TRAINING COURSE ON
FIELD CALIBRATION OF IRRIGATION OUTLETS

Hakra 4-R and Sirajwah Distributaries
Fordwah Eastern Sadiqia Irrigation and Drainage Project

Haroonabad, 22 October to 1 November, 1995

Technical Report

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CHAPTER 1: INTRODUCTION

The Fordwah Eastern Sadiqia (FES) Irrigation and Drainage Project is located in the southeastern portion of the Province of Punjab adjoining the boundary with India covering around 30-50% of the Fordwah and Eastern Sadiqia Canal command areas. The Suleimanki Headworks is located on the left side of the Sutlej River, which supplies the Fordwah Canal and the Eastern Sadiqia Canal.

In the Fordwah Canal Command serves 100,000 hectares (ha) of cultivated land is referred to as FES (North). The upper portion of the Eastern Sadiqia Canal provides irrigation water to 105,000 ha called FES (South), which includes the Hakra 4-R Distributary command area and all cultivated land upstream.

The Government of Pakistan and the World Bank are jointly funding irrigation and drainage activities within the FES (South) boundary. The Phase I effort (1993-981 consists of a geomembrance lining for many distributaries and a combination of surface drains, interceptor drains along branch canals, and some experimental subsurface drains. There are two monitoring components and a highly significant research component that are designed to provide necessary insights for preparing the Phase II documentation.

Under the research and monitoring components, there are five organizations who have a need for measuring the discharge rate passing through many of the moghas (outlets) from distributaries and minors (1) Watercourse Monitoring and Evaluation Directorate (WMED), Water and Power Development Authority (WAPDA), (2) On-Farm Water Management (OFWM) Directorate, Department of Agriculture, Province of Punjab; (3) International Waterlogging and Salinity Research Institute (IWASRI), WAPDA, (4) International Sedimentation Research Institute, Pakistan (ISRIP) and (51 International Irrigation Management Institute (IIMI).

There was a recognition that the various moghas could be calibrated, many of them quite easily, but some would prove difficult. Also there was a shared feeling that it would be advantageous to jointly calibrate these moghas and then share this information with all interested organizations.

At the same time, IWASRI and the International Sedimentation Research Institute, Pakistan (ISRIP), WAPDA had been conducting ponding tests, as well as inflow-outflow seepage tests on same distributaries in FES (South). This field data was being collected prior to canal lining as part of the research subcomponent, "Performance Evaluation of Canal Lining". This data was also highly important to the

Department of Irrigation and Power, Province *of* Punjab and their international consultants, Mott MacDonald.

The Punjab Department of Irrigation and IIMI jointly collaborated in organizing this training course, with the assistance of WAPDA. The Hakra 4-R Distributary and the Sirajwah Distributary were selected as the field sites for conducting this training because all of the participating organizations were working in these command areas. A classroom was provided by Government Degree College at Haroonabad for lectures and data analysis. Most of the training course was conducted at the field sites, with each organization providing transport vehicles. Meals were served at the WAPDA Lodge in Haroonabad.

On the day of the training course, a description of the physical system was given by the Executive Engineer, Hakra Division, Mr. Rana Nazir Ahmad, while a special lecture on outlets was delivered by Mr. M.H. Siddiqi of the Punjab Irrigation Department.

CHAPTER 2: METHODOLOGY FOR FIELD CALIBRATION OF IRRIGATION OUTLETS (MOGHAS)[1]

2.1 Introduction to Irrigation Flow Control Structure

There is a growing awareness around the world of the necessity to improve the agricultural productivity of existing irrigated lands. A major thrust in these efforts is to improve irrigation water management practices. In order to do so, some knowledge is required about the resource being managed. This would imply that, as a minimum, the water must be measured at strategic locations within the system. The ideal situation would be to measure the discharge of every division in the system, including the quantity of water delivered to each farmer.

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

In most irrigation systems, there are numerous structures that can be calibrated for the purpose of water measurement as illustrated in Figure 2.1. Usually, the most common constriction in the irrigation delivery network is a gate structure, with some systems having hundreds of gate structures for flow control. Other common irrigation structures that can be calibrated are culverts, inverted siphons, drop structures, weirs, and wasteways. In fact, any type of structure that constricts the flow (i.e. causing a backwater effect and subcritical flow upstream) can be field calibrated for discharge measurernent.

The installation of standardized primary flow measuring devices, such as laboratory calibrated flumes and weirs, is advantageous in that field calibrations are not necessary unless: (1) the dimensions of the device are incorrect; or (2) the installation does not correspond with the conditions under which the laboratory rating was developed. Major disadvantages of using these devices are, first of all, expense, but often it is the added head loss in the channel which results in higher water levels upstream from the device that may even result in lower discharge capacity for the irrigation channel. In some cases, flow measurements have disclosed that the discharge capacity of an irrigation channel is considerably less than the design capacity, partly due to backwater effects from open channel constrictions, but mostly due to inadequate maintenance of the channel. Also, neglected structural maintenance can have a significant effect on the discharge ratings for such structures.

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[1]Taken from the report "Field Calibration of Irrigation Flow Control Structures", by Gaylord V. Skogerboe, Gary P. Merkley, M.S.Shafique, and Carlos A. Gandarillas. International Irrigation Center, Dept. of Agricultural and Irrigation Engrg., Utah State University, Logan, Utah 84322-4150 (Jan 1992)

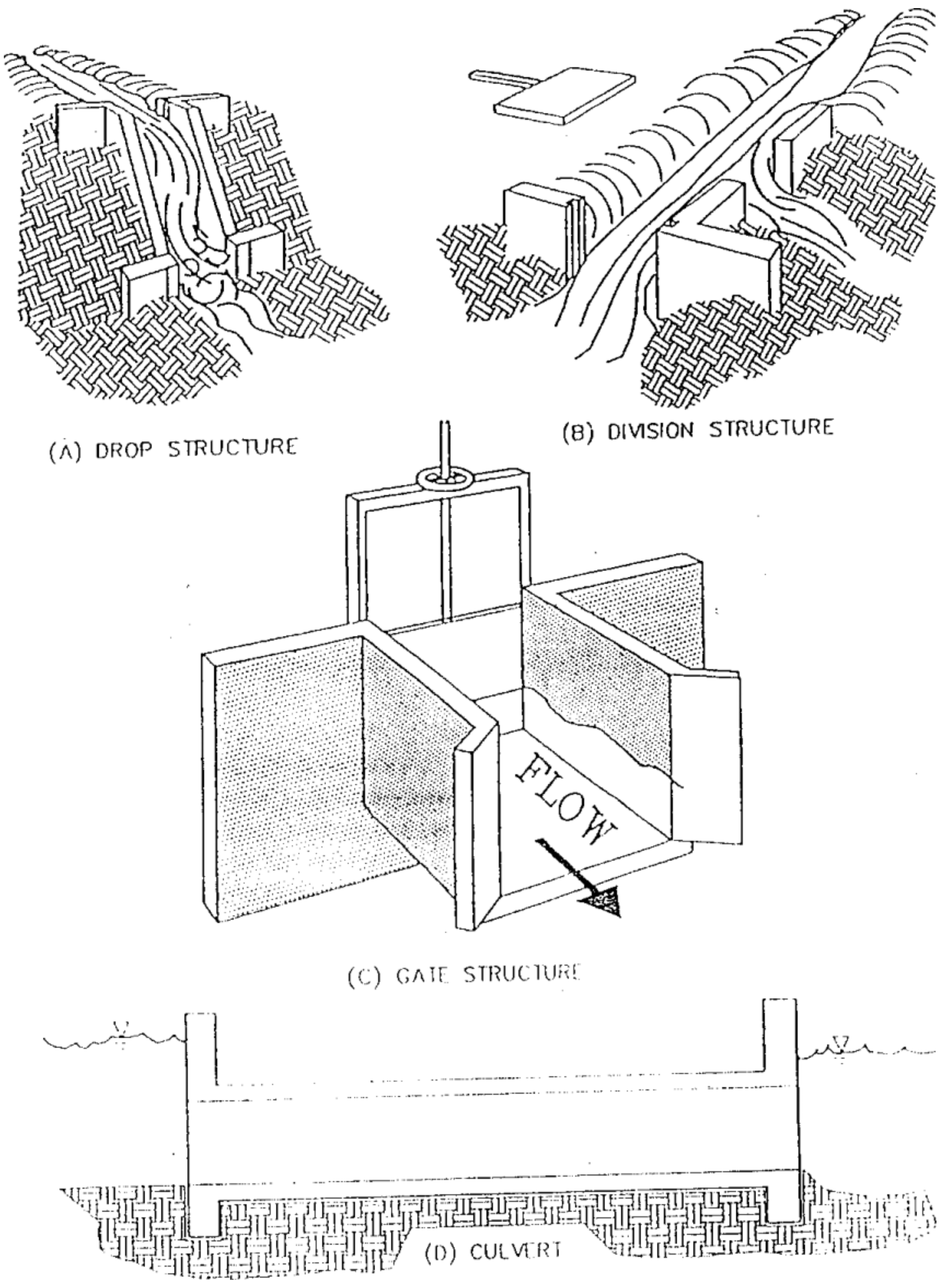


Figure 2.1 Typical irrigation structures that can be field calibrated for discharge measurement.

2.2 Field Measurements

For all of the irrigation structures described herein, the discharge equation can be written in terms of one or two flow depths, and a structure opening. Thus, during the field calibration procedures, one or two flow depths must be measured, along with the discharge rate and the opening. After completion of the field calibration, only the flow depth(s) and setting need to be measured, and the discharge can be calculated directly.

2.2.1 Discharae Measurement for Field Calibration

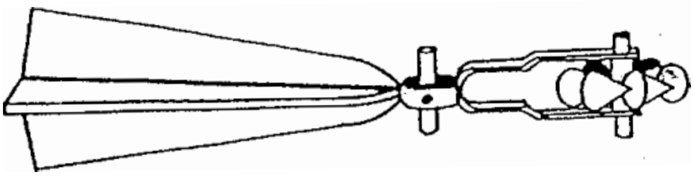
The two most common methods of collecting discharge measurements in irrigation networks are: (1) current metering; and (2) the use of flow measuring flumes, such as Parshall or Cutthroat flumes (see Figure 2.2). A current meter is usually used for discharges greater than ***500*** lps, and often for flow rates larger than 200 lps. In contract, temporarily installed flow measuring flumes are usually used for discharge rates less than about 300-500 lps.

Sometimes, for large discharge rates (greater than about $20m^3/s$), the dye dilution technique can be used. With improved dyes and instruments that measure in parts per billion (ppb) rather than parts per million (ppm), this technique is becoming increasingly useful. The greatest difficulty is in thoroughly mixing the dye with the water. Thus, injecting the dye at a steady rate upstream of an inverted siphon or drop structure is particularly helpful.

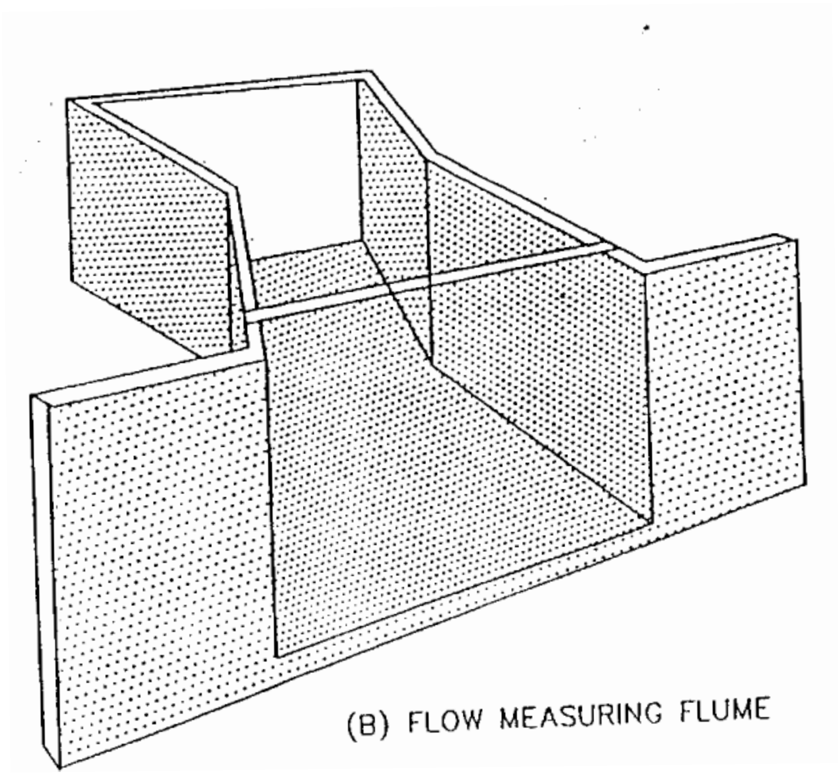
Another useful technique for measuring discharge rates is to make volumetric measurements. For example, a small pan or bucket can be used to determine the discharge rate over a small portion of weir overflow structure. By taking a series of such measurements over the crest width, the total discharge rate can be determined. Lots of ingenuity can be employed in developing various configurations of volumetric pans or containers (see Figure 2.3).

For measuring very small flow rates (less than 1 lps), a plastic bag can be used. After collecting the water for many seconds or minutes, the water can be repeatedly poured into a graduated volumetric container to determine the total volume of water. This is a useful method for measuring leakage from gate structures when they are closed.

Books and manuals on water measurement should be consulted for the variety of techniques and devices that could be used to measure discharge rates during field calibrations. Although current meters and flumes are most commonly used for developing field discharge ratings of irrigation structures, there is a multitude of devices and techniques that can be employed.



(A) CURRENT METER



(B) FLOW MEASURING FLUME

Figures 2.2. Typical devices for discharge measurement in irrigation channels.

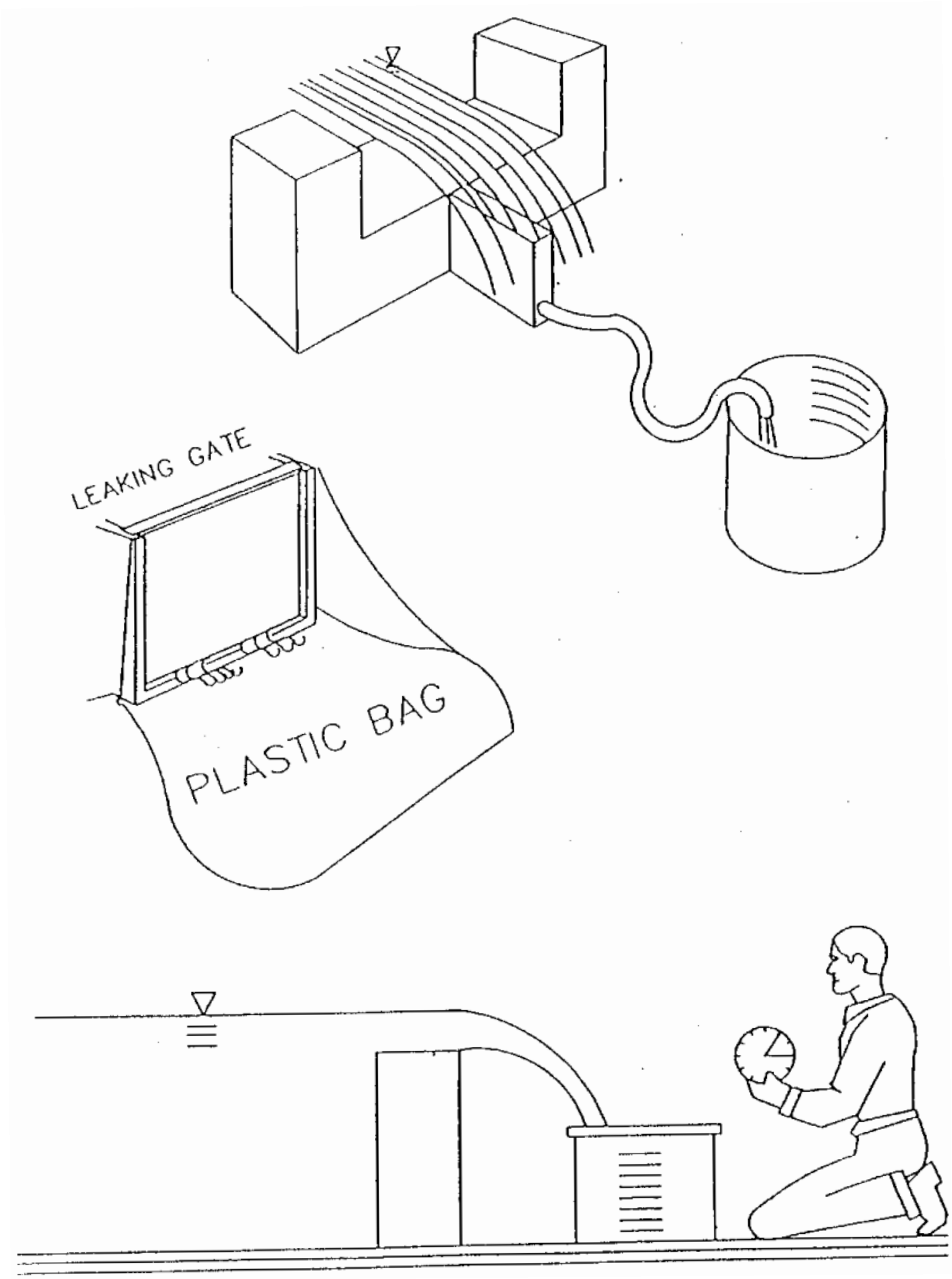


Figure 2.3. Some *of* the possibilities for volumetric discharge measurements.

2.2.2 Flow Depth Measurements

Staff gauges are most commonly used to measure flow depths. The staff gauge is placed against the wall of an irrigation structure or on a post located in the middle *of* the irrigation channel. The primary advantage of a staff gauge is that everybody can read it, including the farmers. For this reason, it is desirable to have the staff gauge read directly in liters per second (lps) rather than depth, but: **(1)** this is only possible for structures operating under free flow (modular) conditions ***(see*** discussion below); and, (2) a staff gauge must be specially made for each rated structure ***(see*** Figure 2.4), and the rating itself may change with time.

The primary disadvantage of staff gauges is that they must be repainted every year or two because the markings below the water surface become obliterated. For this reason, plus the expense of installing staff gauges, another reasonable alternative is to measure from a benchmark on the wall of an irrigation.structure downward to the water surface using a tape measure. The benchmark must be referenced to the appropriate zero flow depth level for the particular irrigation structure being calibrated so that each tape measurement can be corrected to give flow depth. The benchmark(s) should **be** carefully marked by etching or painting. **Also** the field notes prepared during the calibration procedureshould include a good sketch of the location of each benchmark. **A** typical example is shown in Figure 2.5. When preparing field notes, a good guideline to keep in mind is that anyone reading the notes ten years later should be able to easily understand all of the detailed procedure and should be able to duplicate the field work, including the relocation of each benchmark.

It the water surface is not smooth where the flow depth(s) are being measured, then it is highly desirable to use a piezometer connected to a stilling well. The piezometer pipe can be placed through the wall of the irrigation structure or can be extended into the irrigation channel ***(see*** figure 2.6). Commonly, piezometer openings of 5-10 mm diameter are used, whereas the piezometer pipe can be this same size, or larger. The more turbulent the flow, or the greater the fluctuations in the water surface, the smaller the diameter of the piezometer openings that should be used. The disadvantages in using very small piezometer openings are: **(1)** clogging of the openings; and (2) slower response times within the stilling well when the flow depth is rapidly changing.

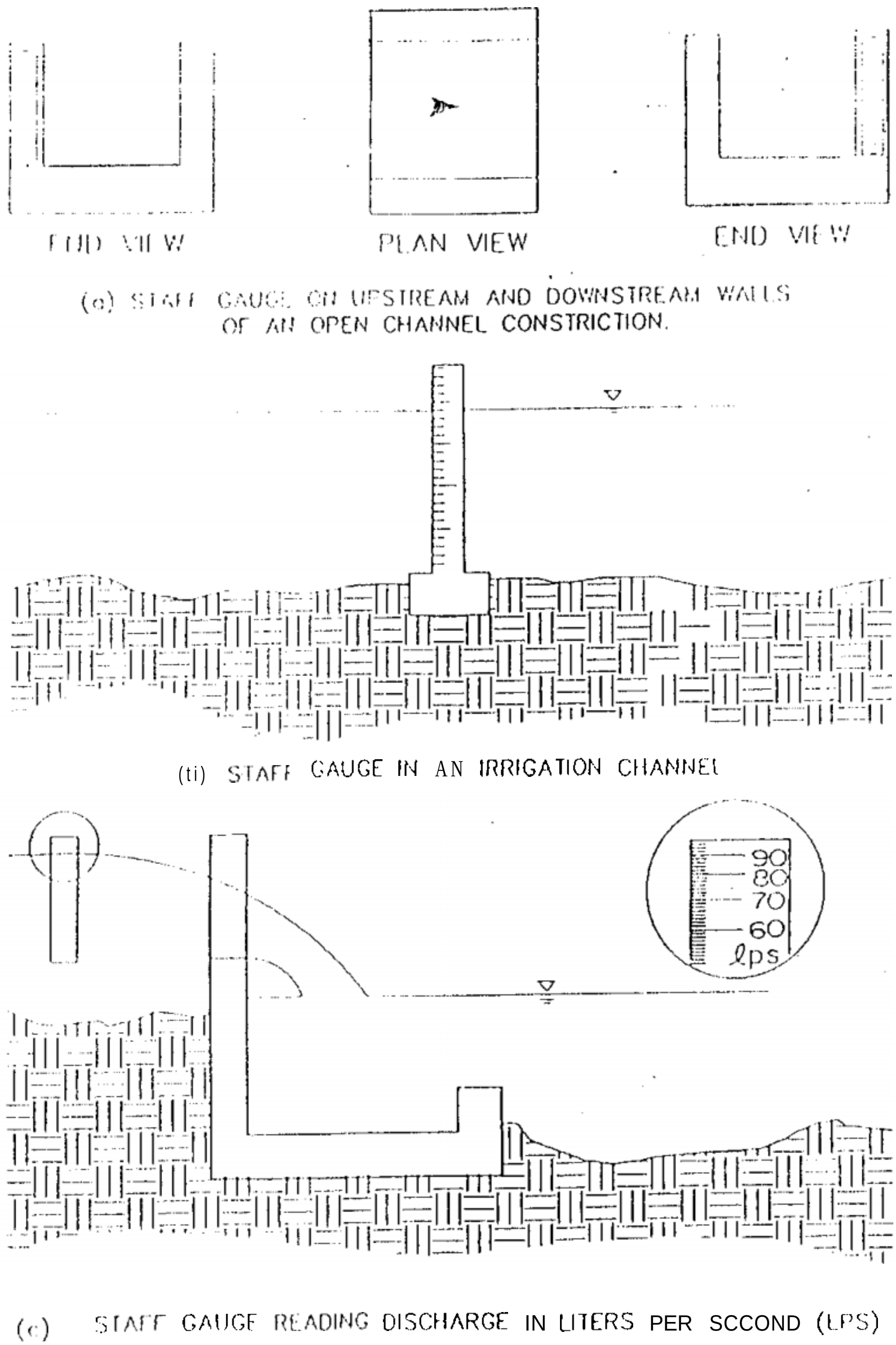


Figure 2.4. Various uses of staff gauges.

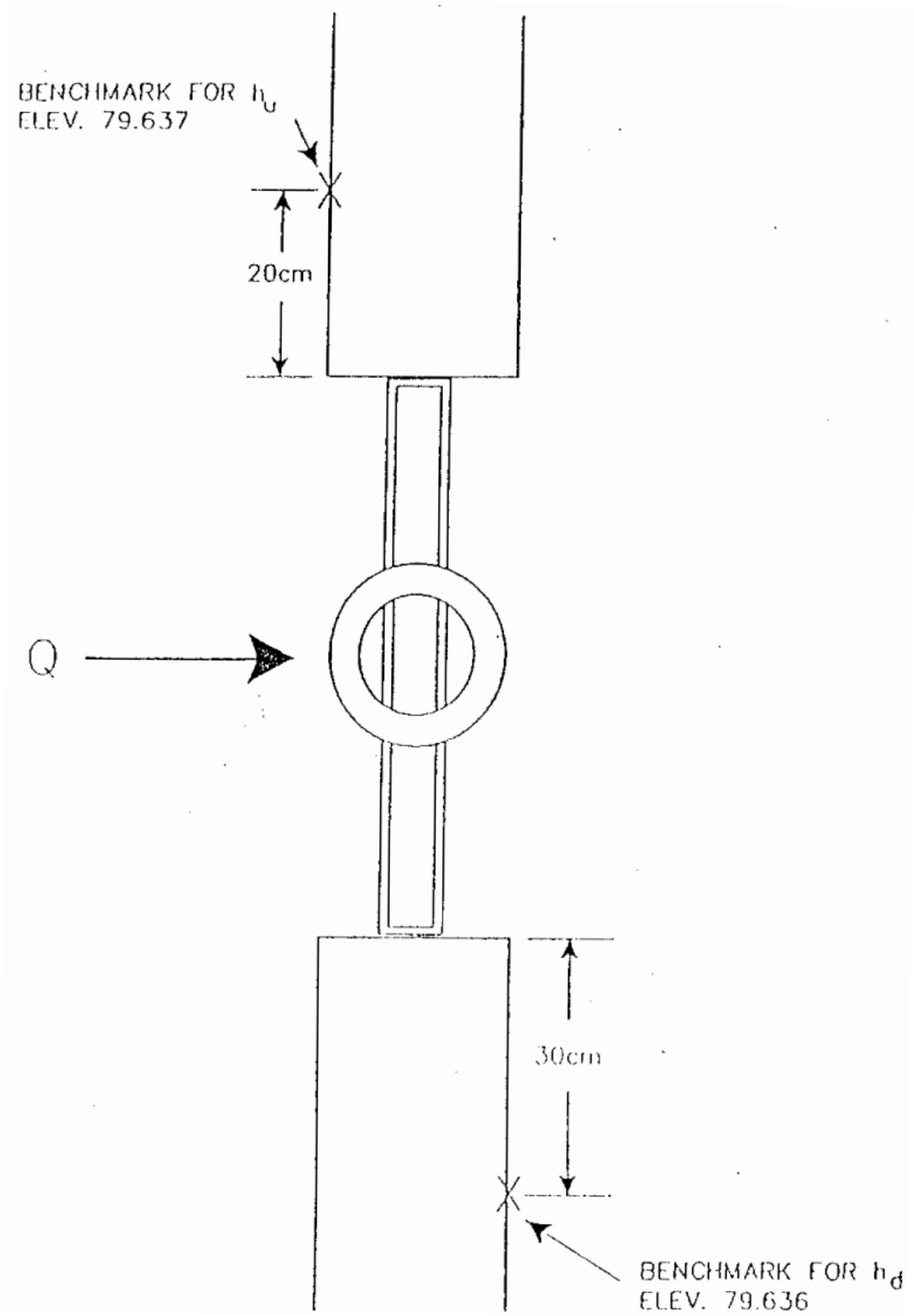


Figure 2.5. Fieldbook sketch locating benchmarks for upstream and downstream flow depth measurements at a gate structure.

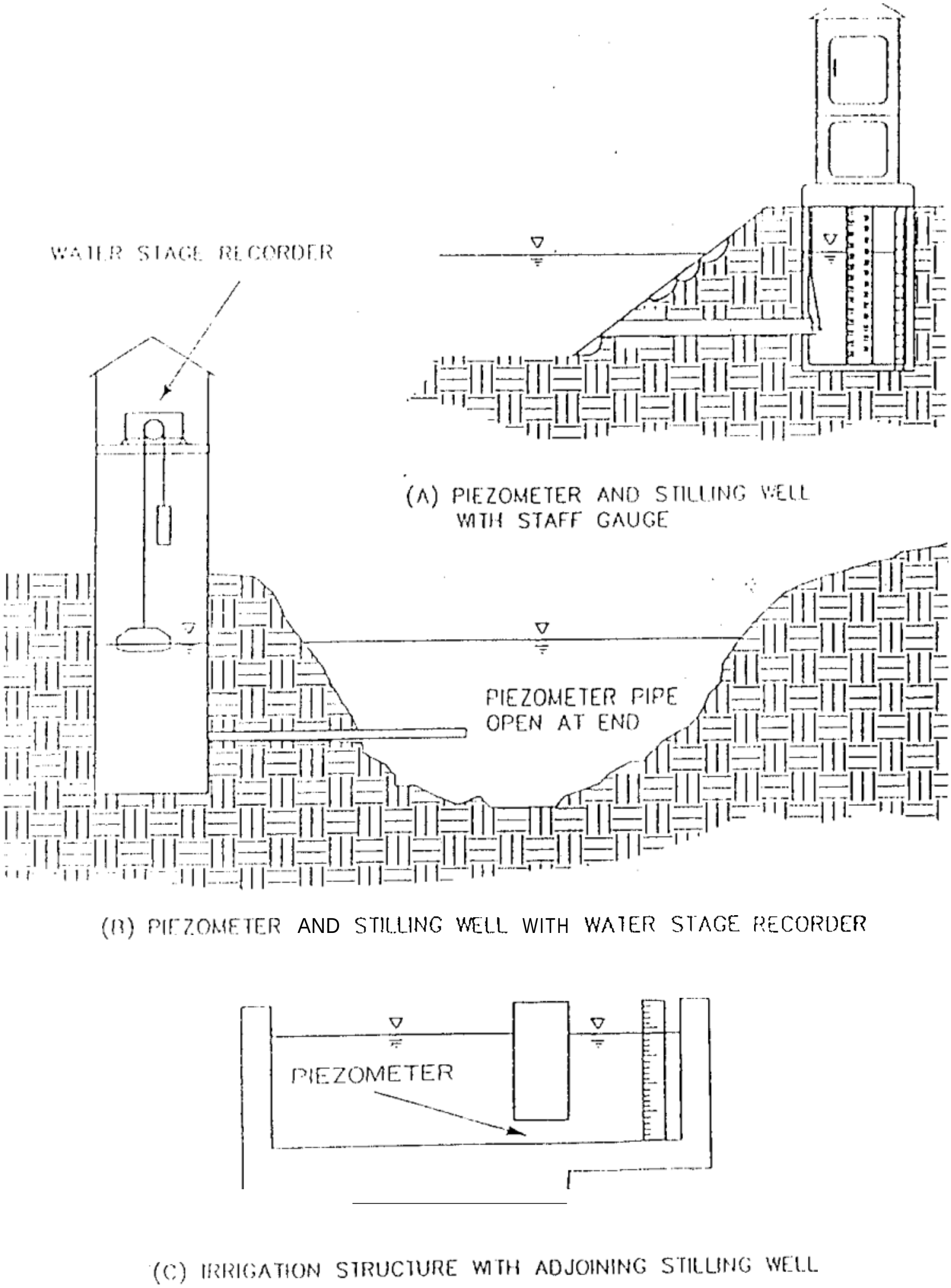


Figure 2.6. Typical piezometer installations.

2.3 **Flow** Conditions

2.3.1 Backwater Effects

A simple open channel constriction is show in Figure 2.7. 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 2.7, 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 the flow has been re-established in the channel.

2.3.2 Free Flow and Submeraed 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 or "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 condition 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:

$$Q_f = f(h_u) \tag{2.1}$$

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 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.

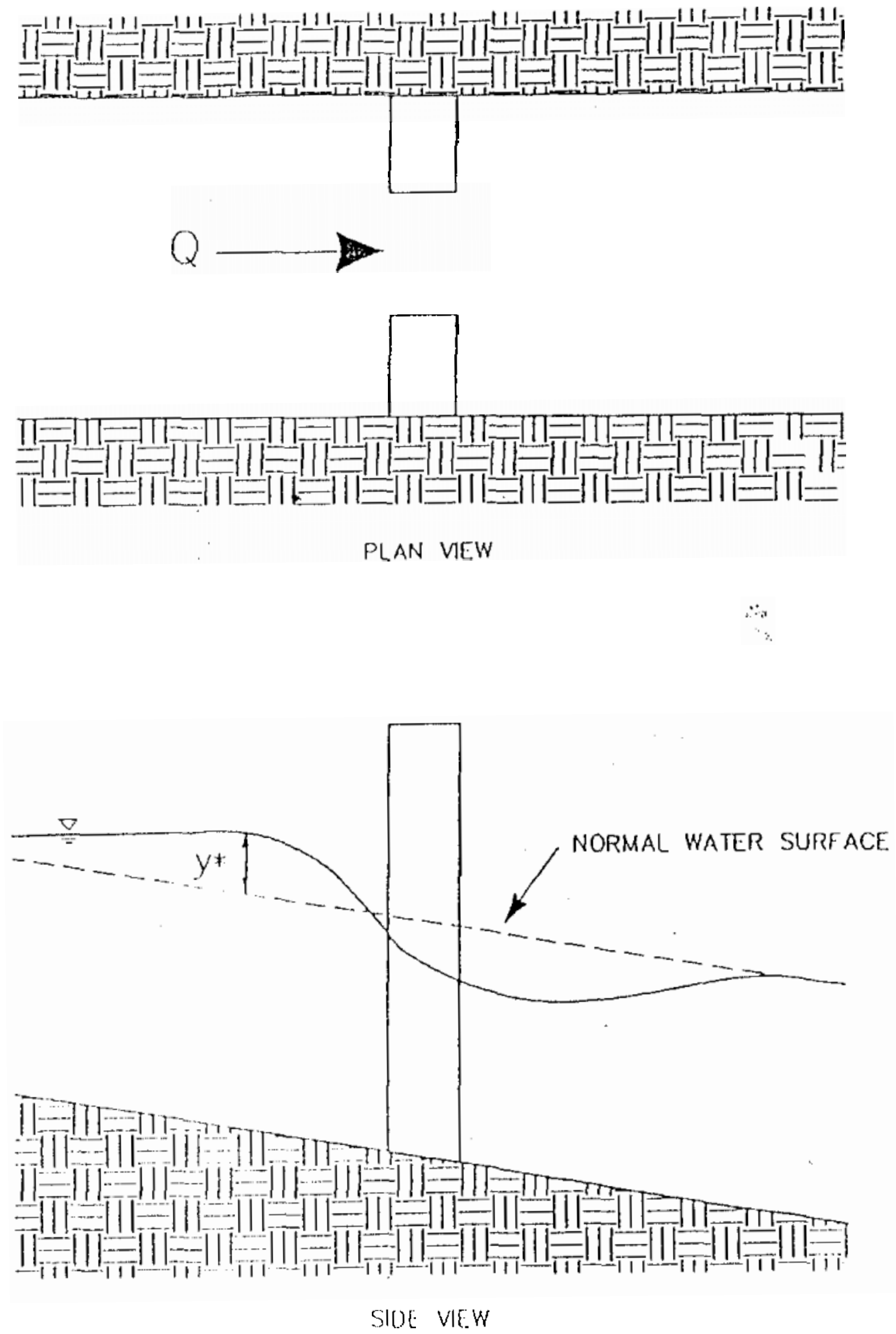


Figure 2.7. Definition sketch for backwater effects **from** an open channel constriction.

The definition given to submergence, S, is:

$$S = \frac{h_d}{h_u} \tag{2.2}$$

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 $(h_d - h_u)$, and submergence:

$$Q_s = f(h_u, h_d) = f(h_u - h_d, S) \tag{2.3}$$

Ofttimes, constrictions designed initially to operate under free flow conditions becomes 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 2.8. 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 tailwater depth has also increased the depth of flow at the upstream station.

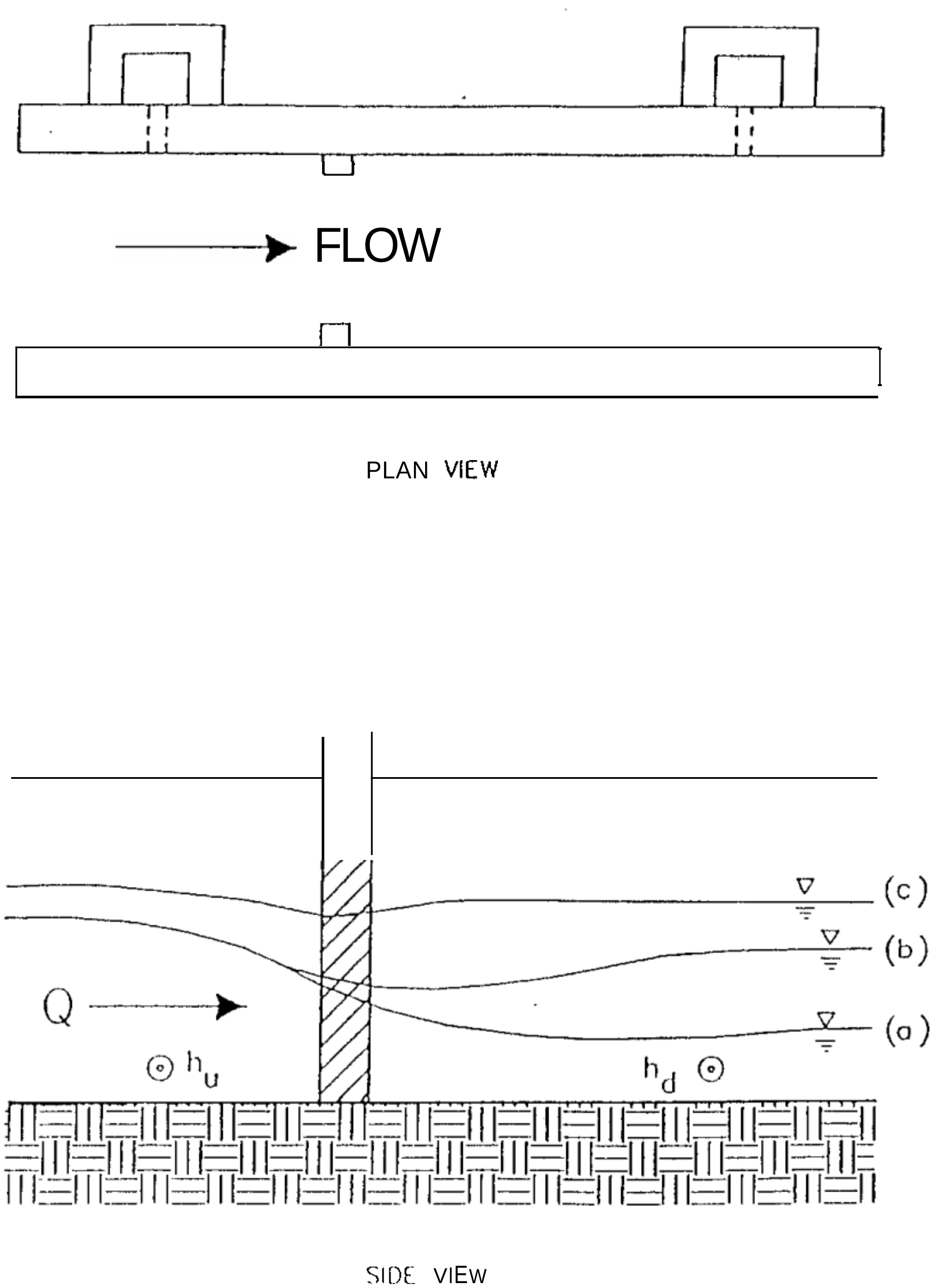


Figure 2.8. Illustration of flow conditions in an open channel constriction.

2.4 Rating Open Channel Constrictions

2.4.1 Free Flow

The general form of the free flow equation is:

$$Q_f = C_f h_u^{n_f} \tag{2.4}$$

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 in 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.

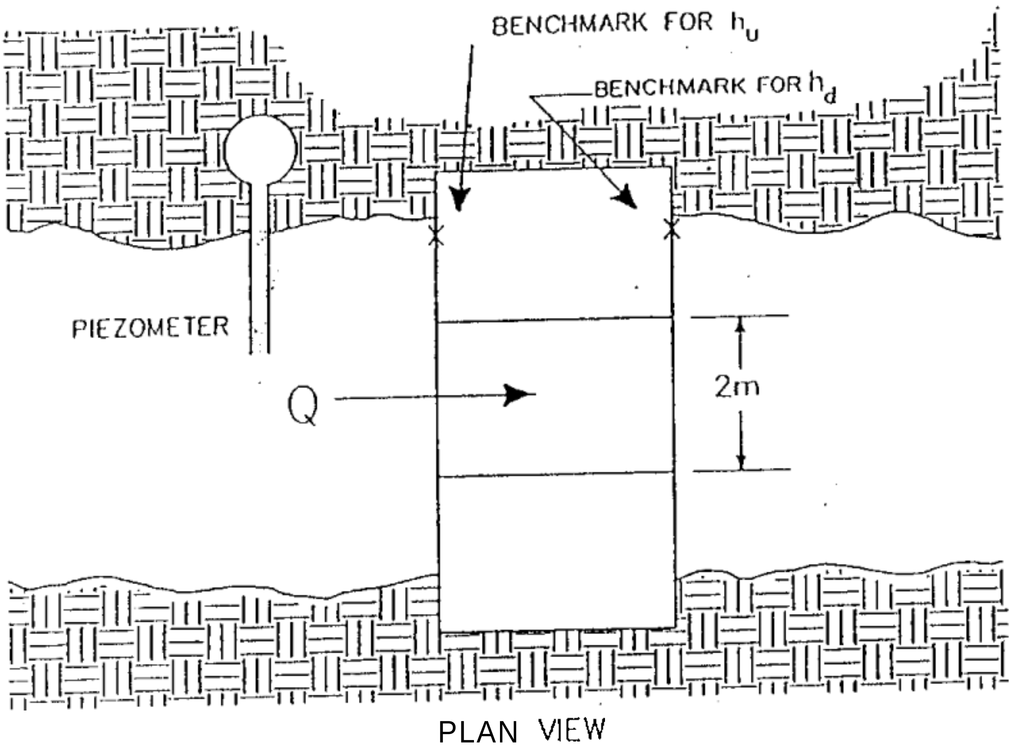
A hypothetical example of developing the field discharge rating for a rectangular open channel constriction is illustrated in Figure 2.9, and the field data is listed in Table 2.1. The discharge rate in the constriction was determined by taking current meter readings at a location upstream, and again 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 2.2) is shown in Figure 2.10 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

$$Q_f = C_f (1.0)^{n_f} = C_f \tag{2.5}$$

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

$$Q_f = 0.72(h_u)_{sw}^{1.53} \tag{2.6}$$



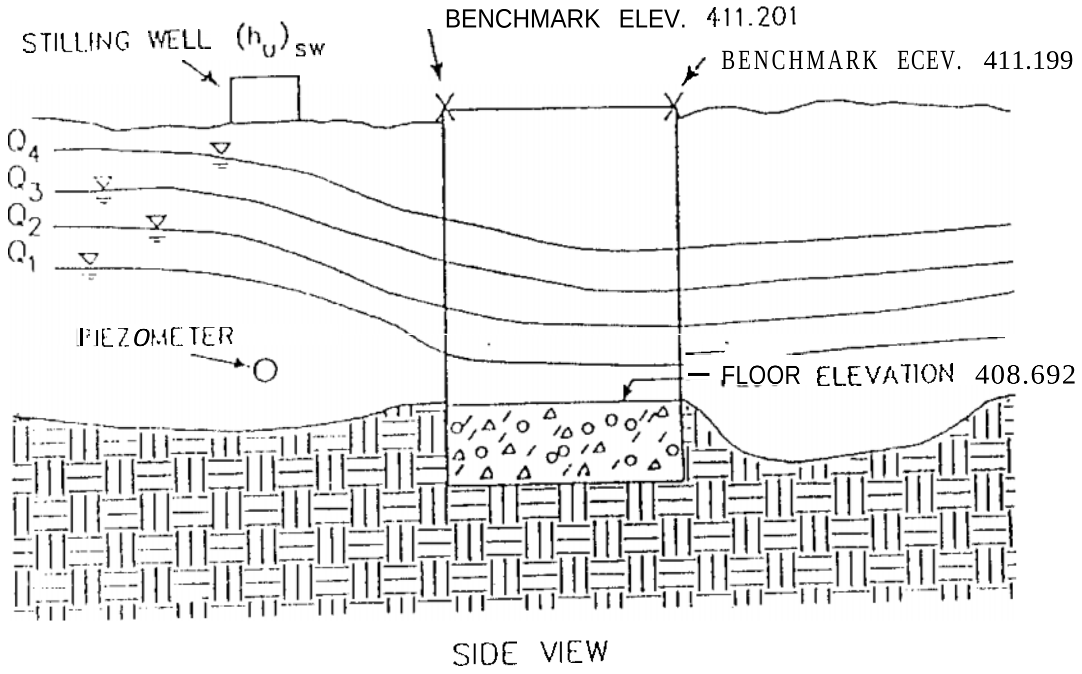


Figure 2.9. Example of free flow water surface profiles for an open channel constriction.

TABLE 2.1. 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

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
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 408.692m. 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.

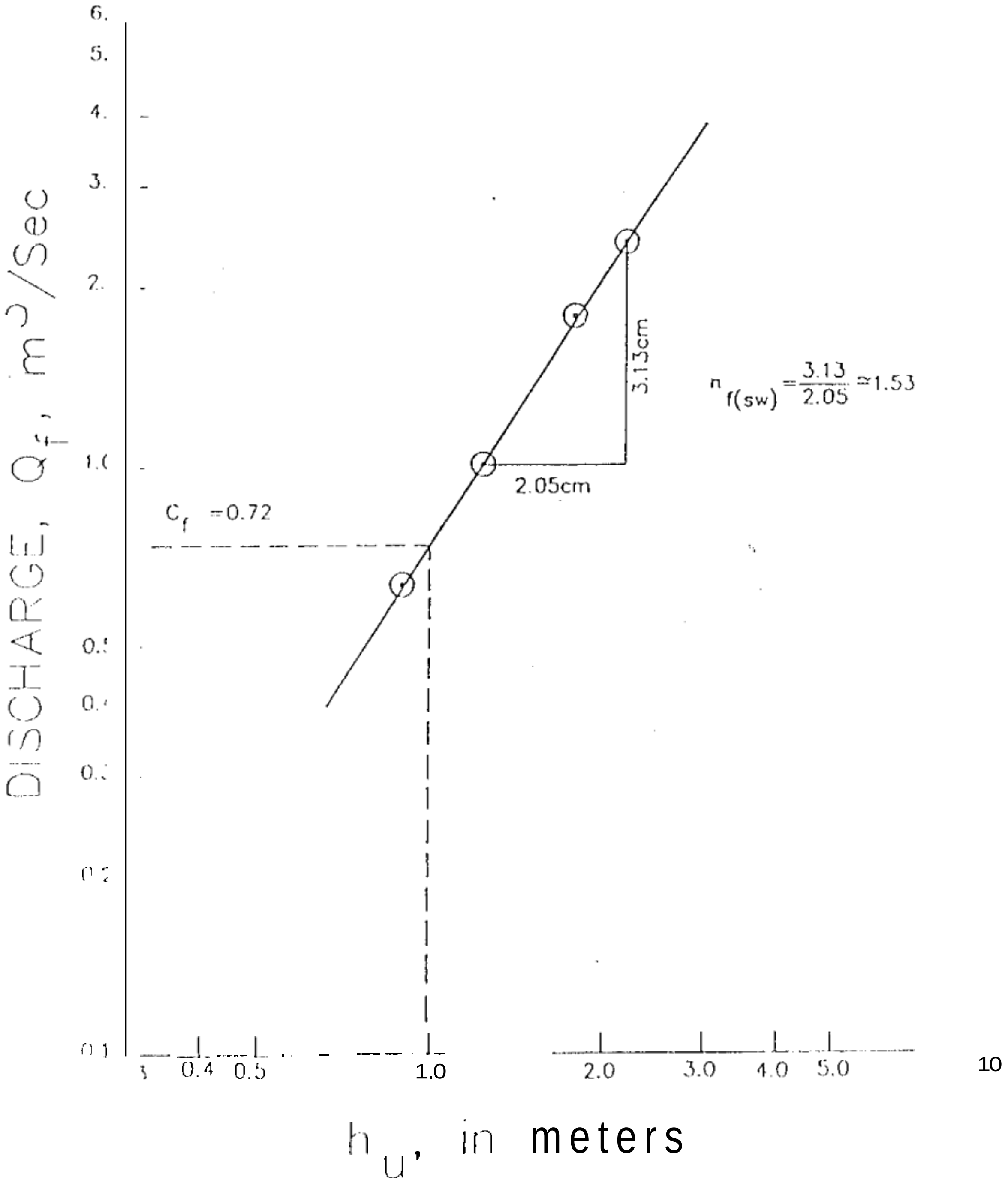


Figure 2.10. Free *flow* discharge rating using the example data.

A comparison of the free flow ratings for the stilling well flow depths and the flow depths along the headwall measured from the benchmark are shown in Figure 2.11. The free flow equation for the flow depths measured below the benchmark is:

$$Q_f = 0.74(h_u)_x^{1.55} \tag{2.7}$$

If a regression analysis is done with the free flow data using the theoretical value of n, = 3/2:

$$Q_f = 0.73(h_u)_{sw}^{1.5} \tag{2.8}$$

$$Q_f = 0.75(h_u)_x^{1.5} \tag{2.9}$$

A comparison of Equations and Equation and are shown in Table . The discharge error resulting from using n, = 3/2 varies from -1.91 percent to +2.87 percent.

2.4.2 Submerqed flow

The form of the submerged flow equation is:

$$Q_s = \frac{C_s(h_u - h_d)^{n_f}}{(-\log S)^{n_s}} \tag{2.10}$$

Where the subscric s denotes submerged flow, so thε Q_s is the submerged flow discharge, C_s is the submerged flow coefficient, and n_s is the submerged flow exoonent. Note that the free flow exponent, n„ is used with the term, h_u - h_d. Consequently, n, is determined from the free flow rating, while C_s and n_s must be evaluated using submerged flow data. The theoretical variation in n_s is between 1.0 and 1.5 (Skogerboe and Hyatt 1967).

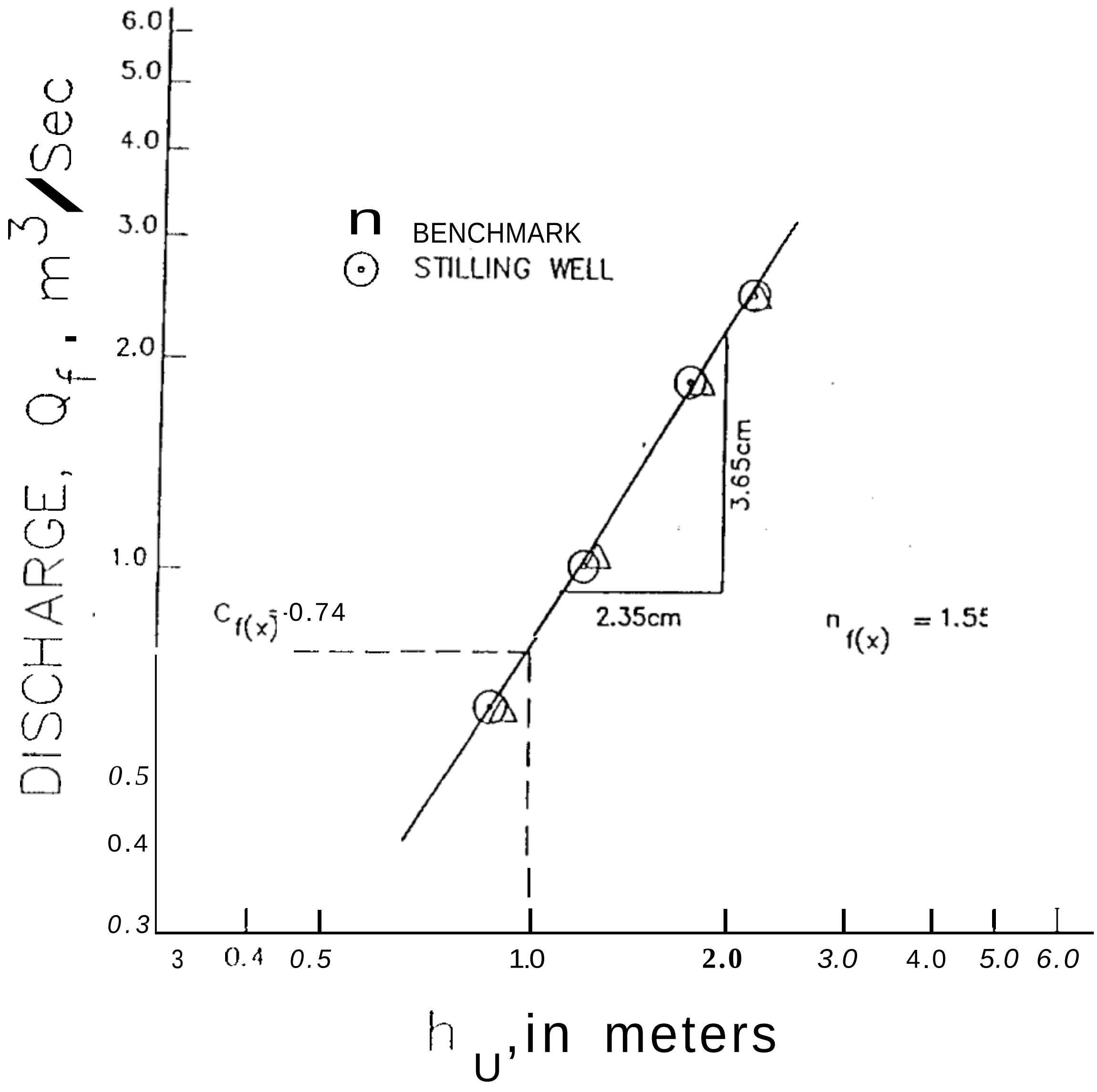


Figure 2.11. Comparison of free flow discharge ratings for stilling well and benchmark flow depths.

TABLE 2.3. Errors in discharge ratings assuming n_f = 1.5 for the example rectangular open channel constriction.

Measured Discharge m^3/s	Measured Depth, $(h_u)_{sw}$ m	$Qf = 0.72(h_u)_{sw}^{1.53}$ m^3/s	Percent Error	$Qf = 0.73h_u)_{sw}^{1.50}$ 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
					-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	$Qf = 0.72(h_u)_x^{1.55}$ m^3/s	Percent Error	$Qf = 0.73h_u)_x^{1.50}$ 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
2.412	2.151				

The value of the free flow exponent, n_f, can also be determined through an iterative procedure from only submerged flow data. An initial value for n_f can be assumed (e.g., the theoretical value), then repeatedly adjusted as the approximating submerged flow equation better fits the field or laboratory data. Such a procedure may be necessary when a constriction is to be calibrated in the field and only operates under submerged flow conditions. This procedure is best applied using a programmable calculator or computer, in which case the solution can be obtained rapidly.

The hypothetical example illustrated in Figure 2.9 will be used to demonstrate the procedure for developing the submerged flow discharge rating. The **two** benchmarks shown in Figure will be used for measuring h_u and h_d. In this case, a constant discharge was diverted into the irrigation channel and a check structure with gates located 120 m downstream was used to continually increase the flow depths. Each time that the gates were changed, it took 2-3 hours for the water surface elevations upstream to stabilize. Thus, it took one day to collect the data for a single discharge rate. The data listed in Table 2.4 was collected in two consecutive days. The data reduction is listed in Table 2.5.

TABLE 2.4. 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
22 Jun 86	0.824	1.390	1.479
22 Jun 86			
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

Qs m^3/s	$(h_u)_x$ rn	$(h_d)_x$ m	S	-log S	$Q_{\Delta h=1}$
0.813	1.061	0.832	0.784	0.1057	7.986
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	1.639	1.597	0.975	0.0110	175.371

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