Open Channel Flow Sessional
(Lab Manual)

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Preface

Flow in rivers and canals are the examples of open channel flow. In water resources engineering, in designing any structure on the river or canal or for flood mitigation process discharge is the primary information needed. Discharge measurement in open channel is different from closed conduit. So the main objective of this course is to teach the student how to measure discharge in an open channel and also to give an idea about some terms and phenomena of an open channel flow which will be used by them in future in practical field.

This Lab manual was prepared with the help of some famous books written by renowned authors on Open Channel Flow, Lab manual of Open Channel Flow Sessional of Bangladesh University of Engineering and Technology (BUET) and some other colleagues motivated us to update the lab manual.

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Experiment No. 1

DETERMINATION OF STATE OF FLOW AND CRITICAL DEPTH IN OPEN CHANNEL

1.1 General

The state of open channel flow is mainly governed by the combined effect of viscous and gravity forces relative to the inertial forces. This experiment mainly deals with determination of the state of flow in an open channel at a particular section. The state of flow is very important, as the flow behavior depends on it. In order to construct different structures in rivers and canals and to predict the river response, the state of flow must be known. The experiment also deals with determination of critical depth, which is very useful in determining the types of flow in practice.

1.2 Theory

1.2.1 State of flow

Depending on the effect of viscosity relative to inertia, the flow may be laminar, turbulent or transitional. The effect of viscosity relative to the inertia is expressed by the Reynolds number, given by

$$Re = \frac{VR}{v} \qquad (1.1)$$

Where, V is the mean velocity of flow, R is the hydraulic radius (=A/P), A is the wetted cross-sectional area, P is the wetted perimeter and v is the kinematic viscosity of water. Kinematic viscosity varies with temperature. The values of kinematic viscosity of water at different temperatures are given in Table 1.1. The value of υ at 20^0C ($=1.003\times10^{-6}$ m^2/s) is normally used to compute the Reynolds number of open channel flow.

When, $Re < 500$ the flow is laminar

$500 \le Re \le 12{,}500$ the flow is transitional

$Re > 12{,}500$ the flow is turbulent.

Most open channel flows including those in rivers and canals are turbulent. The Reynolds number of most open channel flows is high, of the order of 10^6, indicating that the viscous forces are weak relative to the inertia forces and do not play a significant role in determining the flow behavior.

When the flow is dominated by the gravity, then the type of flow can be identified by a dimensionless number, known as Froude Number. Given by

$$Fr = \frac{V}{\sqrt{gD}} \qquad (1.2)$$

Where, V is the mean velocity of flow, D is the hydraulic depth (= A/T), A is the cross- sectional area, T is the top width and g is the acceleration due to gravity (= 9.81 m/s^2).Depending on the effect of gravity relative to inertia, the flow may be subcritical, critical or supercritical-

When, Fr<1 the flow is subcritical
Fr =1 the flow is critical
Fr>1 the flow is supercritical

The flow in most rivers and canals is subcritical. Supercritical flow normally occurs downstream of a sluice gate and at the foot of drops and spillways. The Froude number of open channel flow varies over a wide range covering both subcritical and supercritical flows and the state or behavior of open channel flow is primarily governed by the gravity force relative to the inertia force. Therefore, the Froude number is the most important parameter to indicate the state or behavior of open channel flow.

Depending on the numerical values of Reynolds and Froude numbers, the following four states of flow are possible in an open channel:

i) Subcritical laminar Fr<1, Re<500
ii) Supercritical laminar Fr>1, Re<500
iii) Subcritical turbulent Fr<1, Re>12,500
iv) Supercritical turbulent Fr>1, Re>12,500

The first two states of flow, subcritical laminar and supercritical laminar, are not commonly encountered in applied open channel hydraulics. Since the flow is generally turbulent is open channel, the last two states of flow are encountered is engineering problems.

Table 1.1 Kinematic viscosity of water at different temperatures

Temperature, ^{0}C	Kinematic viscosity, $v \times 10^{-6}$, m^2/s
0	1.781
5	1.518
10	1.307
15	1.139
20	1.003
25	0.890
30	0.798
40	0.653
50	0.547
60	0.466
70	0.404
80	0.354
90	0.315
100	0.282

1.2.2 Critical depth

Flow in an open channel is critical when the Froude number of the flow is equal to unity.

Critical flow in a channel depends on the discharge and the geometry of channel section. For a rectangular section, the critical depth is given by

$$y_c = \sqrt[3]{\frac{Q^2}{gB^2}} \qquad (1.3)$$

Where, y_c is the critical depth, Q is the discharge and B is the width of the channel.

When the depth is greater than the critical depth, the flow is subcritical. When the depth is less than the critical depth, the flow is supercritical.

1.3 Objectives of the experiment

1) To measure water depth both upstream and downstream of a weir.
2) To determine the Reynolds number (Re) and the Froude number (Fr).
3) To determine the state of flow.
4) To determine critical depth (y_c).
5) To observe the subcritical and the supercritical flows.

1.4 Experimental setup

To develop different states of flow, the following laboratory setup is used.

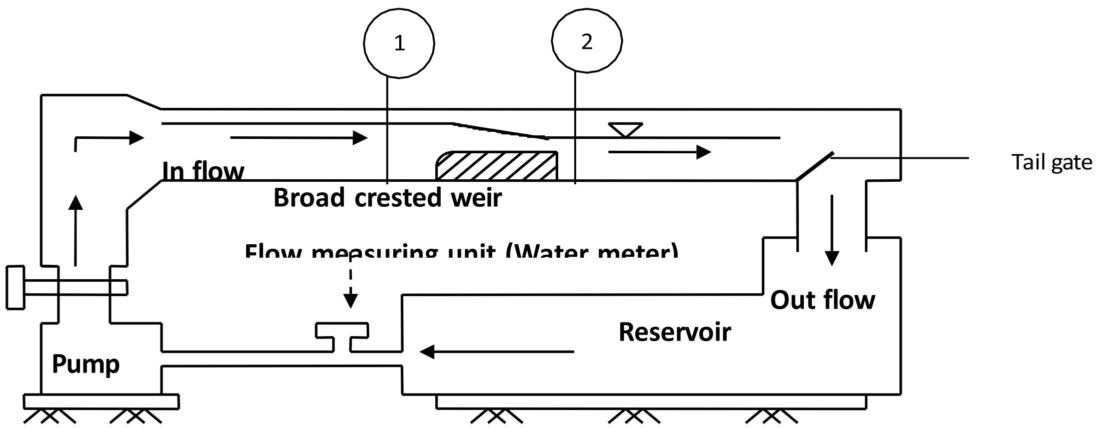


Fig. 1.1 Schematic diagram of experimental setup

1.5 Procedure

i) Measure the depth of flow at sections 1 and 2 by a point gage.
ii) Take the reading of discharge.
iii) Calculate the velocity at both the sections.
iv) Calculate Re and Fr for both the sections using Eqs. (1.1) and (1.2) and determine the state of flow.
v) Calculate the critical depth y_c using Eq. (1.3).

1.6 Assignment

1. Why the state of flow and the critical depth of a river or canal need to be determined?
2. How can you determine that the flow in a river is subcritical, critical or supercritical without taking any measurement?
3. State why the Froude number is more significant than the Reynolds number to determine the state of open channel flow.

DATA SHEET

Experiment Name :
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Discharge, Q = cm^3/s Flume width, B = cm

Critical depth, y_c = cm Temperature = °C

Kinematic viscosity, v = cm^2/s

Section	Depth of flow y (cm)		Area A=B y (cm^2)	Perimeter P=(B+2y) (cm)	Hydraulic Radius R=A/P (cm)	Hydraulic Depth D=A/T (cm)	Velocity V=Q/(By) (cm/s)	Froude number Fr	Reynolds number Re	State of flow
1										
2										

Experiment No. 2

FLOW OVER A BROAD-CRESTED WEIR

2.1 General

A broad-crested weir is an overflow structure with a truly level and horizontal crest. It is widely used in irrigation canals for the purpose of flow measurement as it is rugged and can stand up well under field conditions. But practically some problems arise with the weir, as there exists a dead water zone at the upstream of the weir and the head loss is more comparable to other devices. By virtue of being a critical depth meter, the broad crested weir has the advantage that it operates effectively with higher downstream water levels than a sharp crested weir. This experiment deals with measurement of discharge using the broad- crested weir and also calibration of the weir

2.2 Theory

2.2.1 Description of the weir

The broad-crested weir has a definite crest length in the direction of flow. In order to maintain a hydrostatic pressure distribution above the weir crest, i.e. to maintain the streamlines straight and parallel, the length of the weir is designed such that $0.07 \leq H_1/L \leq 0.50$ where H_1 is the head above the crest and L is the length of the weir (Fig. 2.1). Under this condition, critical flow occurs over the weir at section A-A and the weir provides an excellent means of measuring discharge in open channels based on the principle of critical flow. The upstream corner of the weir is rounded in such a manner that flow separation does not occur.

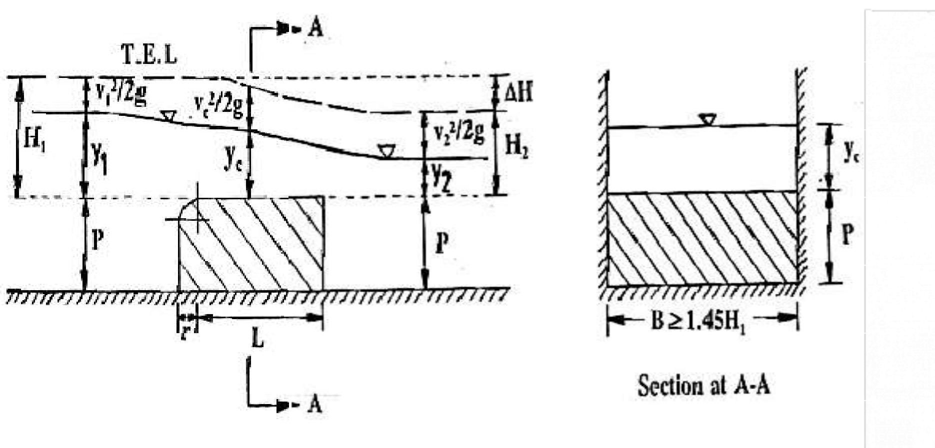


Fig. 2.1 Flow over a broad-crested weir

2.2.2 Theoretical discharge

Consider a rectangular broad-crested weir shown in Fig. 2.1. Based on the principle of critical flow (Fr = 1), the theoretical discharge Q_t over the weir is given by

$$Q_t = \sqrt{g} B y^{1.5} \qquad (2.1)$$

Where, B is the width of the weir, y_c is the critical depth and g is the acceleration due to gravity.

The usual difficulty in using Eq. (2.1) for computing discharge lies in locating the critical flow section and measuring the critical depth accurately. This difficulty is, however, overcome by measuring the depth of flow upstream of the weir where the flow is not affected

by the presence of the weir. With reference to Fig. 2.1, neglecting the frictional losses and applying the energy equation between the upstream section and the critical flow section, we obtain

$$H_1 = y_c + \frac{V_c^2}{2g}$$

Where, Vc is the critical velocity. Since at the critical state of flow, the velocity head is equal to one-half of the hydraulic depth (D) and for a rectangular channel D = y, the above equation gives

$$H_1 = y_c + \frac{V_c^2}{2g} = y_c + \frac{D_c}{2} = y_c + \frac{y_c}{2} = \frac{3}{2} y_c$$

so that

$$y_c = \frac{2}{3} H_1$$

and Eq.(2.1) becomes

$$Q_t = (2/3)^{1.5} \sqrt{g}\, BH_1^{1.5} \qquad (2.2)$$

2.2.3 Coefficient of discharge

Due to the assumptions made in the derivation of the governing equation, the theoretical discharge and the actual discharge always vary from each other. So, the coefficient of discharge is introduced. If Q_a is the actual discharge, then the coefficient of discharge, C_d, is given by

$$C_d = Q_a/Q_t \qquad (2.3)$$

Then

$$Q_a = C_d (2/3)^{1.5} \sqrt{g}\, BH_1^{1.5} \qquad (2.4)$$

The coefficient of discharge for a broad-crested weir depends on the length of the weir and whether the upstream corner of the weir is rounded or not. Normally, in a field installation it is not possible to measure the energy head H_1 directly and therefore the discharge is related to the upstream depth of flow over the crest, y_1, by the equation

$$Q_a = C_v C_d (2/3)^{1.5} \sqrt{g}\, By_1^{1.5} \qquad (2.5)$$

Where, C_v is the correction coefficient for neglecting the velocity head in the approach channel. Generally the effect of C_v is considered in C_d and finally the governing equation becomes

$$Q_a = C_d (2/3)^{1.5} \sqrt{g}\, By_1^{1.5} \qquad (2.6)$$

and

$$Q_t = (2/3)^{1.5} \sqrt{g}\, By_1^{1.5} \qquad (2.7)$$

2.1.1 Calibration

Calibration is the act of obtaining a definite relationship for the measuring device using the sets of known data. For a broad-crested weir, the Eq.2.7 can be expressed as a relationship between the upstream depth and the discharge, i.e. $Q = ky_1^n$. This relation is known as stage discharge equation for discharge measurement. So calibration deals with determination of coefficient k and exponent n using the sets of experimental data and develop the equation $Q = ky_1^n$ so that the equation can be useful for flow estimation. The plotting of the calibrated equation is known as calibration curve for the measuring device. There are two different ways to develop a calibration equation. These are

i) Plotting best fit line by eye estimation.
ii) Developing best fit line by regression.

2.3 Objectives of the experiment

i) To determine the theoretical discharge of the weir.
ii) To measure the actual discharge and hence to find out the coefficient of discharge.
iii) To calibrate the weir.

2.4 Experimental setup

The experimental setup for this experiment is given below.

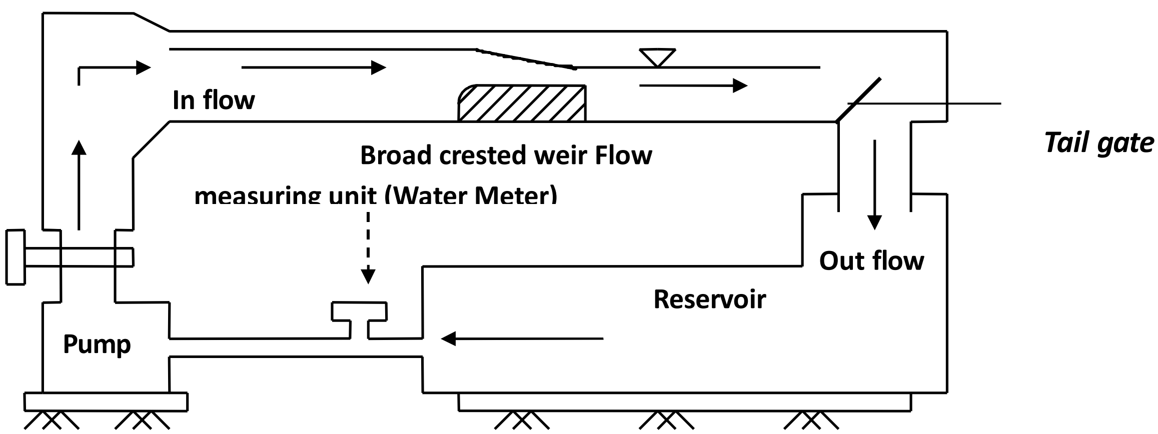


Fig. 2.2 Setup for flow over a broad-crested weir

2.5 Procedure

To determine the theoretical and the actual discharges and the coefficient of discharge

i) Measure the upstream water level over the weir y_1 at three points, then find the average depth and determine the theoretical discharge using Eq. (2.7).
ii) Take the reading of actual discharge and hence find the coefficient of discharge using Eq. (2.3).

2.6 Shape of Q vs y graph

In a plain graph paper the plot of $Q = ky^n$ is non-linear. But in a log log paper $Q = ky^n$ plots as a straight line since $\log Q = \log k + n \log y$ which is an equation of a straight line (of the form $y = mx + c$).

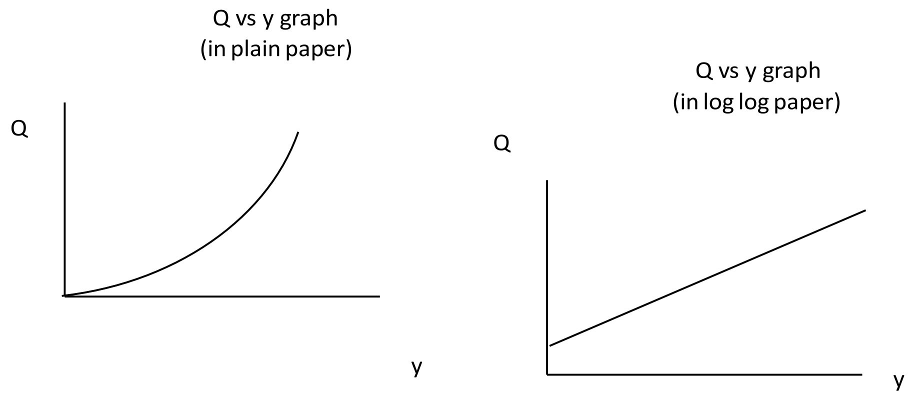


Fig. 2.3: Q (actual discharge) vs y(upstream depth of water above weir) graph

2.7 Assignment

1. What are the advantage, disadvantage and use of a broad-crested weir?
2. Why is it necessary to calibrate a broad-crested weir?
3. A broad-crested weir is designed so that $0.07 \leq H_1/L \leq 0.50$. What do the upper and lower limits of H_1/L signify?

DATA SHEET

Experiment Name :
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Length of the weir, L = cm Width of the weir (or flume), B = cm

Depth of water over weir crest (cm)		Theoretical discharge Q_t (cm^3/s)	Actual discharge Q_a (cm^3/s)	Coefficient of discharge C_d

Calibration of the weir

i) By eye estimation (should be done by students having odd student number)

Actual discharge ,Q_a (cm^3/s)	Depth of water above weir crest, y_1 (cm)

Experiment No. 3

FLOW THROUGH A VENTURI FLUME

3.1 General

Although weirs are an effective method of artificially creating a critical section at which the flow rate can be determined, a weir installation has at least two disadvantages. First, the use of weirs results in relatively high head loss. Second, most weirs create a dead water zone upstream of it which can serve as a settling basin for sediment and other debris present in the flow. Both of these disadvantages can be overcome with an open flume having a contraction in width which is sufficient to cause the flow to pass through a critical depth. Venturi flume is an open flume used widely in irrigation canals for measuring discharge But Venturi flumes have a disadvantage that there is relatively small head difference between the upstream section and the critical section, especially at low Froude numbers. This experiment deals with measurement of discharge using a Venturi flume and also calibration of the flume.

3.2 Theory

3.2.1 Description of the flume

Venturi flume has a converging section, a throat section and a diverging section. The bed level is kept horizontal. The streamlines run parallel to each other at least over a short distance upstream of the flume.

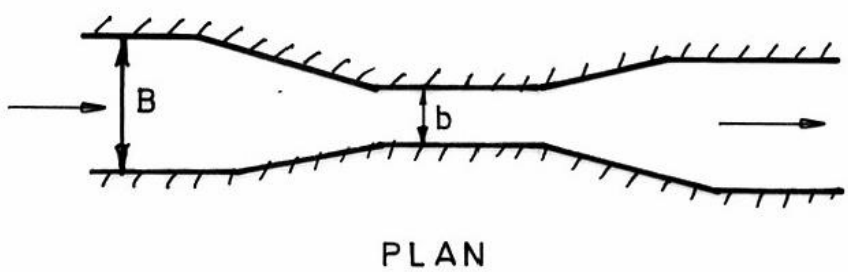


Fig. 3.1 Flow through a Venturi flume

3.2.2 Theoretical discharge at free flow condition

Considering that critical flow occurs at the throat section of the flume, the theoretical discharge at free flow is given by

$Q_{tf} = AV = A_c V_c$

Where, A_c and V_c are the area and velocity at the critical flow section of the flume. At the critical state of flow

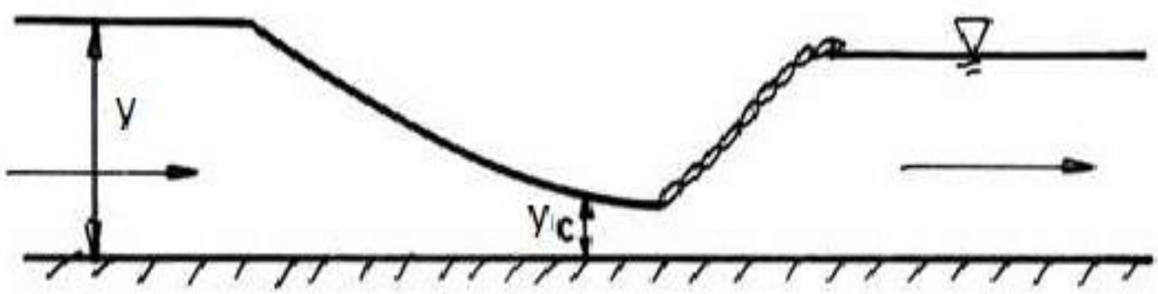
$$Fr = 1$$

or

$$\frac{V_c^2}{gD_c} = 1$$

or

$$V_c = \sqrt{gD_c}$$



Now, for a rectangular flume, $A_c = by_c$ and $D_c = y_c$, where b is the width of the Venturi flume at the throat section. Hence, the theoretical discharge at free flow given by

$$Q_{tf} = A_c V_c = by_c \sqrt{gy_c} \quad (3.1)$$

For a rectangular channel at critical condition there exists a relationship between total head and the critical depth as

$$H = \frac{3}{2} y_c$$

Hence, putting

$$y_c = \frac{2}{3} H$$

in Eq.(3.1), we obtain

$$Q_{tf} = (2/3)^{1.5} \sqrt{g}\, bH^{1.5} \quad (3.2)$$

Where, H is the head measured sufficiently upstream of the flume.

3.2.2 Theoretical discharge at submerged flow condition

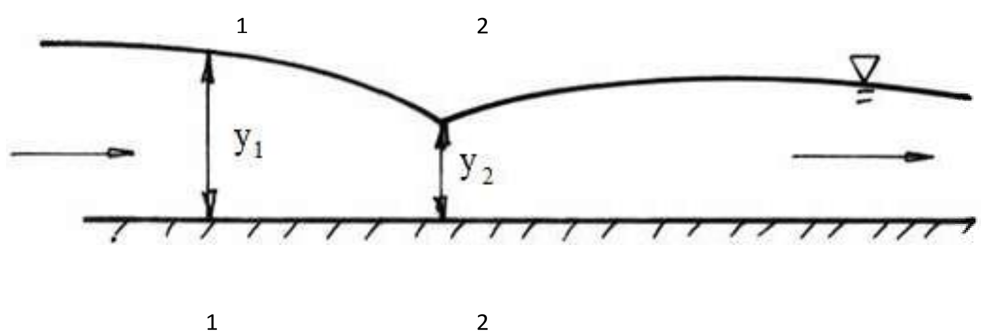


Fig. 3.3 Submerged flow condition

No critical flow section exists at submerged flow condition. Considering Fig 3.3, applying the energy equation between sections 1and 2 neglecting frictional losses, we obtain

No critical flow section exists at submerged flow condition. Considering Fig 3.3, applying the energy equation between sections 1and 2 neglecting frictional losses, we obtain

$$y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g}$$

which gives

$$V_2^2 (1 - \frac{V_1^2}{V_2^2}) = 2g(y_1 - y_2)$$

If A and a are the wetted areas at sections 1 and 2, respectively, then using the continuity equation

$AV_1 = aV_2$

we obtain

$$\frac{V_1}{V_2} = \frac{a}{A}$$

If we assume

$$M = \frac{V_1}{V_2} = \frac{a}{A}$$

Then

$$V_2^2 (1 - M^2) = 2g(y_1 - y_2)$$

so that

$$V_2 = \sqrt{\frac{2g(y_1 - y_2)}{1 - M^2}}$$

Hence, the theoretical discharge at submerged flow condition

$$Q_{ts} = aV_2 = a\sqrt{\frac{2g(y_1 - y_2)}{1 - M^2}} \qquad (3.3)$$

3.2.4 Coefficient of discharge

Due to the assumptions made in the derivation of the governing equation, the theoretical discharge and the actual discharge always vary from each other. So, the coefficient of discharge C_d is introduced. If Q_a is the actual discharge, then the coefficient of discharge at free flow condition, C_{df}, is given by

$$C_{df} = Q_a/Q_{tf} \qquad (3.4)$$

Normally, in a field installation it is not possible to measure the energy head H directly and therefore the discharge is related to the upstream depth of flow y_1 by the equation

$$Q_a = C_v C_{df} (2/3)^{1.5} \sqrt{g}\, b y_1^{1.5} \qquad (3.5)$$

Where, C_v is the correction coefficient for neglecting the velocity head in the approach channel. Generally the effect of C_v is considered in C_d and finally the governing equations become

$$Q_a = C_{df} (2/3)^{1.5} \sqrt{g}\, b y_1^{1.5} \qquad (3.6)$$

and

$$Q_{tf} = (2/3)^{1.5} \sqrt{g}\, b y_1^{1.5} \qquad (3.7)$$

The coefficient of discharge at submerged flow condition, C_{ds} is given by

$C_{ds} = Q_a/Q_{ts}$ (3.8)

3.2.5 Calibration

Calibration is the act of obtaining a definite relationship for the measuring device using the sets of known data. For a broad-crested weir there is a definite relationship between the upstream depth and the discharge, i.e. $Q = ky_1^n$. This relation is known as the calibration equation for the device. So calibration deals with determination of k and n and develop the equation $Q = ky_1^n$. The plotting of the calibration equation is known as calibration curve. There are two different ways to develop a calibration equation. These are

i) Plotting best fit line by eye estimation.
ii) Developing best fit line by regression.

3.3 Objectives of the experiment

i) To determine the theoretical discharge of the flume at free flow and submerged flow conditions.
ii) To measure the actual discharge and hence to find out the coefficient of discharge at free flow and submerged flow conditions.
iii) To calibrate the flume.

3.4 Experimental setup

The experimental setup for this experiment is given below.

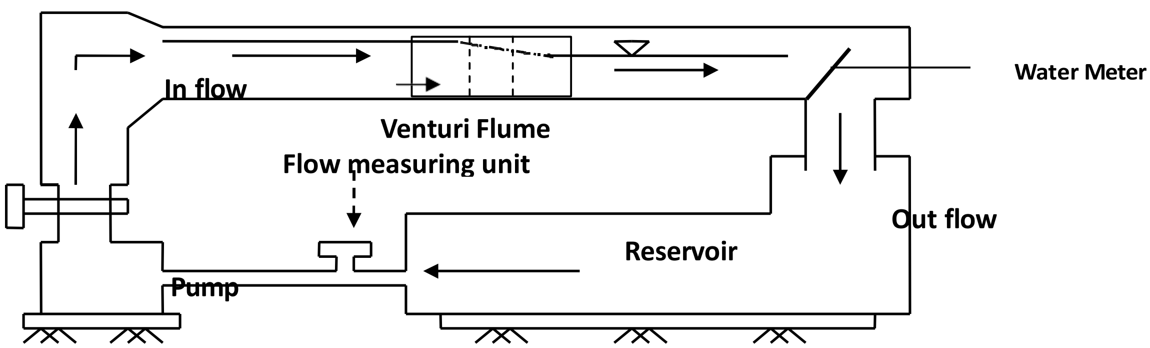


Fig. 3.4 Setup for flow through a Venturi flume

3.5 Procedure

To determine the theoretical and the actual discharges and the coefficient of discharge at free flow condition

i) Measure the depth of flow sufficiently upstream of the flume and determine the theoretical discharge using Eq.(3.7).
ii) Take the reading of actual discharge and hence find the coefficient of discharge using Eq. (3.4).

3.6 Shape of Q vs y graph

In a plain graph paper the plot of $Q = ky^n$ is non-linear. But in a log log paper $Q = ky^n$ plots as a straight line since $\log Q = \log k + n \log y$ which is an equation of a straight line (of the form $y = mx + c$).

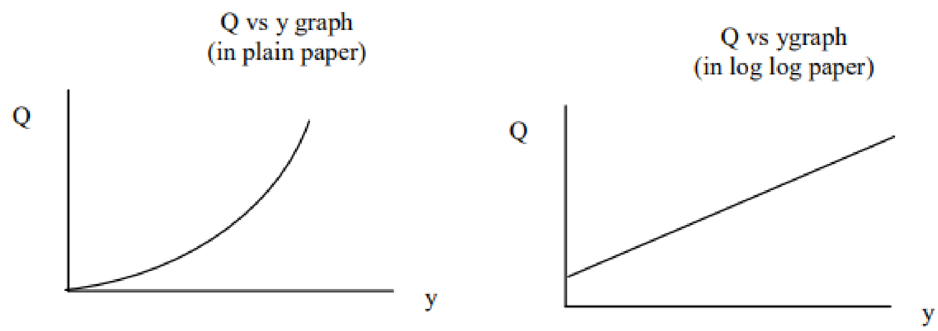


Fig. 3.5: Q (actual discharge) vs y(upstream depth of water) graph

3.7 Assignment

1. What are the advantage, disadvantage and use of a Venturi flume?
2. What is the difference between free and submerged flows? How can you produce submerged flow in a laboratory flume? What is the effect of submergence on the flow?

DATA SHEET

Experiment Name :
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Channel width, B = cm Throat width, b = cm

Actual discharge Q_a (cm^3/s)	Free flow condition			Submerged flow condition				
	y_1 (cm)	Q_{tf} (cm^3/s)	C_{df}	y_1 (cm)	y_2 (cm)	M	Q_{ts} (cm^3/s)	C_{ds}

Calibration of the flume:

i) By eye estimation (should be done by students having odd student number)

Actual discharge ,Q_a (cm^3/s)	Depth of water at upstream, y_1 (cm)

Experiment No. 4

FLOW THROUGH A PARSHALL FLUME

4.1 General

The problem with a Venturi flume is that there is a relatively small head difference between the upstream section and the critical section. This problem can be overcome by designing a flume which has a contracted throat section in which critical flow occurs followed by a short length of supercritical flow and a hydraulic jump at the exit section. A flume of this type was designed by R.L. Parshall and is widely known as the Parshall flume. Practically this type of flume is used in small irrigation canals for flow measurement purpose. It is better than all other devices discussed before as it is more accurate, can withstand a relatively high degree of submergence over a wide range of backwater condition downstream from the structure and it acts as a self-cleaning device due to the fact that high velocity washes out the debris and sediments present in the flow. However, when a heavy burden of erosion debris is present in the stream, the Parshall flume becomes invalid like weir, because deposition of debris will produce undesirable result. Another problem which arises with this flume is that the fabrication is complicated and also fabrication should be done as per requirement. This experiment deals with the measurement of discharge using a Parshall flume.

4.2 Theory

4.2.1 Description of the flume

A Parshall flume consists of a broad flat converging section, a narrow downward sloping throat section and an upward sloping diverging section. The reason of downward sloping throat section is to increase the head difference between the upstream section and the critical section. The upward slope in the diverging section is given to produce a high tailwater depth which reduces the length of the supercritical flow region.

4.2.2 Theoretical discharge

The Parshall flume is a calibrated device i.e. there exists a definite depth-discharge relationship for the flume. So, analytic determination of theoretical discharge is not required for this flume. Similar to other types of device, the discharge through a Parshall flume is given by

$$Q_t = KH_a^n \qquad (4.1)$$

where K is a constant which depends on the system of units used, n is an exponent and H_a is the upstream depth measured at the location shown in Fig. 4.1.

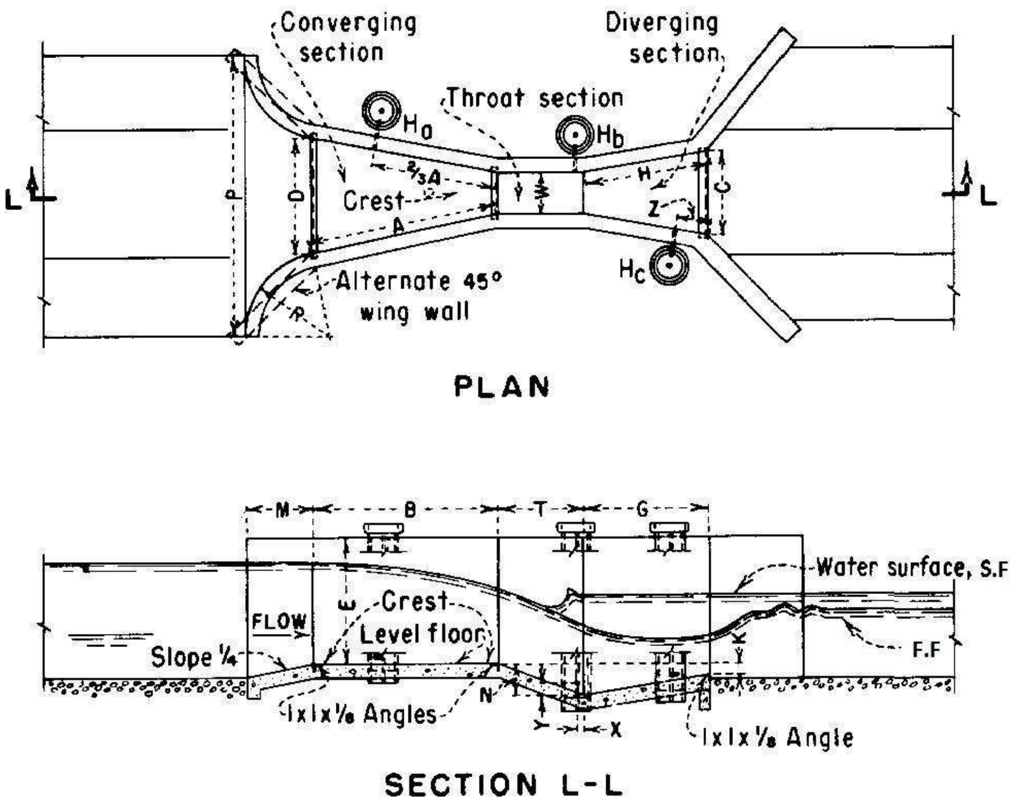


Fig. 4.1 Flow through Parshall flume

The values of K and n depend on the throat width and are given in Table 4.1.

According to this table, for free flow condition, the depth-discharge relationship of a Parshall flume of $6''$ throat width which is normally used in the laboratory, as calibrated empirically, is given by

$$Q_{tf} = 2.06\ H_a^{1.58}$$

Where, Q_{tf} is in ft^3/s and H_a is in ft.

Table 4.1 Values of K and n for different throat widths

Throat width	Equation
3″	$Q = 0.992\ H_a^{1.547}$
6″	$Q = 2.06\ H_a^{1.58}$
9″	$Q = 3.07\ H_a^{1.53}$
12″ to 8′	$Q = 4WH^{1.552W^{0.026}}$
10′ to 50′	$Q = (3.6875W + 2.5)\ H_a$

In the above equation, Q is the free discharge in cfs, W is the width of the throat in ft and H_a is the gage reading in ft.

4.2.3 Coefficient of discharge

The actual discharge always varies with the theoretical discharge of the flume. So the introduction of a coefficient of discharge is necessary. If the actual discharge Qa is measured by the water meter, the coefficient of discharge is given by

$C_{df} = Q_a/Q_{tf}$ (at free flow condition) (4.3)

$C_{ds} = Q_a/Q_{ts}$ (at submerged flow condition) (4.4)

4.2.4 Percentage of submergence

The percentage of submergence for the Parshall flume is given by $100H_b/H_a$, where H_b is the downstream depth measured from the invert datum. When the percentage of submergence exceeds 0.6, the discharge through the Parshall flume is reduced. The discharge of Parshall flume then determined from figure 4.3.

4.3 Objectives of the experiment

i. To determine the theoretical discharge at the free flow condition.
ii. To determine the theoretical discharge at the submerged flow condition.
iii. To determine the coefficient of discharge C_d for both the free and submerged flow conditions.
iv. To verify the values of K and n.

4.4 Experiment setup

The experiment setup is given below.

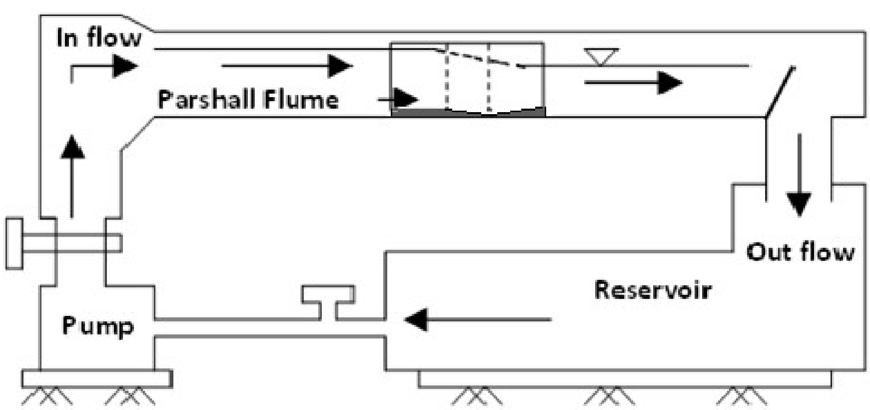
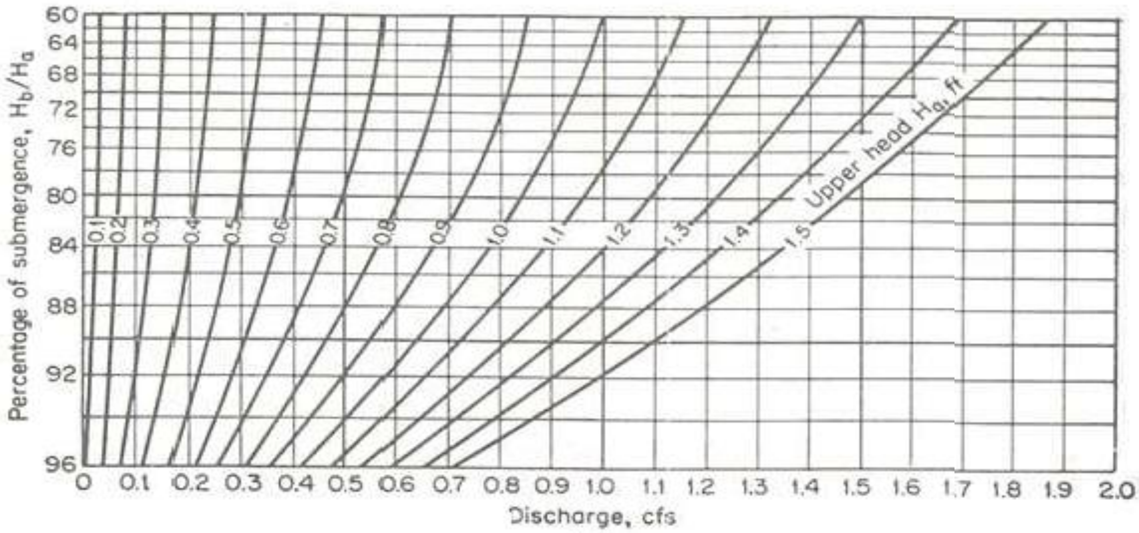
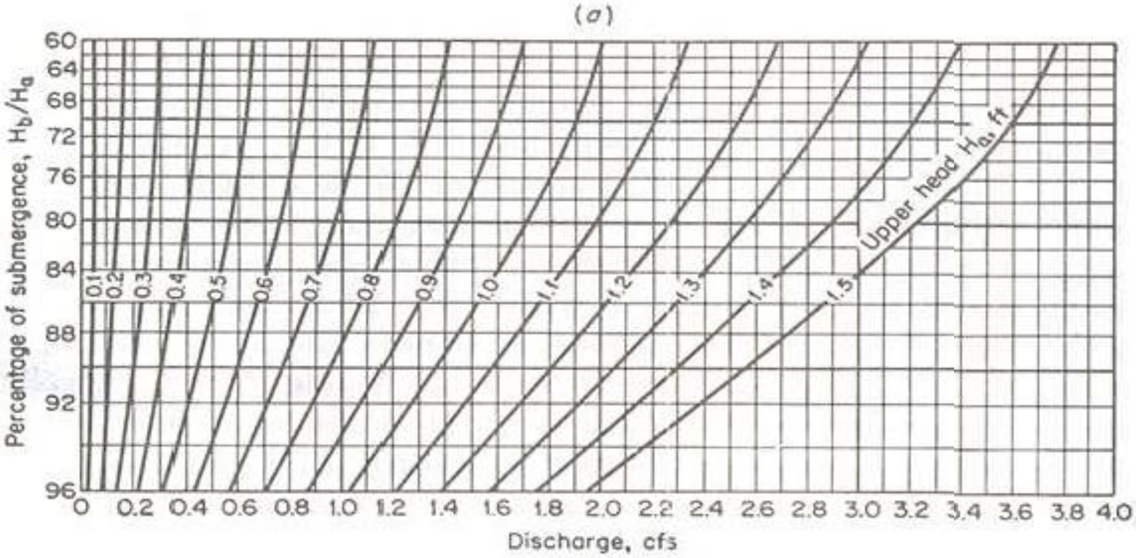


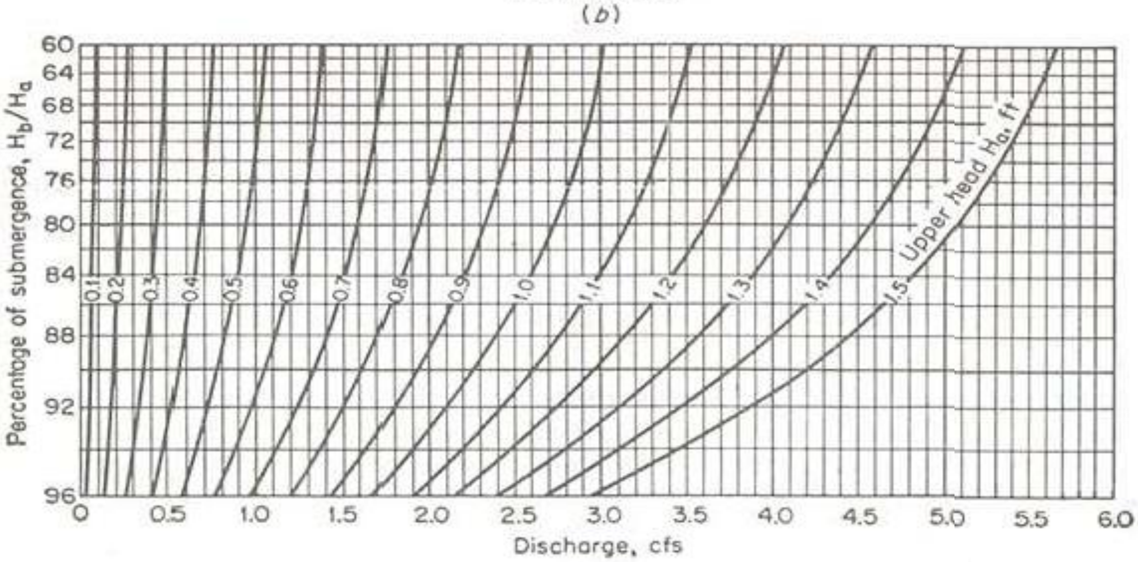
Fig.4.2 Setup for flow through a Parshall flume


Percentage of submergence, H_b/H_a
Discharge, cfs
Upper head H_a, ft
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5

(a)


Percentage of submergence, H_b/H_a
Discharge, cfs
Upper head H_a, ft
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5

(b)


Percentage of submergence, H_b/H_a
Discharge, cfs
Upper head H_a, ft
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5

(c)

4.5 Procedure

To determine the theoretical discharge at the free flow condition

i) Measure the head H_a.

ii) Compute Q_{tf} using Eq.(4.2).

To determine the theoretical discharge at the submerged flow condition

i) Measure the heads H_a and H_b.

ii) Compute Q_{tf} using Eq.(4.2).

iii) Find the % of submergence, $100H_b/H_a$.

iv) If the % of submergence exceeds 60%, find the discharge from Fig. 4.3.

To determine the coefficient of discharge, measure the actual discharge from the water meter and calculate C_{df} and C_{ds} using Eqs.(4.3) and (4.4).

To verify the values of K and n

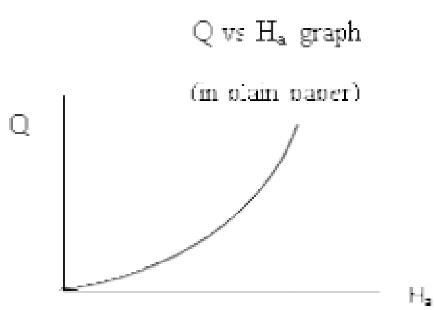
i) Plot Q_a vs H_a in a log log paper.

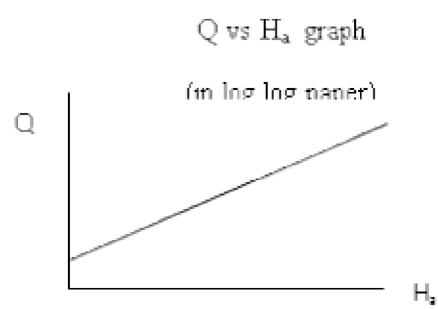
ii) Slope of the plotted line gives the value of n.

iii) Using the value of n for any set of values of Q_a and H_a, find K using Eq.(4.1).

4.6 Shape of Q vs H_a graph

In a plain graph paper the plot of $Q = KH_a^{\ n}$ is a non-linear. But in a log log paper $Q = KH_a^{\ n}$ plots as a straight line since $\log Q = \log K + n \log H_a$ which is the equation of a straight line (of the form y = mx + c).





4.7 Assignment

1. What are the advantage, disadvantage and use of a Parshall flume?

2. Why a downward narrow section and an upward diverging section are provided in a Parshall flume?

DATA SHEET

Experiment Name : Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Throat width, W = in Actual discharge, Q_a = ft^3/s

Free flow condition			Submerged flow condition				
H_a (ft)	Q_{tf} (ft^3/s)	C_{df}	H_a (ft)	H_b (ft)	% Submergence $=(H_b/H_a)$ *100%	Q_{ts} (ft^3/s)	C_{ds}

Verification of K and n

Actual discharge ,Q_a (ft^3/s)	H_a (ft)

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