurs in the flume as compared with free flow. This reduction in head loss may mean that the channel banks upstream from the flume do not have to be raised to enable the same maximum flow capacity in the channel that existed prior to the installation of the flume. When a flat-bottomed Cutthroat Flume is installed to operate under submerged flow conditions, the flume floor may be placed at the canal bottom. This placement will allow quicker drainage of the canal section upstream from the flume, particularly for flow rates which are less than the maximum discharge; however, this may result in high values of submergence and the error in the discharge measurement will be greater. Thus, there is a significant advantage in placing the flume floor as high as conditions permit. The following procedure should be used in placing a Cutthroat Flume to operate under submerged flow conditions.

1.   Determine the maximum flow rate, Q_s, to be measured

2.   On the channel bank, where the flume is to be installed, locate the high water line to determine the maximum flow depth.

3.   Giving consideration to the amount of freeboard in the channel at maximum discharge and maximum flow depth, determine how much higher the water surface can be raised in the channel upstream from the flume location.

4.   With the floor of the flume being placed at essentially the same elevation as the bottom of the channel, the maximum depth of flow (Step 2) becomes h and the additional amount that the water surface in the canal can be raised (Step 3) becomes h_u - h Using this information, the submergence, h_d/h_u, can be computed.

41

Maximum Allowable Freeboard Level

1·19'

Q$_f$ ⟶

24" × 4 5' Cutthroat Flume
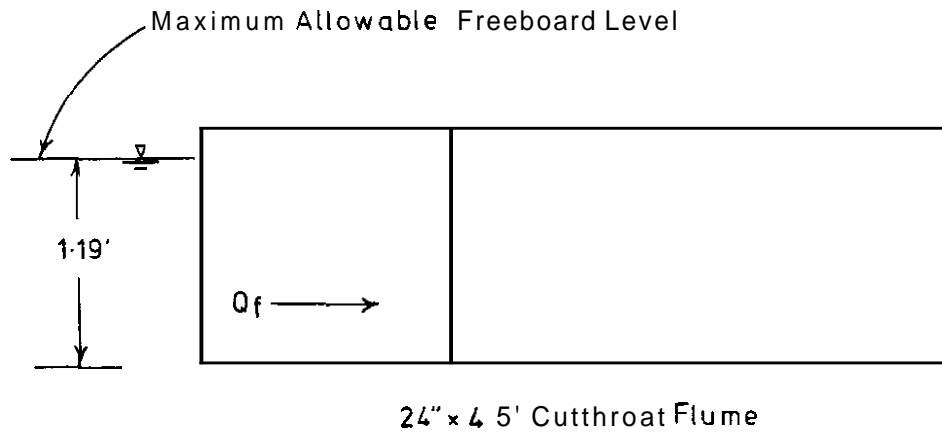
Fig. 3.8. **Floor elevation placement for free flow operation of 24in x 4.5 ft Cutthroat Flume in example problem.**

5. Select an appropriate size of Cutthroat Flume by trial-and-error. Knowing Q_s, S and h_u is important in guiding the procedure.

   a. First, the submerged flow rating tables would be consulted. The submerged flow multiplication factor, Q_s/Q_f, could be read for each flume size for the known value of submergence, S. Actually, by already knowing (or having an estimate of) Q_s will indicate to some extent the range of flume sizes that might be appropriate.

   b. Then, the estimated or known value of Q_s can be divided by the submerged flow multiplication factor, Q_s/Q_f, for each flume size to arrive at a required value of the free flow discharge rate, Q_f.

   c. Now, the known maximum value of the upstream flow depth, h_u, can be used in the free flow discharge rating tables for each flume size being investigated in order to determine whether the value of Q_f in the rating table equals or exceeds the required value of Q_f calculated in Step 5b.

   d. Based on the results from Step 5c, the most appropriate size of Cutthroat Flume can be selected.

3.5 DISCHARGE RATING TABLES

Most of the Cutthroat Flumes used in Pakistan have a flume length, L, of 3 feet. Most commonly, throat widths of 8 inches and 12 inches have been fabricated. For a flume length of 3 feet, four throat widths, W, are shown in Table 3.6; namely, 4 inches, 8 inches and 12 inches. The free flow discharge ratings for these four sizes of Cutthroat Flume were presented earlier in

Table 3.6. Sizes of Cutthroat Flume commonly used in Pakistan.

Flume size	C_f	n_f	S_t	C_s	n_s	$Q_{f(max)}$ cusecs
4"* 3'	1.404	1.84	0.580	0.942	1.384	1.40
8"* 3'	2.858	1.826	0.674	1.600	1.489	2.86
12''' 3'	4.330	1.811	0.754	2.048	1.567	4.33

Table 3.3 where the discharge is listed in cubic feet per second (cfs). or cusecs. Note that the maximum value of h_u in Table 3.3 is 1.00 foot, which corresponds to the recommended value of h_u for free flow conditions being L/3 (3 feet/3 = 1.00). The discharge rates are listed to three decimal places in order to facilitate the interpolating of h_u values intermediate to those listed in the first column of Table 3.3; however, the field discharge measurement will not be this accurate. For example, for a flume length of 3 feet and a throat width of 8 inches (8" * 3' CTF), if the measured value of h_u is 0.77 foot, then the discharge rate listed in Table 3.3 would be 1.77 cusecs in the field records.

For submerged flow conditions, the submerged flow multiplication factors (Q_s/Q_f) are listed in Table 3.4 for the four Cutthroat Flume sizes having a flume length of L = 3 feet. Note that the maximum value of submergence listed in Table 3.4 is 0.950, (95 percent). For values of submergence greater than 0.95, the coefficients and exponents listed in Table 3.3 can be used to calculate the discharge rate, but the accuracy is rapidly deteriorating as the submergence approaches 1.00.

A comparison of the transition submergence, S_t, listed in Table 3.5 with the submerged flow multiplication factors in Table 3.4 shows that the deviation between free flow and submerged flow is very slight when the submergence is only a few percentage points greater than S_t, but this difference rapidly increases as the submergence, S, increases. For example, the transition submergence for a 8" * 3' Cutthroat Flume is 0.674 (Table 3.5), whereas a one percent difference between the free flow discharge and submerged flow discharge requires a submergence of 0.73 (submerged flow multiplication factor of 0.99) as interpolated from Table 3.4. Yet, a one percent difference in submergence from 0.94 to 0.95 results in a five percent change in the submerged flow discharge rate (from Table 3.4, [1 - 0.680/0.717] = 0.0516 = 5.16 percent).

For example, using the 8"*3' Cutthroat Flume, the free flow discharge for h_u = 0.77 feet is 1.774 cusecs. If the downstream flow depth, h_d, had been 0.69, then the submergence, S, is equal to 0.857 (0.69/0.77). For this value of submergence, the submerged flow multiplication factor (Q_s/Q_f) in Table 3.4 can be interpolated as 0.8973, which results in a submerged flow discharge rate of 1.59 cusecs (1.774*0.8973 = 1.592).

CHAPTER 4. MEASUREMENT OF DISCHARGE BY CONVENTIONAL CURRENT METER METHOD

4.1 INTRODUCTION

A current meter is an instrument used to measure the velocity of flowing water. The principle of operation is based on the proportionality between the velocity of the water and the resulting angular velocity of the meter rotor. It is widely used. The accuracy in results by using a current meter depends upon the proper operation. adjustment and maintenance of this instrument.

4.2 TYPES OF CURRENT METER

There are many countries that manufacture good quality current meters. One of the more recent innovations is the electro - magnetic current meter that digitally displays the velocity measurement. The electronic types of current meters will be used much more in the future.

Current meters with a rotating unit that is sensing the water velocity are either vertical-shaft or horizontal-shaft types. The vertical-axis current meter has a rotating cup with a bearing system that is simpler in design, more rugged, and easier to service and maintain than horizontal- shaft (axis) current meters.

Because of the bearing system, the vertical-shaft meters will operate at lower velocities than horizontal-axis current meters.
The bearings are well protected from silty water, the bearing adjustment is usually less sensative, and the calibration at lower velocities where friction plays an important role is more stable (Hagan,1989).

Two of the commonly used vertical-axis current meters are the Price Type **A** Current Meter and the Pygmy Current Meter, which is used for shallow flow depths and low velocities. **A** diagram for the Price Type **A** Current Meter is shown in Figure 4.1. In addition, there are some rugged, high quality horizontal-axis current meters that give excellent results.

The horizontal-shaft current meters use a propeller. These horizontal-axis rotors disturb the flow less than the vertical-axis cup rotors because of axial symmetry with the flow direction. Also, the horizontal-shaft current meters are less sensitive to the vertical velocity copmonents. Because of its shape, the horizontal-axis current meter is less susceptible to becoming fouled by small debris and vegetative material moving with the water (Hagan, 1989).
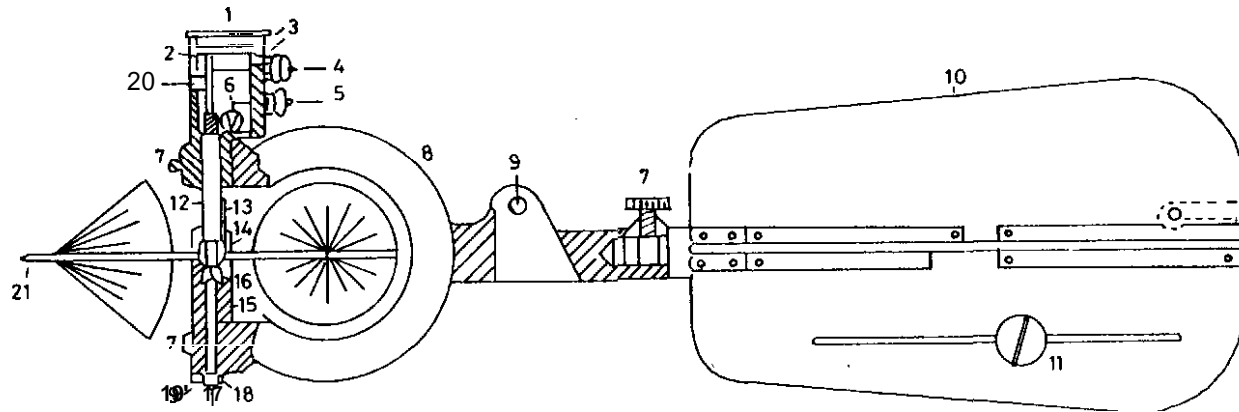
EXPLANATIONS

1	Cap for contact chamber	11	Balance weight
2	Contact chamber	12	Shaft
3	Insulating bushing for contact binding post	13	Bucket wheel hub
		14	Bucket wheel hub nut
4	Single-contact binding post (upper)	15	Raising nut
5	Penta-contact binding post (lo er)	16	Pivot bearing
6	Penta-gear	17	Pivot
7	Set screws	18	Pivot adjusting nut
8	Yoke	19	Keeper screw for pivot adjusting nut
9	Hole for hanger screw	20	Bearing lug
10	Tailpiece	21	Bucket wheel

Source,: Don M. Corbett et al 1962

Fig. 4.1.   Assembly Diagram for a Price Type A Current Meter.

Some common horizontal-axis current meters are the Ott(German),the Neyrpic (France)and the Hoff (U.S.A). Some recent models have proven to be both accurate and durable when used in irrigation channels.

4.3 GENERAL DESCRIPTION OF CURRENT METER MEASUREMENTS

A current meter measurement is the summation of the products of the subsection areas of the stream cross-section and their respective average velocities. The continuity equation is used:

$$Q = \sum (a * v) \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (4.1)$$

where:

Q is total discharge,
a is an individual subsection area,
v is thee corresponding mean velocity of the flow normal to the subsection.

In the mid-section method of computing a current meter measurement, it is assumed that the velocity sample at each vertical represents the mean velocity in a rectangular subsection. The subsection area extends laterally from half the distance from the preceding observation vertical to half the distance to the next, and the vertical from the water surface to the sounded depth. .

The cross-section is defined by depths at verticals 1,2,3,4,...n in Figure 4.2. At each vertical, the velocities are sampled using a current meter to obtain the mean velocity of each subsection. The subsection discharge is then computed for any subsection at vertical **x** by use of the equation,

$$q_x = v_x [\frac{(b_x - b_{(x-1)})}{2} + \frac{(b_{(x+1)} - b_x)}{2}] d_x \quad \cdots\cdots\cdots\cdots\cdots \quad (4.2)$$

$$q_x = v_x [\frac{b_{(x+1)} - b_{(x-1)}}{2}] d_x \quad \cdots\cdots\cdots\cdots\cdots \quad (4.3)$$

where:

q_x — discharge through subsection **x**;
v_x — mean velocity at vertical x;
b_x = distance from initial point to vertical **x**;
$b_{(x-1)}$ = distance from initial point to preceding vertical;
$b_{(x+1)}$ = distance from initial point to next vertical ; and
d_x — depth of water at vertical **x**.
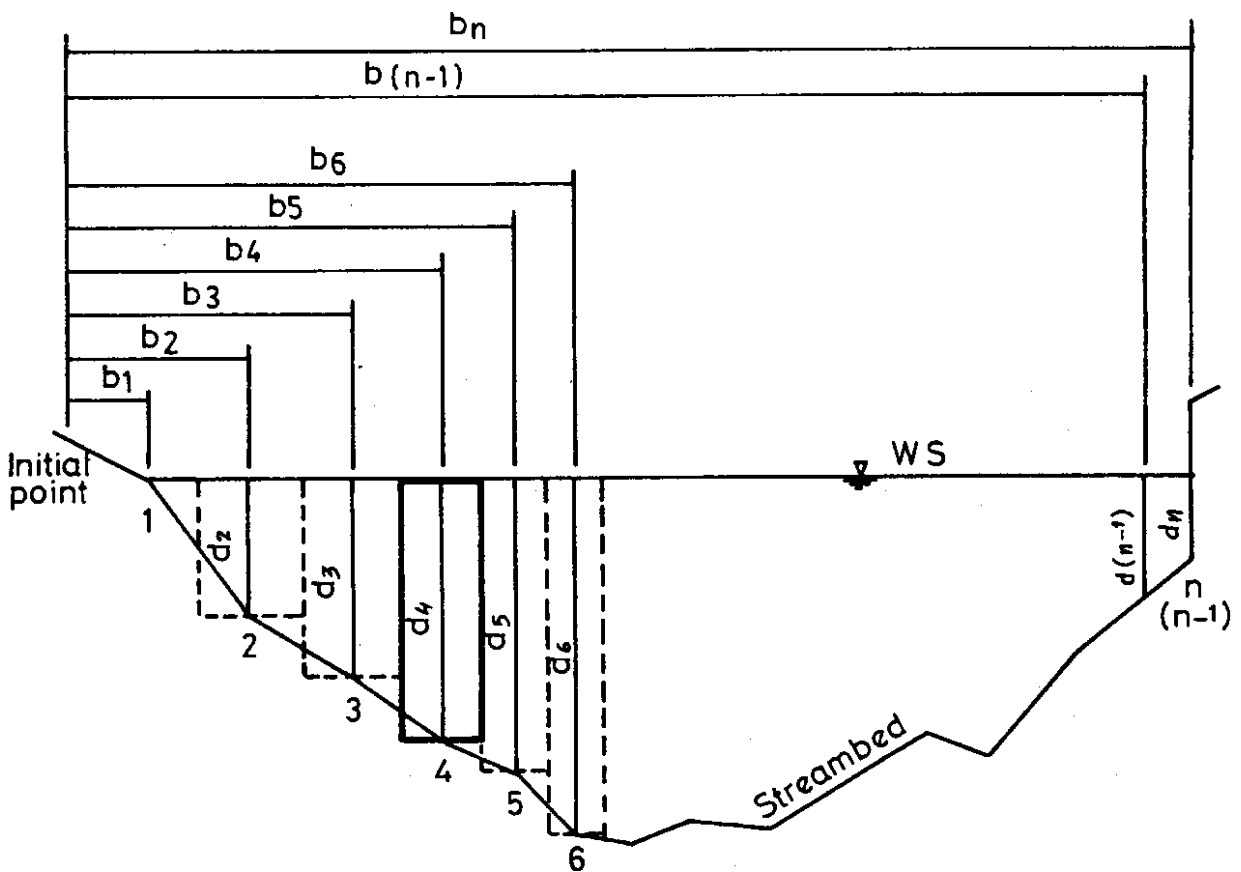
1, 2, 3........n   Observation verticals

b1, b2, b3   bn   Distance, in feet or meters, from the initial point to the observation vertical

d1, d2, d3   dn   Depth of water, in feet or meters, at the observation vertical

‒ ‒ ‒ ‒ ‒   Boundaries of subsections, on heavely outlined is discussed in text

Fig. 4.2. Definition sketch of mid-section method of computing cross-section area for discharge measurements.

Thus, for example, the discharge through Subsection **4** (heavily outlined in Figure **4.2)** is

$$q_4 = v_4 [\frac{4-4}{2}] d_4 \quad \cdots \cdots \qquad (4.4)$$

The procedure is similar when **x** is at an end section. The "preceding vertical" at the beginning of the cross section is considered coincident with vertical 1; the "next vertical" at the end of the cross section is considered coincident with vertical n. Thus (Rantz 1982),

$$q_1 = v_1 [\frac{b_2 - b_1}{2}] d_1 \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots \qquad (4.5)$$

$$q_n = v_n [\frac{b_n - b_{(n-1)}}{2}] d_n \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots \qquad (4.6)$$

4.4   VELOCITY MEASUREMENT METHODOLOGY

4.4.1   <u>Vertical Velocity Method</u>

The most complete method for establishing the mean velocity at a vertical is to take a series of current meter velocity measurements at various depths in the vertical. Often, the current meter is placed below the water surface at one-tenth of the water depth and a velocity measurement is made, then the current meter is placed at two-tenths of the water depth; this procedure is continued until the velocity has finally been measured at nine-tenths of the water depth below the water surface.

Of particular importance are the velocity measurements at relative water depths of 0.2, 0.6 and 0.8 because they are used in the simpler standard methods (see Figure 4.3).

When the above field procedure has been completed for a number of the verticals in the cross-section, the data is plotted on rectangular coordinate graph paper. The relative water depth, which varies from zero at the water surface to unity at the channel bed,
is plotted on the ordinate starting with zero at the top of the ordinate scale and unity at the bottom of the ordinate scale. Velocity is plotted on the abscissa. **A** smooth curve can be fitted on the data points for each vertical, from which the mean velocity for the vertical can be determined. **Also,** the relative water depth(s) corresponding with the mean velocity on the velocity profile can be compared between each vertical.
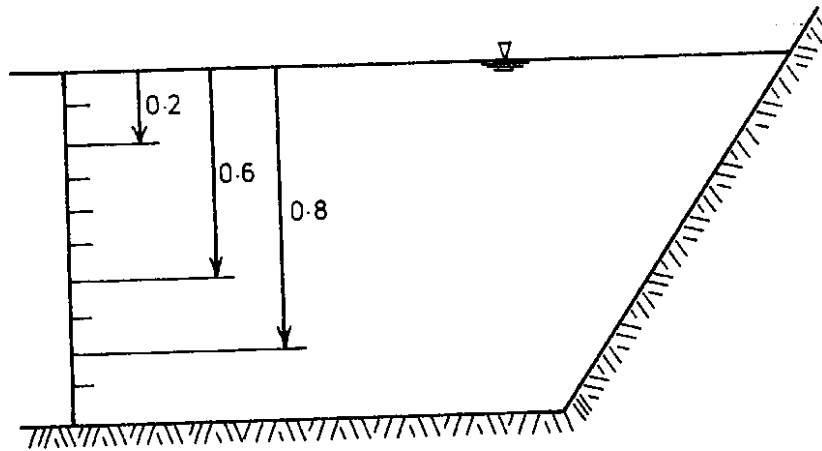
Fig. 4.3. Vertical velocity method.

Because the field procedure and data analysis are time consuming, simpler methods are commonly used which are described in the following sections. However, the Vertical Velocity Method provides an opportunity to determine whether or not the simpler procedures are valid, or if some adjustments are required.

4.4.2   Two Points Method

The most common methodology for establishing the mean velocity in a vertical is the Two Points Method. Based on many decades of experience , a current meter measurement is made at two relative water depths– 0.2 and 0.8 below the water surface. The average of the two measurements is taken as the mean velocity in the vertical.

In some field cases, it can be quite obvious that the velocity profile is distorted. For example , measurements taken downstream from a structure may have very high velocities near the water surface that can be visually observed, or near the channel bed which can be sensed by the hydrographer when using the Wading Method. If there is any suspicion that an unusual velocity profile might exist in the cross-section, then the Vertical Velocity Method should be used to establish an appropriate procedure for determining the mean velocity in a vertical for that particular cross-section.

4.4.3   Six-tenths Depth Method

For shallow water depths, say less than 75 cm for the larger current meters and **45** cm for the small current meters, the Six-tenths Depth Method is used. However, shallow is a relative term that is dependent on the type (size) of current meter being used, as well as irregularities in the channel beds (e.g.,rocks and boulders). **A** single current meter measurement is taken at a relative water depth of 0.6 below the water surface and the resulting velocity is used as the mean velocity in the vertical.

In irrigation canals, this method is commonly used at the first vertical from each bank, while the two points method is used at all of the other verticals in the cross-section. Frequently, the first vertical from each bank has a low velocity so that the discharge in each section adjacent to the left and right(looking downstream) banks represents a very small portion of the total discharge in the cross-section. In situations where shallow flow depths exist across **most** of the cross-section, and the Six-tenths Depth Method must be used because of the type of current meter that is available, then it can be expected that there will likely be considerable error, say more than ten percent.

4.5   VELOCITY AT VERTICAL WALLS

Vertical walls are frequently encountered in irrigation systems. Usually, this occurs in rectangular channels lined with concrete or brick-and-mortar. Even earthen canals will likely have some structures with a rectangular cross-section. In some cases, there may be a vertical retaining wall along only one side of the canal to stabilize the embankment. In such cases, visual observation will usually disclose that the velocity at the vertical wall is significantly greater than zero.

Hagan (1989) reports some laboratory data that is useful in estimating the mean velocity at a smooth vertical wall. This data is plotted in Figure **4.4.** For example, if the water depth at the vertical wall is denoted by D, and current meter measurements are made in a vertical located at a distance D from the wall, then the mean velocity at the wall will be the ratio 0.65 multiplied by the mean velocity measured in the vertical at the distance D from the wall.

The accuracy of the estimated mean velocity at the wall will be enhanced by measuring the mean velocity in a vertical located as close to the vertical wall as the current meter equipment will allow. Thus, if a current meter measurement could be made at a distance $D/4$ from the wall, then the estimated mean velocity at the vertical wall would be the mean velocity measured at $D/4$ from the wall multiplied by the ratio $0.65/0.90$, which is obtained from Figure **4.5.**

4.6   SELECTION OF MEASURING CROSS-SECTION

The most commonly used criterion in selecting a channel cross-section for current meter measurements is that it be located in a straight reach where parallel streamlines exist. In addition, cross-sections having large eddies and excessive turbulence would be avoided. **Also,** a cross-section having stagnant water near one of the banks would be avoided, if possible. Other important criteria are avoiding cross-sections where the flow depths are shallow (except near the banks) and the flow vetocities are too low. Rantz(1982) recommends that the flow depths should exceed **15** cm and the flow velocities should exceed **15** cm/sec.

A cross- section is needed that has no aquatic growth that can foul the current meter. Finally, a-cross-section is preferred where the channel bed is not highly irregular so that the area of the cross-section can be accurately determined; also, an irregular bed will affect the velocity profiles.
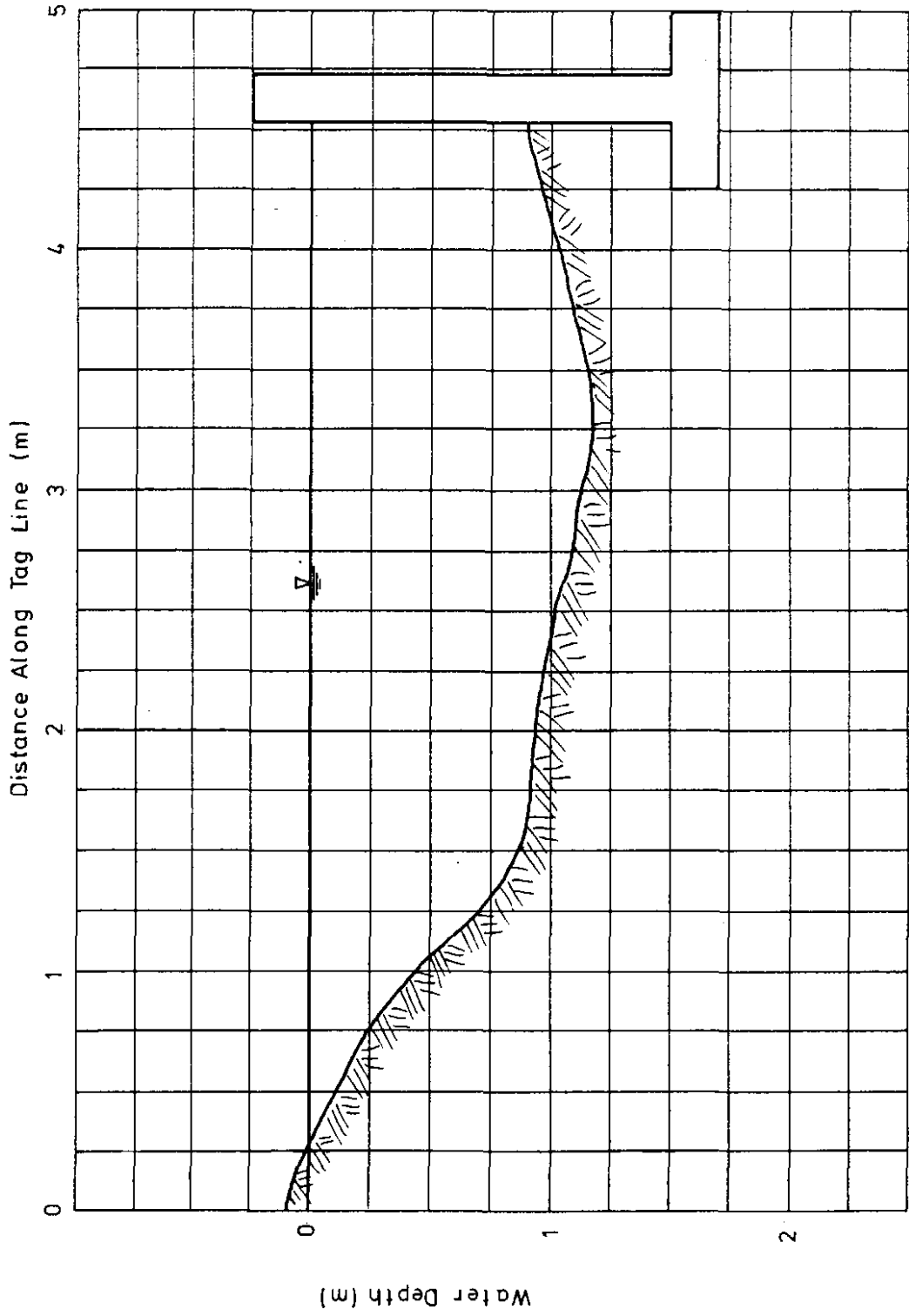
Fig. 4.4. Example Earthen Canal Cross-Section Showing Location of Verticals
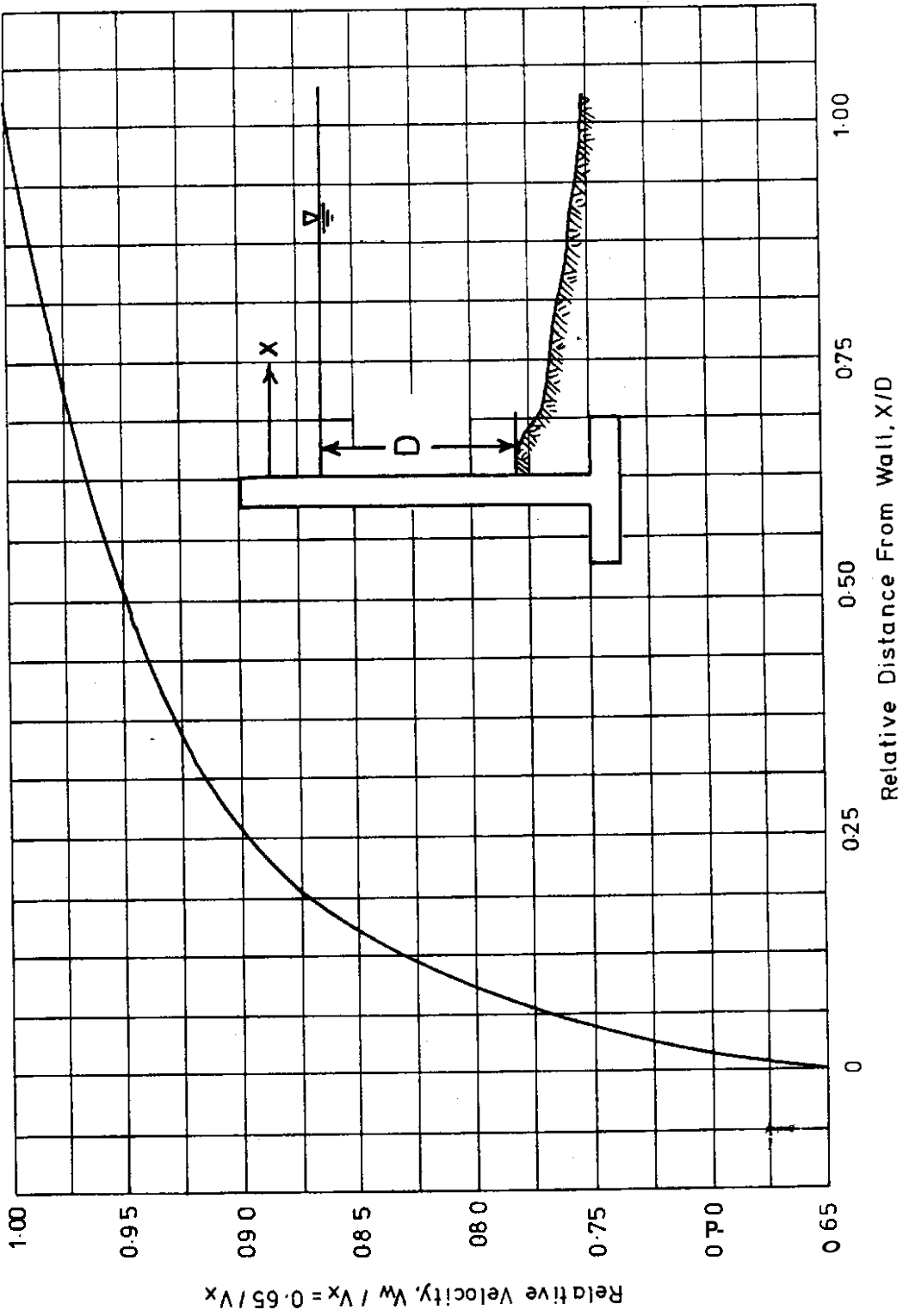for Defining the Flow Area.

Fig. 4.5.   Relative Mean Velocity Near a Vertical Wall.

54

4.7   PROCEDURE FOR CURRENT METER DISCHARGE MEASUREMENT

The discharge measurement with a current meter is obtained using the steps described below.

- Selection of location should be away from the head structures and culverts. The section should be straight.
- Divide the section into sub-sections, preferably 20-30 sections, but the spacing does not have to be less then one feet (30 cm) (see Figure 4.6).
- Measure the total depth of flowing water in the channel by using a wading rod.
- Calculate the *2/70, 6/10* and *8/10* of total water depth in the channel. These depths are termed as observation depths.
- Assemble the equipment properly after recording the information related to sectional width and depth.
- Fix the Current meter on the wading rod at the required depth. The observation depth in a vertical should be measured from the water surface, not the channel bed.
- Hold the wading rod at the first vertical.
- Keep the head phone and stop watch under ready condition.
- Before starting, the observer should ensure that the whole system is working properly.
- Keeping the wading rod at the first section, push the button of the stop watch and start counting the revolutions of the current meter through the sound of clicks with the help of a head phone. The standard range of revolutions and time periods is 5 to 100 revolutions and 40 to 70 seconds, respectively. The number of revolutions and time period taken should preferably be within these ranges.
- Using a constant number of revolutions with the same observation depth, note the variation in the time period (seconds) at each sub-section of the channel.
- Calculate the velocity of water at wall using graph paper containing the relative distance from the wall (X/D) on the horizontal axis and the relative velocity (V_w/V_x = 0.65/ V_x) on the vertical axis (Figure **4.5).**
- Calculate the total area of the section by subtracting the two tape measurements to obtain the width and averaging the two depths to obtain the mean depth.
- Calculate the discharge of each sub-section by multiplying the values of area and mean velocity of that section.
- Finally, compute the total discharge by adding the discharges for all sub- sections.

Example of Discharge Measurement Procedure:

Following the above procedure the discharge measurement was made using the current meter. The sample example for the computation of discharge for the Heran Distributary of Sanghar is the following Table **4.1.**

_____Table 4.1   (Continued)

Date: **25** June 1996   Channel: Heran Distributary   Station: Sanahar

Distance from Initial point	Depth	Depth of Observation	Revolutions	Time in seconds	Velocity			Area	Mean Depth	Width	Discharge
					At point	Mean in verical	Mean in section				
							1.715	3.54	3.54	1.00	6.07
8.00	3.55	0.2	80	40.0	1.98	1.755					
		0.8	80	52.0	1.53						
							1.773	3.54	3.54	1.00	6.27
9.00	3.54	0.2	90	44.0	2.02	1.791					
		0.8	80	51.0	1.56						
							1.795	3.54	3.54	1.00	6.35
10.00	3.53	0.2	90	44.0	2.023	1.799					
		0.8	80	50.4	1.575						
							1.815	3.5	3.5	1.00	6.35
11.00	3.52	0.2	90	43.0	2.073	1.831					
		0.8	80	49.8	1.59						
							1.821	6.14	3.51	1.75	11.18
12.75	3.50	0.2	80	40.4	1.96	1.815					
		0.8	80	47.5	1.67						
							1.77	6.03	3.45	1.75	10.67
14.50	3.40	0.2	80	42.0	1.89	1.725					
		0.8	80	51.0	1.56						
							1.685	3.36	3.36	1.00	5.66
15.50	3.32	0.2	80	44.0	1.80	1.645					
		0.8	60	40.0	1.49						

Observers: _____

No.____of_____pages   Computations _____   Checked _____

Table 4.1   (Continued)

Date: <u>25 June 1996</u>   Channel: <u>Heran Distributary</u>   Station: <u>Sanghar</u>

Distance from Initial point	Depth	Depth of Observation	Revolutions	Time in seconds	Velocity At point	Velocity Mean in verical	Velocity Mean in section	Area	Mean Depth	Width	Discharge
							1.64	3.3	3.3	1.00	5.412
16.50	3.28	0.2	80	45.0	1.76	1.64					
		0.8	80	52.4	1.52						
							1.555	3.24	3.24	1.00	5.038
17.50	3.20	0.2	80	50.0	1.59	1.475					
		0.8	60	44.0	1.36						
							1.46	3.1	3.1	1.00	4.526
18.50	3.00	0.2	80	49.4	1.61	1.45					
		0.8	60	46.6	1.29						
							1.422	2.9	2.9	1.00	4.123
19.50	2.80	0.2	60	41.6	1.44						
		0.8	60	44.0	1.36						
							1.323	2.775	2.775	1.00	3.671
20.50	2.75	0.2	60	42.4	1.41	1.25					
		0.8	60	55.0	1.09						
							1.213	2.62	2.62	1.00	3.184
21.50	2.50	0.2	60	50.0	1.20	1.18					
		0.8	60	52.0	1.16						
22.50	2.2	0.2	60	57.0	1.06	1.03	1.105	2.35	2.35	1.00	2.596
		0.8	40	40.0	1.00						

Observers: _____

No.____of_____pages   Computations _____   Checked _____

59

Table 4.1   (Complete)

Date: <u>25 June 1996</u>   Channel: <u>Heran Distributary</u>   Station: <u>Sanghar</u>

Distance from Initial point	Depth	Depth of Observation	Revolutions	Time in seconds	Velocity			Area	Mean Depth	Width	Discharge
					At point	Mean in verical	Mean in section				
							0.93	1.975	1.975	1.00	1.836
23.50	1.75	0.2	40.0	50.4	0.803	0.831					
		0.8	40.0	47.0	0.859						
				.			0.648	1.375	1.375	1.00	0.891
24.50	1.00	0.2	20	40.0	0.516	0.467					
		0.8	20	5.0	0.419						
							0.35	0.5	0.5	1.00	0.175
25.50				50%		0.233					
										EQ=	114.00

Observers: _____

No ____ of ____ **pages**   Computations _____   Checked _____

CHAPTER 5. DISCHARGE RATING OF FLOW CONTROL STRUCTURES[2]

5.1   INTRODUCTION

Most common open channel flow control structures can also serve as flow measurement devices when properly calibrated

The improvement of water management practices invariably hinges upon on the ability to measure flow rates and volumes at key locations in an irrigation system. Water measurement is a fundamental basis for evaluating the performance of water management practices, and for quantifying the effects of improvements in those practices. Flow rate information can be used to calculate various performance indices, such as efficiency terms, from which comparative evaluations can be made for different years and among other irrigation systems.

In the majority of irrigation projects, discharge measurements are only made at the canal headworks, which may be an outlet structure from a dam or a structure that diverts water from a river. However, there are also irrigation projects in which the water delivered to each user (farmer) is measured. The technology for measuring irrigation water is rather simple and has been available for many decades. Unfortunately, this technology has not been incorporated into the routine maintenance and operation practices of many irrigation projects.

In most irrigation systems, there are numerous structures that can be calibrated for the purpose of water measurement. Usually, the most common constriction in the irrigation delivery network is a gate structure, with some systems having hundreds of gate structures for flow control. Other common irrigation structures that can be calibrated are culverts, inverted syphons, drop structures, weirs and wasteways. In fact, any type of structure that constricts the flow (i.e. causing a backwater effect and subcritical flow upstream) can be field calibrated for discharge measurement.

The installation of standardized primary flow measuring devices, such as laboratory calibrated flumes and weirs, is advantageous in that field calibrations are not necessary unless: (1) the dimensions of the device are incorrect; or (2) the installation does not correspond with the conditions under which the laboratory rating was developed. Major disadvantages of using these devices are, first of all, expense, but often it is the added head loss in the channel which results in higher water levels upstream from the device

---

[2]   Material for this chapter is taken from the technical report of the training course on "Field Calibration of Irrigation Structures, Fordwah Canal, Fordwah Eastern Sadiqia Irrigation and Drainage Project", Report T-3, August 1995.