Bahawalnagar, 28 May to 6 June, 1995

Technical Report

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CHAPTER 1: Introduction

1.1. Background

IIMI was asked in 1989 by the Secretaries of Irrigation & Power and Agriculture of the Government of Punjab to commence work in the Fordwah/Eastern Sadiqia area, given the fact that a number of development projects would be initiated in the area. The objective of IIMI's research is to develop and pilot test in collaboration with national research and line agencies *alternative* irrigation management practices to optimize agricultural production and *mitigate* problems *of salinity/sodicity.* The research is carried out at various levels of the irrigation system, from main system operations to field level irrigation application.

The main system component of the research is carried out in collaboration with the Punjab Irrigation & Power Department and aims to develop tools to assist irrigation managers to take better founded decisions on operations and maintenance. Within the framework of this programme, a field calibration training course was organized in the Fordwah Canal Division on the request of the Secretary Irrigation & Power Department, Punjab.

The training course had four main components:

1. Classroom and field site lectures on hydraulic principles of rating of structures and the use of the current meter

2. Rating of distributary head regulators by current meter (wading method)

3. Inflow-outflow test to determine seepage losses

4. Rating of major structures and cross-regulators by current meter (suspension method, boat)

The participants of the training course calibrated all hydraulic structures of the Chishtian sub-division (Fordwah Branch RD 199-371). Upon completion of the rating of structures (component 2), an inflow-outflow test was conducted to estimate the seepage losses in Fordwah Branch (RD 199-371). Finally, component 4 was entrusted to the International Sedimentation Research Institute, Pakistan (ISRIP). The training course was organized from 28 May to 6 June in Bahawalnagar.

Limitations

To develop the rating of hydraulic structures, measurements have to be taken with a range of discharges (e.g. 20%, 40%, 60%, 80%, 100% and 120% of the full supply discharge). Thus, the training course was organized during a period in which supplies were not at their maximum. During this ten-day period, 1-2 measurements were taken for each structure. Although this will give a good idea of the rating of these structures, more measurements will be required to develop full rating curves.

1.2. Description of the System

Fordwah canal off-takes from the left abutment of Suleimanki Headworks on the Sutlej river and conveys water to MacLeod Ganj and Fordwah Branch canals at its tail at RD 44850 (see map). Fordwah Branch canal runs for about 75 miles (tail RD is 371650) and has a full supply discharge! of 2603 cusecs. The head of the system is non-perennial and receives supplies from 15 April to 15 October only. At RD 129 of Fordwah Branch, the Sadiq-Ford feeder supplies water to Fordwah Branch from 15 October *to* 15 April in order to feed the perennial canals of the Fordwah system. The Fordwah Division is divided into three sub-divisions, Minchinabad (from the head of Fordwah canal *to* RD 77 of Fordwah Branch), Bahawalnagar (from RD 77 to RD 199 of the Fordwah Branch) and Chishtian (from RD 199 to 371 of Fordwah Branch). See the index plan on the next page.

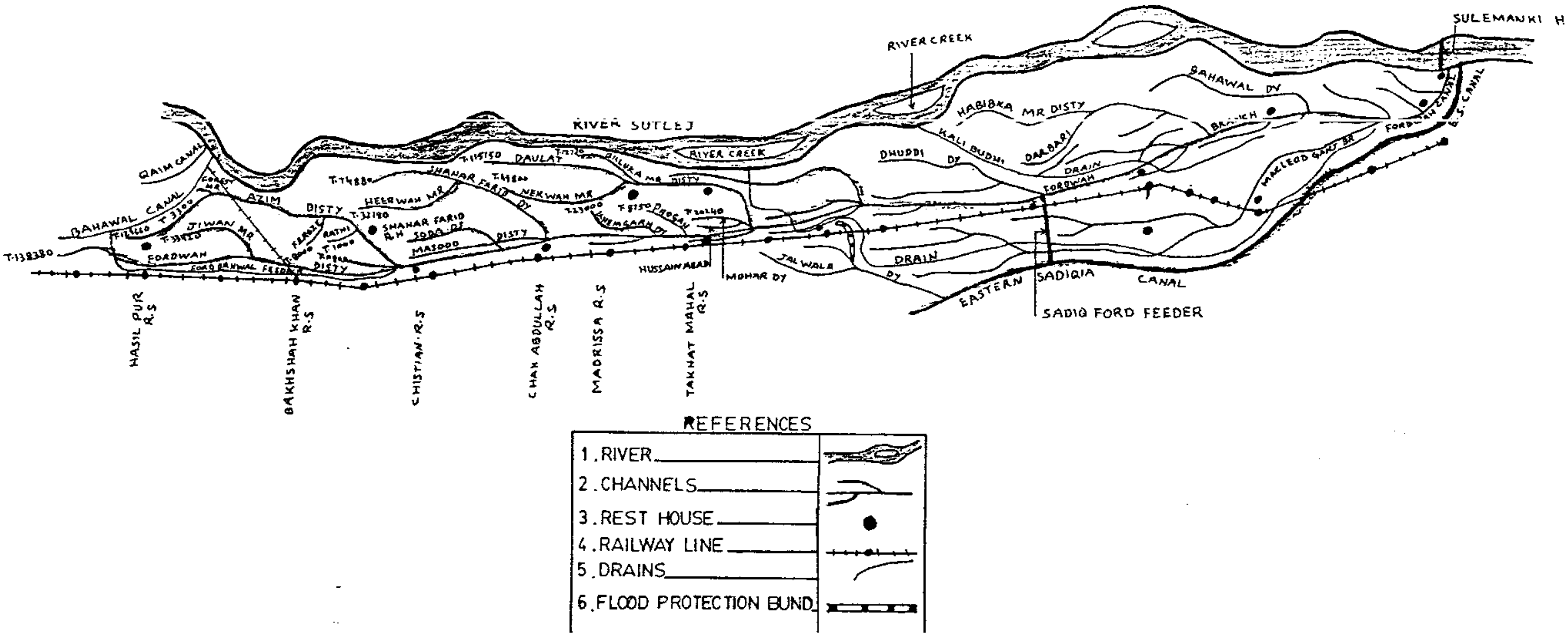
1.3. Units

During the training course the participants were trained *to* convert easily from the metric system to the imperial system and back. Although the metric system has been officially adopted in Pakistan, imperial units are very much in use in Pakistan's irrigation system. It is for this reason that while the units in the more generic cheapter 2 on methodology are kept in the metric system, units in the results chapter are presented in the imperial system. Conversions are given in the table below for an easy reference of the reader.

Conversion Factors

1 foot (= 12 inches)	0.3048 m = 30.48 cm
1 foot/second	0.3048 m/s
1 (cubic) foot/second (cusec)	0.0283 m^3/s = 28.3 l/s
1 metre (=100 cm)	3.28 feet
1 m/s	3.28 feet/s
1 m^3/s (cumec) (=1000 l/s)	35.31 cusec

INDEX PLAN OF FORDWAH DIVISION



2.1. *Backwater Effects*

A simple open-channel constriction is shown in Figure 1. The flow through such constrictions is most often in the tranquil range, and produces gradually varied flow far upstream and a 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, an "M1" or "M2" backwater profile occurs. The maximum backwater effect, denoted by y^* in Figure 1, occurs a relatively short distance upstream. 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, the flow expansion process begins and continues until the normal regime of flow has been re-established in the channel.

2.2 Free-Flow and Submerged-Flow

The two most significant flow regimes under which any open channel constriction may operate are free-flow and submerged-flow. Other terms for free-flow are critical-depth flow and modular flow, while other terms for submerged-flow are drowned flow and non-modular flow. The distinguishing difference between the two flow conditions is the occurrence of critical velocity in the vicinity of the constriction (usually a very short distance upstream of the narrowest portion of the constriction). When this critical flow control occurs, the discharge is uniquely related to the depth of "head" upstream of the critical section. Thus, measurement of a flow depth at some specified location upstream, h_u, from the point of the critical conditions is all that is necessary to obtain the free-flow discharge, Q_f. Consequently, Q_f can be expressed as a function of h_u:

$$Q_f = f(h_u) \tag{1}$$

When the flow conditions are such that the downstream flow depth is raised to the extent that the flow velocity at every point through the constriction becomes less than the critical value, then the constriction is operating under submerged-flow conditions. With this flow regime, an increase in tailwater flow depth, Δh_d, will increase the head upstream of the constriction by Δh_u (Δh_u will be less than Δh_d). Both the upstream depth, h_u, and the downstream depth, h_d, must be measured to determine the discharge through a calibrated constriction operating under submerged-flow conditions.

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[1] The material of this Chapter was taken from a manual by the same title published by the International Irrigation Center, Utah State University dated January 1992 and authored by Gaylord V. Skogerboe. Gary P. Merkley, M.S. Shafique and Carlos A. Gandarillas.

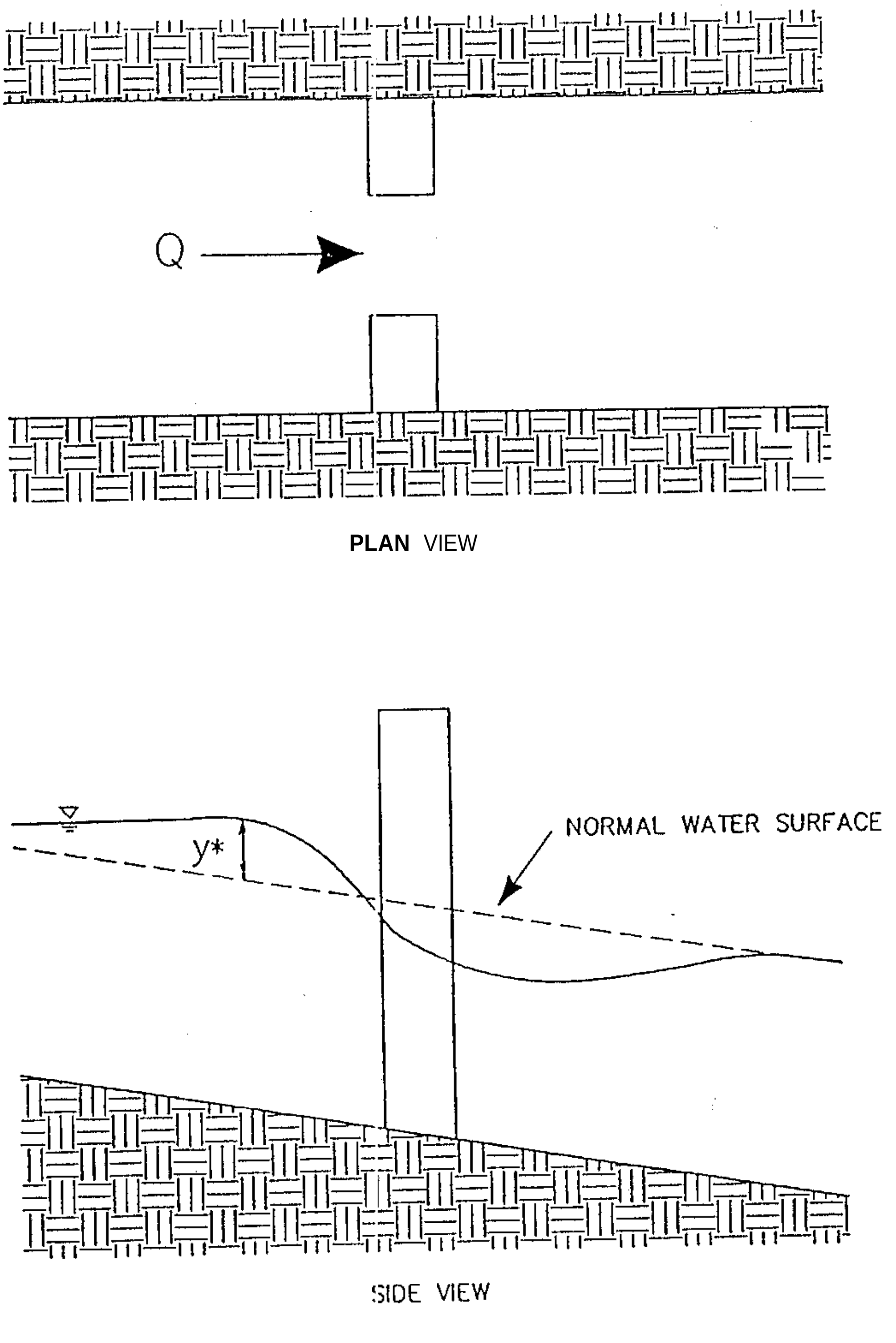


Figure 1. Definition sketch for backwater effects from an open-channel constriction

The definition given to submergence, S, is:

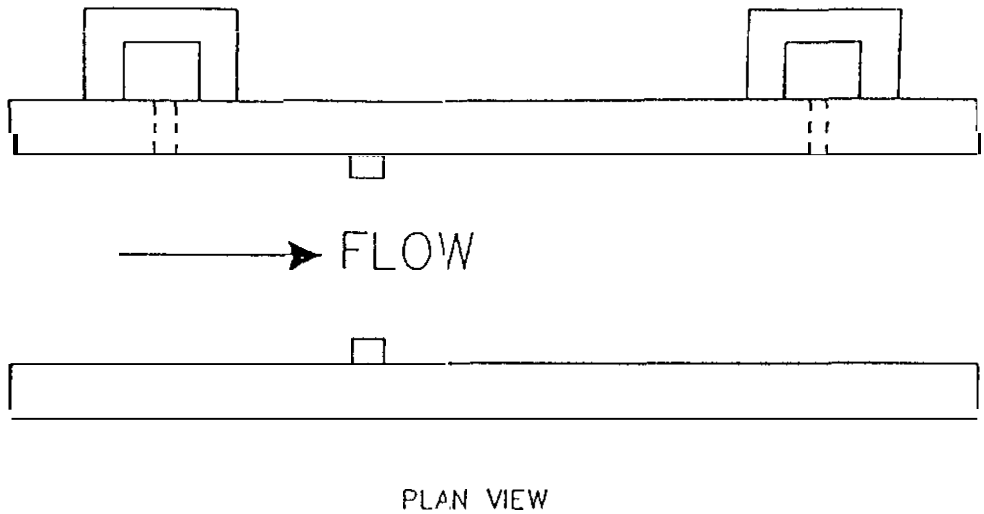
$$S = \frac{h_d}{h_u} \tag{2}$$

The submergence may also be represented in percent. The submerged-flow discharge, Q_s, is a function of h_u and h_d and the governing relationship is generally written in terms of discharge, head loss ($h_u - h_d$), and submergence:

$$Q_s = f(h_u, h_d) = f(h_u - h_d, S) \tag{3}$$

Ofttimes, constrictions designed initially to operate under free-flow conditions become submerged as a result of unusual operating conditions, or the accumulation of moss and vegetation in the open channel. Care should always be taken to note the operating condition of the constriction in order to determine which rating should be used. The value of submergence marking the change from free-flow to submerged-flow, or vice versa, is referred to as the transition submergence, S_t. At this condition, the discharge given by the free-flow equation is exactly the same as that given by the submerged-flow equation. Hence, if discharge equations are known for both free-flow and submerged-flow conditions, a definite value of the transition submergence can be obtained by setting the equations equal to one another and solving for S_t. It should be noted that this derived value of S_t is highly sensitive to slight errors in the coefficients or exponents of either equation (Skogerboe, Hyatt and Eggleston 1967).

The difference between free-flow, the transition state, and submerged-flow water surface profiles is illustrated for a simple channel constriction in Figure 2. Water surface profile (a) illustrates free-flow, and (b) indicates the transition submergence condition. Both profiles (a) and (b) have the same upstream depth, with profile (b) having the maximum Submergence value for which the free-flow condition can exist. The submerged-flow condition is illustrated by profile (c), where an increase in the tailwater depth has also increased the depth of flow at the upstream station.



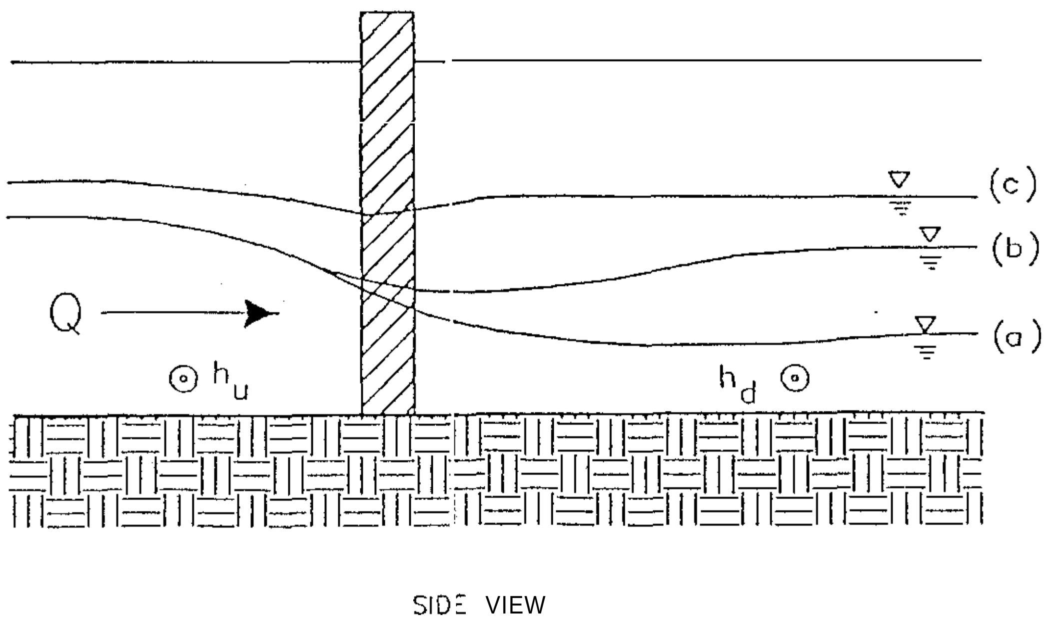


Figure 2. Illustration of flow conditions in an open-channel constriction.

2.3 Rating **Open** Channel Constrictions

2.3.1 <u>Free Flow</u>

The general form of the free-flow equation is:

$$Q_f = C_f h_u^{n_f} \tag{4}$$

where the subscript f denotes free flow, so that **Q_f** is the free-flow discharge, C_f is the free-flow coefficient, and n_f is the free-flow exponent. The value of C_f increases as the size of the constriction increases, but the relationship is usually not linear. The value of n_f is primarily dependent upon the geometry of the constriction with the theoretical values being ***3/2*** for a rectangular constriction and ***5/2*** for a triangular constriction. **A** trapezoidal constriction would have a free-flow exponent of 3/2 at extremely shallow flow depths and 5/2 for extremely deep flow depths; thus, n_f increases with depth at a trapezoidal constriction. The theoretical values of n_f are modified by the approach velocity, so that n_f increases as the approach velocity increases. However, the measured values correspond very well with the theoretical values for very low approach velocities

A hypothetical example of developing the field discharge rating for a rectangular open-channel constriction is illustrated in Figure 3, and the field data is listed in Table 1. The discharge rate in the constriction was determined by taking current meter readings at a location upstream, and again at another location downstream. This is a good practice because the flow depths upstream and downstream are often significantly different, *so* that the variation in the measured discharge between the two locations is indicative of the accuracy of the current meter equipment and the methodology used by the field staff.

A logarithmic plot of the free-flow data (see Table 2) is shown in Figure **4** for the stilling well flow depths, $(h_u)_{sw}$. Note that n_f is the slope of the straight line and C_f is the value of **Q_f** for $(h_u)_{sw}$ = 1.0, since

$$Q_f = C_f(1.0)^{n_f} = C_f \tag{5}$$

The slope, n_f, must be determined using a scale as illustrated. The resulting free-flow equation is:

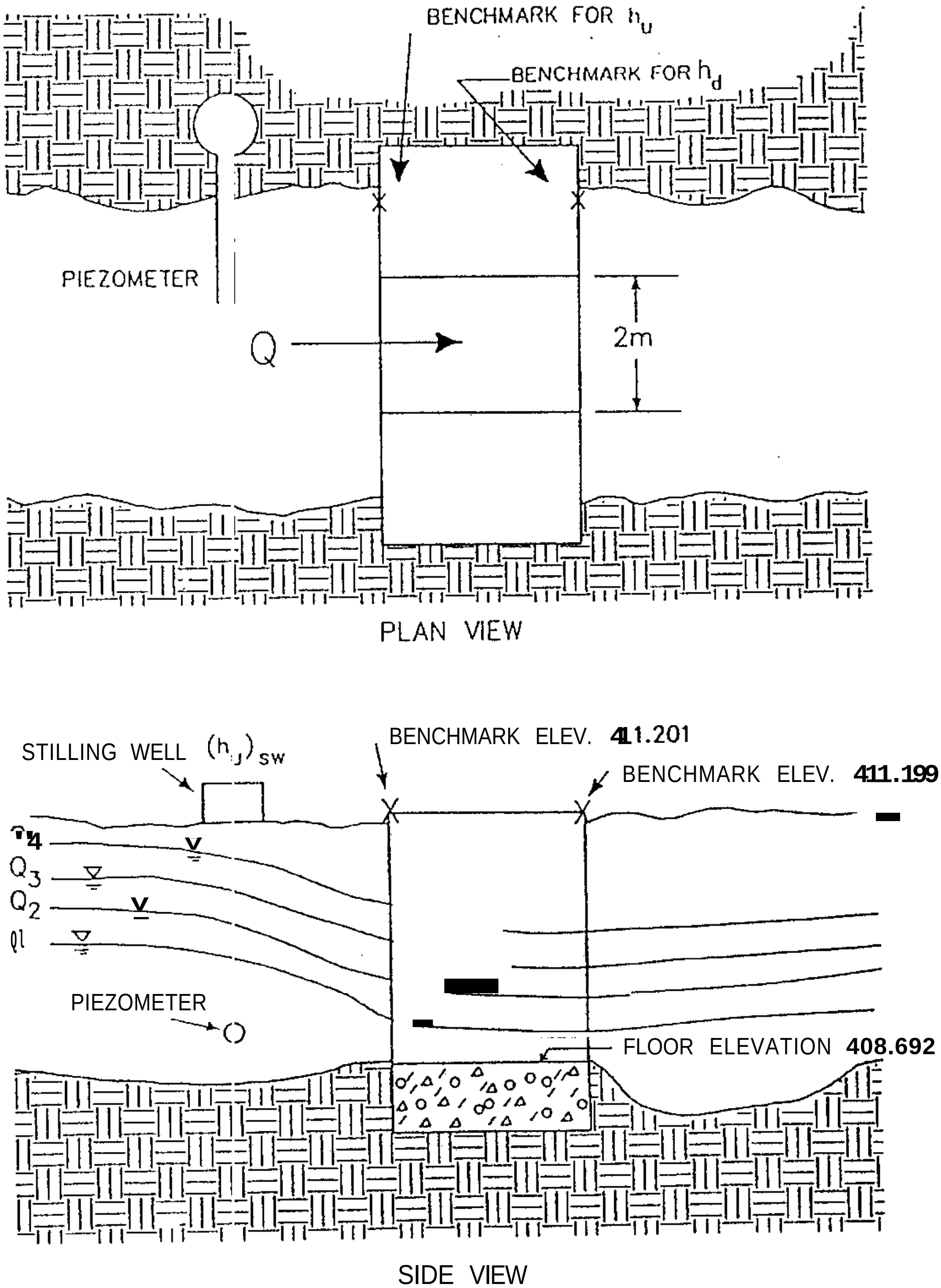


Figure 3 Example of free flow water surface profiles for an open-channel constriction.

TABLE 1. Free-flow field data for example open-channel constriction

Date	Discharge m^3/s	Water Surface Elevation in Stilling Well, m	Tape Measurement from Benchmark, m
21 Jun 86	0.628	409.610	1.604
21 Jun 86	1.012	409.935	1.294
21 Jun 86	1.798	410.508	0.734
21 Jun 86	2.409	410.899	0.358

Note: The listed discharge is the average discharge measured with a current meter at a location 23 **m** upstream of the constriction, and at another location 108 **m** downstream.

TABLE 2. Free-flow data reduction for example open-channel constriction.

Discharge m^3/s	Water Surface Elevation, m	$(h_u)_{sw}$ m	Tape Measurement m	$(h_u)_x$ m
0.628	409.610	0.918	1.604	0.905
1.009	409.935	1.243	1.294	1.215
1.797	410.508	1.816	0.734	1.775
2.412	410.899	2.207	0.358	2.151

Note: The third column values equal the values in the second column minus the floor elevation of 408.692 m. The values in the last column equal the benchmark elevation of 411.201 m minus the floor elevation of 408.692 m minus the values in column four.

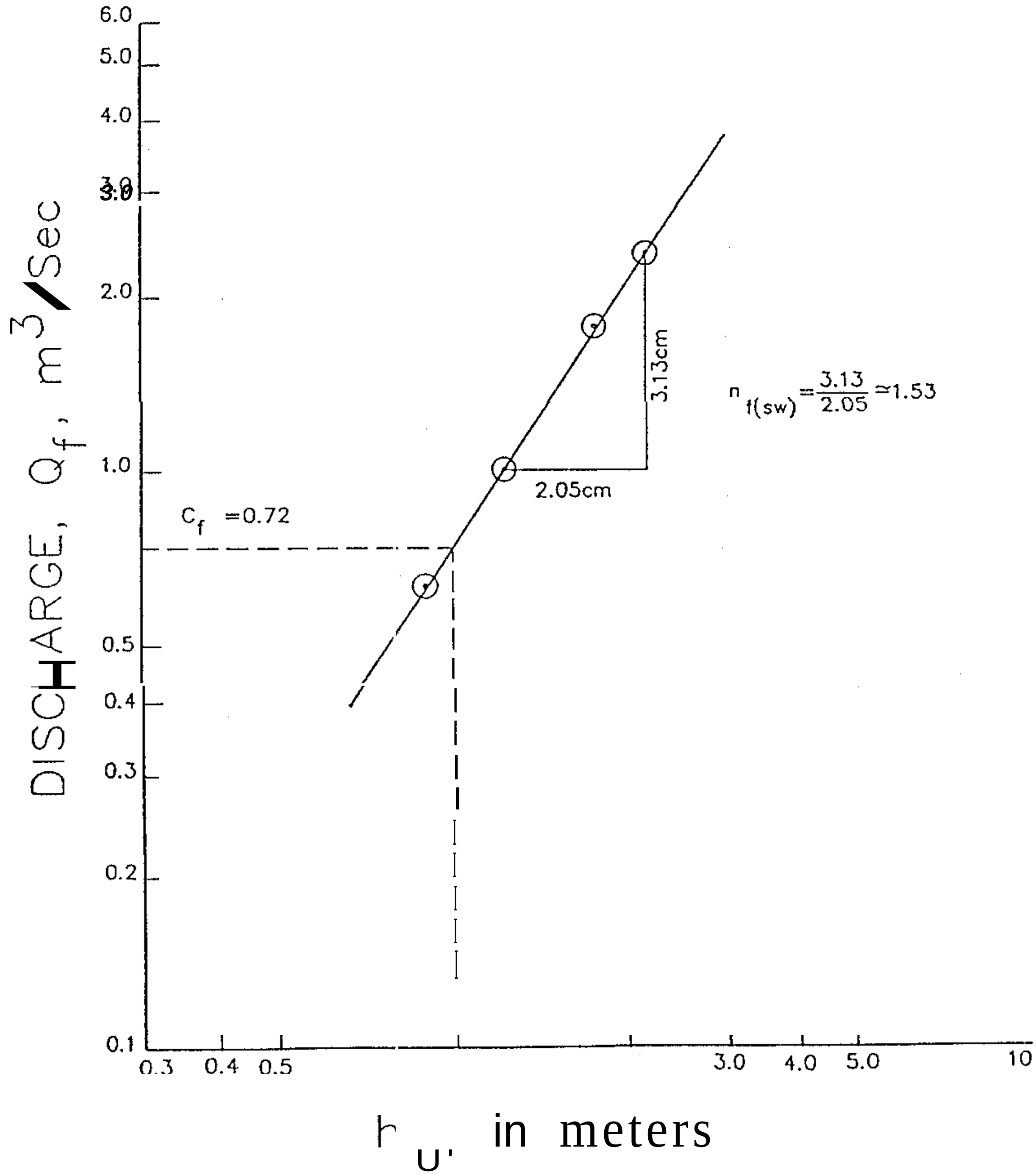


Figure **4.** Free-flow discharge rating using the example data.

$$Q_f = 0.72\,(h_u)_{sw}^{1.53} \tag{6}$$

A comparison of the free-flaw ratings for the stilling well flow depths (Equation 6) and the flow depths along the headwall measured from the benchmark are shown in Figure 5. The free-flow equation for the flow depths measured below the benchmark is:

$$Q_f = 0.74\,(h_u)_{sw}^{1.55} \tag{7}$$

If a regression analysis is done with the free-flow data using the theoretical value on n_f = 3/2,

$$Q_f = 0.73\,(h_u)_{sw}^{1.5} \tag{8}$$

$$Q_f = 0.75\,(h_u)_x^{1.5} \tag{9}$$

A comparison of Equations 6 and 8 and Equations 7 and 9 are shown in Table 3. The discharge error resulting from using n_f = 3/2 varies from -1.91 percent to +2.87 percent.

2.3.2 Submerued Flow

The general form of the submerged-flow equation is:

$$Q_s = \frac{C_s\,(h_u - h_d)^{n_f}}{(-\log S)^{n_s}} \tag{10}$$

Where the subscript *s* denotes submerged flow, so that Q_s is the submerged-flow discharge, C_s is the submerged-flow coefficient, and n_s is the submerged-flow exponent. Note that the free-flow exponent, n_f, is used with the term h_u - h_d. Consequently, n_f is determined from the free-flow rating, while *C*, and n_s is between 1.0 and 1.5 (Skogerboe and Hyatt 1967).

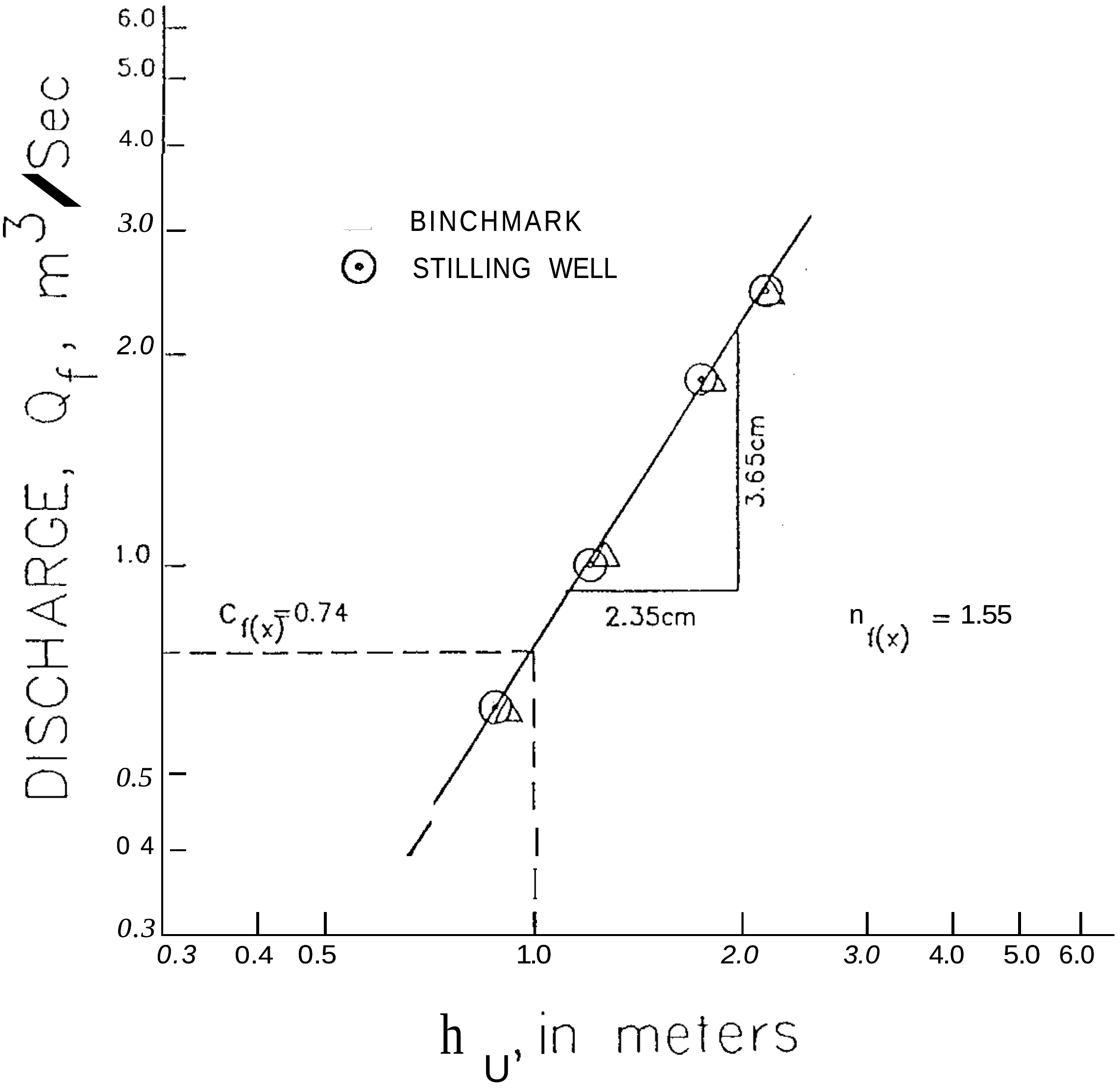


Figure 5. Comparison of free-flow discharge ratings for stilling well and benchmark flow depths.

Measured Discharge m^3/s	Measured Depth, $(h_u)_{sw}$ m	$Q_f=0.72(h_u)_{sw}^{1.53}$ m^3/s	Percent Error	$Q_f=0.73(h_u)_{sw}^{1.50}$ m^3/s	Percent Error
0.628	0.918	0.632	0.64	0.642	2.23
1.009	1.243	1.004	-0.50	1.012	0.30
1.797	1.816	1.794	-0.17	1.786	-0.61
2.412	2.207	2.417	0.21	2.393	-0.80

Measured Discharge m^3/s	Measured Depth, $(h_u)_x$ m	$Q_f=0.74(h_u)_x^{1.55}$ m^3/s	Percent Error	$Q_f=0.75(h_u)_x^{1.50}$ m^3/s	Percent Error
0.628	0.905	0.634	0.96	0.646	2.87
1.009	1.215	1.001	-0.79	1.004	-0.50
1.797	1.775	1.801	-0.22	1.774	-1.28
2.412	2.151	2.426	0.58	2.366	-1.91

TABLE 4. Submerged-flow field data for example open-channel constriction.

Date	Discharge m^3/s	Tape Measurement from U/S Benchmark	Tape Measurement from D/S Benchmark
22 Jun 86	0.813	1.448	1.675
22 Jun 86	0.823	1.434	1.605
22 Jun 86	0.825	1.418	1.548
23 Jun 86	0.793	1.335	1.376
23 Jun 86	1.427	0.983	1.302
23 Jun 86	1.436	0.966	1.197
23 Jun 86	1.418	0.945	1.100
23 Jun 86	1.377	0.914	1.009
23 Jun 86	1.241	0.871	0.910

Q_s m^3/s	$(h_u)_x$ m	$(h_d)_x$ m	S	-log S	$Q\Delta_h$=1
0.823	1.075	0.902	0.839	0.0762	12.486
0.825	1.091	0.959	0.879	0.0560	19.036
0.824	1.119	1.028	0.919	0.0367	33.839
0.793	1.174	1.131	0.963	0.0164	104.087
1.427	1.526	1.205	0.790	0.1024	8.305
1.436	1.543	1.310	0.849	0.0711	13.733
1.418	1.564	1.407	0.900	0.0458	25.005
1.377	1.595	1.498	0.939	0.0273	51.220
1.241	1.638	1.597	0.975	0.0110	175.371

A logarithmic plot of the submerged-flow data is shown in Figure *6.* Each data point in Figure 12 can have a line drawn at a slope on **n,** = 1.55, which can be extended to where it intercepts the abscissa at h_u **-** h_d = **1.0;** then, the corresponding value of discharge can be read on the ordinate, which is listed as $Q_{\Delta h = 1.0}$ in Table 5. The value of $Q_{\Delta h = 1.0}$ can also be solved analytically because a straight line on logarithmic paper is a power function having the simple form:

$$Q_s = Q_{\Delta h = 1.0} \, (h_u - h_d)^{n_f} \tag{11}$$

or

$$Q_{\Delta h=1.0} = \frac{Q_s}{(h_u - h_d)^{n_f}} \tag{12}$$

where $Q_{\Delta h = 1.0}$ **has** a different value for each value of the submergence, S.

Using the term $Q_{\Delta h = 1.0}$, implies that h_u - h_d = 1.0 by definition, **so** that Equation 10 reduces to:

$$Q_{\Delta h=1.0} = \frac{C_s(1.0)^{n_f}}{(-\log S)^{n_s}} = C_s(-\log S)^{-n_s} \tag{13}$$

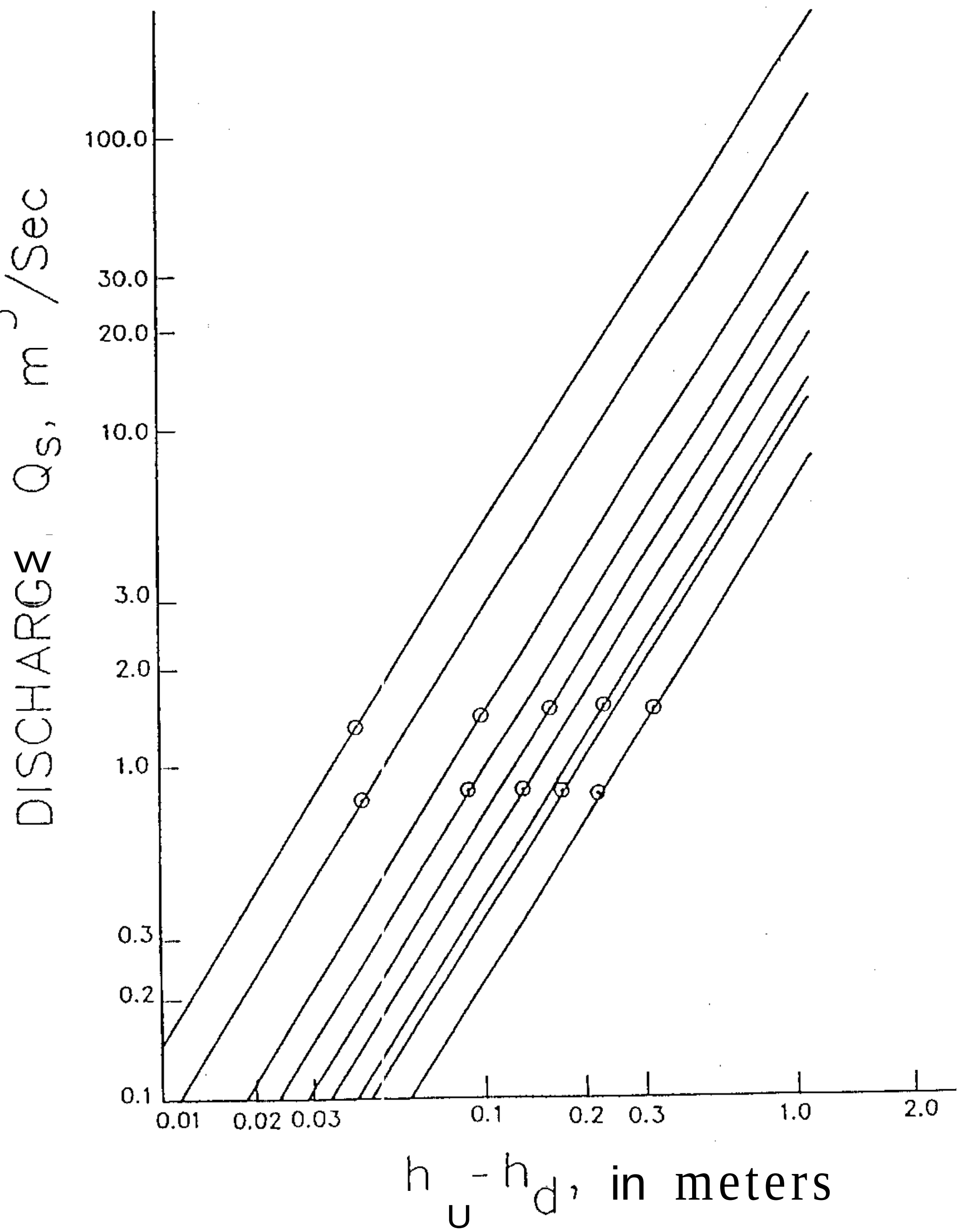


Figure 6. Logarithmic plot of submerged-flow data for the example open-channel constriction.

Again, this is a power function where $Q_{\Delta h = 1.0}$ can be plotted against (-logS) on logarithmic paper to yield a straight line relationship. The submerged-flow data in Table **5** is plotted in Figure 7. Note that the straight line has a negative slope ($-n_s$) and that C_s is the value of $Q_{\Delta h = 1.0}$ when (-logS) is equal to 1.0. For this particular data, the submerged-flow equation is:

$$Q_s = \frac{0.367(h_u - h_d)^{1.55}}{(-logS)^{1.37}} \tag{14}$$

By setting the free-flow discharge equation equal to the submerged-flow discharge equation (Equations 7 and 14), the transition submergence, S_t, can be determined:

$$0.74h_u^{1.55} = \frac{0.367(h_u - h_d)^{1.55}}{(-logS)^{1.37}} \tag{15}$$

and,

$$0.74(-logS)^{1.37} = 0.367(1-S)^{1.55} \tag{16}$$

The value of *S* in this relationship is S_t provided the coefficients and exponents have been accurately determined. Again, small errors will dramatically affect the determination of S_t.

$$0.74(-logS_t)^{1.37} = 0.367(1-S_t)^{1.55} \tag{17}$$

Equation 17 is solved by trial-and-error to determine S_t, which in this case is 0.82. Thus, free flow exists when S < 0.82 and submerged flow exists when the submergence is greater than 82 percent.

A final free-flow and submerged-flow discharge rating is plotted on logarithmic paper in Figure 8. Also, Table 6 is a typical free-flow rating based on Equation 7 for the example open-channel constriction. The submerged-flow discharge can be calculated using the reduction factors in Table 7 to multiply by the free-flow rating corresponding to the measured value of h_u. This reduction factor is obtained by calculating the ration Q_s/Q_f:

$$\frac{Q_s}{Q_f} = \frac{0.367(h_u - h_d)^{1.55}}{(-logS)^{1.37}} \frac{1}{0.74h_u^{1.55}} \tag{18}$$

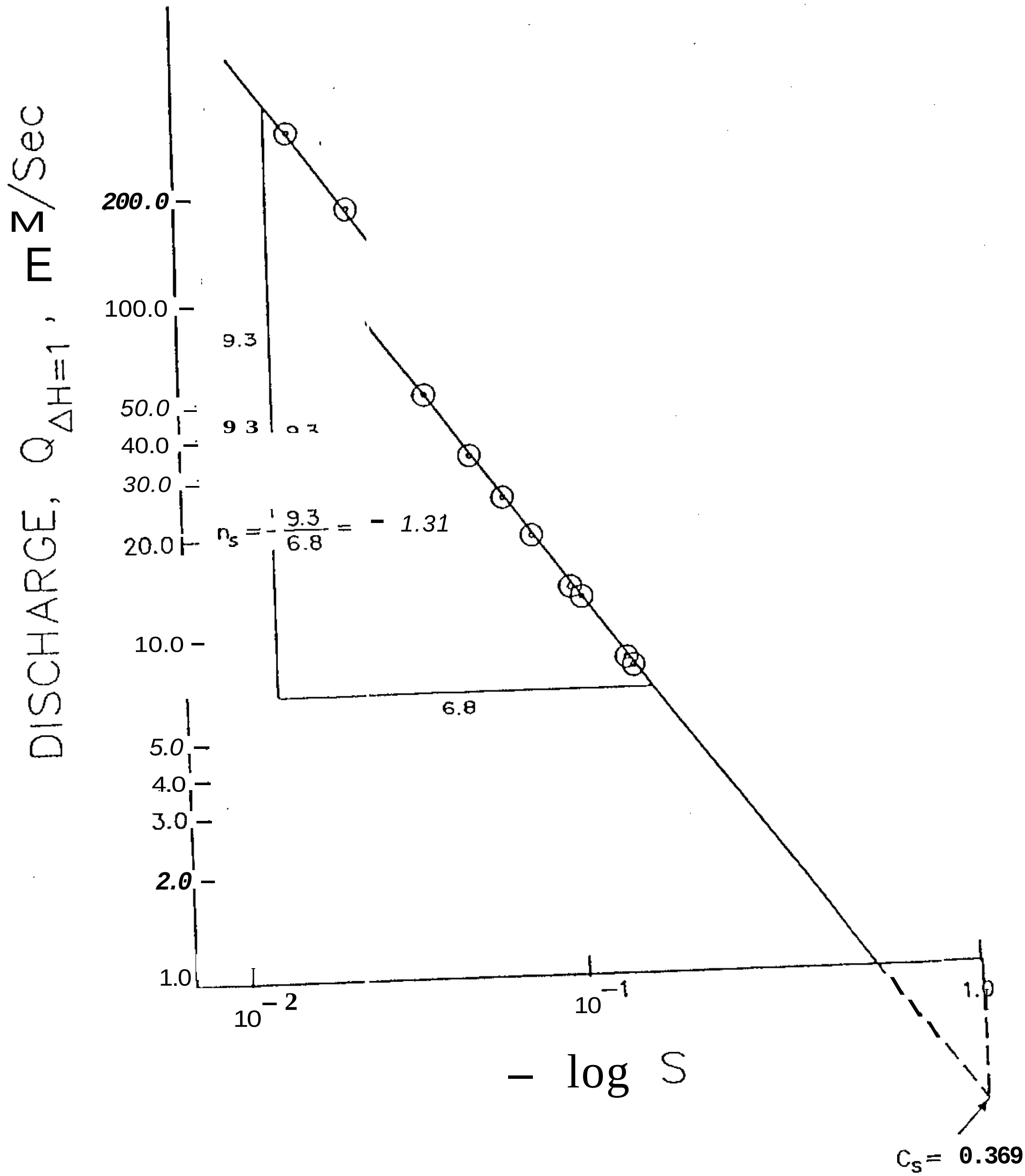


Figure 7. Logarithmic plot for determining the submerged-flow coefficient, C_s, and the submerged-flow exponent, n_s, for the example open-channel constriction.

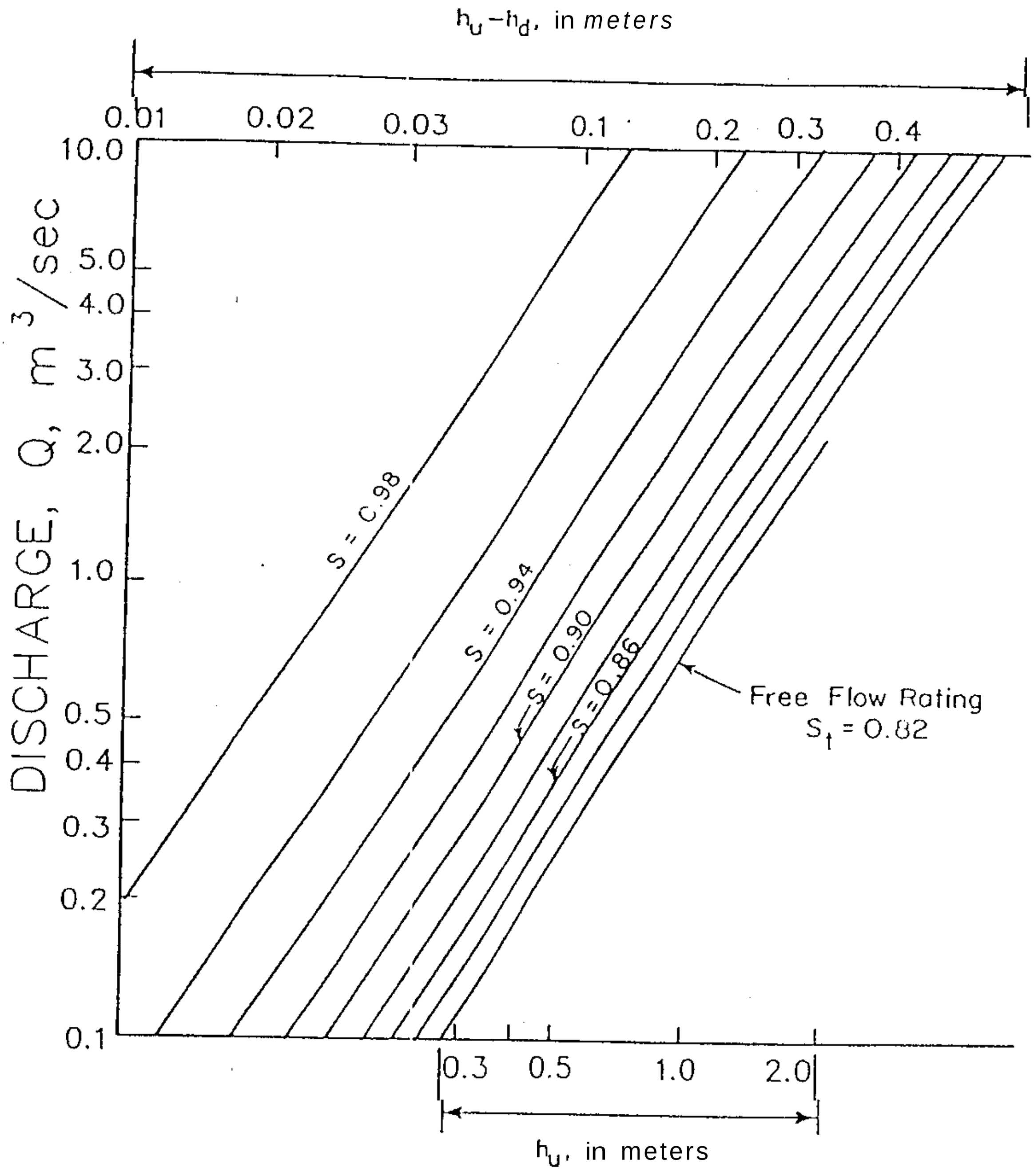


Figure 8. Free-flow and submerged-flow rating for the example open-channel constriction.

TABLE 6. Free-flow discharge rating for **example** open-channel constriction.

h_u m	Q_f m³/s	h_u m	Q_f m³/s	h_u m	Q_f m³/s	h_u m	Q_f m³/s
0.810	0.534	1.110	0.870	1.410	1.260	1.710	1.700
0.820	0.544	1.120	0.882	1.420	1.274	1.720	1.715
0.830	0.554	1.130	0.894	1.430	1.288	1.730	1.731
0.840	0.565	1.140	0.907	1.440	1.302	1.740	1.746
0.850	0.575	1.150	0.919	1.450	1.316	1.750	1.762
0.860	0.586	1.160	0.931	1.460	1.330	1.760	1.777
0.870	0.596	1.170	0.944	1.470	1.345	1.770	1.793
0.880	0.607	1.180	0.956	1.480	1.359	1.780	1.809
0.890	0.618	1.190	0.969	1.490	1.373	1.790	1.825
0.900	0.629	1.200	0.982	1.500	1.387	1.800	1.840
0.910	0.639	1.210	0.994	1.510	1.402	1.810	1.856
0.920	0.650	1.220	1.007	1.520	1.416	1.820	1.872
0.930	0.661	1.230	1.020	1.530	1.431	1.830	1.888
0.940	0.672	1.240	1.033	1.540	1.445	1.840	1.904
0.950	0.683	1.250	1.046	1.550	1.460	1.850	1.920
0.960	0.695	1.260	1.059	1.560	1.474	1.860	1.936
0.970	0.706	1.270	1.072	1.570	1.489	1.870	1.952
0.980	0.717	1.280	1.085	1.580	1.504	1.880	1.969
0.990	0.729	1.290	1.098	1.590	1.518	1.890	1.985
1.000	0.740	1.300	1.111	1.600	1.533	1.900	2.001
1.010	0.752	1.310	1.125	1.610	1.548	1.910	2.018
1.020	0.763	1.320	1.138	1.620	1.563	1.920	2.034
1.030	0.775	1.330	1.151	1.630	1.578	1.930	2.050
1.040	0.786	1.340	1.165	1.640	1.593	1.940	2.067
1.050	0.798	1.350	1.178	1.650	1.608	1.950	2.083
1.060	0.810	1.360	1.192	1.660	1.623	1.960	2.100
1.070	0.822	1.370	1.205	1.670	1.638	1.970	2.117
1.080	0.834	1.380	1.219	1.680	1.654	1.980	2.133
1.090	0.846	1.390	1.233	1.690	1.669	1.990	2.150
1.100	0.858	1.400	1.247	1.700	1.684	2.000	2.167

TABLE 7. Submerged-flow reduction factors for the example open-channel constriction.

S	Q_s/Q_f	S	Q_s/Q_f
0.82	1.000	0.91	0.9455
0.83	0.9968	0.92	0.9325
0.84	0.9939	0.93	0.9170
0.85	0.9902	0.94	0.8984
0.86	0.9856	0.95	0.8757
0.87	0.9801	0.96	0.8472
0.88	0.9735	0.97	0.8101
0.89	0.9657	0.98	0.7584
0.90	0.9564	-	-

Which is also equal to

$$\frac{Q_s}{Q_f} = \frac{0.496(1-S)^{1.55}}{(-\log S)^{1.37}} \tag{19}$$

For example, if h_u and h_d are measured and found to be 1.430 and 1.337, the first step would be to compute the submergence, *S*,

$$S = \frac{1.337}{1.430} = 0.935 \tag{20}$$

Thus, the submerged-flow condition exists in the example open-channel constriction. From Table 6, the value of **Q, is** 1.288 m^3/s for h_u = 1.430 m. Then, entering Table 7, the value **of** Q_s/Q_f can be found by interpolating halfway between *S* = 0.93 and *S* = 0.94. Thus, Q_s/Q_f = 0.9077. Consequently, Q_s can be determined from

$$Q_s = Q_f\left(\frac{Q_s}{Q_f}\right) = 1.288(0.9077) = 1.169 m^3/s \tag{21}$$

Finally, it should be noted that all of the preceding graphical solutions to the calibration of open-channel constrictions can also be obtained numerically through logarithmic transformations and linear regression. The graphical solutions have the advantage of being more didactic; however, for experienced persons, the numerical solution is usually more convenient. It is always useful to plot the results, either by hand or using computer software, to reduce the possibility of errors or inclusion of erroneous data values. Obvious errors tend to be more apparent in plots than in numerical calibration results.

2.4. Rating Orifices

Any type of opening in which the upstream water level is higher than the top of the opening is referred to as an orifice. In this case, if the jet of water emanating from the orifice discharges freely into the air or downstream channel without backwater or tailwater effect, then the orifice is operating under free-flow conditions. If the upstream water level is below the top of the opening, then the opening is hydraulically performing as a weir structure. For free-flow conditions through an orifice, the discharge equation is:

$$Q_f = C_d \, C_v \, A \sqrt{2gh_u} \tag{22}$$

where C_d is the dimensionless discharge coefficient, C_v is the dimensionless velocity head coefficient, A is the cross-sectional area of the orifice opening, g is the acceleration due **to** gravity, and h_u is measured from the centroid of the orifice ***to*** the upstream water level as shown in Figure 9a.

The upstream depth, h_u, can also be measured from the bottom ***of*** the orifice opening if the downstream depth is taken to be about 0.611 times the vertical orifice opening, which takes into account the theoretical flow contraction just downstream of the orifice. Otherwise, the inferred assumption is that the downstream depth is equal **to** one-half the opening, and h_u is effectively measured from the area centroid of the opening. Either of these two assumptions may be adequate in rating the orifice for free-flow conditions, and in defining the governing equation, but it should be noted that the choice will affect the value of the discharge coefficient.

If the downstream water level **is** also above the top of the orifice (see Figure 9b), then submerged conditions exist and the discharge equation becomes:

$$Q_s = C_d \, C_v \, A \sqrt{2g(h_u - h_d)} \tag{23}$$

Where $h_u - h_d$ is the difference in water surface elevations upstream and downstream of the submerged orifice.

The velocity head coefficient, C_v, approaches unity as the approach velocity **to** the orifice decreases **to** zero. In irrigation systems, C_v can usually be assumed to be unity since most irrigation channels have very flat gradients and the flow velocities are low (usually much less than **1** m/s).

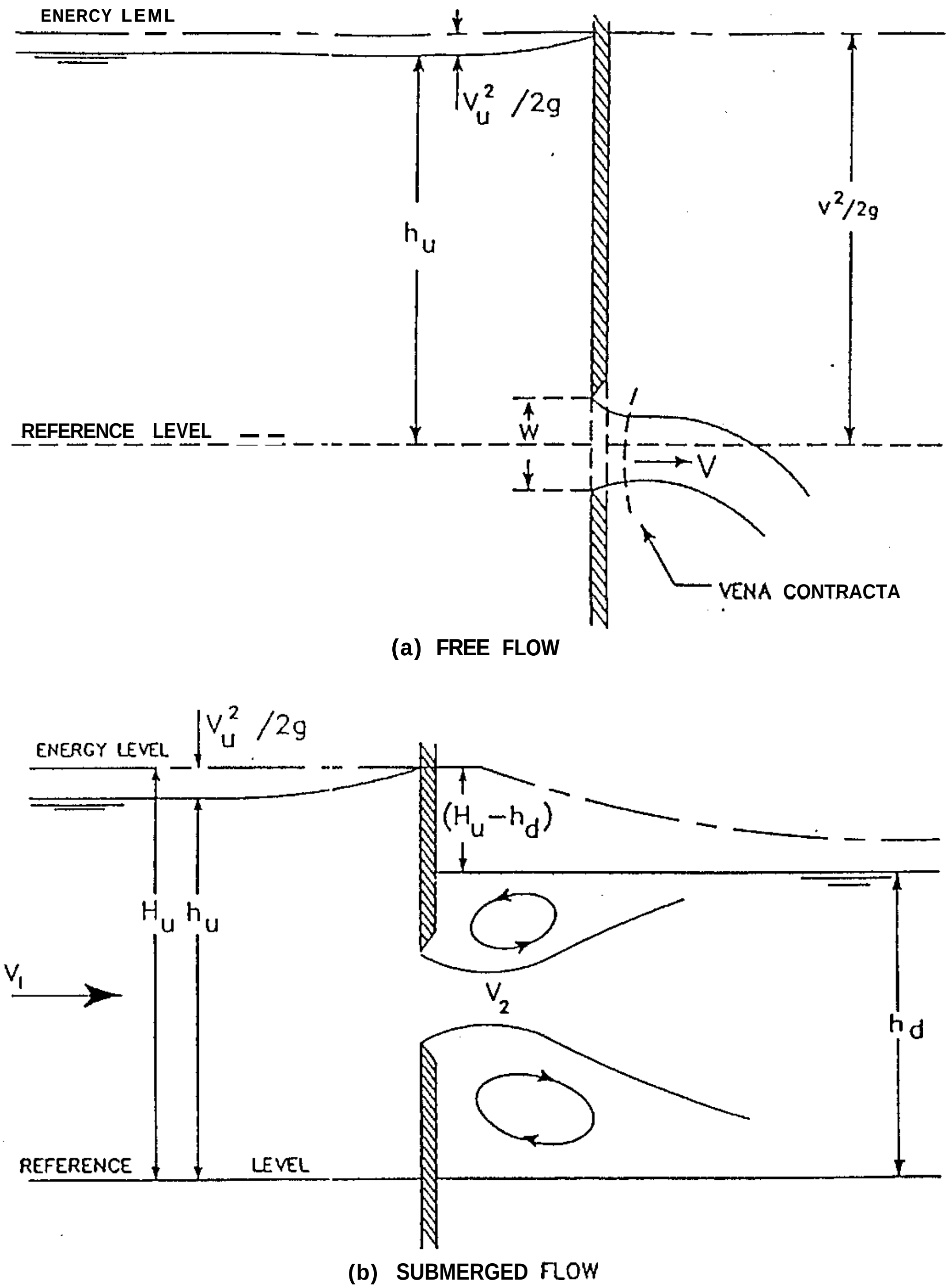


Figure 9. Definition **sketch** of **orifice flow.**

An orifice can be used as a highly accurate flow measuring device in an irrigation system. If the orifice structure has not been previously rated in the laboratory, then it can easily be rated in the field. The hydraulic head term, h_u or $(h_u - h_d)$, can be relied upon to have the exponent 1/2, which means that a single field rating measurement, if accurately made, will provide an accurate determination of the coefficient of discharge, C_d. However, the use of a single rating measurement implies the assumption of a constant C_d value, which is not the case in general. Adjustments to the basic orifice equations for free- and submerged-flow are often made to more accurately represent the structure rating as a function of flow depths and gate openings. The following sections present some alternative equation forms for taking into account the variability in the discharge coefficient under different operating conditions. Orifices usually have C_d values of about 0.60 to 0.80, depending on the geometry and installation of the structure, but values ranging from about 0.3 to 0.9 have been measured in the field.

2.4.1 Free-Flow Rectangular Gate Structures

A definition sketch for a rectangular gate structure having free orifice flow is shown in Figure 10. For a rectangular gate having a gate opening, G_o, and a gate width, W, the free-flow discharge equation can be obtained from Equation 22, assuming that the dimensionless velocity head coefficient is unity.

$$Q_f = C_d \, G_o \, W\sqrt{2g(h_u - G_o/2)} \qquad (24)$$

where G_o is the vertical gate opening, W is the gate width, and G_oW is the area, A, of the orifice opening.

The upstream flow depth, h_u, can be measured anywhere upstream of the gate, including the upstream face of the gate. The value of h_u will vary a small amount depending on the location chosen for measuring h_u. Consequently, the value of the coefficient of discharge, C_d, will also vary according to the location selected for measuring h_u.

One of the most difficult tasks in calibrating a gate structure is obtaining a highly accurate measurement of the gate opening, G_o. For gates having a threaded rod that rises as the gate opening is increased, the gate opening is read from the top of the handwheel to the top of the rod with the gate closed, and then set at some opening, G_o. This very likely represents a measurement of gate opening from where the gate is totally seated, rather than a measurement from the gate sill; therefore, the measured value of G_o from the threadrod will usually be greater than the true gate opening, unless special precautions (described below) are taken to calibrate the threadrod.

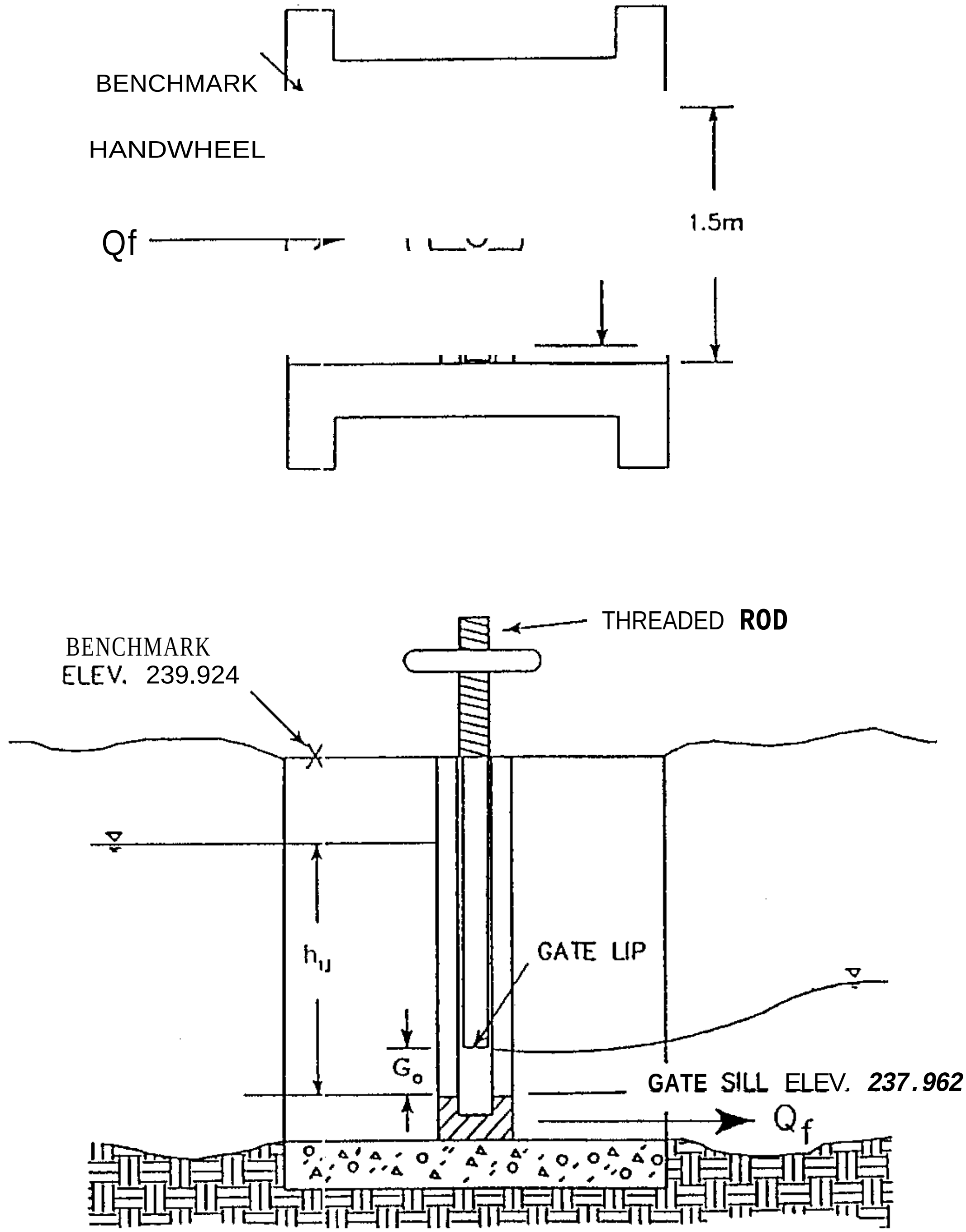


Figure 10. Definition sketch and example problem for a rectangular gate structure having free-orifice flow.

Likewise, when the gate lip is set at the same elevation as the gate sill, there will undoubtedly be some flow or leakage through the gate. This implies that the datum for measuring the gate opening is below the gate sill. In fact, there is often leakage from a gate even when it is totally seatecl (closed) because *of* inadequate maintenance. An example problem will be used to illustrate the procedure for determining an appropriate zero datum for the gate opening.

For the rectangular gate structure shown in Figure 10, the calibration data listed in Table 8 was collected. The data reduction is listed in Table 9 where the coefficient of discharge, C_d was calculated from Equation 24.

A rectangular coordinate plot of C_d versus the gate opening, G_o, listed in Table **9** is shown graphically in Figure 11. The value of C_d continues to decrease with larger gate openings. To determine if a constant value of C_d can be derived, Equation 24 can be rewritten in the following format:

$$Q_f = C_d (G_o + \Delta G_o) W \sqrt{2g \left((h_u)_{\Delta G_o} - \frac{G_o + \Delta G_o}{2} \right)} \qquad (25)$$

Where ΔG_o is a measure of the zero datum level below the gate sill, which

$$(h_u)_{\Delta G_o} = h_u + \Delta G_o \qquad (26)$$

is shown in Figure 12. An appropriate value of ΔG_o will be determined by trial-and error for the example problem. Assuming values *of* ΔG_o equal to 1 mm, 2 mm, 3 mm, etc., the computations for determining C_d can be made using Equation 25. The results for ΔG_o equal to 1 mm, 2 mm, 3 mm, 4 mm, 5 mm, 6 mm, 7 mm, 8 mm and 12 mm (gate seated) are listed in Table 10. The best results are obtained from ΔG_o of 3 mm; this result is plotted in Figure 13, which shows that C_d varies from 0.582 to 0.593 with the average value of C_d being 0.587. For this particular gate structure, the discharge normally varies between 200 and 300 lps, and the gate opening is normally operated between 40-60 mm, so that a constant value of C_d = 0.587 can be used when the zero datum for G_o and h_u is taken as 3 mrn below the gate sill (another alternative would be to use a constant value *of* C_d = 0.575 for ΔG_o = **4** mm and G_o greater than 30 mm).

Discharge, Q_t m³/s	Gate Opening, G_o m	Upstream Benchmark Tape Measurement m
0.0646	0.010	0.124
0.0708	0.020	1.264
0.0742	0.030	1.587
0.0755	0.040	1.720
0.0763	0.050	1.707
0.0767	0.060	1.825

TABLE 9. Data reduction for example rectangular gate structure having free orifice flow.

Q_t m³/s	G_o m	h_u m	C_d (see note below)
0.0646	0.010	1.838	0.756
0.0708	0.020	0.698	0.677
0.0742	0.030	0.375	0.654
0.0755	0.040	0.242	0.635
0.0763	0.050	0.175	0.625
0.0767	0.060	0.137	0.620

Note: The discharge coefficient, C_d, was calculated using the following equation:

$$Q_t = C_d G_o W \sqrt{2g\,(h_u - G_o/2)}$$

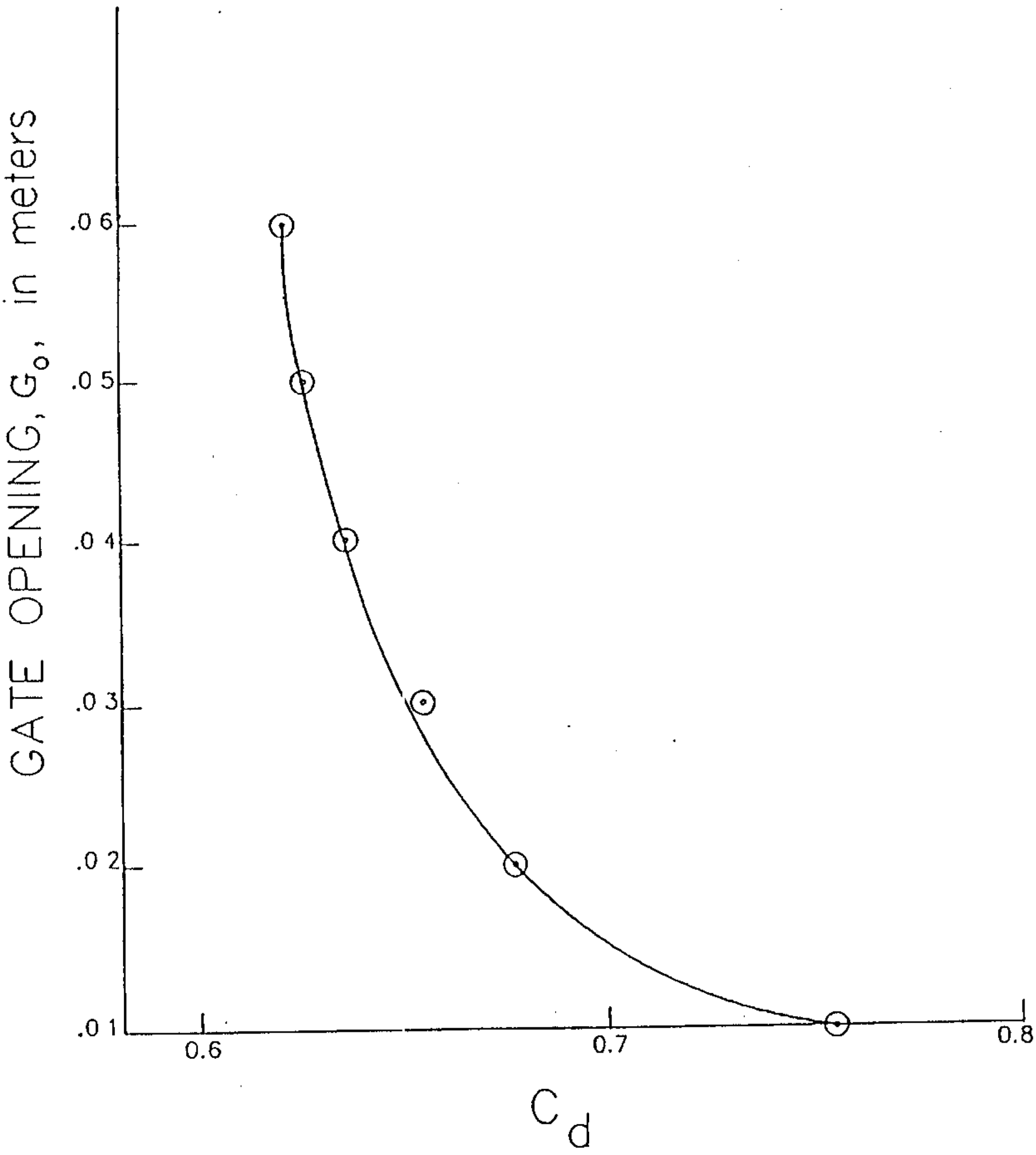


Figure 11. Variation in the discharge coefficient, C_d, with gate opening, G_o, for the example rectangular gate structure with free orifice flow.

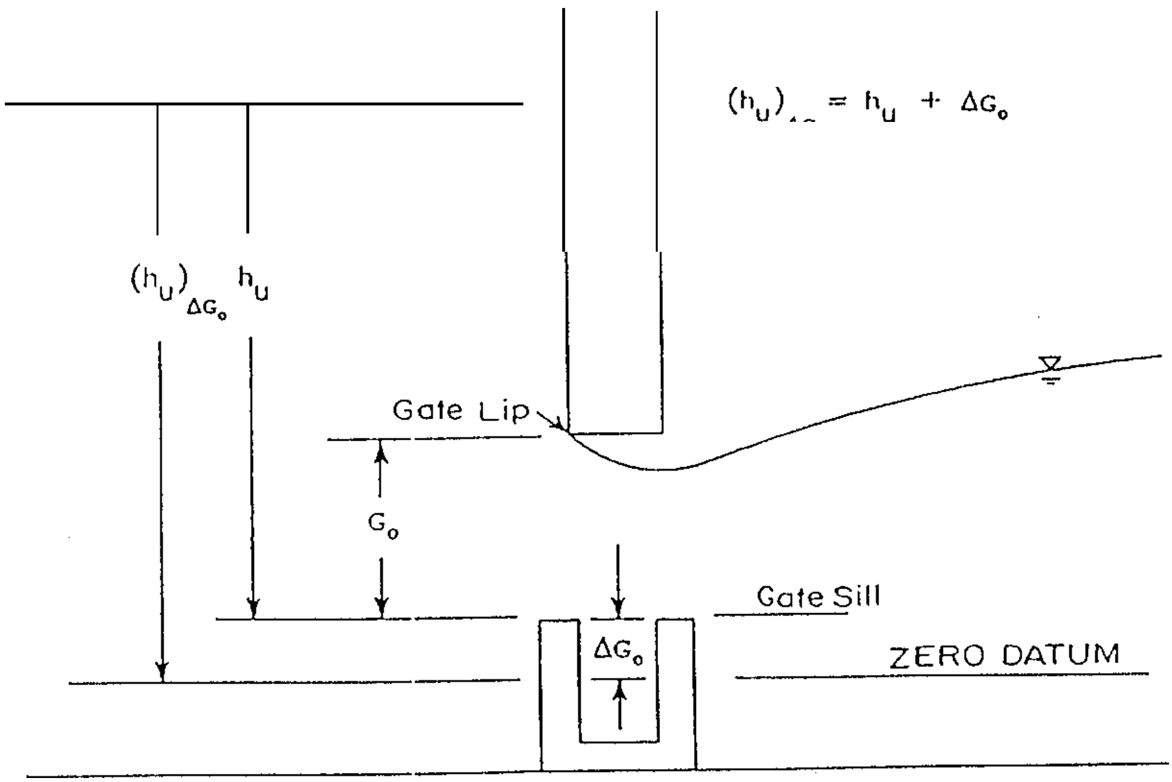


Figure 12. Definition sketch of the zero datum level for gate opening and upstream flow depth for a rectangular gate structure.

TABLE 10. Computation of the discharge coefficient, C_d, for adjusted values of gate opening and upstream flow depth for the example rectangular gate structure having free orifice flow.

Q_f m^3/s	G_o m	h_u m	Discharge Coefficient, C_d (see note below)									
			ΔG_o 0 mm	ΔG_o 1 mm	ΔG_o 2 mm	ΔG_o 3 mm	ΔG_o 4 mm	ΔG_o 5 mm	ΔG_o 6 mm	ΔG_o 7 mm	ΔG_o 8 mm	ΔG_o 12 mm
0.0646	0.010	1.838	0.756	0.688	0.630	0.582	0.540	0.504	0.472	0.445	0.420	0.344
0.0708	0.020	0.698	0.677	0.644	0.615	0.588	0.563	0.540	0.519	0.500	0.482	0.425
0.0742	0.030	0.375	0.654	0.632	0.612	0.593	0.575	0.558	0.542	0.527	0.513	0.471
0.0755	0.040	0.242	0.635	0.619	0.604	0.589	0.575	0.561	0.549	0.536	0.525	0.495
0.0763	0.050	0.175	0.625	0.611	0.599	0.586	0.575	0.563	0.552	0.542	0.531	0.514
0.0767	0.060	0.137	0.620	0.608	0.597	0.586	0.575	0.565	0.556	0.546	0.537	0.531

Note: The last column with ΔG_o = 12 mm is for the gate totally seated (closed). The discharge coefficient, C_d, was calculated from:

$$Q_f = C_d (G_o + \Delta G_o) W \sqrt{2g\left((h_u)_{\Delta G_o} - \frac{G_o + \Delta G_o}{2}\right)}$$

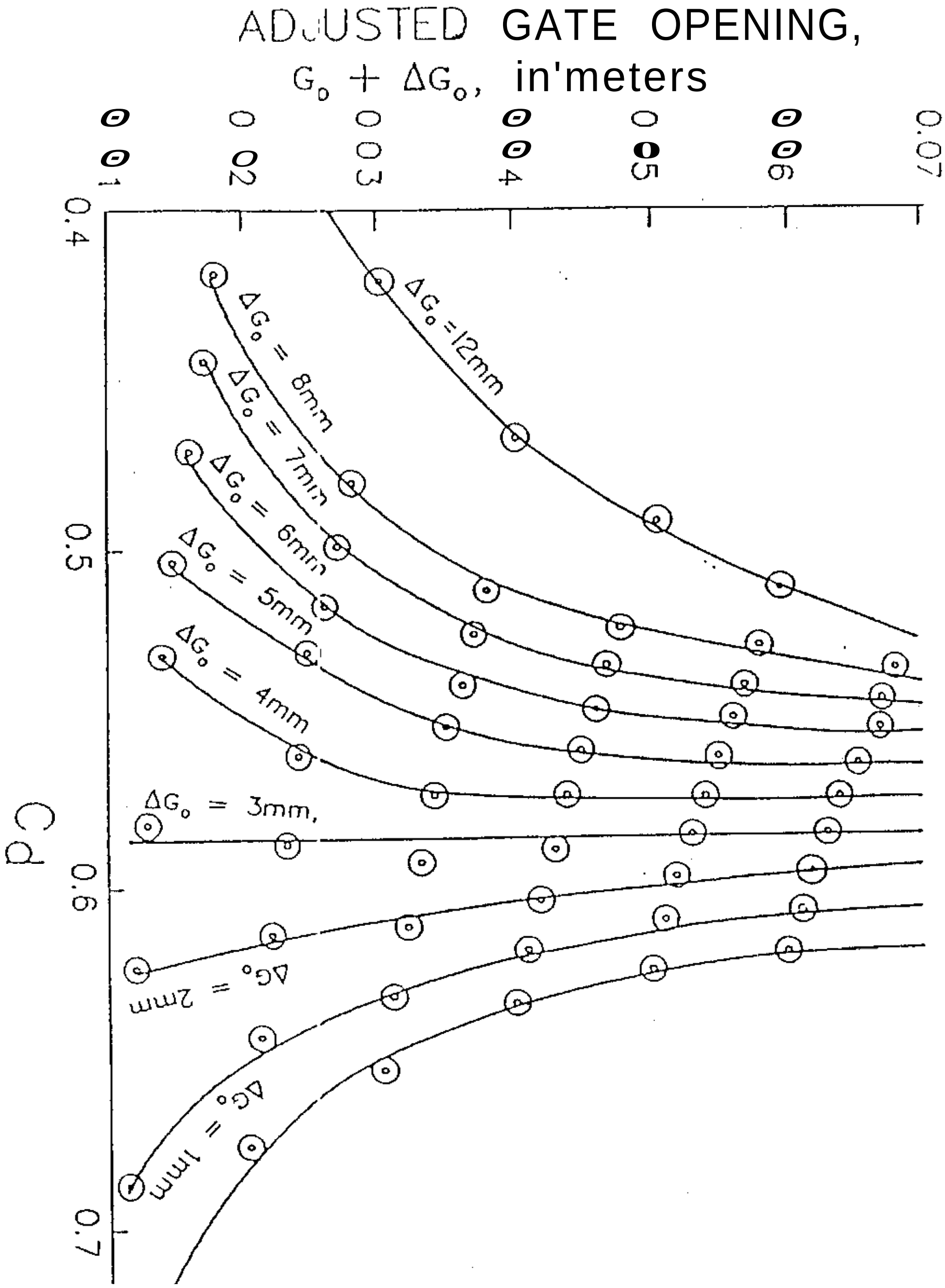


Figure 13. Variation in the discharge coefficient, C_d, with the adjusted gate opening, $G_o + \Delta G_o$, for the exarnple rectangular gate structure with free orifice flow.

2.4.2 Submeraed-Flow Rectanaular Gate Structures

Submerged-flow gate structures are the most common constrictions employed in irrigation networks. The gates are used to regulate the water levels upstream and the discharge downstream For this reason, they are very important structures that need to be field calibrated. Fortunately, they are one of the easiest structures to field calibrate for discharge measurement.

A definition sketch for a rectangular gate structure having submerged orifice flow is shown in Figure 14. Assuming that the dimensionless velocity head coefficient in Equation 23 is unity, the submerged-flow discharge equation for a rectangular gate having an opening G_o, and a width W, becomes:

$$Q_s = C_d \, G_o \, W\sqrt{2g \, (h_u - h_d)} \tag{27}$$

where G_oW is the area, A, of the orifice.

The upstream flow depth, h_u, can be measured anywhere upstream of the gate, including the upstream face of the gate. Likewise, the downstream flow depth, h_d can be measured anywhere downstream of the gate, including the downstream face of the gate. Many times, h_u and h_d will be measured at the gate because only one reference benchmark is needed on top of the gate structure in order to make tape measurements down to the water surface. This is satisfactory if the water surfaces on the gate are smooth. If not, h_u and h_d should be measured at locations where the water surface is smooth, not turbulent and fluctuating.

All of the information in the previous section regarding the measurement of gate opening, G_o , applies equally well for submerged gates.

For the rectangular gate structure shown in Figure 14, the field calibration data is listed in Table 11. Note that for this type of slide gate, the gate opening was measured both on the left side $(G_o)_L$, and the right side, $(G_o)_R$, because the gate lip is not always horizontal. The data reduction is listed in Table 12 where the coefficient of discharge, C_d was calculated from Eiquation 27. The variation in C_d with gate opening, G_o, is plotted in Figure 15.

As in the case of the free-flow orifice calibration in the previous section, a trial-and-error approach can be used to letermine a more precise zero datum for the gate opening (see Figure 12). In this case, Equation 27 would be rewritten as:

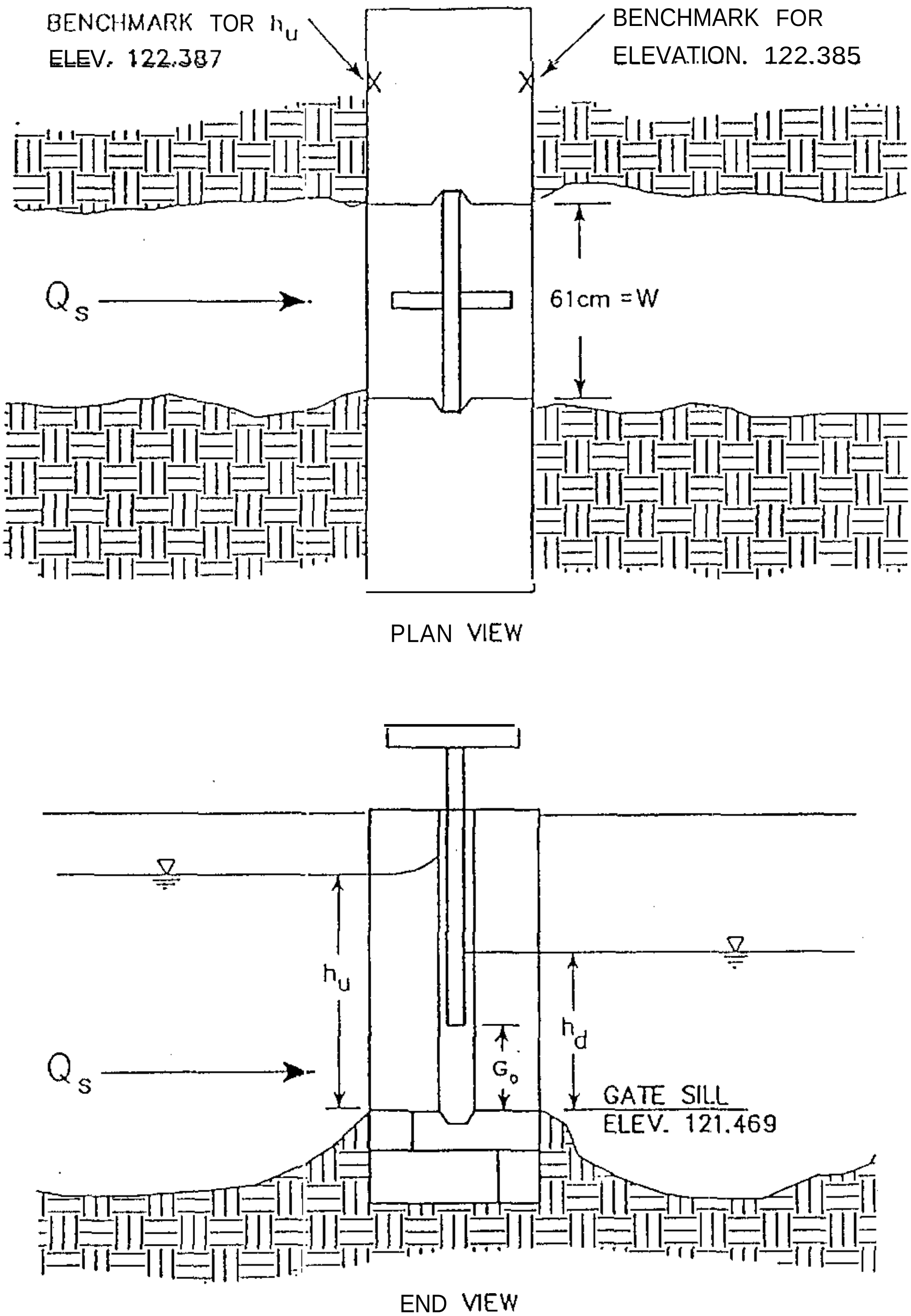


Figure 14. Definition sketch and example problem for a rectangular gate structure having submerged orifice flow.

TABLE 11. Example field calibration data for a rectangular gate structure having submerged orifice flow.

Discharge, Q_s m³/s	Gate Opening $(G_o)_L$ m	Gate Opening $(G_o)_R$ m	Benchmark Tape Measurements Upstream m	Benchmark Tape Measurements Downstream
0.079	0.101		0.095	0.273
[illegible]	[illegible]		0.099	0.283
0.111	0.139	0.139	0.102	0.296
0.126	0.161	0.163	0.105	0.290
0.141	0.180	0.178	0.108	0.301
0.155	0.199	0.197	0.110	0.301

TABLE 12. Data reduction for example rectangular gate structure having submerged orifice flow.

Q_s m³/s	G_o m	h_u m	h_d m	h_u - h_d m	C_d
0.079	0.102	0.823	0.643	0.180	0.676
0.095	0.121	0.819	0.633	0.187	0.674
0.111	0.139	0.816	0.620	0.196	0.668
0.126	0.162	0.813	0.626	0.187	0.666
0.141	0.179	0.810	0.615	0.195	0.660
0.155	0.198	0.808	0.615	0.193	0.659

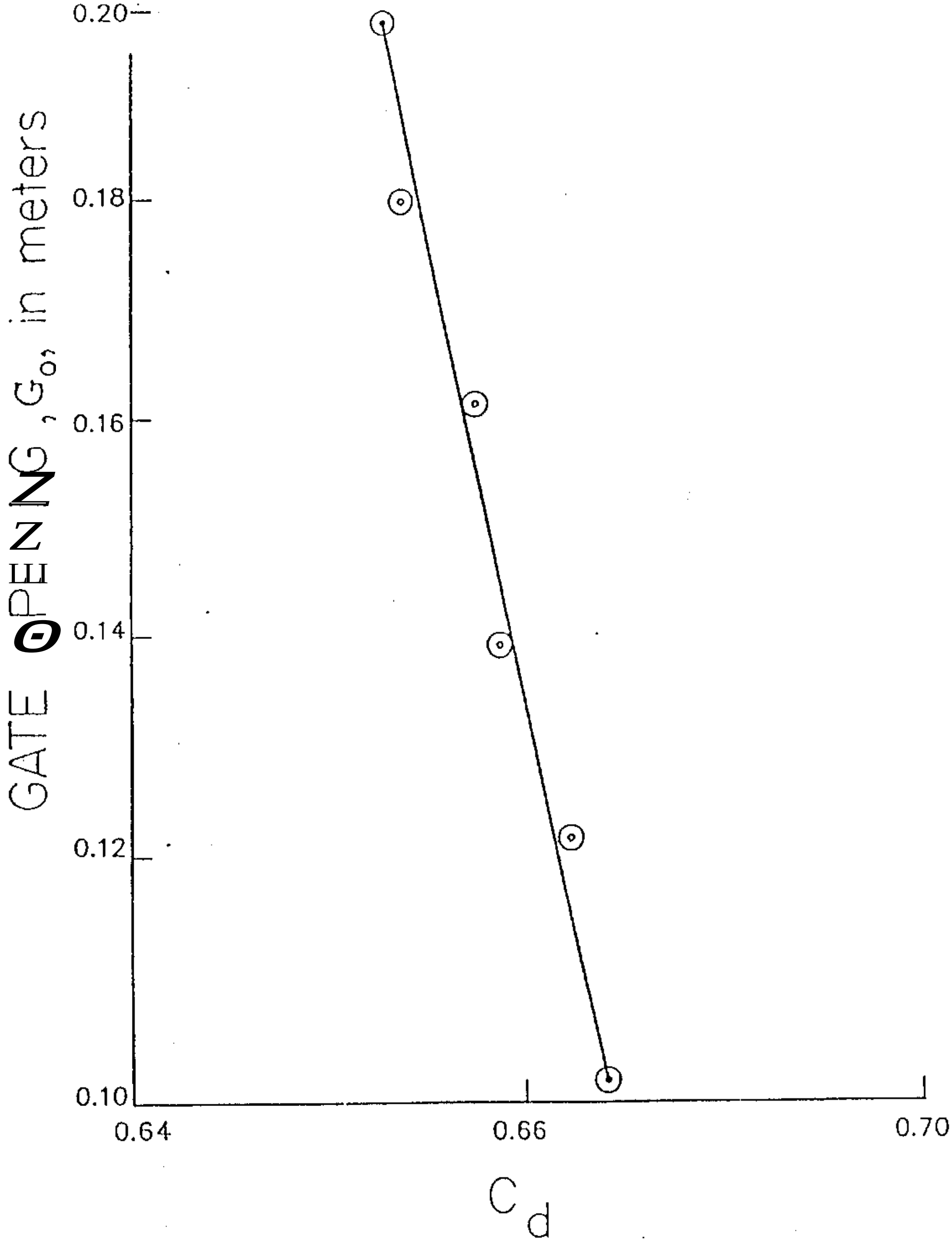


Figure **15.** Variation in the discharge coefficient, C_d, with gate opening, G_o, for the example rectangular gate structure with submerged orifice flow.

$$Q_s = C_d \ (G_o + \Delta G_o) \ W \sqrt{2g \ (h_u - h_d)} \tag{28}$$

Q_s m³/s	G_o m	h_u - h_d m	C_d (see note below) ΔG_o = 4 mm	ΔG_o = 6 mm	ΔG_o = 8 mm
0.079	0.102	0.1801	0.650	0.638	0.626
0.095	0.121	0.1865	0.651	0.641	0.631
0.111	0.139	0.1960	0.649	0.640	0.631
0.126	0.162	0.1869	0.650	0.642	0.635
0.141	0.179	0.1949	0.646	0.639	0.632
0.155	0.198	0.1931	0.646	0.640	0.634

$$Q_s = C_d (G_o + \Delta G_o) \ W \sqrt{2g (h_u - h_d)} \tag{29}$$

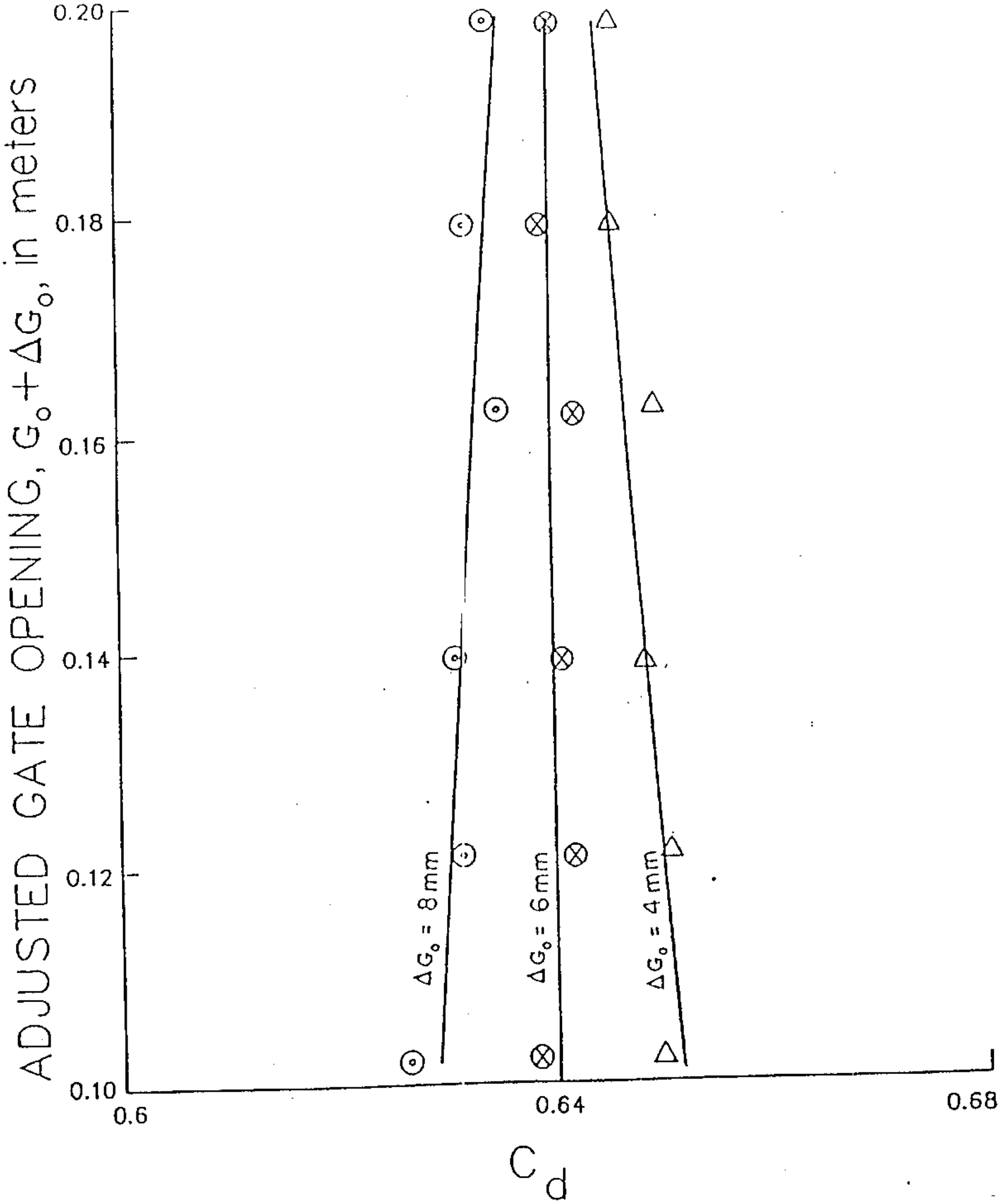


Figure **16.** Variation in the discharge coefficient, C_d, with the adjusted gate opening, $G_o + \Delta G_o$, for the example rectangular gate structure with submerged orifice flow.

2.4.3 Calibratins Larse Gate Structures

The authors have successfully calibrated large gate structures in irrigation canals for free and submerged orifice flow. The basic hydraulic principles described above for orifice flow apply equally well to large structures. However, there are some significant differences in the field calibration methods and rating equations.

Current metering can be used in large canals to determine discharge, and both conventional (propeller) or magnetic meters can give good results. Magnetic current meters have the advantage that the job can be finished relatively fast, and with very good accuracy. However, conventional current meters can be more useful when the flow velocities fluctuate due to turbulence. Under these conditions, the conventional current meters tend to do a better job of integrating the velocity over a period of one or two minutes; however, this advantage has lessened as improved models of the magnetic current meters are introduced. Discharge may be the most difficult rating parameter to measure in large canals because it cannot be measured directly, as is the case for flow depths and gate dimensions. Discharge measurements may also present a significant element of personal danger during current metering when the work is being done on the upstream side of a culvert or siphon.

Current metering on the upstream side of a gate structure can give better results because the flow is much less turbulent than on the downstream side. Turbulence on the downstream side tends to continue for a significant distance downstream of the gate, yet it is preferable to measure the discharge near the gate to avoid errors due to leakage and seepage along the canal. Thus, the discharge some distance downstream or upstream of the gate may be sigriificantly different than the discharge through the gate. Nevertheless, as with smaller gate structures, it is desirable to measure discharge on both the upstream and downstream sides of the gate (when possible) for the purpose of comparison and double-checking the work.

Upstream and downstream flow depths may be measured adjacent to the gate, or further upstream and downstream of the gate. For free-flow, the downstream depth need not be measured, but for submerged flow only the difference in upstream and downstream water elevations is required. For submerged-flow ratings, the upstream and downstream depths may be measured independently from a common datum, which may be using staff gauges that have been referenced to the same elevation (e.g. mean sea level). Greater accuracy can be obtained by using an engineers' level and taking elevation measurements of the upstream and downstream water surfaces. Nevertheless, subsequent field application of the structure rating will require the use of staff gauges or stilling wells; otherwise, use of the structure as a flow measurement device will be less convenient. It is important to base the depth measurements during rating on the same depth measurements that will be used to later determine discharge.

A different form of the subrnerged-flow rating equation has been used with excellent results on many different orifice-type structures in large canals. The differences in the equation involve consideration of the gate opening and the downstream depth as influential factors in the determination of the discharge coefficient. Thus, the discharge coefficient is not constant, but rather depends upon the gate opening and downstream depth. The equation is as follows:

$$Q_s = C_s\, h_s\, W \sqrt{2g(h_u - h_d)} \qquad (30)$$

and,

$$C_s = a \left(\frac{G_o}{h_s}\right)^{\beta} \qquad (31)$$

where h_s is the downstream depth referenced to the bottom of the gate opening, alpha and beta are empirically-fitted parameters, and all other terms are as described in the previous sections of this manual. The value of the exponent beta is usually very close to unity, and in fact, for beta equal to unity the equation reverts to that of a constant value of C_s = alpha (the h_s term cancels).

Table 14 shows some example field calibration data for a large canal gate, and the results of the regression and calibration are plotted in Figure 17. The solution to the example calibration is: alpha = 0.796, and beta = 1.031. This particular data set indicates an excellent fit to Equation 30, and it is typical of other large gate structures operating under submerged-flow conditions.

A similar equation can be used for free-flow through a large gate structure, with the upstream depth, h_u replacing ti-e term h_s, and with $(h_u - G_o/2)$ replacing $(h_u - h_d)$. The free-flow equation can also be calibrated using $(h_u - 0.611\, G_o)$ instead of $(h_u - G_o/2)$.

TABLE 14. Field data for submerged-flow calibration of a large gate.

Data Set	Discharge (m^3/s)	G_o (m)	Δh (m)	h_s (m)	G_o/h_s	C_s
1	8.38	0.60	3.57	2.205	0.272	0.206
2	9.08	0.70	3.00	2.010	0.348	0.268
3	5.20	0.38	3.31	1.750	0.217	0.168
4	4.27	0.30	3.41	1.895	0.158	0.125
5	5.45	0.40	3.43	2.025	0.198	0.149
6	12.15	0.95	2.63	2.300	0.413	0.334
7	5.49	0.38	3.72	1.905	0.199	0.153
8	13.52	1.10	2.44	2.405	0.457	0.369
9	14.39	1.00	3.84	2.370	0.422	0.318
10	16.14	1.13	3.79	2.570	0.440	0.331
11	6.98	0.50	3.70	1.980	0.253	0.188
12	11.36	0.58	7.64	2.310	0.251	0.183
13	7.90	0.42	6.76	2.195	0.191	0.142
14	7.15	0.38	6.86	2.110	0.180	0.133
15	7.49	0.51	3.98	2.090	0.244	0.184
16	10.48	0.70	3.92	2.045	0.342	0.266
17	12.41	0.85	3.76	2.205	0.385	0.298
18	8.26	0.55	3.91	2.065	0.266	0.208

Note: the data is for two identical gates in parallel, both having the same opening for each data set, with a combined opening width of 2.20 m.

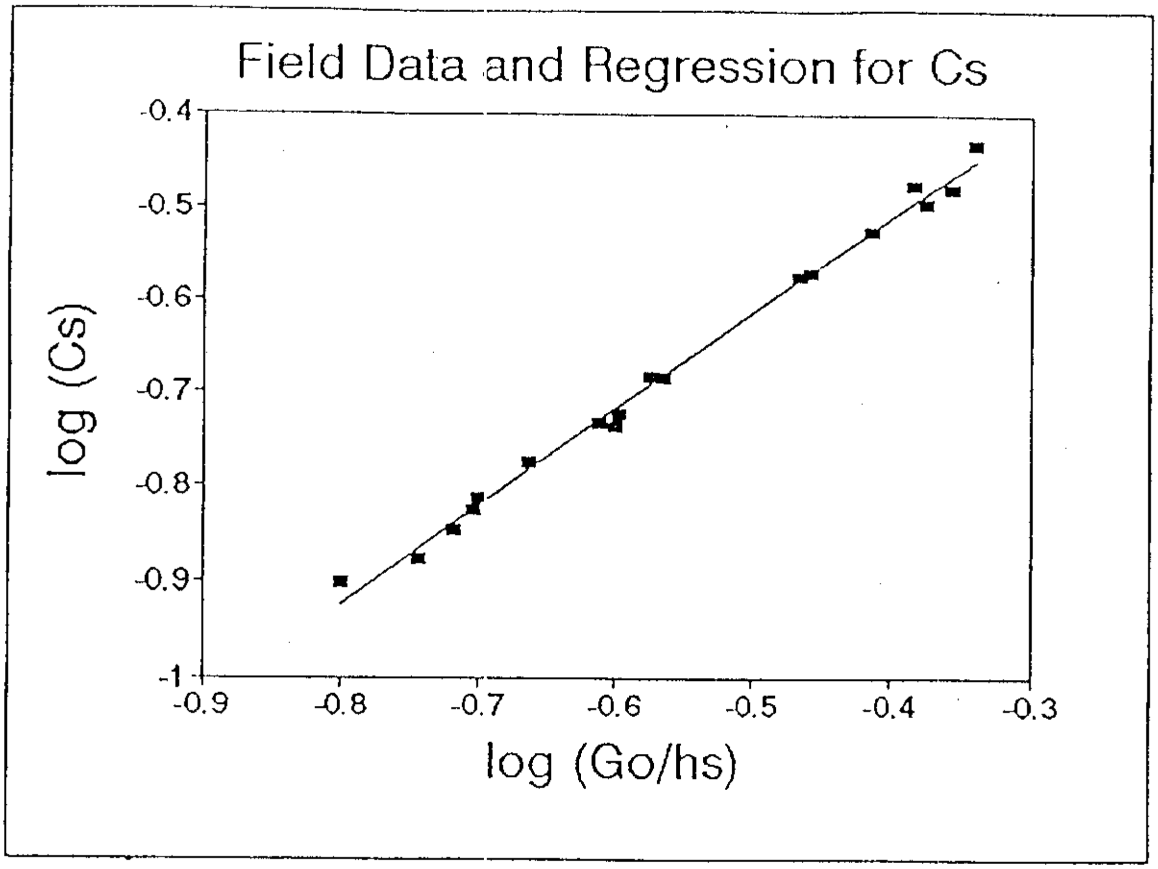


Figure 17. Calibration Data for the Example Large Gate Structure,

2.5. Current Meters for Discharge Measurement

2.5.1 Types of Current Meters

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

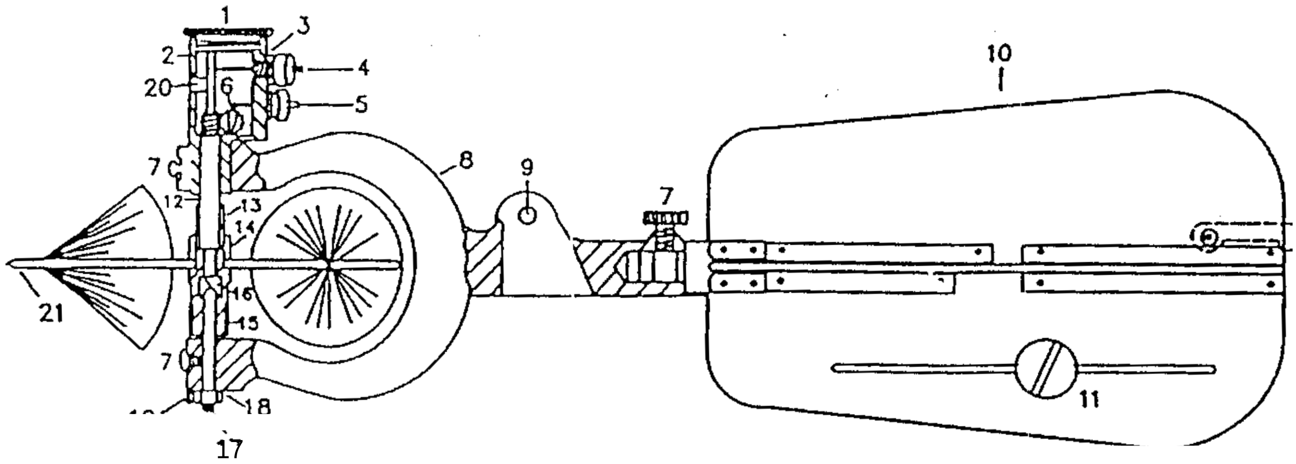
Current meters with a rotating unit that is sensing the water velocity are either vertical-shaft or horizontal-shaft types. The vertical-axis current meter has a rotating cup with a bearing system that is simpler in design, more rugged, and easier to service and maintain than horizontal-shaft (axis) 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 sensitive, 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 Price Pygmy Current Meter, which is used for shallow flow depths and low velocities. A diagrarn for the Price Type A Current Meter is shown in Figure **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 horizorital-shaft current meters are less sensitive to the vertical velocity components. 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).

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.

Electronic (electro-magnetic) current meters are now available that contain a sensor with the point velocity being digitally displayed. Some of the earlier models had considerable electronic noise under turbulent flow conditions. Fortunately, present models yield more stable velocity readings and have been used in irrigation channels. Undoubtedly, these instruments will be further improved in the near future.



EXPLANATION

1 CAP FOR CONTACT CHAMBER
2 CONTACT CHAMBER
3 INSULATING BUSHING FOR CONTACT BINDING POST
4 SINGLE-CONTACT BINDING POST (UPPER)
5 PENTA-CONTACT BINDING POST (LOWER)
6 PENTA-GEAR
7 SET SCREWS
8 YOKE
9 HOLE FOR HANGER SCREW
10 TAILPIECE
11 BALANCE WEIGHT
12 SHAFT
13 BUCKET WHEEL HUB
14 BUCKET WHEEL HUB NUT
15 RAISING NUT
16 PIVOT BEARING
17 PIVOT
18 PIVOT ADJUSTING NUT
19 KEEPER SCREW OFR PIVOT ADJUSTING NUT
20 BEARING LUG
21 BUCKET WHEEL

Source: Don M. Corbett et al. **1962.**

Figure 18. Assembly Diagram for a Price Type **A** Current Meter

2.5.2. Care of Equipment

Accuracy in velocity measurements can only be expected when the equipment is properly assembled, adjusted, and maintained. The current meter should be treated as a delicate instrument that needs meticulous care and protective custody, both when being used and when being transported. The required treatment of a current meter is analogous to a surveyors careful attention with a transit or level.

The current meter necessarily receives a certain amount of hard usage that may result in damage, such as a broken pivot, chipped bearing, or bent shaft that will result in the current meter giving velocity readings that are lower than actual velocities. Observations of velocities near bridge piers and abutments, water depth readings taken at cross-sections having irregular bed profiles with the current meter attached to the measuring line, and the periodic occurrence of floating debris represent the greatest hazards to the equipment (Corbett and others, 1943).

Damage to current meter equipment during transportation is generally due to careless packing or negligence in protection. A standard case is provided by all manufacturers of current meter equipment, which should always be employed before and after taking a discharge measurement. In particular, the equipment case should always be used when transporting the current meter, even when the distance is relatively short. Transportation of assembled equipment from one location to another is one of the most common sources of damage (Corbett and others, 1943).

In some countries, it is common to see current meter equipment without a case. Also, during transport, this equipment will be placed on the floor of the vehicle. In one case, there was a 30 percent variation in velocity measurements among seven current meters as a result of improper care and protection.

2.5.3. Current Meter Ratings

Usually, a current meter is calibrated in a towing tank. The current meter is attached to a carriage that travels on rails (tracks) placed on the top of the towing tank. Then, a series of trials are conducted wherein the current meter is towed at different constant velocities. For each trial, the constant velocity of the carriage is recorded, as well as the revolutions per second (rev/s) of the current meter. This data is plotted on rectangular coordinate graph paper to verify that a straight-line relation exists; then, the equation is determined by regression analysis. Table **1** is an example of a velocity rating based on the rating equation for one particular current meter:

$$\text{Velocity (m/s)} = 0.665 \text{ (rev/s)} + 0.009$$

Table 15. Velocity Rating for an Example Current Meter

Velocity in meters per second (m/s)

Time Seconds	REVOLUTIONS										
	5	10	15	20	25	30	40	50	60	80	100
40	0.092	0.175	0.258	0.342	0.425	0.508	0.674	0.840	1.007	1.339	1.672
41	0.090	0.171	0.252	0.333	0.415	0.496	0.658	0.820	0.982	1.307	1.631
42	0.088	0.167	0.247	0.326	0.405	0.484	0.642	0.801	0.959	1.276	1.592
43	0.086	0.164	0.241	0.318	0.396	0.473	0.628	0.782	0.937	1.246	1.556
44	0.085	0.160	0.236	0.311	0.387	0.462	0.614	0.765	0.916	1.218	1.520
45	0.083	0.157	0.231	0.305	0.378	0.452	0.600	0.748	0.896	1.191	1.487
46	0.081	0.154	0.226	0.298	0.370	0.443	0.587	0.732	0.876	1.166	1.455
47	0.080	0.151	0.221	0.292	0.363	0.434	0.575	0.716	0.858	1.141	1.424
48	0.078	0.148	0.217	0.286	0.355	0.425	0.563	0.702	0.840	1.117	1.394
49	0.077	0.145	0.213	0.280	0.348	0.416	0.552	0.688	0.823	1.095	1.366
50	0.076	0.142	0.209	0.275	0.342	0.408	0.541	0.674	0.807	1.073	1.339
51	0.074	0.139	0.205	0.270	0.335	0.400	0.531	0.661	0.791	1.052	1.313
52	0.073	0.137	0.201	0.265	0.329	0.393	0.521	0.648	0.776	1.032	1.288
53	0.072	0.135	0.197	0.260	0.323	0.385	0.511	0.636	0.762	1.013	1.264
54	0.071	0.132	0.194	0.255	0.317	0.378	0.502	0.625	0.748	0.994	1.241
55	0.070	0.130	0.190	0.251	0.311	0.372	0.493	0.614	0.735	0.976	1.218
56	0.068	0.128	0.187	0.247	0.306	0.365	0.484	0.603	0.722	0.959	1.197
57	0.067	0.126	0.184	0.242	0.301	0.359	0.476	0.592	0.709	0.942	1.176
58	0.066	0.124	0.181	0.238	0.296	0.353	0.468	0.582	0.697	0.926	1.156
59	0.065	0.122	0.178	0.234	0.291	0.347	0.460	0.573	0.685	0.911	1.136
60	0.064	0.120	0.175	0.231	0.286	0.342	0.452	0.563	0.674	0.896	1.117
61	0.064	0.118	0.173	0.227	0.282	0.336	0.445	0.554	0.663	0.881	1.099
62	0.063	0.116	0.170	0.224	0.277	**0.331**	**0.438**	0.545	0.653	0.867	1.082
63	0.062	0.115	0.167	0.220	0.273	0.326	0.431	0.537	0.642	0.853	1.065
64	0.061	0.113	0.165	0.217	0.269	0.321	0.425	0.529	0.632	0.840	1.048
65	0.060	0.111	0.163	0.214	0.265	0.316	0.418	0.521	0.623	0.828	1.032
66	0.059	0.110	0.160	0.211	0.261	0.311	0.412	0.513	0.614	0.815	1.017
67	0.059	0.108	0.158	0.208	0.257	0.307	0.406	0.505	0.605	0.803	1.002
68	0.058	0.107	0.156	0.205	0.254	0.302	0.400	0.498	0.596	0.791	0.987
69	0.057	0.105	0.154	0.202	0.250	0.298	0.395	0.491	0.587	0.780	0.973
70	0.057	0.104	0.152	0.199	0.247	0.294	0.389	0.484	0.579	0.769	0.959

2.6. Methods of Employing Current Meters

2.6.1 Wading

The wading method involves having the hydrographer stand in the water holding a wading rod with the current meter attached to the rod. The wading rod is graduated so that the water depth can be measured. The rod has a metal foot pad which sets on the channel bed. The current meter can be placed at any height on the wading rod and is readily adjusted to another height by the hydrographer while standing in the water.

A tag line is stretched from one bank to the other, which can be a cloth or metal tape. This tag line is placed perpendicular to the flow direction. The zero length on the tag line does not have to correspond with the edge of the water on one of the banks. This tag line is used to define the location of the wading rod each time that a current meter measurement is made.

The wading rod is held at the tag line. The hydrographer stands sideways to the flow direction and is thus facing towards one of the banks. The hydrographer stands 5-10 cm downstream from the tag line and approximately 50 cm aside from the wading rod. During the measurement, the rod needs to be held in a vertical position and the current meter must be parallel with the flow direction. Usually, the notekeeper can signal to the hydrographer whether or not the rod is vertical in relation to the flow direction.

If the flow velocity at the bank is not zero, then this velocity should be estimated as a percentage of the velocity at the nearest measuring point (vertical). Thus, the nearest measuring point should be as close to the bank as possible in order to minimize the error in the calculaied discharge for the section adjacent to the bank (Corbett and others, 1943).

2.6.2. Bridge

Many of the larger irrigation channels have bridges at various locations, such as headworks and check structures (cross regulators), but they may not be located at an appropriate section for current meter measurements. However, culverts often prove to be very good locations, with the current meter measurements usually being made on the downstream end of the culvert where parallel streamlines are more likely to occur. In some cases, wooden foot bridges have been placed by nearby inhabitants in order to cross the canal. Bridges often have piers, which tend to collect debris on the upstream face, that should be removed prior to undertaking current meter measurements.

Either a hand line or a reel assembly may be used from a bridge. In either case, a weight is placed at the bottom of the line, which sets on the channel bed in order that the line does not move as a result of the water flow. The current rneter is then placed at whatever location is required for each measurement.

For a hand line assembly, the weight is lowered from the bridge to the channel bed and the reading on the graduated hand line is recorded; then, the weight is lifted until it is setting on the water surface and the difference in the two readings on the hand line is recorded as the water depth. Afterwards, the current meter is placed at the appropriate location on the hand line in order to make the current meter measurement.

If a weight heavier than 10-15 kg is required in order to have a stable, nearly vertical, cable line, then a crane-and-reel assembly is used. The reel is mounted on a crane designed to clear the hanclrail of the bridge and to guide the meter cable line beyond any interference with bridge members. The crane is attached to a movable base for convenience in transferring the equipment from one measuring point (vertical) to another (Corbett and others, 1943).

2.6.3. Cablewav

For very wide canals, or rivers, with water depths exceeding 150 cm, a cable is placed above the water with vertical supports on each bank that are heavily anchored for stability. The cable supports a car (box) that travels underneath the cable using pulleys. This car carries the hydrographer and the current meter equipment. The cable has markers so that the location across the channel is known. A hand line or a cable reel assembly is used depending on the size of the weight that must be used.

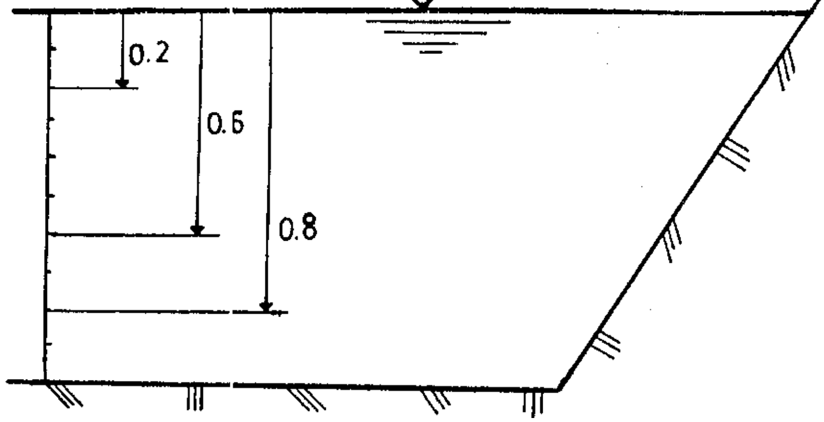
2.6.4. Boat

For some extremely wide channels, such as encountered in the Indian Subcontinent, the installation of a cableway is a significant expense, so that a boat is commonly employed instead. Either a hand line or a cable reel assembly is used.

2.7. Velocity Measurement Methodologies

2.7.1. Vertical Velocitv Method

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 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 methods (see sketch on next page).



When the above field procedure has been completed for a number of 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,

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.

2.7.2 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. 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.

2.7.3 Six-Tenths Depth Method

For shallow water depths, say less ihan 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.

2.8. Velocity at Vertical Wails

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 **2.** 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 D14 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 2.

2.9. 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 velocities 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.

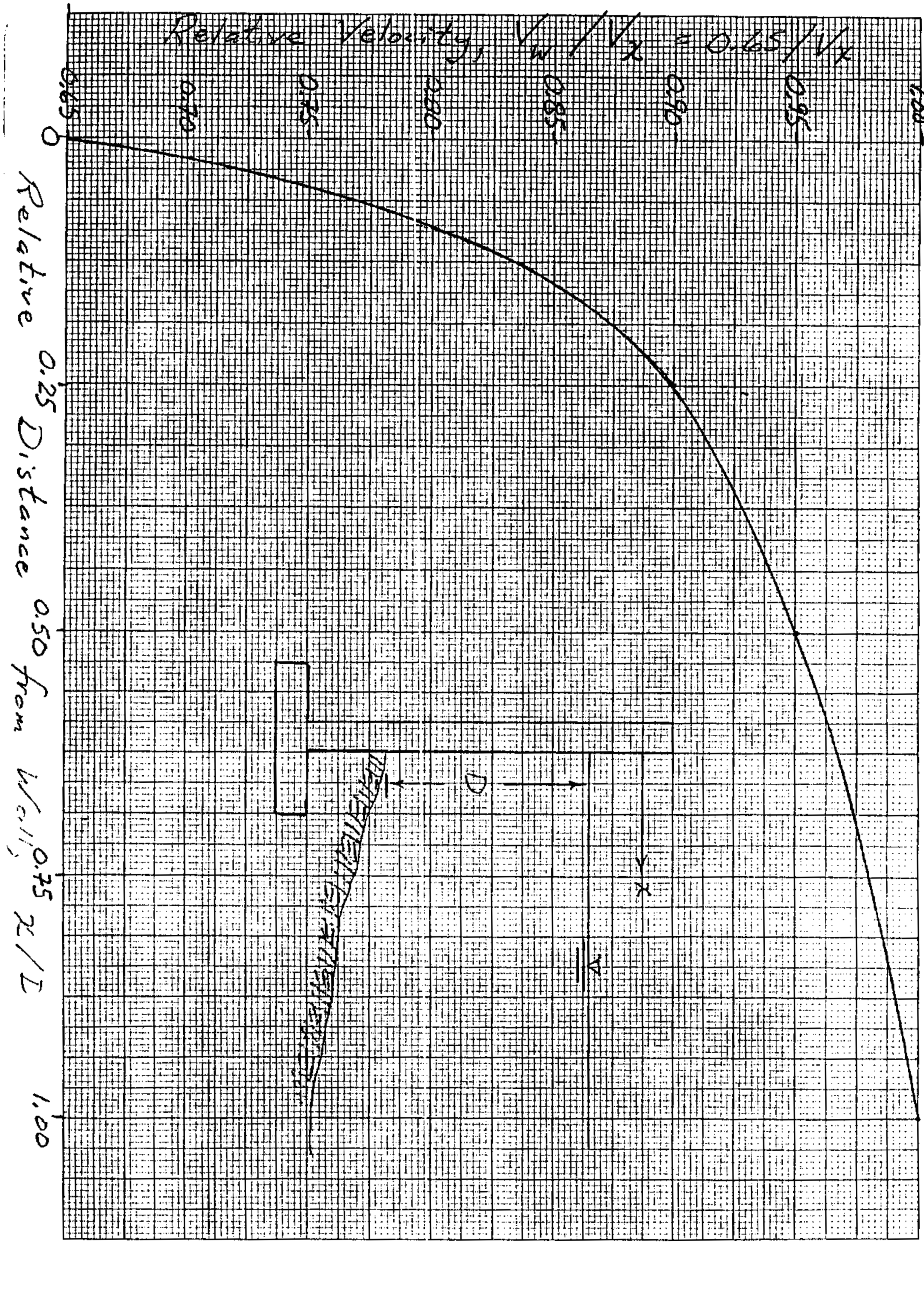


Figure **19.** Relative Mean Velocity Near a Vertical Wall.

2.10. Subdivision of Cross-Section into Verticals

The current meter is used to measure the mean velocity of each vertical in the cross-section. In addition, the spacing of the verticals is used in determining the cross-sectional area of each section, where a section is defined as the cross-sectional area of flow between two verticals.

The measuring cross-section should be subdivided into twenty or more verticals for a relatively smooth channel bed. For an irregular channel bed, more verticals are needed, not only to better define the cross-sectional area of flow, but also because an irregular bed causes more variation in the velocity distribution. At the same time, verticals do not need to be spaced closer than 0.3 m (Corbett and others, 1943).

An example earthen canal cross-section is illustrated in Figure 3. The most important verticals for defining the cross-sectional area of flow are shown.

2.11. Measurement Of Water Depths

The water depth must be known at each vertical in order to calculate the cross-sectional area of flow for two sections, one on each side of the vertical. Accurately determining the flow areas is just as important as accurate velocity measurements.

The greatest sources of error in measuring the depth of water are: (1) an irregular channel bed; and (2) a channel bed that is soft so that a weight or a wading rod sinks into the soft material thereby indicating a water depth greater than actually exists. Another source of error can occur when reading a wading rod because water piles up on the upstream edge and is much lower on the downstream edge, thereby requiring the hydrographer to sight across the rod, looking both upstream and downstream, in order to obtain an appropriate reading.

2.12. Recording of Data

Hagan (1989) discusses the various formats for the recording of data and computational procedures. A highly preferred format is illustrated in Figure 4. This format is used for calculating the discharge rate in the cross-sectional area of flow between two verticals.

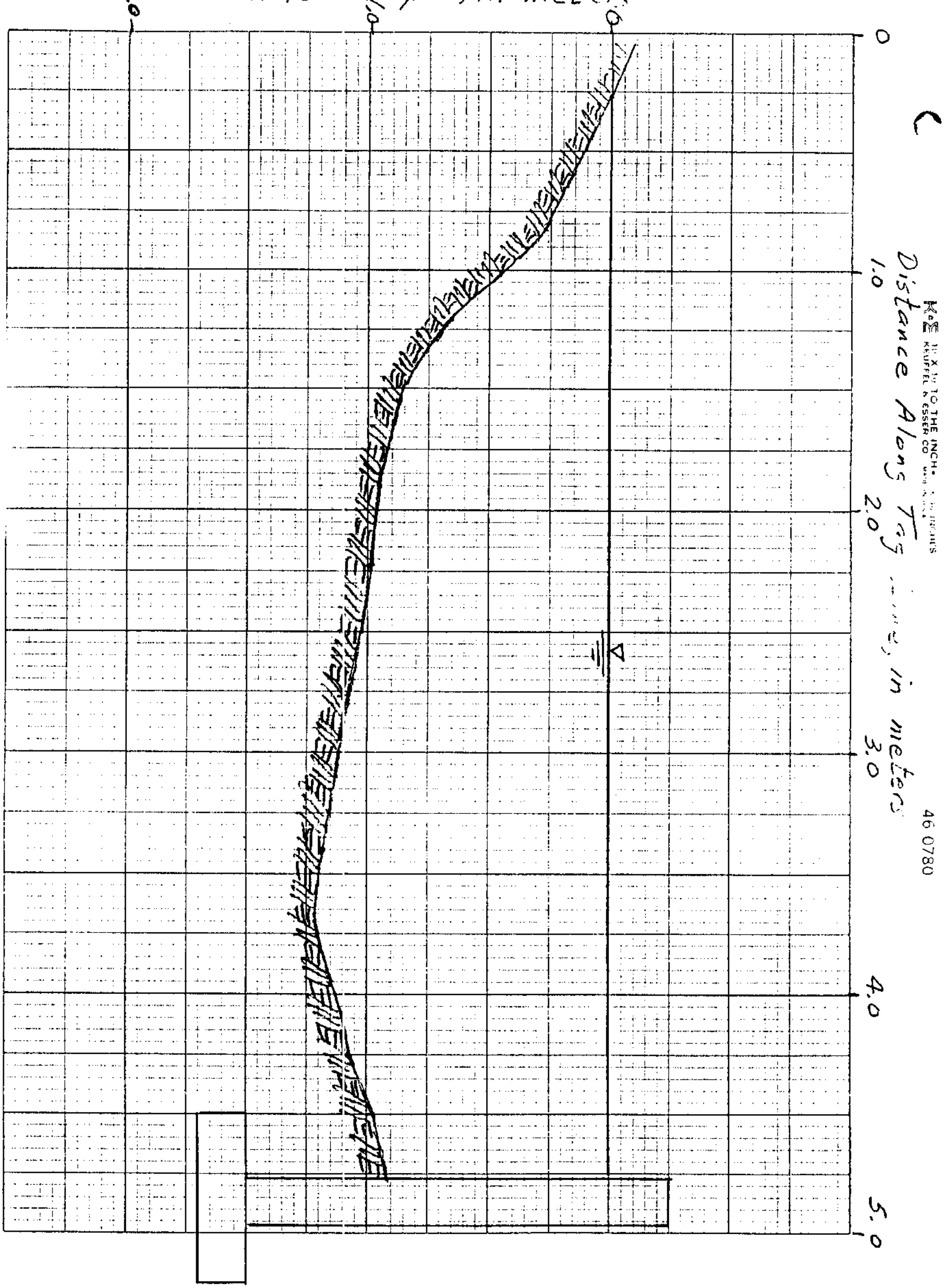


Figure 20. Example Earthen Canal Cross-Section Showing Location of Verticals for Defining the Flow Area.

Date:-________________ Channel:____________________ Station:_____________________

Distance from initial point	Depth	Depth of observation	Revolutions	Time in seconds	Velocity			Area	Mean Depth	Width	Discharge
					At point	Mean in vertical	Mean in section				

Observers:____

No.________of________pages Computations ____________________________ Checked______________________________

Figure 21. Format for Recording Current Meter Measurement Data and Calculating Discharge.

2.13. Unsteady Flow Conditions

The canals in Pakistan normally operate under unsteady flow conditions. At any cross-section, the discharge is continually changing, with fluctuations of 10-20 percent occurring in a single day. Sometiines, the daily fluctuations are even greater. The literature on current meter discharge measurements pertains to steady-state flow conditions. Thus, there is a need to modify the usual procedures in order to accommodate unsteady flow conditions.

Usually, current meter measurements in an irrigation channel are undertaken in order to calibrate flow control structures. The first step in developing a procedure for unsteady flow is to identify the type of flow condition passing through the flow control structure, such as listed in Table 2. If free flow conditions exist, then the upstream flow depth, h, needs to be monitored periodically (say every 15-20 minutes) during the current meter measurement, whereas both h_u and the downstream flow depth, h_d, needs to be monitored for submerged flow conditions. The ratio of the hydraulic head (h, or $h_u - h_d$) at the beginning $_o$ of the test to the hydraulic head at any later time, t, can be used to adjust the current meter velocity measurement at time t to conform with the discharge at the beginning of the test.

The downstream flow depth, h_d, should always be monitored, even under free flow conditions, in order to adjust the flow depth readings obtained during the current meter measurements if taken downstream from the structure being calibrated. If the current meter measurements are taken upstream from the flow control structure, then the monitored values of h would be used to adjust the flow depths measured while conducting the current meter discharge measurement.

2.14. Computational Procedure

The computational procedure for a current meter discharge measurement is illustrated in Table 3. Figure 3 was used in this example, with the major verticals being at readings of 0.82, 1.23, 2.22, 3.70 and **4.50** meters along the tag line. Intermediate verticals were selected as listed in Table 3.

Note in Figure 3 that the water surface is contained between 0.27 and 4.77 meters along the tag line. The first cross-section is contained between 0.27 and 0.82 meter along the tag line. In Table 3, the velocity at the bank is roughly estimated to be 10 percent of the mean velocity in the vertical at 0.82 meter along the tag line; oflen, the velocity at the bank is listed as zero. Because of the shallow water depth at 0.82 meter, the Six-Tenths Depth Method was used in making the current meter measurement, which resulted in a velocity of 0.208 m/s that was obtained from Table 1. Since this first section has a triangular section, the appropriate velocity occurs at the centroid, which in this case is 2(0.02+0.208)/3. The discharge in this cross-section is less than 0.6 percent of the total discharge.

For the last cross-section, which contains a vertical wall, a set of current meter measurements were made at 4.50 meters along the tag line, with the mean velocity in the vertical being 0.553 m/s. The distance, L, from the vertical wall is 0.27 meter (4.77-4.50). The depth of water, D, at Ihe vertical wall is 0.92 meter. Thus, LID is 0.27 *m10.92* m = 0.29. From Figure 2, the relative mean velocity in the vertical is 0.91, whereas the relative mean velocity at the wall is 0.65. The equation used for calculating the mean velocity at the vertical wall is shown at the bottom of Table 3.

The monitoring data for the flow depths upstream and downstream from the flow structure are listed in Table 2. The last two colums contain the calculated head ratios and h_d corrections, respectively. the head ratio has been plotted against clock time in Figure 5.

The current meter measurement was taken 126 meters downstream from the flow control structure. After completing the current meter measurements, the average velocity can be calculated using the actual point velocity measurements in Table 3, which is 0.486 m/s. Thus, the lag time between the flow control structure and the cross-section where the current meter measurements were taken becomes:

$$\textit{Time Lag} = \frac{126 \textit{ meters}}{0.486 m/s (60 sec/min)} = 4.5 \textit{ minutes}$$

To obtain the appropriate head ratio requires that the time lag be subtracted from the recorded clock time, then this adjusted clock time for each current meter measurement is used in Figure 22 to obtain the appropriate head ratio, which is recorded in Table 17. Finally, the actual point velocity measured by the current meter is multiplied by the head ratio to obtain the corrected point velocity in Table 17.

The mean depth is recorded in Table 17 under the third column from the right. This mean depth takes into account the h correction in Table 16. Again, the recorded clock time in Table 17 minus the lag time is used to obtain the appropriate h, correction in Table **16.**

In this particular example, the difference between assuming steady-state versus unsteady flow conditions amounts **to** 2.7 percent of the discharge, which is not *so* significant, but still important. Often, the variation will be much greater.

Table 16. HEAD RATIO AND FLOW DEPTH CORRECTIONS

Date: 17 May 1995 Channel: Sunburn Distry Station: 0+126

Structure: Gate Flow Conditions: Submeraed Orifice,

Crest Elev: 162.132 H_u **BM** Elev: 163.083 h_d **BM** Elev: 162.857

Flow Condition	Head Ratio
Free Orifice Flow	$[(h_u)_o/(h_u)_t]^{0.5}$
Submerged Orifice Flow	$[(h_u - h_d)_o/(h_u - h_d)_t]^{0.5}$
Free Open Channel Flow	$[(h_u)_o/(h_u)_t]^{1.5}$
Submerged Open Channel Flow	$[(h_u - h_d)_o/(h_u - h_d)_t]^{1.5}$

Clock	Tape/Gauge	h_u	h_d Tape/Gauge	h_d	$h_u - h_d$	Head Ratio	h_d Corr-ection
09:15	0.093	0.775	0.321	0.404	0.37	1.000	0.000
09:30	0.088	0.780	0.320	0.405	0.375	0.995	-0.001
09:46	0.097	0.771	0.322	0.403	0.368	1.004	+0.001
10:00	0.103	0.765	0.324	0.401	0.364	1.010	+0.003
10:14	0.110	0.758	0.326	0.399	0.359	1.017	+0.005
10:28	0.114	0.754	0.327	0.398	0.356	1.021	+0.006
10:46	0.118	0.750	0.328	0.397	0.353	1.025	+0.007
11:02	0.119	0.749	0.328	0.397	0.352	1.027	+0.007
11:18	0.121	0.747	0.329	0.396	0351	1.028	+0.008

Observers ______________________________

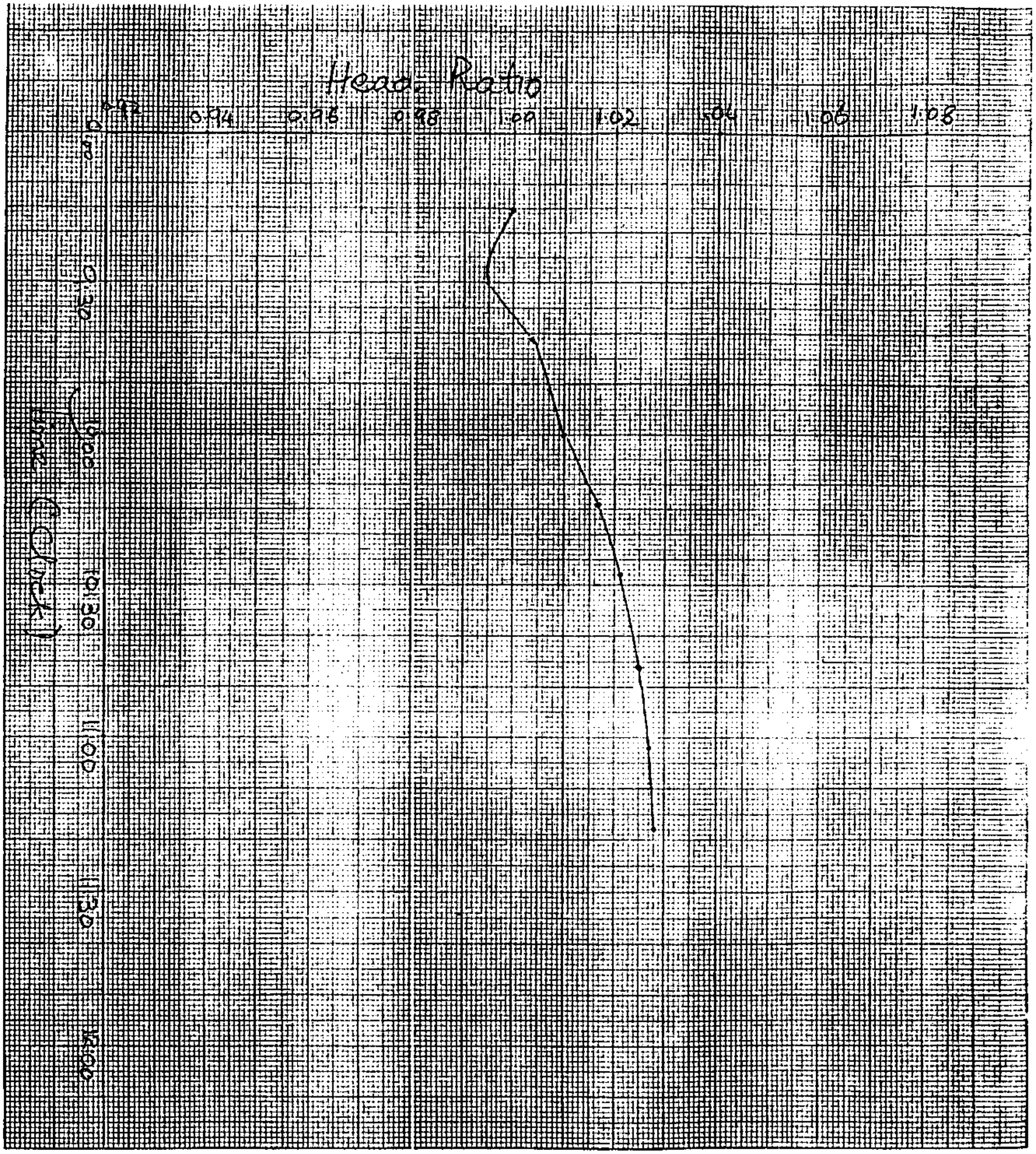


Figure 22. Example of Head Ratio Variation at a Flow Control Structure During a Current Meter Discharge Measurement.

Table 17. Example of Computational Procedure for a Current Meter Discharge Measurement during Unsteady **Flow** Conditions.

Date: 17 May 1995 Channel: Sunburn Distv. Station: 0+126 m Page 1 of 3

Distance from initial point	Depth	Clock Time in minutes	Depth of obser-vation	Revolu-tions	Time in sec-onds	Velocity					Section			
						At point			Mean in vertical	Mean in section	Area	Mean Depth Corr-ected	Width	Disch arge
						Actual	Head Ratio	Corr-ected						
						10%			.02					
										.152	.074	.135	.55	.011
0.82	0.27	9:24	0.6	20	67	.208	.998	.208	.208					
										.224	.197	.48	.41	.044
1.23	0.69	9:29	0.2	20	54	.255	.997	.254						
		9:31	0.8	20	61	.227	.996	.226	.240					
										.257	.238	.745	.32	.061
1.55	0.86	9:36	0.2	25	58	.296	.995	.294						
		9:38	0.8	25	68	.254	.996	.253	.274					
										.301	.317	.905	.35	.095
1.90	0.95	9:43	0.2	25	50	.342	.998	.341						
		9:45	0.8	30	65	.316	1.000	.316	.328					
										.358	.309	.965	.32	.111
2.22	0.98	9:49	0.2	30	49	.416	1.004	.418						
		9:51	0.8	30	57	.356	1.004	.360	.389					

Observers:____________________

Computations ________________ Checked________________

Table 17. (Continued).

Date: 17 May 1995 Channel: Sunburn Disty. Station: 0+126 m Page 2 of 3

Distance from initial point	Depth	Clock Time in minutes	Depth of obser-vation	Revolu-tions	Time in sec-onds	Velocity: At point: Actual	Velocity: At point: Head Ratio	Velocity: At point: Corr-ected	Velocity: Mean in vertical	Velocity: Mean in section	Section: Area	Section: Mean Depth Corr-ected	Section: Width	Section: Disch arge
2.22	0.98								.389					
										.434	.282	1.006	.28	.122
2.25	1.02	9:56	0.2	40	53	.511	1.007	.515						
		9:58	0.8	40	62	.438	1.008	.442	.478					
										.494	.317	1.058	.30	.157
2.80	1.08	10:02	0.2	40	49	.552	1.009	.557						
		10:05	0.8	40	59	.460	1.010	.465	.511					
										.540	.335	1.116	.30	.181
3.10	1.13	10:10	0.2	40	43	.628	1.013	.636						
		10:12	0.8	40	56	.484	1.014	.491	.568					
										.579	.352	1.172	.30	.204
3.40	1.18	10:18	0.2	50	52	.648	1.017	.659						
		10:21	0.8	50	66	.513	1.018	.522	.590					
										.612	.366	1.221	.30	.224
3.70									.633					

Observers:

Computations ______________ **Checked** ______________

Table 17. (Complete)

Date: 17 Mav 1995 Channel: Sunburn Disty. Station: 0+126 m Page 3 of 3

Distance from initial point	Depth	Clock Time in minutes	Depth of obser-vation	Revolu-tions	Time in sec-onds	Velocity					Section			
						At point			Mean in vertical	Mean in section	Area	Mean Depth Corr-ected	Width	Disch arge
						Actual	Head Ratio	Corr-ected						
3.70	1.22	10:27	0.2	50	49	.688	1.019	.701						
		10:29	0.8	50	61	.554	1.020	.565	.633					
										.624	.360	1.200	.30	.225
4.00	1.13	10:34	0.2	50	51	.661	1.021	.675						
		10:36	0.8	50	62	.545	1.022	.557	.616					
										.602	.282	1.130	.25	.167
4.25	1.08	10:40	0.2	50	55	.614	1.023	.628						
		10:43	0.8	50	63	.537	1.024	.550	.589					
										.578	.262	1.049	.25	.151
4.50	0.97	10:49	0.2	40	47	.575	1.025	.589						
		10:52	0.8	40	51	.531	1.025	.544	.566					
										.482	.262	0.970	.27	.126
4.77	0.92	11:03	V = 0.556 (0.65/0.91) = 0.397											
														1.879
													Say	**1.88**

Observers:

Computations **Checked**

3.1. Physical characteristics of structures

The **14** distributary head slructures in the Chishtian sub-division are a mixture of gated orifices (9) and broad-crested weirs (5). There are 5 cross-regulators, which are all gated orifices. In addition to that, 2 broad-crested weirs are present. In Figure 23, a schematic view of the irrigation system of the Chishtian sub-division is given.

All structures in the Chishtian sub-division have been provided with benchmarks (so-called white marks) to measure h_u, h_d (both with reference to the crest) and G_o, see Figure 24. The reference levels of these white marks (WM) with respect to the crest are presented in table 18. The spindle zero indicates the tape measurement reading when the gate is closed.

LAY-OUT OF FORDWAH BRANCH

(Chishtian sub-division)

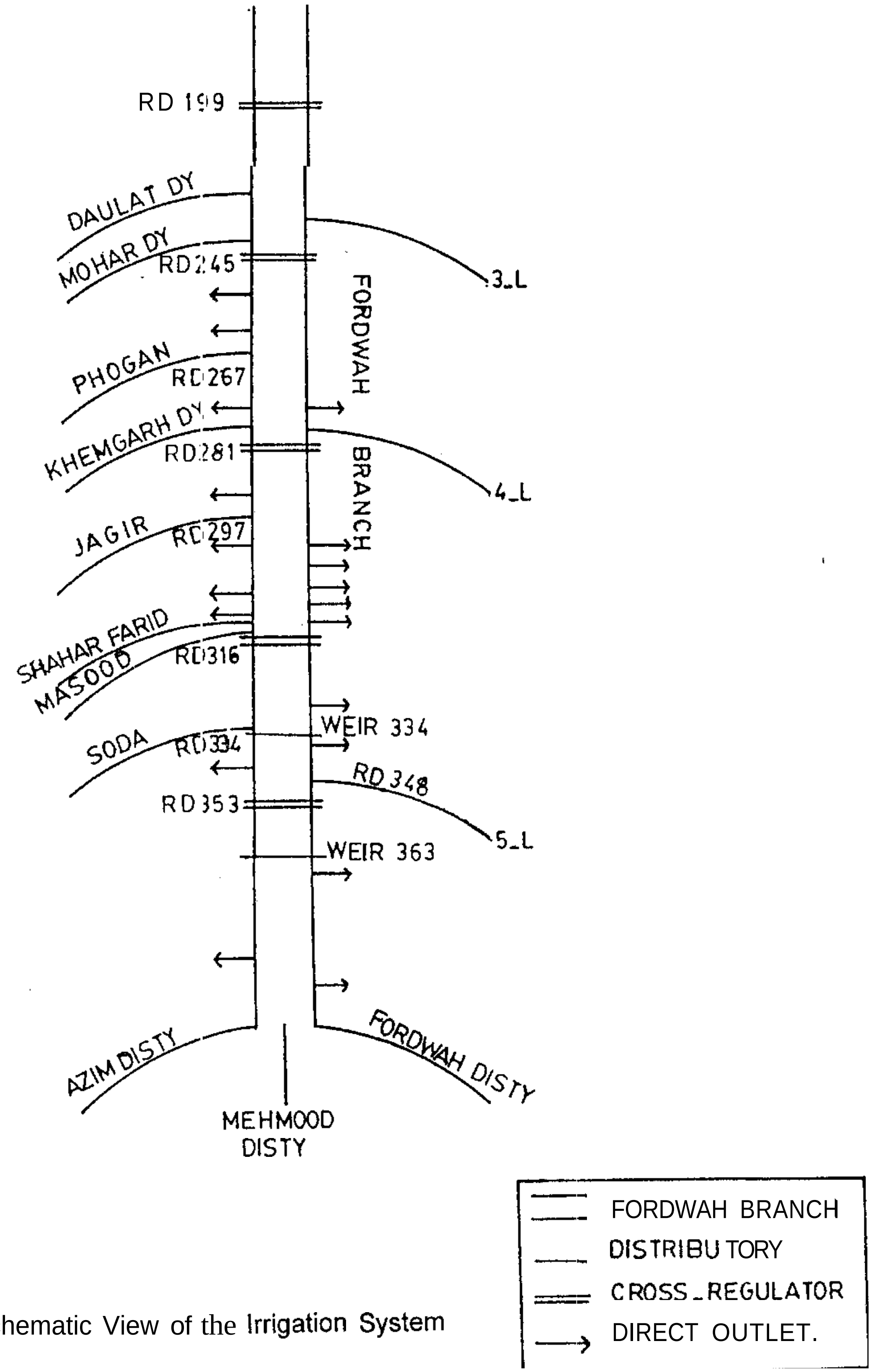


Figure **23** A Schematic View of the Irrigation System

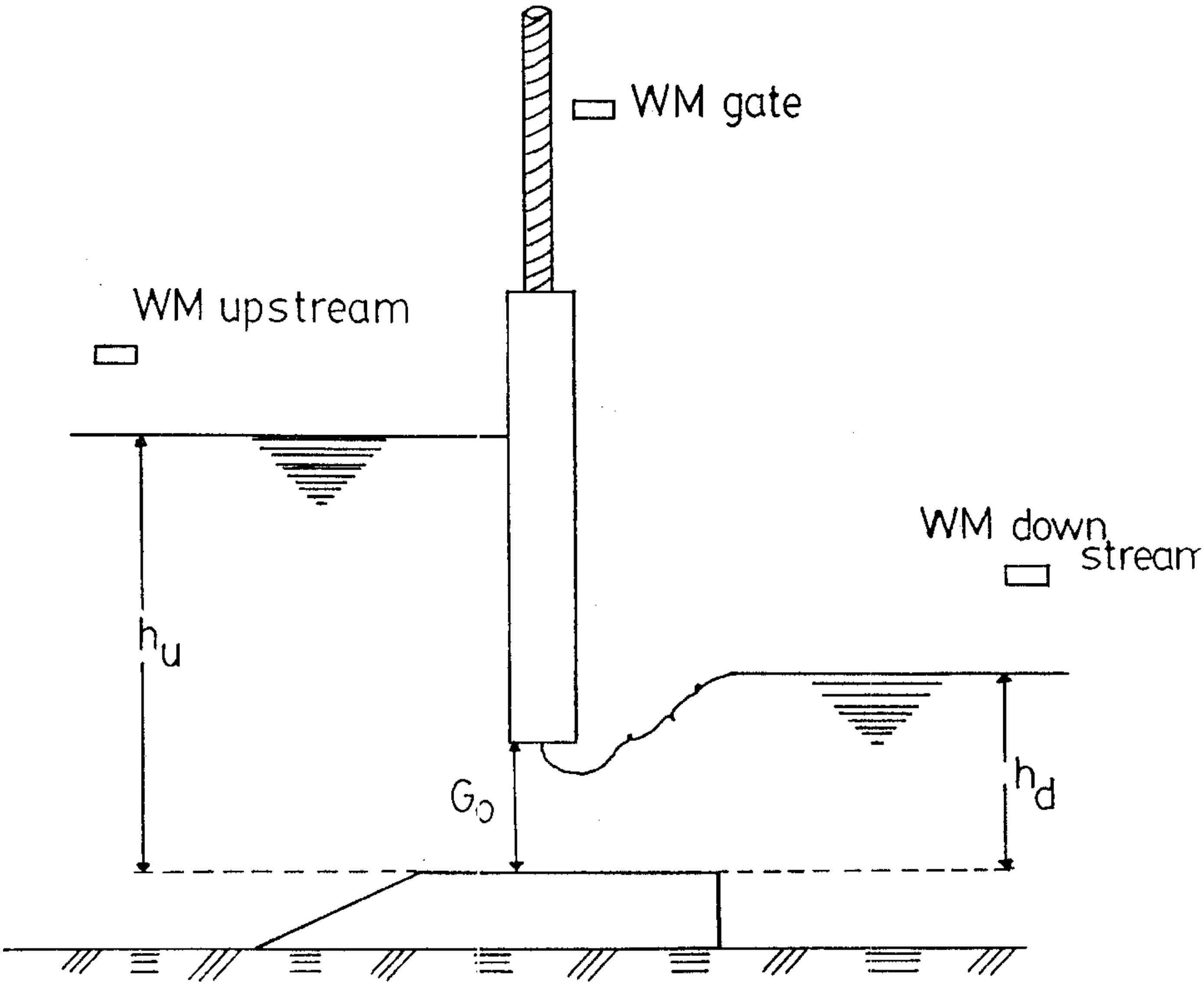


Figure 24 White Marks on Structures in the Chishtian Sub-Division

Table 18. Dimensions of structures and reference levels

Structures	Dimensions (ft)		WM ab. crest (ft)		gate WM	Spindle Zero
	width of opening	gate height	U/S	D/S	ab. crest	
D199	7.51	5.74	-0.16 *	-9.28 *	9.12	3.38
	7.51	5.74	-0.16 *	-9.28 *	9.02	3.28
	7.54	5.74	-0.16 *	-9.28 *	9.35	3.61
	7.51	5.74	-0.16 *	-9.28 *	8.99	3.25
	7.48	5.74	-0.16 *	-9.28 *	9.15	3.41
	7.45	5.74	-0.16 *	-9.28 *	9.09	3.35
Daulat	7.45	5.61	5.18	2.85 *	9.41	3.80
	7.41	5.61	5.15	2.82 *	8.46	2.85
Mohar	3.97	4.49	5.77	5.28	6.99	2.49
3L	3.97	-	4.17	4.13	-	-
D245	7.38	8.43	9.12	7.38	10.04	1.61
	7.48	8.43	9.09	7.31	9.97	1.54
	7.45	8.43	9.15	7.41	10.10	1.67
	7.48	8.43	9.12	7.38	10.04	1.61
Phogan	2.26	-	2.92	1.89	-	-
Khem Gahr	3.87	4.62	3.37	2.61	7.93	3.31
4L	3.12	-	4.52	4.51	-	-
D281	14.96	6.49	5.98	5.31	11.48	4.99
	15.09	6.49	5.98	5.31	11.05	4.56
Jagir	4.92	5.61	6.88	6.28	8.92	3.31
Shahr Farid	8.53	5.97	7.37	-0.34 *	7.38	1.41
Masood	5.94	4.82	7.03	7.03	8.99	4.17
D316	7.45	6.76	8.30	-1.28 *	10.53	3.77
	7.45	6.76	8.36	-1.21 *	10.33	3.58
	7.45	6.76	8.33	-1.25 *	10.63	3.87
Soda	4.69	-	4.47	2.58	-	-
5L	2.46	-	3.84	3.94	-	-
D 353	20.01	6.10	4.56	2.10	9.18	3.08
Fordwah	5.90	4.49	6.59	4.59	9.15	4.66
	5.97	4.49	6.59	4.59	9.12	4.62
Azim	5.94	4.99	6.56	3.18	9.61	4.62
	5.97	4.99	6.56	3.18	9.61	4.62
Mehmud	4.10	2.39	4.99	4.76	5.58	3.54

* gauge

3.2. ISRIP Calibration

The ISRIP team took measurements of the main structures and cross-regulators of the Fordwah system as well as the inlet structure of Eastern Sadiqia during the same time period as the field calibration training. The results are presented in table 19.

Table 19. Results of discharge, measurements of main structures in Fordwah Canal Division

Structure[2]	Type	Design Q_d cusecs	Observed Q_o cusecs	Flow condition	Cd	G_o	h_u feet	h_d feet
Head Fordwali Canal	overflow	3447	2168.9	free flow		-	3.5	1.65
			2427.8			-	3.6	2.38
Head E. Sadiqia Canal	overflow	6100	5683.4	free flow		-	5.3	2.55
			5538.5			-	5.3	2.9
Head Fordwah Branch	weir	2603	1670.3	free flow		-	-	
			1991.7	free flow		-	-	
Fordwah Branch RD 77500	weir	1833	1315.9	free flow		-	-	
			1478.5	free flow		-	-	
Fordwah Branch RD 199812	orifice	1287	760.3	free flow	0.56	1.84	5.14	0.36
	x-reg		879.4	free flow	0.55	2.17	5.18	0.62
Fordwah Branch RD 245650	orifice	900	545.3	submerged	0.70	2.13	7.91	5.54
	x-reg		664.5	submerged	0.72	2.65	8.15	5.98
Fordwah Branch RD 281000	orifice	820	410.5	submerged	0.74	1.77	5.36	3.68
	x-reg		610.3	submerged	0.94	3.46	5.43	4.85
			669.5	submerged	1.13	4.17	5.40	5.07
Fordwah Branch RD 316380	orifice	510	319.3	free flow	0.56	1.36	6.04	3.65
	xreg		406.4	submerged	0.81	2.09	6.41	4 60
Fordwah Branch RD 354000	orifice	410	324.4	free flow	0.58	2.16	3.70	-0.08

Open channel flow - free flow : $Q_f = C_f \times W \times 2g^{0.5} \times h_u^{nf}$

Open channel flow - submerged flow: $Q_s = \dfrac{C_s\ W\ 2g^{0.5}(h_s - h_d)^{n_f}}{(-\log S)^{n_s}}$

Orifice - free flow: formula $Q_f = C_d\ G_o\ W\sqrt{2g(h_u - G_o/2)}$

Orifice - submerged flow: formula $Q_s = C_d\ G_o\ W\sqrt{2g(h_u - h_d)}$

2 Structure dimensions and benchmark levels are not available with us at the time of publishing for the inlet structures of Fordwah and Eastern Sadiqia canals and the weirs at RD 0 and 77 of Fordwah Branch.

3.3. IIMI-ID Calibration

During the training course, discharge measurements were taken for 13 out of 14 distributary head structures. Shahr Farid distributary was closed during this period and could, therefore, not be included. The results of the measurements are presented in table 20. To determine the Cd, the formulas, described on page *67* were applied. In order to establish a full rating curve, more measurements will be needed at different discharge rates. This applies particularly to the orifices, as the Cd changes with the gate opening (see Chapter 2). Some specimen of the discharge measurements executed in the field are given in Annex-4.

Table 20. Results of discharge measurements of distributary head structures.

Channel	Design discharge	Measured discharge	Gate Opening	Type of structure	Flow condition	Cd	h_u	h_d
Daulat	209	155.7	2.09	orifice	free flow	0.35	4.29	2.32
Mohar	36	31.5	1.47	orifice	submerged	0.72	4.84	3.98
		28.3	1.32	orifice	submerged	0.51	4.80	3.85
3_L	18	12.4		weir	submerged	0.039	3.32	3.27
Phogan	17.5	18.4		weir	free flow	0.86	2.17	1.11
Khenigarh	30	26.9	0 96	orifice	submerged	2.16[5]	2.75	1.64
4_L	16	13.8		weir	submerged	0.025	3.825	3.72
Jagir	28	28.7	1.25	orifice	submerged	0.75	5.76	5.17
Shahr Farid	153			orifice	free flow	-		
Masood	35	30.1	1.62	orifice	submerged	0.54	5.33	4.82
Soda	77	63.9		weir[3]	free flow	1.00	1.42	-
		65.3		weir"	free flow	1.08	1.37	-
5_L	4	10.1		culvert	submerged	-		
Fordwah	158	197.5	2.47	orifice	submerged	0.87	4.75	3.84
		199.0	2.40		submerged	0.98	4.62	3.83
Mehmood	8.25	21.1	0.99	orifice	submerged	1.28	2.99	2.74
Azim	244	175.4	1.62	orifice	free flow	0.53	4.51	-

3 Horizontal stop logs (karrees) were present on top of the structure during the measurements. Actual crest was taken to be the top of the stoplogs. However, leakage has occurred through the stoplogs.

4 -do-

5 The gate of Khem Garh has been damaged

There are a total of 19 direct outlets in Fordwah Branch. All but one of them are pipe outlets with diameters ranging from 3 to 12 inches[6]. Discharge measurements were taken for these pipe outlets during two days of the training course. The Cd coefficient was determined using the formula:

$$Q = C_d \times A \times \sqrt{(2g \times (h_u - h_d)}$$

The results of the discharge measurements as well as the discharge rating are presented in table 21.

Table 21. Results of discharge measurements for direct outlets in the Chishtian Sub-division.

Direct outlets	Measured discharge cusecs	Type	Flow condition	Size	Cd
RD 260115-R	0.73	pipe	submerged	dia: 5"	0.53
RD 263186-R	2.53	pipe	submerged	dia: 6"	
RD 272600-R	2.25	pipe	submerged	dia: 6.7"	0.68
RD 273200-L	2.08	pipe	submerged	dia: 9"	0.57
RD 296500-R	5.68	pipe	free outlet	dia: 8"	1.42
RD 303000-L	1.23	pipe	submerged	dia: 4.7"	0.68
RD 305500-L	1.87	pipe	submerged	dia: 5"	0.94
RD 308855-L	2.06	pipe	submerged	dia: 5.5"?	1.36
RD 311620-R	3.38	pipe	submerged	dia: 8.5	0.64
RD 313384-L	1.31	pipe	submerged	dia: 7.5	0.57
RD 314050-R	2.60	pipe	submerged	dia: 5.5"	1.22
RD 316250-L	3.33	pipe	submerged	dia: 12"	0.74
RD 316350-L	closed	pipe		dia: 9"	
RD 333500-L	2.10	pipe	submerged	dia: 7.5"	0.75
RD 342275-L	2.29	pipe	submerged	dia: 6.5	2.09
RD 352700-R	3.14	APM		b=.28 y=.8?	1.07
RD 363500-L	0.53	pipe	free outlet	dia: 3"?	0.16
RD 368000-R	8.83	pipe	submerged	dia: 12"	0.66
RD 370742-L	1.58	pipe	submerged	dia: 5.8	0.38
Total	47.52				

[6] The diameters of DO 308855-L and DO 363500-L and the dimensions of DO 352700-R have been estimated. All others were established during the canal closure in January 1995.

3.4. Rating structures with $KD^{5/3}$ formula

Traditionally structures are provided with rating tables, based on a Q-h, relationship. The formula used is $Q = K * D^{5/3}$, in which K is a constant and D is the water depth (water level with reference to bed level) in feet. In fact this is a simplified version of Mannings' equation. The downstream gauge (whose 0 level is at the design bed level) is read by the gauge reader and the corresponding discharge can be obtained from the rating table. Since irrigation canals in Pakistan carry a substantial sediment load and silting/scouring are usual phenomena, the actual bed level of a canal can differ substantially from the original bed level. The relation between the downstream gauge reading and the discharge has to be corrected for this difference. One way of doing this is by taking soundings to establish the difference between actual and design bed level This value then has to be added/deducted from the reading obtained from the downstream gauge. Regular soundings will be required to guarantee accuracy of conversion of water levels into discharges. Periodically, the discharge ratings of structures need to be checked by taking discharge observations. It is advocated here that the results of the structure calibrations, which are less subject to change than the calibration based on the downstream gauge, are used for this periodic recalibration. The main advantage of the Q-h, relationship, i.e. its ease in use for the gauge reader and other irrigation staff, can be kept, while being able to update the formula in a straightforward manner.

The results obtained by the ISRIP team on the main structure were used to calculate the constant K for the $KD^{5/3}$ formulas of the structures, see table 22. D was established by taking the average water depth at the location were the discharge measurements were carried out, which was always within 1000 feet of the downstream gauge.

Canal	Location	Observed Q	D/s gauge	D feet	Correction	K
Fordwah Canal	Head	2168.9	7.80	7.0	- 0.80	84.7
		2427.8	8.13	7.5	- 0.63	84.5
E. Sadiqia Canal	Head	5683.4	8.75	17.0	+ 8.25	50.6
Fordwah Branch	Head	5538.5	8.70	17.0	+ 8.30	49.3
		1670.3	NA	5.7		91.8
Fordwah Branch	D77	1991.7	NA	6.4		90.3
		1315.9	6.70	5.8	- 0.90	70.3
Fordwah Branch	D199	1478.5	7.00	6.4	- 0.60	67.0
		760.3	9.64	3.6	- 6.04	89.9
Fordwah Branch	D245	879.4	9.90	3.9	- 6.00	91.0
		545.3	6.50	4.4	- 2.10	46.2
Fordwah Branch	D281	664.5	6.97	4.8	- 2.17	48.7
		410.5	5.50	3.9	- 1.60	42.5
		610.3	6.70	5.2	- 1.50	39.1
Fordwah Branch	D316[7]	669.5	6.92	5.3	- 1.62	41.6
		319.3	4.90	3.6	- 1.30	37.8
Fordwah Branch	D354	406.4	5.85	5.3	- 0.55	25.2
		324.4	5.70	3.5	- 2.20	40.2

The results that were obtained from the discharge measurements on the distributary head structures were also used to calculate the coefficient K in the formula $Q = K*D^{5/3}$. The water depth D was obtained from the cross-section that was taken while undertaking discharge measurements. The results are given in table 23.

[7] At D316 the cross-section is sandy and highly irregular, possibly explaining the significant difference in the correction factor between the two readings that were taken at this location.

Table 23. Establishing the coefficient K for head distributary structures in Chishtian sub-division

Structure	Observed Q	D/s gauge	D feet	Correction	K
Daulat	155.7	2.94	2.95	+ 0.01	25.7
Mohar	31.5	2.30	1.50	- 0.80	16.0
	28.3	2.17	1.40	- 0.77	16.2
3_L	12.4	1.70	1.45	- 0.25	6.7
Phogan	18.4	1.01	1.00	- 0.01	18.4
Khemgarh	26.9	missing	1.65		11.7
4_L	13.8	2.35	1.25	- 1.10	9.5
Jagir	28.7	bended	1.50		14.6
Shahr Farid	-	-			
Masood	30.1	3.69	1.50	- 1.19	15.3
Soda	63.9	2.67	2.00	- 0.67	20.1
	65.3	2.86	2.10	- 0.76	19.0
5_L	10.1	3.62	1.00	- 2.62	10.1
Fordwah	197.5	4.30	3.25	-1.05	27.7
	199.0	4.29	3.20	- 1.09	28.6
Mehmood	21.1	2.51	2.50	-0.01	4.6
Azim	175.4	2.81	2.50	-0.31	38.1

Fordwah Branch Canal (Chishtian Subdivision RD 199 to RD 371:6 June 1996)

The Inflow-Outflow Method is the preferred technique fur determining the seepage from an irrigation channel. The reason is because seepage is being evaluated under normal operating conditicins for the irrigation channel. However, the seepage rate must be larger than the error in the discharge ratings in order to have sufficient accuracy; otherwise, the Ponding Method should be used, which also has some disadvantages.

The Inflow-Outflow Method is basically a water balance methodology. An irrigation channel is usually subdivided into reaches with a water balance conducted for each reach. The nodal points for each reach are usually a flow control structure (e.g. cross regulator) that can be calibrated in the field for discharge measurement. A discharge rating is developed for each inflow and outflow structure prior to conducting the inflow-outflow test.

An alternative is to conduct discharge measurements at each inflow and outflow point while conducting the inflow-outflow test. The principal disadvantage of this technique *is* that more time is required while conducting the test. Since the irrigation channels in Pakistan experience significant discharge fluctuations throughout the day, this must be taken into account in conducting any inflow-outflow test.

For this particular inflow-outflow test, discharge ratings had been developed for all of the inflow and outflow structures. Thus, only water levels had to be measured, with the discharge rating being used to calculate the discharge rate.

The lower end of the Forclwah Branch Canal from RD 199 to the tail at **RD** 371 was subdivided into five reaches:

(1) RD 199 to RD **245;**
(2) RD 245 to RD 281;
(3) RD 281 to RD 316;
(4) **RD** 316 to RD 353; and
(5) RD 353 to RD 371.

Each nodal point has a Cross Regulator (CR) except the tail of Fordwah Branch Canal, which consists of three Distributary Head Regulators.

The layout of the Fordwah Branch Canal is shown in the accompanying figure. The team assignments are listed in the accompanying table, which shows the time frame for observing water levels at each inflow and outflow structure. The listed times take into account the time lag from the inflow structure to the downstream ouflow structures. The first two reaches (RD 199-RD 245 and RD 245-RD 281) are synchronized taking into account the time lags from the RD 199 CR to all of the downstream structures until reaching the RD 281 CR. Afterwards, because of time constraints, each reach was treated as an independent reach. Details of the team assignments are presented in Annex-3

CHAPTER 5: *Results: Inflow-Outflow Test*

Canals in the Punjab are subject to frequent fluctuations in water levels over relatively short periods of time, impacting on the discharges supplied to off-takes (Kuper et al., 1994. Bhutta and Vander Velde, 1990). When undertaking an inflow-oufflow test, a steady state flow period (SFP) is highly desirable in order to minimize problems of storage and drainage in canal reaches (and thus obtaining results that do not represent true seepage losses). During this test, the canal was in unsteady state with water levels rising during the day. The results of the test are likely to be affected by this. In particular the weirs (3_L, 4_L, 5_L, Phogan, Soda) are sensitive to water level fluctuations, influencing the off-taking discharge.

Discharge measurements were undertaken in the field prior to the inflow-oufflow test, to determine the discharge coefficient C_d for the different hydraulic structures. The measurement error will be within 5 % of the discharge. Also, measurement errors in reading the water levels during the inflow-outflow test can occur. An overall error limit of 5 % of the inflow is assumed for a test that is undertaken during an SFP. For tests that are undertaken during unsteady state, results are likely to contain a larger error.

date	6 June, 1995
canal	Fordwah Branch RD 199-371
length	172,000 feet (52.4 km)
design discharge	1286 cusecs (inflow point)
average actual discharge	1000 cusecs (kharif)

Table **24.** Results inflow-outflow **test for Fordwah Branch RD 199-371**

Structure[8]	h_u[9] feet	h_d feet	G_o feet	Inflow cusecs	Outflow cusecs
RD 199	5.14	0.82	2.22	885.2	
Daulat	4.28				closed[10]
Mohar	4.885				30.1
3_L	3.285	3.200	-		13.9
DO 260115-R					closed
DO 263186-R					2.7
Phogan	2.40	0.96	-		25.9
DO 272600-R					2.5
DO 273200-L					2.4
Khemgarh	2.745	1.745	0.55		36.7
4_L	3.895	3.805	-		10.3
DO 296500-R			-		6.0
Jagir	6.27	5.28	1.29		38.0
DO 303000-L			-		1.2
DO 305500-L			-		1.9
DO 308855-L			-		2.1
DO 311620-R			-		3.2
DO 313384-L			-		closed
DO 314050-R			-		2.6
DO 316250-L			-		3.1
Shahr Farid	5.37		1.41		141.8
Masood	5.27	5.00	1.72		23.0
Soda	1.42[11]		-		63.7
DO 333500-L			-		2.4
DO 342275-L			-		2.3
5_L	3.155	2.805	-		11.5
DO 352700-R			-		3.2
DO 363500-L					closed
DO 368000-R					6.9
DO 370742-L					1.7
Fordwah	4.72	2.38	0.67		84.9
Azim	4.69	1.26	2.12		204.9
Mehmood	3.12	2.69	0.99		27.4
Total				885.2	766.3

8 DO means direct outlet

9 h_u is the upstream water depth over the crest, h_d is the downstream water depth over the crest and G_o is the gate opening

10 There was some leakage and overflow. A value of 10 cusecs is assumed.

11 This represents the water depth over the karrees

The total inflow is 885.2 cusecs. Total outflow to distributaries and direct outlets amounts to 766.3 cusecs.

The wetted area is calculated using the simulation results of a hydraulic model SIC[12]. By running the steady state flow module, SIC computes the volumes of water in each reach as well as the water surface level (which will give us the water depth). We thus have the volume of water V, the length of each reach L, and the average depth of water D in a reach. As canals in the area are generally very wide and flat, we can assume a trapezoidal section in which the bed width B >> a (see sketch). The side slope is assumed to be 45°.

If $D = \frac{a}{\sqrt{2}}$ and the wetted perimeter $P = B + 2*a$

then $P = B + 2\sqrt{2}*D = B + 2.83*D$

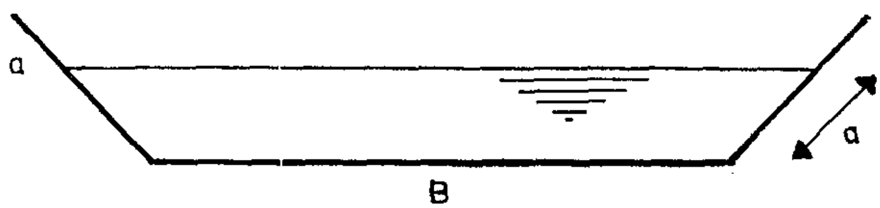
The area $S = B * D + D^2$

By replacing B by $P - 2.83*D$ we find:

$S = P * D - (2.83 - 1) * D^2 = P * D - 1.83*D^2$

Since we know V (= S * L), we can then compute P * L for each reach

The total wetted area is 12.4 million square feet (msf). Total seepage is, therefore, 118.9 cusecs (= 9.6 cusecs per msf) or 13.4 %.



[12] SIC is a mathematical model that can simulate the flow in a canal. SIC was calibrated for Fordwah Branch during the first few days of June, enabling us to use the results for calculation of the wetted area.

Recommendations field calibration training course

Rating tables

1. Traditionally, discharges are calculated by gauge readers using the formula $Q = KD^{5/3}$. The results of the training course confirm the utility of this formula. This formula needs to be regularly updated due to siltation/scouring in the alluvial channels of Pakistan. This can be done by cross-referencing the KD formula monthly with the more permanent structure formula that has been used in this training programme.

2. Replace rating tables for all structures in the Chishtian sub-division and cross-regulators in the Fordwah division using the K values obtained during the training course.

3. Carry out additional measurements to determine C_d for the permanent ratings. This can be a joint exercise of IIMI and concerned ID staff, who are now fully trained.

Gauges

1. Downstream gauges of Shahar Farid, Khemgarh, Phogan, Jagir need replacement.

2. Several gauges need painting and maintenance (e.g. d/s Daulat).

3. Tail gauges need to be replaced.

Inflow-outflow tests

1. In future, when inflow-outflow tests are done, a steady flow period should be ensured for more accurate results.

Eauipment

1. Current meters should be made available to the officers and sub-engineers of the I & P department.

IMIS

1. The results of the calibrations should be used to implement IMIS in the Chishtian sub-division. New K coefficients can now be used to calculate the discharges and analyze the water distribution in Chishtian.

2. **A** similar exercise, i.e. rating of structures and implementation of IMIS should be carried out in a perennial canal system.

Acknowleduements

We would like to thank Ch. Muhammad Shafi, Chief Engineer Bahawalpur Zone for the encouragement given to organize this training course. Thanks are also due to the Punjab Irrigation and Power Department, the Government Engineering Academy Punjab, the International Waterlogging & Salinity Research Institute (IWASRI), the International Sedimentation Research Institute-Pakistan (ISRIP), Watercourse Monitoring and Evaluation Directorate (WMED) for making available their staff to attend the training course. The efforts of ISRIP in co-organizing this training course are gratefully achnowledged.

Mr. Abdul Majeed Bhatti, Executive Engineer, Lining and Mr. Javed Qureshi, Executive Engineer, Fordwah Division have provided valuable comments on a first draft of the report.

Finally, a word of thanks to the participants who have cheerfully worked under quite extreme climatic conditions with temperature running up to 47° C.

1.1. Report No. 1

Field Calibration of Irrigation Flow Control Structures

Muhammad Aslam
Abdul Samad Ashgar
Muhammad Ali Pasha
Muhammad Ramzan
Rana Akhtar Raza
S. Qasim Ali Shah

Introduction

Pakistan is an agricultural country having the largest canal irrigation system on the world globe. To irrigate our **sweat** and fertile land, the Provincial Irrigation Department are responsible for the operation and maintenance of canals. In the land **of** five rivers Punjab, there is a huge canal network system irrigating 20 million acres per year. But, unfortunately due to economical or other reasons, this system is going to be deteriorated day by day.

Training in any field plays a vital role in improving the existing system and opens the door for new thoughts Keeping in view the present situation of canals the International Irrigation Management Institute, Pakistan conducted an excellent training program of 10 days for Irrigation Engineers and relevant field engineers, officers and officials at Bhawalnagar from May 28 to June 6, 1995. The participating engineers and officers were from Government Engineering Academy Punjab, Lahore. I&P Department, Punjab and WAPDA.

Conduction

The course was conducted in highly appreciable manner including lectures, field observations and calibration of existing canal gauges. Group discussions in the field and exchanging the experience!: and views were very informative and will be helpful in the professional life of every participant.

Observations

It has been observed during the discharge observations, the canal gauges/gates structures etc. are not well maintained. Some distributries were taking (drawing) more discharge (water) than the recorded one. It has been felt that I&P Department has no adequate calibrating gauges for the proper operation of the system.

Suggestions and Recommendations

In order to improve the deficiencies the following suggestions are presented:

i. Calibrating instruments must be provided by the I&P department to the staff.

ii. Staff training with modern instruments and techniques, specially the Sub-Engineers must get training of such type.

With the coordination of various departments there may be some advance training programs for the Senior Engineers also, which will enhance the knowledge and skills of the trainees.

At the end of this report, we are grateful to Prof. Skogerboe, Director IIMI Pakistan, Mr. Marcel Kuper (IIMI), Mr. Mushtaq Khan (IIMI) and all the staff members of IIMI Pakistan, Mr. Barkat Ali (Engineer Wapda) and the staff for the cooperation they extended during the discharge measurements. We congratulate the organizers on conducting a very successful course.

1.2. Report No.2

Field Calibration Training Report

Khadim Hussein
Shahid Iftikhar
Salamat Ali
Ghulam Rasool Shauq
Barkat Ali

The IIMI arranged a Field Calibration Training Program at Bhawalnagar and invited various departments especially the Irrigation Department to participate in this training course.

The participants came at Bhawalnagar from various cities of Punjab and attended the course on May 28, 1995. The training was started in the conference room of the Lining Circle Office at Chishtian road. Professor Skogerboe gave the two manuals to each participant and delivered the lecture to corraborate and understand the topics of flow conditions, discharge sites etc. Similarly on May **29** the lectures was given and the problems were solved in the office. But on third day i.e 30-5-95 all the participants were brought to the Cross Regulator at RD **245** FDW branch.

At that site the full lecture was delivered on the current meters, its types, shape and working etc. All the participants took a great interest in understanding and working with the current meter and then calculated the discharge of Mohar Distry with this apparatus in the group of two.

Similarly from 31-5-95 onward the participants measured the discharges *of* various channels in the form of groups with the current meters with a keen learning attitude because most of the participants had no practical knowledge to measure with the current meter.

Also the Head correction was taken into account due to fluctuation of water levels upstream and downstream of the canals for best accuracy. IIMI also arranged to visit Sulmanki Head Works at river Sutluj from where three main canals Sadiqia. Fordwah on the left side and one Pakpattan canal offtakes from the right side of the river. Participants also learned a lot from this tour as the barrage was constructed during 1924. Mostly the material used is of steel instead of concrete and RCC. Fish ladders were a new thing for some of the participants.

The Prof. Skogerboe invented himself and introduced a throatless meter flume by which the discharge of our outlet can be observed properly and the flume can be transported easily

In short this short training course taught much to the participants. No doubt that the participants took a great interest and worked wholeheartedly. But this training will be of no use if the Irrigation Department does not provide the current meters and other accessories to apply this in the field.

Also it is suggested that this type of short training courses may be carried out at other places of Punjab and Sindh, minimum at Zonal levels, so that most of the other untrained person may get advantage of it for the improvement of the irrigation system as well as for the persons.

1.3. Report **No.** 3

Field Calibiration **of** Hydraulic Structures

Akhtar Riaz Ahmad
Maqbool Ahmad
Shabbir Haider
Qamar Rasool

God says in the Holy Quran:

"Indeed we lead the water to the barren land and therewith bring forth crops where **of** they and their cattle **eat**"

So we can say that the art of applying water to the land dates back to the beginning of human civilization. Pakistan's perennial Irrigation system is the world's third largest irrigation source. About 70% to 80% of population depends upon agriculture directly linked with water. With the world "population explosion" and the demand for additional food, the science of irrigation **is** likely to become the science of survival.

Almost the whole of Pakistan is situated in Arid and Semi Arid zone. ***So*** irrigation science is a necessity for National survival. Irrigation Engineering in Pakistan includes many works such as River Training, dams, head works, barrages, weirs, falls, aqueducts, super passages, canals, distributries, head regulators and outlets etc. etc. Efficiency of all these works has direct impact with distribution and application of water required sufficient knowledge. So keeping in view the importance IIMI arranged the said training. One can say that this ***is*** the 1st rain drop, being dropped by IIMI.

IIMI invited the participants from various engineering departments, for participation in the said course of "Field Calibration of Hydraulic Structures" with new techniques and methods. These were two basic aspects of the training.

1 Calibration of various hydraulic structures for fair distribution of supply with modern ways.

2. to calculate seepage with inflow and out flow method, training was started with detailed and comprehensive lectures of Mr. Skogerboe on the related subject and then later on each and every participants has been provided a chance to work in the field under the guidance of Mr. Skogerboe, Mr. Marcel Kuper, Mr. Mushtaq Khan and IIMI staff.

Fordwah system which was selected for study, is a mix of perennial and non perennial canals. It off-takes from Sulemanki head works with authorized discharge of 3447 cusecs and has its tail at RD 371+000. It irrigates vast land of strip in Bhawalnagar and Bhawalpur districts. It was very astonishing for us as explained by SDO Bhawalnagar that canal at the present stage is taking almost 500 cusces less discharge from head due to siltation problem in head reach. Being professional Irrigation Engineers, we think that if the information delivered to us was correct, thorough study to this aspect must be carried out by I&P Department.

Basic purpose of the training was as explained to carry out study of hydraulic structure with modern and new techniques. During visit to various structures it has been noticed that the gauges fixed by I & P department D/S head regulators were not working properly. Practice was being noticed that discharge table under use were not up to date. They were based on equation Q=K*D *513,* in this system, zero of gauge is fixed with reference to design bedlexisting bed of canals, which required cross check for its accuracy especially for canals having silting problems, otherwise with passage of time gauge reader, might be facing problems to report correct discharge. He will always report the discharge with respect to his own gauge without giving any care to silt deposition to gauge structure. That's why study was introduced by Mr. Skogerboe that whether using the gauge of channel from bed, structure its itself be calibrated with respect to crest level and it will !be more reliable to calculate discharge chance of error will be minimum. For example during study our group visited Daulat Disty off-taking from RD 245 OF FDW Branch with authorized discharge of 209 cs. on arriving at that structure, we asked the gauge reader about discharge. He told that as per D/S gauge, the present discharge in channel is 190cs. But when we measured the discharge with precise AA Current Meter, it comes out 158 cs. There was a difference of 32 cs. The same was the case at tail FDW Discharge reported by I&P gauge reader was 158 cs. where as we measured 199cs. The same situation being observed at the other structures also with new methods and techniques all the participants has been trained to measure discharge and other related matters. **A** detailed lecture on types & use of current meter was delivered by Mr. Barkat who is from ISRIP (International Sedimentation Research Institute, Pakistan) Two booklets was also delivered to each trainee by Mr. Skogerboe for reference. We also worked with pygmy current meter and cut throat flume teams for the discharge of outlets. It was also told that cut throat was designed by Mr. Skogerboe in 1960 era and its working was found very much effective for taking low discharge. Here, we would like to congratulate Mr. Skogerboe for his excellent creation in this field.

As told to us that IIMI is working since 1993 in FDW Division but I think that I&P Incharge Engineer, especially SDO's could not be in a position to spare sometime, to work with IIMI staff for the set righting of daily matters of regulation, that's why in the field control over regulation was found disturbed. Now with this training, the three young engineer's SDO MBD, SDO BWN and SDO CTN with their 6 trained SBE's will left no way out to set right their system. Here, we would also like to request my seniors that at least one perennial system should also be studied on the same lines especially system facing tail storage.

One thing more, which we would like to highlight in this report was that the manner, methods, techniques, attitude with which this training conducted was remarkable. Although we left So many things still left untouched. However, we learned much and get polished ourselves in the related subject. I suggest, that in future, this type of training must be arranged in I & P at least on zonal level but with ***4-5*** week duration.

In the end, we feel morally bound to express deep gratitude to worthy S.E. Bhawalnagar who suggested our names for said training and to Mr. Skogerboe, **Mr.** Marcel Kuper, Mr. Mushtaq Khan with their experienced staff who trained us with patience, personal attention and polishing us in the related field with remarkable behavior and attitude with full facilities and best of their efforts.

May almighty God bless all of us.

Thank you very much.

Annex 1. List of participants

Irriqation & Power Department

Maqbool Ahmed - Sub-Eingineer
Salamat Ali - S.D.O. Minchinabad
Abdul Samad Ashgar - S.D.O. Bahawalnagar
Qamar Rasool Babar - Sub-Engineer Chishtian
M. Shabbir Haider - S.D.O. Malik Branch
Khadim Hussein - S.D.O. Indus River (right bank)
Shahid Iftikhar - Sub-Engineer
M. **Ali** Pasha - Sub-Engineer Takht Mahal
M. Ramzan - Sub-Engineer Bahawalnagar
M. Rashid - S.D.O. Chishtian
Abdul Razaq - Sub-Engineer
Ghulam Rasool Shauq - S.D.O. (rtd)

Government Enqineerina Academy, Puniab

Syed Qasim Ali Shah - Assistant Professor

International Sedimentation Research Institute - Pakistan

Barkat Ali - S.D.O.
Khalid Chatha - Hydrographer
Ghulam Qadir - Field Assistant

International Waterlogging and Salinity Research Institute. WAPDA

M. Aslam - Junior Engineer

Watercourse Monitorina and Evaluation Directorate, WAPDA

Rana Akhtar Raza - Research Officer
Akhtar Riaz - Junior Engineer

International Irriqation Manaqement Institute - Pakistan

Mushtaq Ahmed Khan - Hydraulic Engineer
Marcel Kuper - Associate Expert
G.V. Skogerboe - Director

Annex **2.** Programme **of** the Training **Course**

Field Calibration Training

Bahawalnagar, 28 May to 6 June, 1995

timings: 8.00 am to 1.00 pm
2.00 pm to 6.00 pm

Schedule

May **28**

Venue: Canal Lining Office (Chishtian Road, Bahawalnagar)

AM, PM
Measuring Water Levels
Free Flow in Open Channel Constrictions
Submerged Flow in Open (Channel Constrictions
Classroom problem

May **29**

Venue: Canal Lining Office, RD 199

AM, PM
Rating Orifices
Free Flow Rectangular Gate Structures
Submerged Flow Rectangular Gate Structures
Classroom Problems

May 30

Venue: Canal Lining Office / Mohar distributary

AM, PM
Discharge Measurement with a Current Meter
Field Demonstration of Current Meter Measurements
Field Practice with Current Meter Measurements

May 31

Venue: Canal Lining Office / Daulat

AM
Field Practice with Current Meter Measurements

PM
Comparison of Current Meter Discharge Measurements

June **1**

Venue: Canal Lining Office / **RD** 316, Fordwah, 4_L, Khemgarh

AM
Field Calibrations of Structures with Current Meter Measurements

PM
Computations for Calibration of structures

June **2**

Venue: Suleimanki Headworks / Minchinabad canal resthouse

AM, PM

visit ISRIP measurements
visit to Suleirnanki structures

June **3**

Venue: Canal Lining Office / Masood, Azim, Phogan. 3_L, Mohar

AM
Field Calibrations of Structures with Current Meter Measurements

PM
Computations for Calibration of structures

June **4**

Venue: Canal Lining Office / Soda, Fordwah, Mehmood, Jagir, direct outlets

AM
Field Calibrations of Structures with Current Meter Measurements

PM
Computations for Calibration of structures

June **5**

Venue. Canal Lining Office / direct outlets

AM
Field Calibrations of Structures with Current Meter Measurements

PM
Computations for Calibration of structures
Inflow-Outflow Method for Seepage Measurements

June **6**

Venue: Canal Lining Office / RD 199-245, other reaches

AM
Conduct Inflow-Outflow Measurements

PM
Make a Water Balance for Chishtian sub-division
Closing Ceremony - distribution of certificates to participants

Annex-3

Team 1 : Two Reaches

First Reach	RD 199 to RD 245
Second Reach	ARD 245 to RD 281
RD 199	2:00 - 04:00 am
RD 245 CR 3-L Disty Mohar Disty Daulat Disty	8:30 - 10:30 am
Outlet RD 260 R	10:35 am - 12:35 pm
Outlet RD 263 R	11:00 am - 01:00 pm
Phogan Disty RD 267	11:35 am - 01:35 pm
Outlet RD 273.2 L Outlet RD 272.6 R	12:25 pm - 01:30 pm
RD 281 CR 4L Disty RD 281 Khemgarh Disty RD 281	02:30 pm - 04:30 pm

Team 2: Reach RD 281 to RD 316

RD 281 Cross Regulator	08:45 am - 10:45 am
Outlet RD 296.5 R Jagir Disty. RD 297	11:00 am - 01:00 pm
Outlet RD 303 L	11:50 am - 01:30 pm
Outlet RD 305.5 L	12:10 pm - 01:30 pm
Outlet RD 308.8 L	12:30 pm - 01:30 pm
Outlet RD 311.6 R Outlet RD 3134.4 L Outlet RD 314 R	1:10 pm - 01:30 pm
RD 316 Cross Regulator Masood Disty. RD 316 Shahid Farid Disty Outlet RD 316250 L	1:40 pm - 3:40 pm

Team 3: Reach RD **316 to** RD **353**

RD 316 Cross Regulator Masood Disty. RD 316 Shahid Farid Disty RD 316 Outlet RD 316250 L	09:00 am - 11:00 am
Outlet RD 333.5 L Soda Disty. RD 334	11:30 am - 01:30 pm
Outlet RD 342 L	12:35 pm - 01:30 pm
5-L Disty RD 348	01:25 pm - 03:35 pm
Outlet RD 352.7 R	02:05 pm - 04:05 pm
RD 353 Cross Regulator	02:10 pm - 04:10 pm

Team 4: Rech RD **354 to** RD **371**

RD 354 Cross Regulator	09:30 am - 11:30 am
Outlet RD 363.5L	10:45 am - 12:45 pm
Outlet RD 360 L	11:25 am - 01:25 pm
Outlet RD 370.7 R Azim Disty. RD 371 Mahmood Disty. RD 371 Fordwah Disty. RD 371	11:50 am - 01:30 pm

DATA SETS (SPECIMEN)

Head Ratio Correction

Flow condition	**Head Ratio**
Free Orifice Flow	$[(h_u)_0/(h_u)_t]^{0.5}$
Submerged Orifice Flow	$[(h_u-h_d)_0/(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow ✓	$[(h_u)_0/(h_u)_t]^{1.5}$
Submerged Open Channel Flow	$[(h_u-h_d)_0/(h_u-h_d)_t]^{1.5}$

Channel 4-L Tape measurement gate

Date 1-6-95 Flowcondition Free open channel Flow

Clock Time	h_u Tape/Gauge	h_u	h_d Tape/Gauge	h_d	h_u-h_d	Head Ratio
12:00	0.70	3.815	0.785	3.72	0.095	1
12:30	0.71	3.805	0.790	3.715	0.090	1.004
12:45	0.71	3.805	0.790	3.715	0.090	1.004

Flow Condition	Coefficient of Discharge
Free Orifice Flow	$C_d = Q/A^*[2g^*(h_u-h_d)_t]^{0.5}$
Submerged Orifice Flow	$C_f = Q/G_0{}^*W[2g^*(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow ✓	$C_d = Q/W^*[(h_u)]^{1.5}$
Submerged Open Channel Flow	$C_f = Q^*(-\log S)^{ns}/(h_u-h_d)^{nf}$

$C_d =$ 0.62

Date: 1-6-95 Channel: ____________ Station: ____________

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Velocity: At point: Head Ratio	Velocity: At point: Corrected	Velocity: Mean in vertical	Velocity: Mean in section	Area	Mean Depth	Width	Discharge
2.0	0.8	12:5	0.32	37	120	0.7				0.385	0.3375	0.675	0.5	0.129
2.5	0.9	12:10	0.38	46	"	0.86				0.78	0.425	0.85	0.5	0.33
3.0	1.09	12:15	0.42	50	"	0.93				0.875	0.487	0.975	0.50	0.44
3.5	1.1	12:20	0.44	52	"	0.97				0.95	0.537	1.075	0.50	0.51
4.0	1.2	12:25	0.48	57	"	1.06				1.02	0.58	1.15	0.5	0.58
5	1.25	12 30	0.5	63	"	1.16	1.004			1.11	1.23	1.23	1	1.36
6	1.35	12 35	0.54	70	"	1.29				1.225	1.3	1.3	1	1.59

Sc alamat Ali, Rana Qamar Rasool

Date: 1-6-95 Channel: 4-L Station:

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity					Area	Mean Depth	Width	Discharge
						At point			Mean in vertical	Mean in section				
						Actual	Head Ratio	Corrected						
7	1.40	12:40	0.56	72	120	1.33				1.31	1.37	1.37	1	1.80
8	1.45	12:43	0.58	71	"	1.37				1.32	1.425	1.425	1	1.88
9	1.45	12:45	0.56	64	"	1.18	1.004			1.245	1.45	1.45	1	1.80
9.5	1.4	12:50	0.56	59	"	1.09				1.14	1.43	1.43	1	1.62
10	1.3	12:55	0.42	58	"	1.07				1.08	0.675	1.35	0.5	0.72
10.5	1.3		0.52	48	"	0.89				0.98	0.65	1.3	0.5	0.64
11	0.5		0.20	42	"	0.79				0.84	0.45	0.9	0.5	0.38
11.5	0.15					0.079				0.434	0.163	0.33	0.5	0.7

TOTAL Q=1384 cusecs.

Observers:

No. 1 of 2 pages Computations Checked

Head Ratio Correction

Flow condition	Head Ratio
Free Orifice Flow ✓	$[(h_u)_0/(h_u)_t]^{0.5}$
Submerged Orifice Flow	$[(h_u-h_d)_0/(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow	$[(h_u)_0/(h_u)_t]^{1}$
Submerged Open Channel Flow	$[(h_u-h_d)_0/(h_u-h_d)_t]^{1}$

Channel Daulat Tape measurement gate

Date 31-5-95 Flow condition Free Flow

Clock Time	h_u Tape/Gauge	h_u	h_d Tape/Gauge	h_d	h_u-h_d	Head Ratio
10	0.925	4.29	1.065	2.32	1.97	1.0
10:30	0.920	4.29	1.062	2.33	1.96	0.99
11:00	0.930	4.28	1.065	2.33	1.95	1.00
11:30	0.94	4.27	1.075	2.32	1.95	1.00
12:00	0.95	4.26	1.085	2.31	1.95	1.00
12:30	0.965	4.24	1.11	2.28	1.96	1.01
13:00	0.99	4.22	1.10	2.29	1.93	1.01

Flow Condition	Coefficient of Discharge
Free Orifice Flow ✓	$C_d = Q/A^*[2g^*(h_u-h_d)_t]^{0.5}$
Submerged Orifice Flow	$C_d = Q/G_0^*W[2g^*(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow	$C_d = Q/W^*[(h_u)]^{1.5}$
Submerged Open Channel Flow	$C_d = Q^*(-\log S)^{ns}/(h_u-h_d)^{nf}$

$\boxed{C_d = 0.348}$

Date: 31-5-95 Channel: Daulat Station: 532 feet from head

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Velocity: At point: Head Ratio	Velocity: At point: Corrected	Velocity: Mean in vertical	Velocity: Mean in section	Area	Mean Depth	Width	Discharge
2'	0.73	10:05		40% of next reading					0.50					
										0.85	1.58	1.58	1.0	1.34
3	2.42	10:10	.8d	60	120	1.11								
			.2d	70	"	1.29			1.20					
										1.337	2.67	2.67	1.0	3.57
4	2.92	10:16	.8d	72	"	1.33								
		10:21	.2d	88	"	1.62			1.475					
										1.497	2.93	2.93	1.0	4.387
5	2.95	10:25	.8d	73	"	1.35								
		10:27	.2d	92	"	1.69			1.52					
										1.54	2.98	2.98	1.0	4.59
6'	3.0	10:30	.8d	76	"	1.40								
		10:35	.2d	94	"	1.72			1.56					

Observers: Shabbir, Khadim, Mushtaq, Khan, Maqbool, Razaq, Shahid, Barkat

No. 1 of 6 pages Computations Checked

Date: 31-5-95 **Channel:** area **Station:**

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity At point Actual	Velocity At point Head Ratio	Velocity At point Corrected	Velocity Mean in vertical	Velocity Mean in section	Area	Mean Depth	Width	Discharge
										1.60	3.0	3.0	1.0	4.80
7	3.0	10:40	.8d	80	120	1.47								
		10:43	.2d	99	"	1.81			1.64					
										1.675	3.0	3.0	1.0	5.03
8	3.0	10:45	.8d	82	"	1.51								
		10:49	.2d	104	"	1.91			1.71					
										1.725	3.0	3.0	1.0	5.18
9	3.0	10:54	.8d	85	"	1.56								
		10:57	.2d	105	"	1.92			1.74					
										1.71	3.0	3.0	1.0	5.13
10	3.0	11:00	.8d	78	"	1.44								
		11:05	.2d	105	"	1.92			1.68					
										1.71	2.98	2.98	1.0	5.09
11	2.95	11:10	.8d	73	"	1.45								

Observers:

No. 2 of 6 pages **Computations** **Checked**

Date: [illegible] **Channel:** Daulat **Station:**

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Head Ratio	Corrected	Mean in vertical	Mean in section	Area	Mean Depth	Width	Discharge
		11:15	.2d	111	120	2.03			1.74					
										1.69	5.84	2.92	2.0	9.88
13	2.90	11:18	.8d	78	"	1.44								
		11:21	.2d	101	"	1.85			1.645					
										1.625	5.8	2.9	2.0	9.43
15	2.9	11:25	.8d	70	"	1.29								
		11:28	.2d	105	"	1.92			1.605					
										1.617	5.8	2.9	2.0	9.38
17	2.9	11:31	.8d	71	"	1.31								
		11:34	.2d	105	"	1.92			1.63					
										1.637	5.80	2.90	2.0	9.497
19	2.9	11:37	.8d	75	"	1.38								
		11:40	.2d	104	"	1.91			1.645					
										1.64	5.74	2.87	2.0	9.41

Observers

No. 3 of 6 pages Computations Checked

Date: 31-5-95 Channel: Daulat Station: ____

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Velocity: At point: Head Ratio	Velocity: At point: Corrected	Velocity: Mean in vertical	Velocity: Mean in section	Area	Mean Depth	Width	Discharge
21	2.85	11:44	.8d	73	120	1.35								
		11:47	.2d	105	"	1.92			1.635					
										1.623	5.7	2.85	2.0	9.25
23	2.85	11:50	.8d	73	"	1.35								
		11:54	.2d	102	"	1.87			1.61					
										1.618	5.74	2.87	2.0	9.29
25	2.88	11:57	.8d	74	"	1.36								
		12:00	.2d	103	"	1.89			1.625					
										1.62	5.68	2.84	2.0	9.20
27	2.8	12:3	.8d	75	"	1.38								
		12:6	.2d	101	"	1.85			1.615					
										1.608	5.60	2.80	2.0	9.00
29	2.8	12:9	.8d	70	"	1.29								
		12:12	.2d	104	"	1.91			1.60					

Observers: ____

No. 4 of 6 pages Computations ____ Checked ____

Date: 31-5-95 **Channel:** Daulat **Station:**

Distance from initial point	Depth	Clock Time in minutes	Depth of obser-vation	Revolu-tions	Time in sec-onds	Velocity: At point: Actual	Velocity: At point: Head Ratio	Velocity: At point: Corr-ected	Velocity: Mean in vertical	Velocity: Mean in section	Area	Mean Depth	Width	Disch arge
30	2.8	12:15	.8d	76	120	1.4				1627	2.8	2.8	1.0	4.56
		12:18	.2d	104	"	1.91			1.65					
										1.635	2.84	2.84	1.0	4.64
31	2.88	12:21	.8d	74	"	1.36			1.615					
		12:24	.2d	102	"	1.87								
										1.58	2.84	2.84	1.0	4.49
32	2.80	12:27	.8d	72	"	1.33			1.54					
		12:30	.2d	96	"	1.76								
										1.52	2.8	2.8	1.0	4.26
33	2.8	12:33	.8d	69	"	1.27			1.495					
		12:36	.2d	94	"	1.72								
										1.492	2.8	2.8	1.0	4.18

Observers:

No 5 of 6 pages **Computations** **Checked**

Distance from initial point	Depth	Clock Time in minutes	Depth of obser-vation	Revolu-tions	Time in sec-onds	Velocity					Area	Mean Depth	Width	Disch arge
						At point			Mean in vertical	Mean in section				
						Actual	Head Ratio	Corr-ected						
34	2 8	12:40	·8d	72	120	1.33								
		12:43	·2d	90	″	1.65			1.49					
										1.42	2.8	2.8	1.0	3.98
35	2.8	12:47	·8d	64	″	1.18			1.35					
		12:50	·2d	83	″	1.53								
										1.29	2.72	2.72	1.0	3.52
36	2.65	12:53	·8d	63	″	1.16			1.23					
		12:56	·2d	71	″	1.31								
										1.18	1.63	1.63	1.0	1.93
37	0.62	13:00	·6d	61	″	1.13			1.13					
										0.96	0.765	0.51	1.5	0.734
38.5	·40	13:10	@ 40% of previous			0.452			0.791					

石流 Q= 156 Cu

Observers: ______________________

No. 6 of 6 pages computations ______________ Checked ______________

Head Ratio Correction

Flow condition	**Head Ratio**
Free Orifice Flow	$[(h_u)_o/(h_u)_t]^{0.5}$
Submerged Orifice Flow ✓	$[(h_u-h_d)_o/(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow	$[(h_u)_o/(h_u)_t]^{1.5}$
Submerged Open Channel Flow	$[(h_u-h_d)_o/(h_u-h_d)_t]^{1.5}$

Channel Fordwah Disty Tape measurement gate

Date 1-6-95 Flowcondition Submerged Orifice Flow

Clock Time	h_u Tape/Gauge	h_u	h_d Tape/Gauge	h_d	h_u-h_d	Head Ratio
10:45	1.85	4.79	0.78	3.84	0.91	1.00
11:00	1.75	4.84	0.77	3.85	0.99	0.91
11:15	1.75	4.84	0.77	3.85	0.99	0.91
11:30	1.71	4.88	0.74	3.88	1.00	0.90
11:45	1.76	4.84	0.74	3.88	0.95	0.95
12:00	1.80	4.79	0.75	3.87	0.92	0.98
12:15	1.83	4.77	0.75	3.87	0.89	1.01
12:30	1.81	4.78	0.74	3.88	0.90	1.00
12:45	1.84	4.76	0.74	3.88	0.87	1.03
13:00	1.85	4.75	0.75	3.87	0.87	1.03

Flow Condition	Coefficient of Discharge
Free Orifice Flow	$C_d = Q/A^*[2g^*(h_u-h_d)_t]^{0.5}$
Submerged Orifice Flow ✓	$C_d = Q/G_o^*W[2g^*(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow	$C_f = Q/W^*[(h_u)]^{1.5}$
Submerged Open Channel Flow	$C_d = Q^*(-\log S)^{ns}/(h_u-h_d)^{nf}$

C_d = 0.87

Flow condition	Head Ratio
Free Orifice Flow	$[(h_u)_0/(h_u)_t]^{0.5}$
Submerged Orifice Flow ✓	$[(h_u-h_d)_0/(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow	$[(h_u)_0/(h_u)_t]^{1.5}$
Submerged Open Channel Flow	$[(h_u-h_d)_0/(h_u-h_d)_t]^{1.5}$

Channel *Fordwah Disty* Tape measurement gate ______

Date *1-6-95* Flowcondition *Submerged Orifice Flow*

Clock Time	h_u Tape/Gauge	h_u	h_d Tape/Gauge	h_d	h_u-h_d	Head Ratio
13:15	1.92	4.67	0.76	3.86	0.815	1.0
13:30	2.04	4.56	0.80	3.82	0.74	1.10
13:45	2.04	4.56	0.81	3.81	0.74	1.09

Flow Condition	Coefficient of Oischarae
Free Orifice Flow	$C_d = Q/A^*[2g^*(h_u-h_d)_t]^{0.5}$
Submerged Orifice Flow ✓	$C_d = Q/G_0{}^*W[2g^*(h_u-h_d)_t]^{0.5}$
Free Open Channel Flow	$C_d = Q/W^*[(h_u)]^{1.5}$
Submerged Open Channel Flow	$C_d = Q^*(-\log S)^{ns}/(h_u-h_d)^{nf}$

C_d = *0 87*

Date: 1-6-95 **Channel:** Fordwah Disty **Station:** +340 From Head Regulator

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity At point Actual	Head Ratio	Corrected	Mean in vertical	Mean in section	Area	Mean Depth	Width	Discharge
1		11:00				10% of next			0.093					
										0.51	1.49	1.49	1.0	0.76
2	2.98	11:00	0.596	61	120 sec	0.73			0.93					
		11:03	2.38	39	"	1.13								
										1.15	2.99	2.99	1.0	3.44
3	3.1	11:08	0.62	84		1.20			0.37					
		11:06	2.48	65		1.54								
							0.95	1.29	1.36		3.08	3.08	1.0	3.98
4	3.05	11:12	0.61	64		1.18								
		11:15	2.44	84		1.54								
							0.95	1.30	1.37		3.08	3.08	1.0	4.01

Observers: Marcel Kuper, M Pasha, Shauk, S Qasim, M Ramzan

No. 1 **of** 8 **pages** **Computations** **Checked**

Date: 1-6-95 **Channel:** Fordwah Disty **Station:**

Distance from initial point	Depth	Clock Time in minutes	Depth of obser-vation	Revolu-tions	Time in sec-onds	Velocity					Area	Mean Depth	Width	Discharge
						At point			Mean in vertical	Mean in section				
						Actual	Head Ratio	Corr-ected						
5	3.11	11:19	0.62	66	120 sec	1.22			1.38					
		11:16	2.48	84	"	1.54								
							0.95	1.47		1.55	3.13	3.13	1.0	4.61
6	3.15	11:21	0.63	73	"	1.35	"							
		11:23	2.52	114	"	2.09	"							
								1.58		1.66	3.17	3.17	1.0	5.0
7	3.18	11:28	0.63	71	"	1.31			1.59					
		11:25	2.54	102	"	1.87								
							0.95	1.50		1.58	3.19	3.19	1.0	4.79
8	3.20	11:26	0.64	72	"	1.33			1.57					
		11:31	2.56	99	"	1.81								
								1.57		1.65	3.19	3.19	1.0	5.0

Observers:

No. 2 **of** 8 **pages** **computations** **Checked**

Date: 1-6-95 Channel: Fordwah Disty Station:

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Head Ratio	Corrected	Mean in vertical	Mean in section	Area	Mean Depth	Width	Discharge
9	3.18	11:36	0.63	79	120	1.45			1.72					
		11:33	2.54	108	"	1.98	0.95							
								1.64		1.73	3.19	3.19	1.0	5.24
10	3.20	11:38	0.64	84	"	1.54	0.97		1.75					
		11:42	2.56	107	"	1.96								
							0.97	1.70		1.76	6.46	3.23	2.0	11.02
12	3.26	11:45	0.65	80	"	1.47			1.77					
		11:48	2.61	113	"	2.07								
										1.81	6.5	3.25	2.0	11.64
14	3.25	11:50	0.65	88	"	1.62	0.99	1.79	1.85					
		11:53	2.61	113	"	2.07								
										1.81	6.54	3.27	2.0	11.70

Observers:

No. 3 of 8 pages Computations Checked

Date: 1-6-95 **Channel:** Toxwa Disty **Station:**

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Velocity: At point: Head Ratio	Velocity: At point: Corrected	Velocity: Mean in vertical	Velocity: Mean in section	Area	Mean Depth	Width	Discharge
16	3.28	11:55	0.66	77	120	1.42	0.99	1.79	1.77					
		11:58	2.63	116	"	2.12								
										1.83	6.54	3.27	2.0	11.84
18	3.27	12:00	0.66	89	"	1.63	0.99	1.81	1.88					
		12:04	2.63	117	"	2.14								
										1.91	6.52	3.26	2.0	12.32
20	3.25	12:05	0.65	88	"	1.62	0.99	1.89	1.94					
			2.60	123	"	2.25								
							1.00			1.93	6.58	3.29	2.0	12.70
22	3.3	12:12	0.66	88	"	1.62	1.00		1.93					
		12:17	2.64	122	"	2.23								
										1.87	6.64	3.32	2.0	12.42

Observers:

No. 4 **of** 8 **pages** **Computations** **Checked**

Date: 1-6-95 **Channel:** Fordwah Distr **Station:**

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Head Ratio	Corrected	Mean in vertical	Mean in section	Area	Mean Depth	Width	Discharge
24	3.32		0.67	87	120	1.51			1.82					
		12.21	2.65	117	"	2.14								
										1.82	6.64	3.32	2.0	12.0
26	3.32	12.29	0.67	82	"	1.51								
		12.33	2.66	115	"	2.10								
										1.81	6.72	3.36	2.0	12.16
28	3.40	12:35	0.68	79	"	1.45	1.0	1.81	1.81					
		12:37	2.72	119	"	2.18								
										1.77	3.37	3.37	1.0	5.96
29	3.35	12:40	0.67	79	"	1.45	1.00	1.74	1.74					
		12.44	2.68	111	"	2.03								
										1.74	3.32	3.32	1.0	5.78

Observers:

No. 5 of 8 pages Computations Checked

Date: 1-6-95 Channel: Fordwah Disty Station:

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity					Area	Mean Depth	Width	Discharge
						At point			Mean in vertical	Mean in section				
						Actual	Head Ratio	Corrected						
30	3.28	12:50	0.66	81	120	1.49								
		12:47	2.63	108	"	1.98								
										1.75	3.27	3.27	1.0	5.72
31	3.27	13:00	0.66	77	"	1.42	1.00	1.75	1.75					
		12:50	2.63	114	"	2.09								
										1.83	3.24	3.24	1.0	5.93
32	3.20	13:03	0.64	80	"	1.47	1.00	1.83	1.92					
		13:07	2.56	130	"	2.37								
							1.00	1.74		1.74	3.15	3.15	1.0	5.48
33	3.11	13:11	0.62	76	"	1.40	1.00		1.55					
		13:08	2.49	92	"	1.69								
										1.57	3.06	3.06	1.0	4.80

Observers:

No. 6 of 8 pages Computations Checked

Date: 1-6-95 Channel: Fordwah Disty Station:

Distance from initial point	Depth	Clock Time in minutes	Depth of observation	Revolutions	Time in seconds	Velocity: At point: Actual	Velocity: At point: Head Ratio	Velocity: At point: Corrected	Velocity: Mean in vertical	Velocity: Mean in section	Area	Mean Depth	Width	Discharge
34	3.01	13:14	0.61	80	120	1.47			1.60					
		13:17	2.41	95	"	1.47								
										1.52	3.01	3.01	1.0	4.78
35	3.01	13:22	0.61	71	"	1.31	1.05	1.59	1.44					
		13:19	2.41	85	"	1.56								
										1.40	3.01	3.01	1.0	4.42
36	3.00	13:27	0.60	68		1.25	1.05	1.47	1.36					
		13:31	2.40	80		1.47								
							1.05	1.34		1.28	3.0	3.0	1.0	4.03
37	3.00	13:36	0.60	63	"	1.16	1.05	1.28	1.20					
		13:33	2.40	68	"	1.25								
										1.22	2.5	2.5	1.0	3.20

Observers:

No. 7 of 8 pages Computations Checked

Date: 1-6-95 **Channel:** Fordwah Disty **Station:**

Distance from initial point	Depth	Clock Time in minutes	Depth of obser-vation	Revolu-tions	Time in sec-onds	Velocity					Area	Mean Depth	Width	Disch arge
						At point			Mean in vertical	Mean in section				
						Actual	Head Ratio	Corr-ected						
38	2.0	13:40	0.40	61	120	1.13	1.05	1.01	1.23					
		13:43	1.60	72	"	1.33								
										0.96	0.85	1.7	0.5	0.86
38.5	1.4	1347	0.56	37	"	0.70			0.70					
39.3	—	13:48	—	—										

TOTAL Q= 197.47 Cusecs.

Observers:

No. 8 of 8 pages **Computations** **Checked**