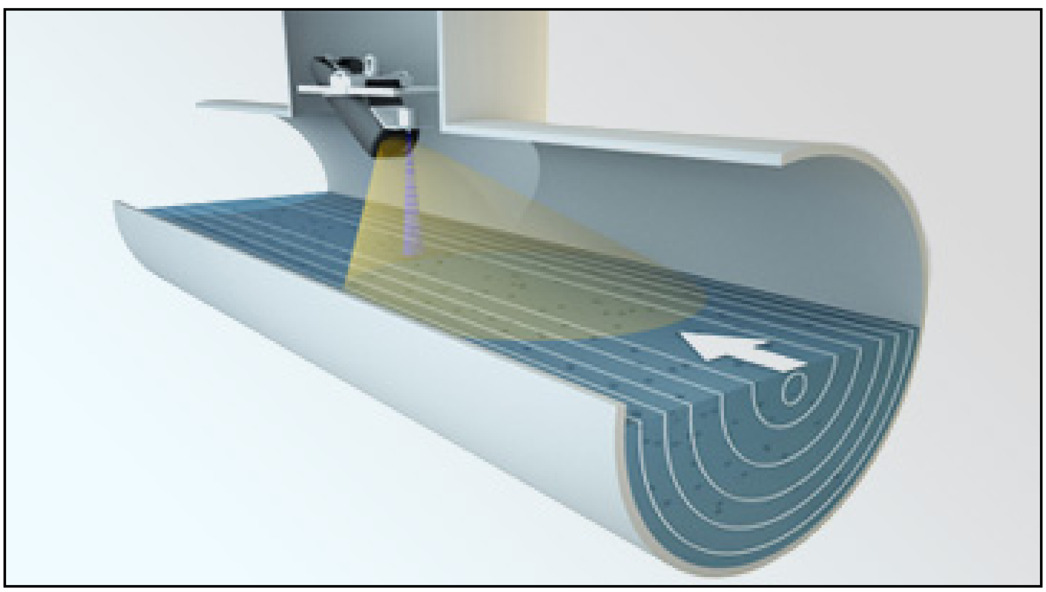
CE 374
Water Resources Engineering Sessional -I
(Lab Manual)



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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

1.2.1State 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

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

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 ν 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 (=1.003×10^{-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, ^0C	Kinematic viscosity, $\nu \times 10^{-6}$, 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 experimen

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.

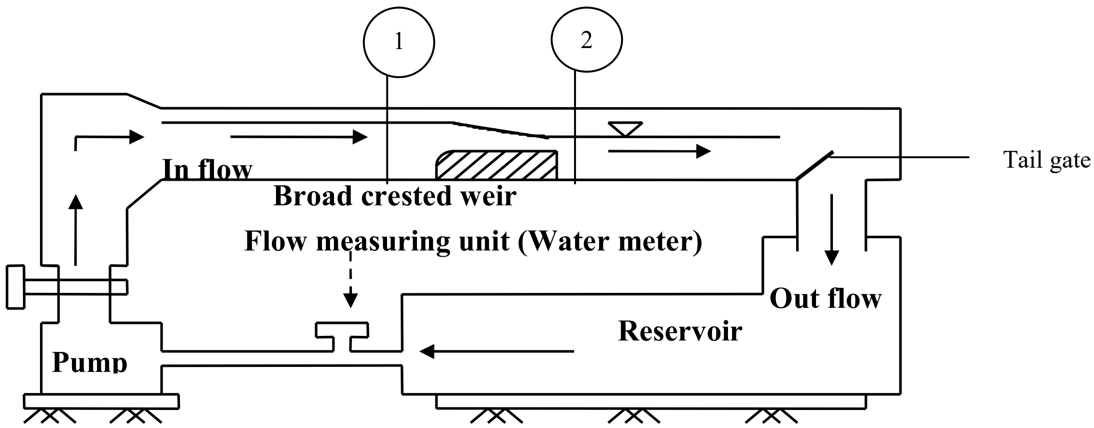


Fig. 1.1 Schematic diagram of experimental setup

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, ν = 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										

Course Teacher :
Designation :

Signature

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.

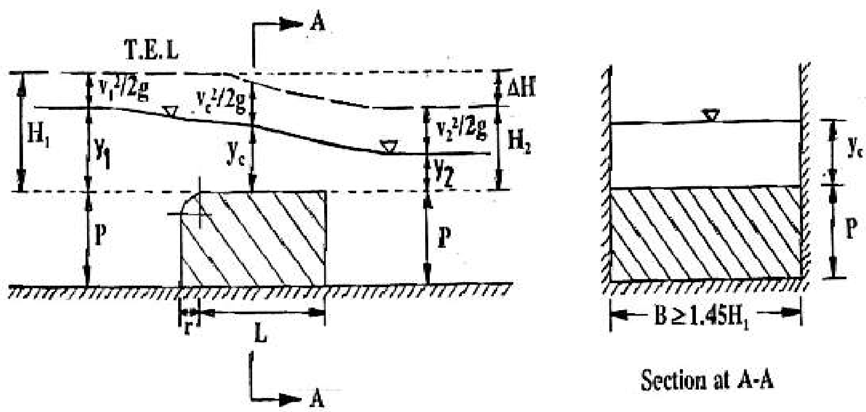


Fig. 2.1 Flow over a broad-crested weir

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

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

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, V_c 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

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

2.2.4 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.

By eye estimation

As $\log Q = \log k + n \log y_1$, so if Q and y_1 are plotted in a log log paper, the line will represent a straight line. Different sets of Q and y_1 are plotted in a log log paper keeping y_1 along the x axis and Q along the y axis. The best fit line is drawn by eye estimation. The slope of the line gives the value of n. Then for any value of y the corresponding value of Q is found from the best fit line. Using these values of y, Q and n, the value of k can be found form the equation $Q = ky_1^n$.

By regression

From $Q = ky_1^n$, we have

$$\log Q = \log k + n \log y_1$$

Putting $\log Q = Y$, $\log k = K$ and $\log y_1 = X$, we obtain

$$Y = K + nX$$

Then

$$n = \frac{N(\sum XY) - (\sum X)(\sum Y)}{N(\sum X^2) - (\sum X)^2}$$

$$K = \frac{\sum Y - n\sum X}{N}$$

$$k = \text{antilog } K$$

where N is the number of sets of Q and y_1 plotted. The correlation coefficient r is given by

$$r = \frac{N\left(\sum XY\right) - \left(\sum X\right)\left(\sum Y\right)}{\left(\sqrt{N\left(\sum X^2\right) - \left(\sum X\right)^2}\right)\left(\sqrt{N\left(\sum Y^2\right) - \left(\sum Y\right)^2}\right)}$$

For a perfect correlation, r = 1.0. If r is between 0.6 and 1.0, it is generally taken as a good correlation.

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.

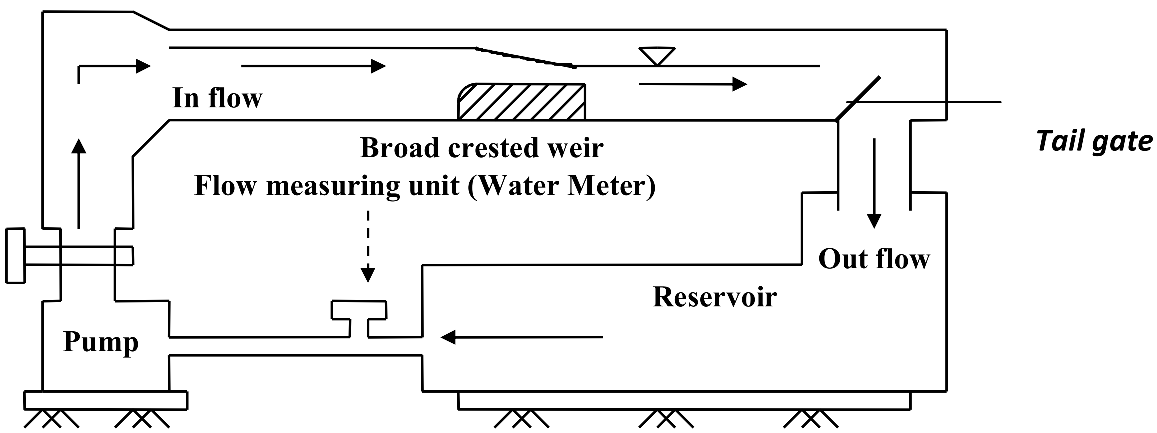


Fig. 2.2 Setup for flow over a broad-crested weir

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).

To calibrate the weir by eye estimation (should be done by students having odd student number)

i) Plot the actual discharges against the corresponding upstream depths in a log log paper and find the value of n and k as discussed in Art. 2.2.4.
ii) Develop the relationship $Q = ky_1^n$.

To calibrate the weir by regression (should be done by students having even student number)

i) Form a table having columns for Q, y_1, X, Y, XY, X^2, Y^2 as discussed in Art. 2.2.4 and find the value of n, k and r .
ii) Compare the equation with that obtained by the eye estimation method.

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 $y = mx + c$).

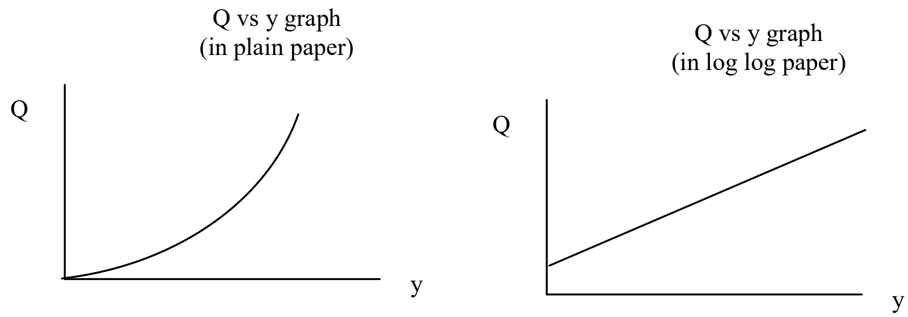


Fig. 2.3: Q (actual discharge) vs y(upstream depth of water above weir) graph

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)

ii) By regression (should be done by students having even student number)

y_1	Q_a	$X = \log y_1$	$Y = \log Q$	XY	X^2	Y^2
		$\sum X=$	$\sum Y=$	$\sum XY=$	$\sum X^2=$	$\sum Y^2=$

Course Teacher :
Designation :

Signature

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.

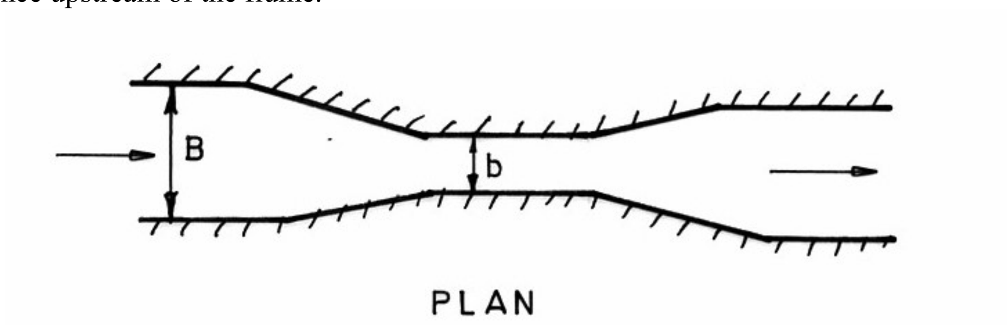


Fig. 3.1 Flow through a Venturi flume

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

$$V_c = \sqrt{gD_c}$$

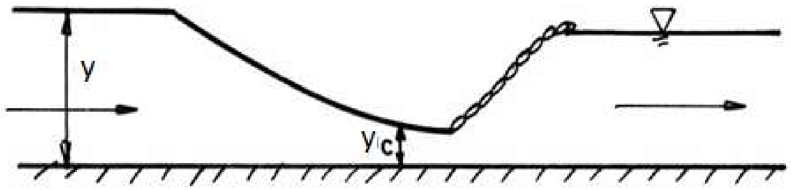


Fig. 3.2 Free flow condition

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

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

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

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

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

3.2.3 Theoretical discharge at submerged flow condition

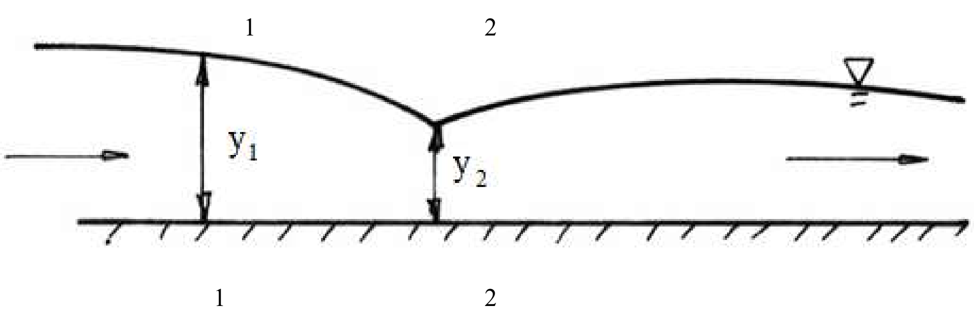


Fig. 3.3 Submerged flow condition

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

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

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

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

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.

By eye estimation

As $\log Q = \log k + n \log y_1$, so if Q and y_1 are plotted in a log log paper, the line will represent a straight line. Different sets of Q and y_1 are plotted in a log log paper keeping y_1 along the x axis and Q along the y axis. The best fit line is drawn by eye estimation. The slope of the line gives the value of n. Then for any value of y the corresponding value of Q is found from the best fit line. Using these values of y, Q and n, the value of k can be found form the equation $Q = ky_1^n$.

By regression

From $Q = ky_1^n$, we have

$$\log Q = \log k + n \log y_1$$

Putting $\log Q = Y$, $\log k = K$ and $\log y_1 = X$, we obtain

$$Y = K + nX$$

Then

$$n = \frac{N(\sum XY) - (\sum X)(\sum Y)}{N(\sum X^2) - (\sum X)^2}$$

$$K = \frac{\sum Y - n\sum X}{N}$$

$$k = \text{antilog } K$$

Where, N is the number of sets of Q and y_1 plotted. The correlation coefficient r is given by

$$r = \frac{N\left(\sum XY\right) - \left(\sum X\right)\left(\sum Y\right)}{\left(\sqrt{N\left(\sum X^2\right) - \left(\sum X\right)^2}\right)\left(\sqrt{N\left(\sum Y^2\right) - \left(\sum Y\right)^2}\right)}$$

For a perfect correlation, r = 1.0. If r is between 0.6 and 1.0, it is generally taken as a good correlation.

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.

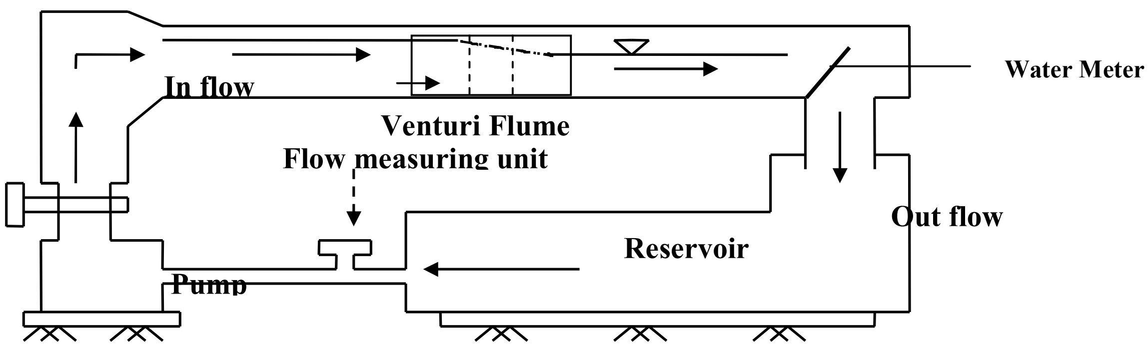


Fig. 3.4 Setup for flow through a Venturi flume

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).

To determine the theoretical and the actual discharges and the coefficient of discharge in submerged flow condition

i) Measure the flow depths at sections 1 and 2 shown in Fig. 3.3 and determine the theoretical discharge using Eq.(3.3).

ii) Take the reading of actual discharge and hence find the coefficient of discharge using Eq. (3.8).

To calibrate the flume (for free flow condition only) by eye estimation (should be done by students having even student number)

i) Plot the actual discharge against the corresponding upstream depth in a log log paper and find the values of n and k as discussed in Art. 3.2.5.

ii) Develop the relationship $Q = ky_1^n$.

To calibrate the flume (for free flow condition only) by regression (should be done by students having odd student number)

i) Form a table having columns for Q, y_1, X, Y, XY, X^2, Y^2 as discussed in Art.3.2.5 and find the values of n, k and r.

ii) Compare the equation with that obtained by the eye estimation method.

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 $y = mx + c$).

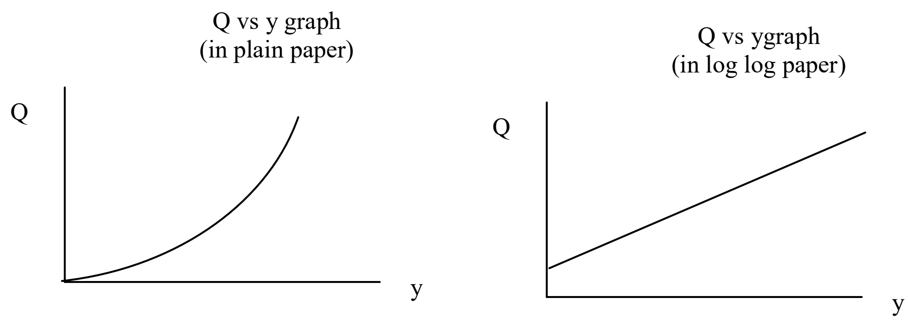


Fig. 3.5: Q (actual discharge) vs y(upstream depth of water) graph

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)

ii) By regression (should be done by students having odd student number)

y_1	Q_a	$X = \log y_1$	$Y = \log Q$	XY	X^2	Y^2
		$\sum X=$	$\sum Y=$	$\sum XY=$	$\sum X^2=$	$\sum Y^2=$

Course Teacher :
Designation :

Signature

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.

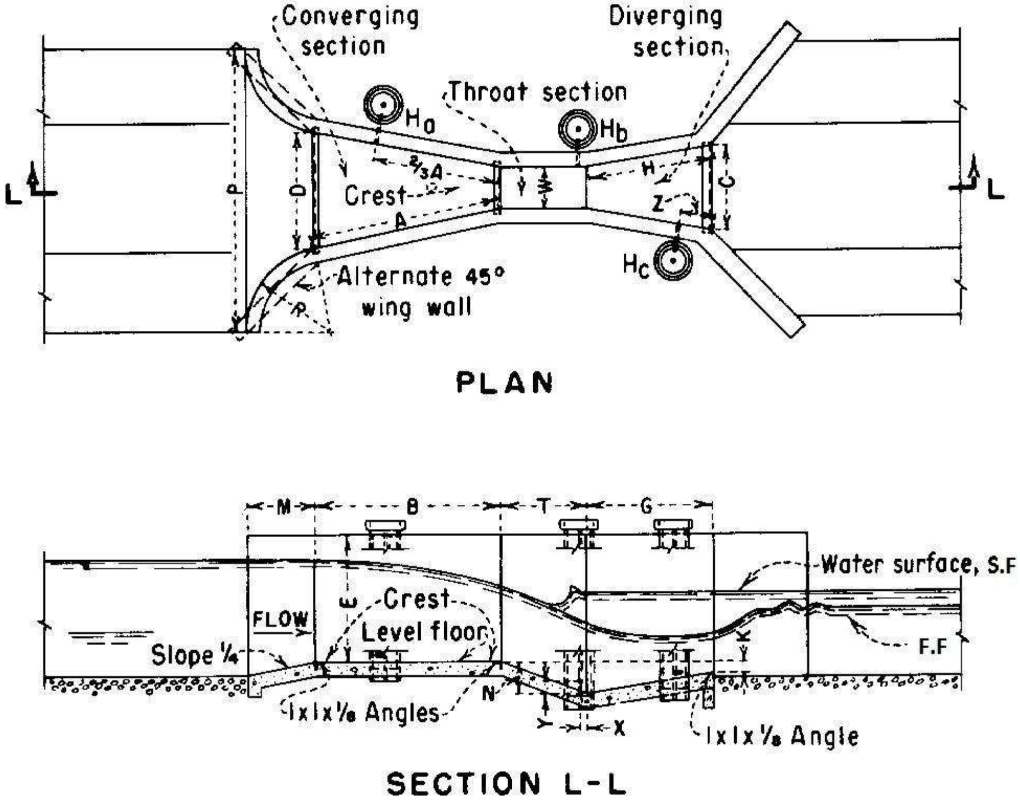


Fig. 4.1 Flow through Parshall flume

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

$$Q_{tf} = 2.06\, H_a^{1.58} \qquad (4.2)$$

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′	$Q = 4WH_a^{1.552W^{0.026}}$
10′ to 50′	$Q = (3.6875W + 2.5)\, H_a^{1.6}$

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.

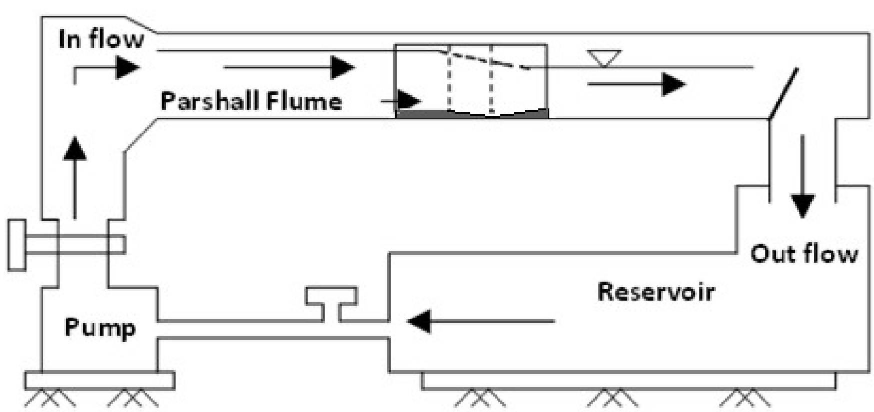


Fig.4.2 Setup for flow through a Parshall flume

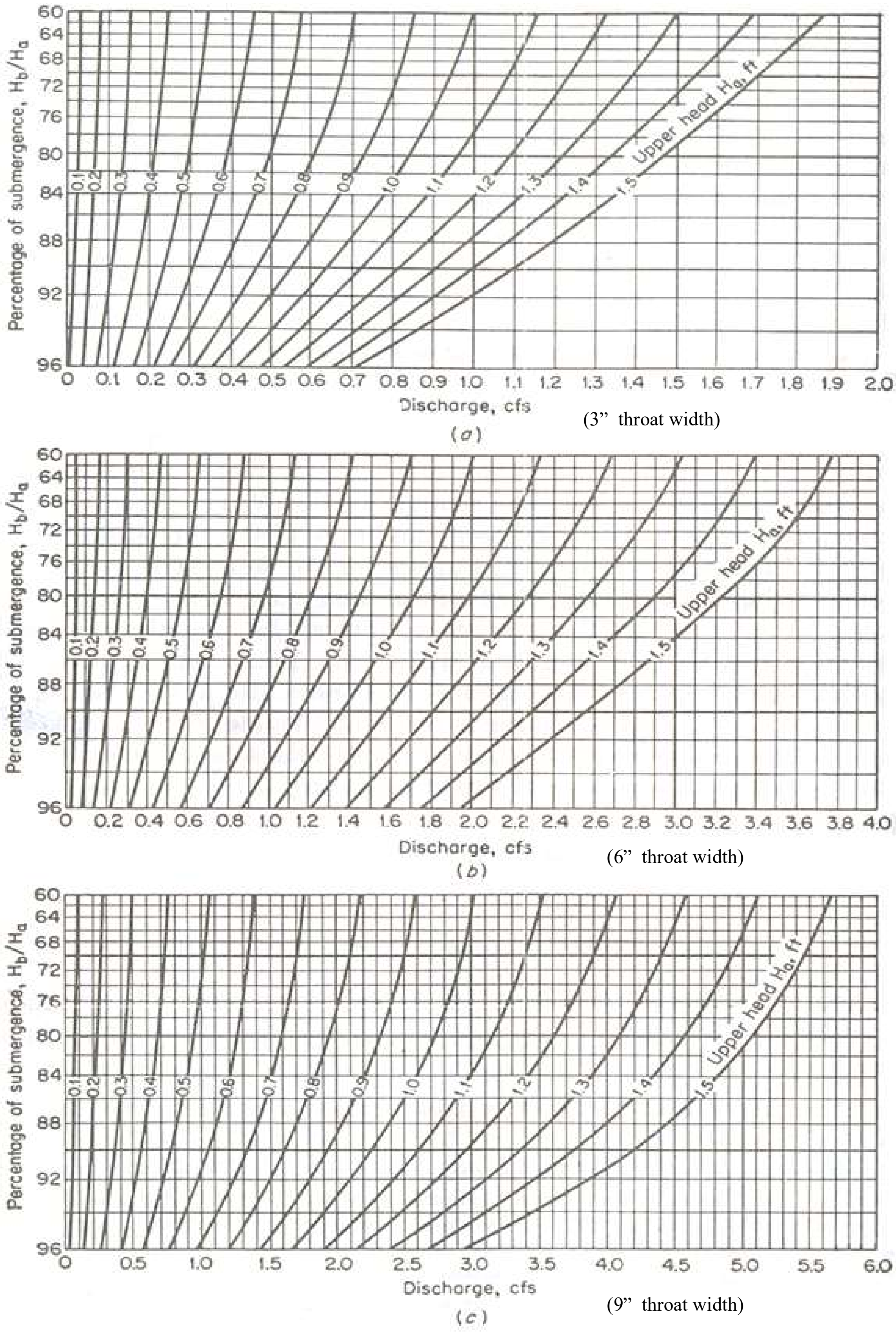


Fig.4.3 The submerged flow for Parshall flumes of various sizes

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 $y = mx + c$).

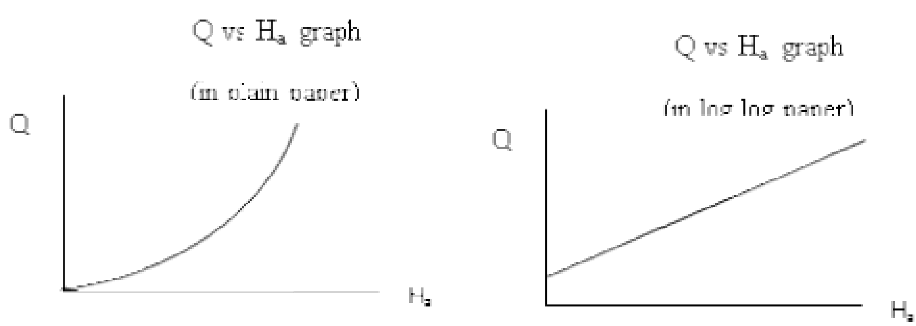


Fig. 4.4 Q (actual discharge) vs H_a (upstream depth of water) graph

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)

Course Teacher :
Designation :

Signature

Experiment No. 5

FLOW THROUGH A CUT-THROAT FLUME

5.1 General

Although a Parshall flume gives very accurate measurement of discharge, the problem of the flume is that the fabrication of such a flume is complicated and also the fabrication should be done as per requirement. The cut throat flume is an attempt to improve on the Parshall flume mainly by simplifying the construction details. So the flume is economical and normally used in straight sections of small irrigation channels for flow measuring purpose. The angles of divergence and convergence remain same for all flumes. So the size of the flume can be changed by merely moving the vertical walls in or out. This experiment deals with the measurement of discharge using a cut throat flume.

5.2 Theory

5.2.1 Description of the flume

The Cut throat flume has a flat bottom, vertical walls and a zero length throat section. The details of the standard shape of a cut throat flume are shown in Fig 5.1. It can operate either as a free or submerged flow structure. Under free flow condition critical depth occurs in the vicinity of the neck. Any flume length from 45 cm to 3 m can be used while neck widths between 2.5 cm and 1.8 m have been investigated.

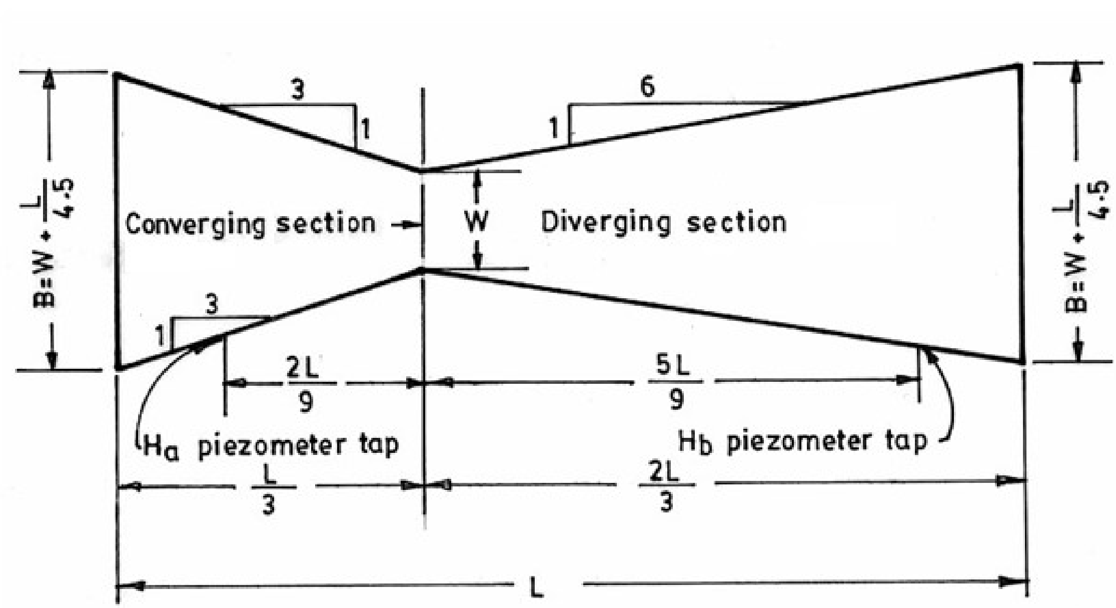


Fig. 5.1 Flow through a cut throat flume

5.2.2 Theoretical discharge at free flow condition

The theoretical discharge through cut throat flume for free flow condition is given by

$$Q_{tf} = CH_a^n \quad (5.1)$$

Where, C is the free flow coefficient given by

$$C = KW^{1.025} \quad (5.2)$$

K is the flume length coefficient, W is the width of the neck, n is an exponent and H_a is the upstream flow depth, measured at a distance of 2L/9 form the throat, as shown in Fig. 5.1. The values of K and n are obtained from Fig.5.3 for a given flume length.

5.2.3 Submergence ratio and submerged flow condition

In order to ensure free flow condition, the ratio between the water depths at the exit and entrance, i.e. the submergence ratio (H_b/H_a) should not exceed a certain limit, called the transition submergence, S_t, which can be determined from Fig. 5.3. If the submergence ratio exceeds the transition submergence, the flow condition is said to be submerged flow condition.

5.2.4 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 Q_a is measured by the water meter, the coefficient of discharge is given by

$C_{df}= Q_a/Q_{tf}$ (at free flow condition) (5.3)

5.3 Objectives of the experiment

i) To determine the theoretical discharge at the free flow condition.
ii) To determine the submergence ratio and to check the effect of submergence.
iii) To determine the coefficient of discharge C_d.
iv) To verify the values of coefficient C and exponent n.

5.4 Experiment setup

The experiment setup is given below.

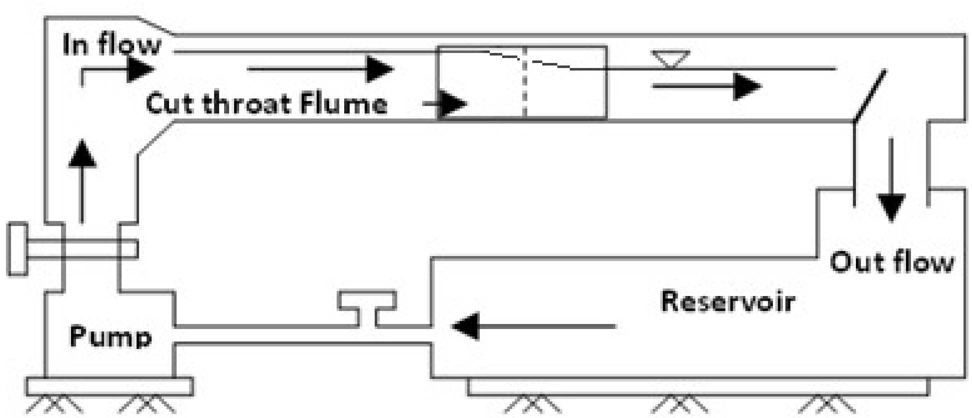


Fig. 5.2 Setup for flow through a cut throat flume

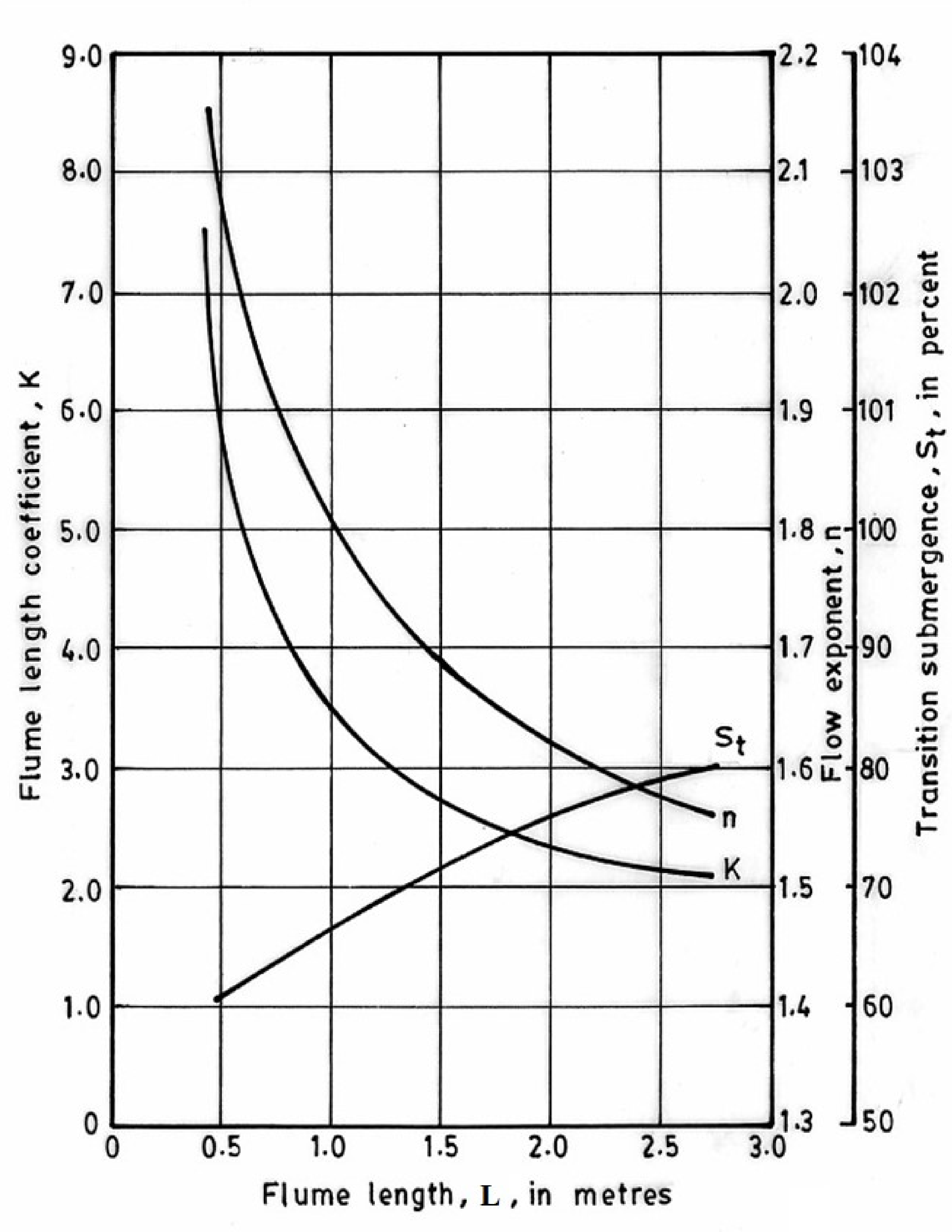


Fig. 5.3 Generalized free flow coefficients and exponents and S_t for cut-throat flumes

5.5 Procedure

To determine the theoretical discharge for the free flow condition

i) Measure the head H_a.
ii) Determine the values of K and n from Fig. 5.2.
iii) Determine the value of C using Eq. (5.3).
iv) Determine the theoretical discharge using Eq. (5.1).

To determine the submergence ratio and check the effect of submergence

i) Measure the heads H_a and H_b and determine the submergence ratio H_b/H_a.
ii) Determine the transition submergence S_t from Fig. (5.3).
iii) If submergence ratio exceeds S_t, the flow is submerged.

To determine the coefficient of discharge, measure the actual discharge from the water meter and calculate C_{df} using Eq. (5.3).

To verify the values of C 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 C using Eq. (5.1).

5.6 Shape of Q vs H_a graph

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

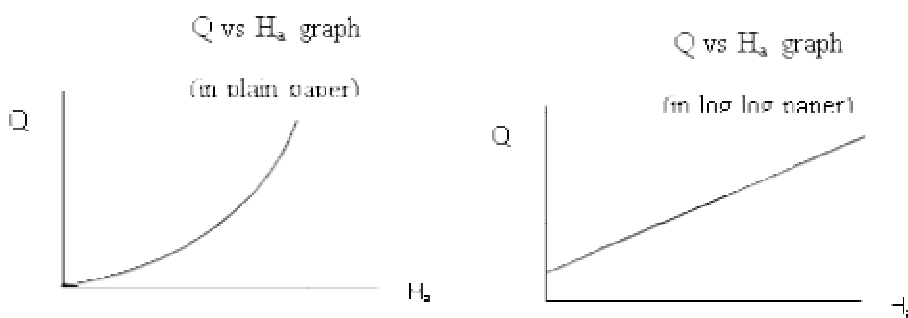


Fig. 5.4 Q (actual discharge) vs H_a (upstream depth of water) graph

5.7 Assignment

1. What are the advantages, disadvantages and use of a cut throat flume?
2. Which one of the four flow measuring devices, viz. broad-crested weir, Venturi flume, Parshall flume and cut throat flume, seems to be the best in an irrigation project of Bangladesh? Justify your answer.

DATA SHEET

Experiment Name :
Experiment Date :

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

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

Flume length, L = m From graph, K = n = S_t =

H_a (m)	C	Q_{tf} (m^3/s)	C_{df}	H_b (m)	Submergence Ratio H_b/H_a	Comments on submergence

Verification of C and n

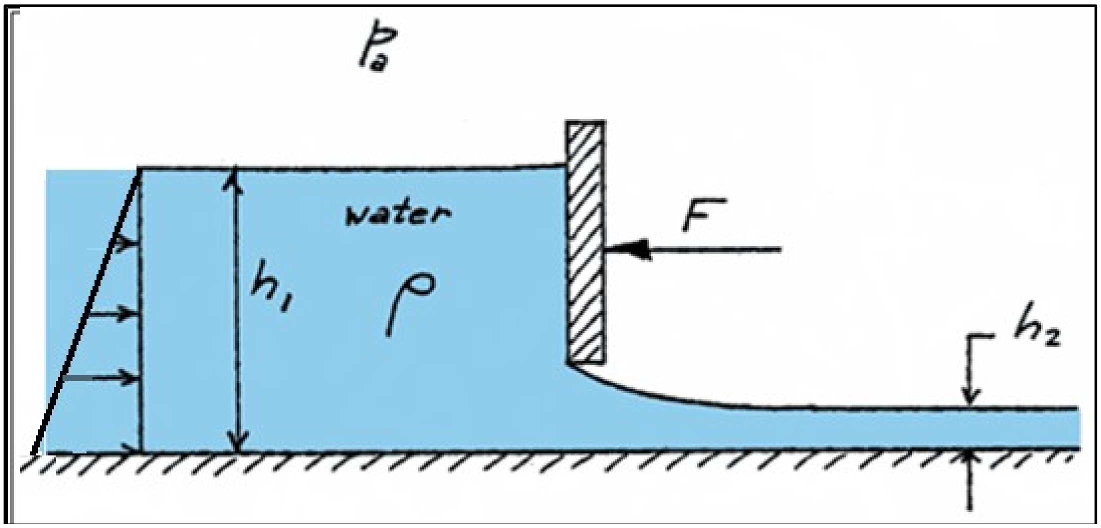
Actual discharge, Q_a (m^3/s)	H_a (m)

Course Teacher :
Designation :

Signature

Experiment No. 6

FLOW BENEATH A SLUICE GATE



6.1 General

Sluice gate is a classical example of the application energy and momentum principle. Sluice gate is used in open channel to control and regulate the flow as well as to measure the discharge in the channel. Sometimes it is used to raise the water level and maintain a constant operating level in irrigation canals. Sluice gate is also used for draining the excess water for both urban areas and rural agricultural areas. This experiment deals with the measurement of discharge beneath a sluice gate.

6.2 Theory

6.2.1 Description of the sluice gate

The simple form of a sluice gate consists of a horizontal channel bed having a vertical gate which can be lifted vertically up and down.

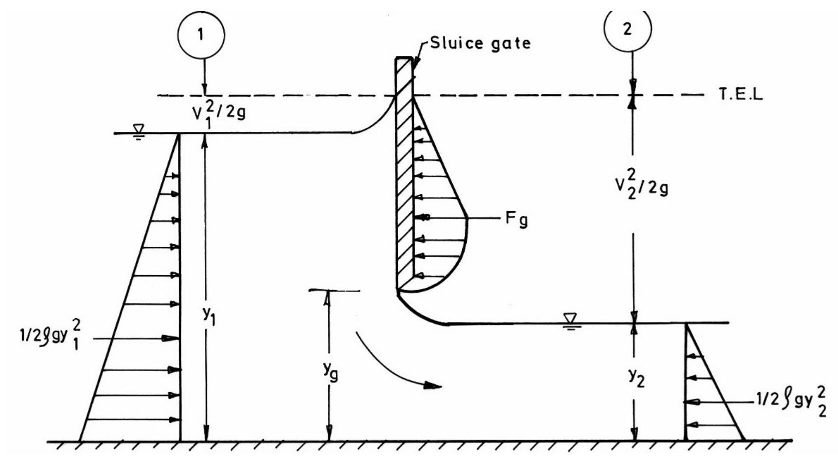


Fig. 6.1 Flow beneath a sluice gate

6.2.2 Theoretical discharge

The Bernoulli equation may be applied in those cases where there is a negligible loss of total head from one section to another or where the magnitude of the head loss is already known. Flow under a sluice gate is an example of converging flow where the correct form of the equation for discharge may be obtained by equating the energies at sections 1 and 2 as shown in Fig. 6.1. As the energy loss between the sections is negligible, we have

$$H_1 = H_2 \qquad (6.1)$$

and therefore

$$y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} \qquad (6.2)$$

Expressing the velocities in terms of Q, the above equation becomes

$$y_1 + \frac{Q^2}{2gb^2y_1^2} = y_2 + \frac{Q^2}{2gb^2y_2^2} \qquad (6.3)$$

Where, b is the width of the sluice gate. Simplifying and rearranging the terms, we obtain

$$Q = by_1\sqrt{\frac{2gy_2}{(y_1/y_2+1)}} \qquad (6.4)$$

or alternatively

$$Q = by_2\sqrt{\frac{2gy_1}{(y_2/y_1+1)}} \qquad (6.5)$$

The small reduction in flow velocity due to viscous resistance between sections 1 and 2 may be allowed for by a coefficient C_v. Then

$$Q = C_v by_2\sqrt{\frac{2gy_1}{(y_2/y_1+1)}} \qquad (6.6)$$

The coefficient of velocity, C_v, varies in the range $0.95 < C_v < 1.0$, depending on the geometry of the flow pattern (expressed by the ratio y_g/y_1) and friction.
The downstream depth y_2 may be expressed as a function of the gate opening, y_g, i.e.

$$y_2 = C_c y_g \qquad (6.7)$$

Where, C_c is the coefficient of contraction whose commonly accepted value of 0.61 is nearly independent of the ratio y_g/y_1. The maximum contraction of the jet occurs approximately at a distance equal to the gate opening. Thus, Eq.(6.6) becomes

$$Q = C_c C_v by_g\sqrt{\frac{2gy_1}{(C_c y_g/y_1+1)}} \qquad (6.8)$$

The above equation can also be written as

$$Q = C_d by_g\sqrt{2gy_1} \qquad (6.9)$$

Where, C_d is the coefficient of discharge and is a function of C_v, C_c, b, y_g, and y_1. Therefore

$$C_d = \frac{C_c C_v}{\sqrt{C_c y_g/y_1+1}} \qquad (6.10)$$

Equation (6.9) may also be written as

$$Q_a = C_d Q_t \qquad (6.11)$$

So that

$$Q_t = by_g\sqrt{2gy_1} \qquad (6.12)$$

Where, Q_t and Q_a are the theoretical and actual discharges, respectively.

6.2.3 Forces on a sluice gate

The momentum equation may be applied to the fluid within any chosen control volume where the external forces are known or can be estimated to a sufficient degree of accuracy. The horizontal components of these forces acting on the fluid within the control volume shown in Fig. 6.1 are the resultants of the hydrostatic pressure distributions at sections 1 and 2, the viscous shear force on the bed and the thrust of the gate. It should be noted that the equation permits the resultant gate thrust (F_g) to be determined even though the pressure distribution along its surface is not hydrostatic. Over a short length of smooth bed the contribution of the shear force may be neglected. The resultant force applied to the fluid within the control volume in the downstream direction is given by

$$F_x = \left[(1/2)\rho g y_1^2 - (1/2)\rho g y_2^2 - F_g\right]b \qquad (6.13)$$

The effect of this force is to accelerate the fluid within the control volume in the downstream direction. Hence

$$F_x = \rho Q_a V_2 - \rho Q_a V_1 \qquad (6.14)$$

Substituting for F_x and gathering terms, we obtain

$$F_g = \frac{1}{2}\rho g y_2^2\left[(y_1/y_2)^2 - 1\right] - \frac{\rho Q_a^2}{b^2 y_2}\left[1 - \frac{y_2}{y_1}\right] \qquad (6.15)$$

Simplifying and eliminating Q_a, we get

$$F_g = \frac{1}{2}\rho g \frac{(y_1 - y_2)^3}{y_1 + y_2} \qquad (6.16)$$

The pressure distribution on the gate cannot be hydrostatic, as the pressure must be atmospheric at both the upstream water level and at the point where the jet springs clear of the gate.

Note that the thrust on the gate, F_H, for a hydrostatic pressure distribution is given by

$$F_H = \frac{1}{2}\rho g\left(y_1 - y_g\right)^2 \qquad (6.17)$$

6.3 Objectives

i) To determine the discharge beneath the sluice gate.
ii) To determine C_v, C_c and C_d.
iii) To plot y_1 vs Q_a for different values of y_g in a plain graph paper.
iv) To determine F_g and F_H and hence to find the ratio F_g/F_H.

6.4 Experimental setup

The experimental setup is given below.

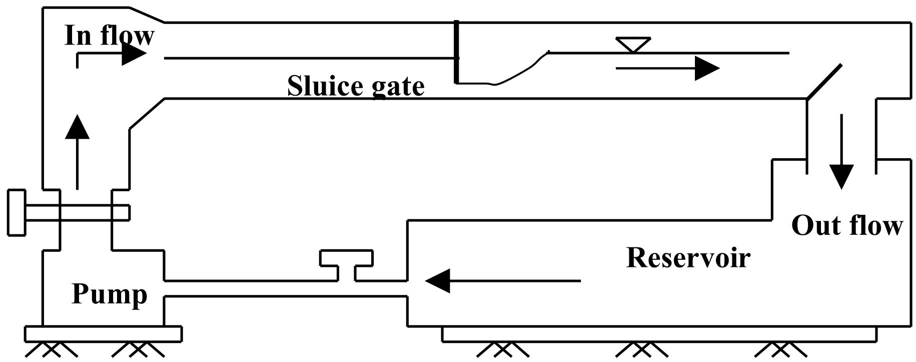


Fig. 6.2 Setup for flow beneath a sluice gate

6.5 Procedure

To determining the discharge beneath the sluice gate

i) Measure y_1 and y_g.
ii) Calculate the theoretical discharge using Eq. (6.12).
iii) Take the reading of actual discharge from the water meter.

To determine C_v, C_c and C_d

i) Calculate C_c using Eq.(6.7).
ii) Using the value of C_c, calculate C_v using Eq.(6.8).
iii) Using the values of C_c and C_v, determine C_d using Eq. (6.10).

Plot $y_1$vsQ_a for different values of y_g in a plain graph paper.

To determine F_g and F_H and hence to find the ratio F_g/F_H

i) Determine y_2.
ii) Determine $F_g$using Eq.(6.16).
iii) Determine F_H using Eq.(6.17) and calculate the ratio F_g/F_H

6.6 Shape of y_1 vsQ_a graph

In a plain graph paper the plot of $Q_a = ky_1^n$ is a parabola. Now, if y_g increases, for same value of y_1, Q increases. So, the $y_1$vsQ_a graph for a higher value of y_g lies below the same graph for a lower value of y_g.

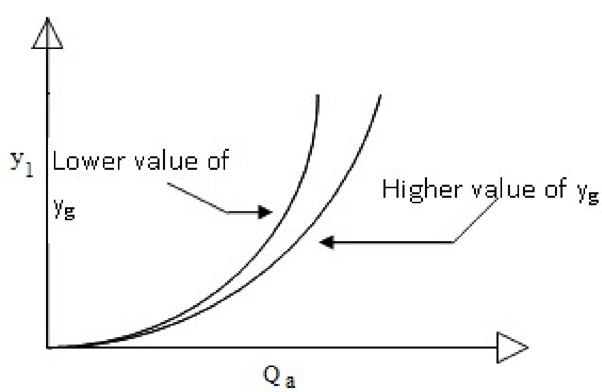


Fig. 6.3 Shape of $y_1$vs$Q_a$graph

6.7 Assignment

1. Explain why the pressure distribution along the surface of the gate is not hydrostatic.
2. What does happen when the gate opening is more than the critical depth?
3. When does the submergence occur and what is its effect on flow beneath a sluice gate?

DATA SHEET

Experiment Name :
Experiment Date :

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

6.9 Data sheet

Width of the sluice gate, b = cm Gate opening, y_g = cm

y_1 (cm)	y_2 (cm)	Q_t (cm^3/s)	Q_a (cm^3/s)	C_v	C_c	C_d	F_g (dyne)	F_H (dyne)	F_g/F_H

$y_1$vs Q graph

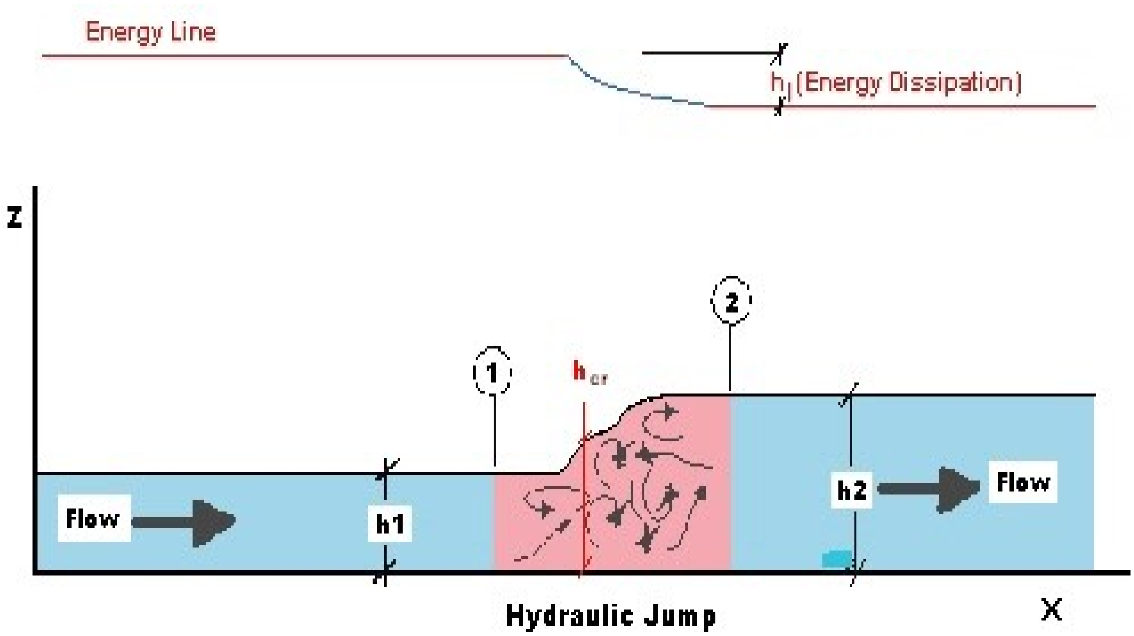
y_g (cm)	y_1 (cm)	Q_a(cm^3/s)

Course Teacher :
Designation :

Signature

Experiment No. 7

STUDY ON HYDRAULIC JUMP



7.1 General

In an open channel when a supercritical flow is made to change abruptly to subcritical flow, the result is usually an abrupt rise of the water surface. This feature is known as the hydraulic jump. It results when there is a conflict between upstream and downstream controls which influence the same reach of the channel. For example, if the upstream control causes supercritical flow and downstream control dictates subcritical flow, then this conflict can be resolved by a hydraulic jump, which passes the flow from one flow regime to other.

This experiment deals with observation of hydraulic jump in a horizontal rectangular channel and development of different relationships between height, length, efficiency and energy loss of a jump. Hydraulic jump is useful in dissipation of excess energy in flows over dams, weirs, spillways and other hydraulic structures to prevent scouring downstream, maintaining high water levels in channels for irrigation and other water distribution purposes, increasing discharge of a sluice gate and thus increasing the effective head across the gate, mixing chemicals for water purification or wastewater treatment, increasing aeration of flows and dechlorination of waste water, identification of special flow conditions, etc.

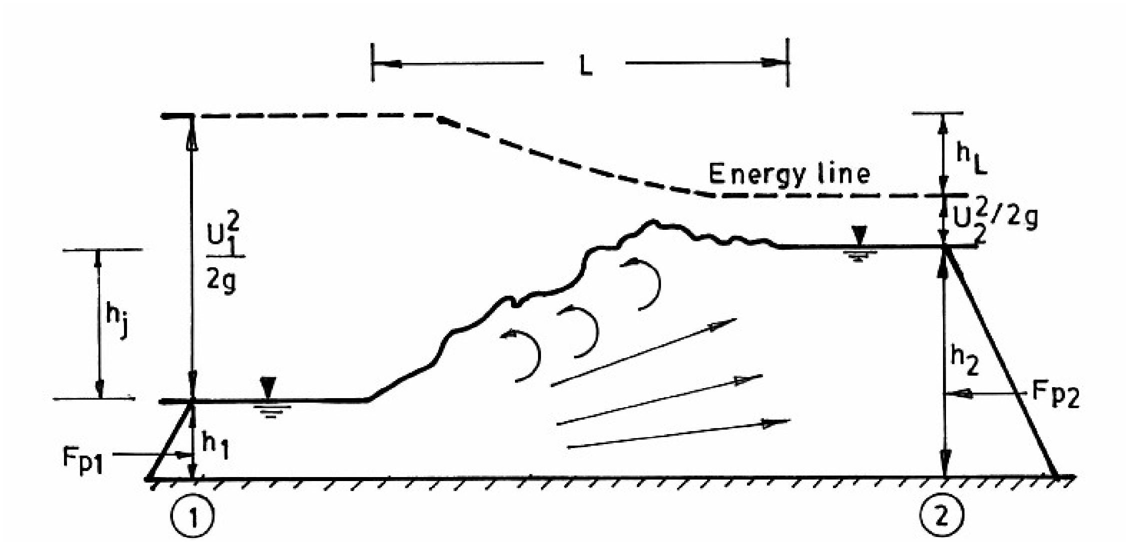


Fig. 7.1 Hydraulic Jump in a horizontal rectangular channel

7.2 Theory

7.2.1 Types of hydraulic jump

Depending on the Froude number before the jump (F_1), the United States Bureau of Reclamation (USBR) classified the hydraulic jumps in horizontal rectangular channels into the following five categories:

Type 1: $F_1 = 1 \sim 1.7$ Undular jump
Type 2: $F_1 = 1.7 \sim 2.5$ Weak jump
Type 3: $F_1 = 2.5 \sim 4.5$ Oscillating jump
Type 4: $F_1 = 4.5 \sim 9.0$ Steady jump
Type 5: $F_1 > 9.0$ Strong jump

7.2.2 Initial and sequent depths

The depth of flow before the jump is known as the initial depth (y_1) and the depth after the jump is known as the sequent depth (y_2). Consider a hydraulic jump occurring in a horizontal rectangular channel (Fig. 7.1). Since the jump takes place in a short reach of the channel, $F_f \approx 0$ and since the channel is prismatic, we can assume that $\beta_1 = \beta_2 = 1$. The hydrostatic forces F_{p1} and F_{p2} may be expressed as

$$F_{p1} = \gamma \bar{z}_1 A_1 \text{ and } F_{p2} = \gamma \bar{z}_2 A_2$$

Where, $\bar{z}_1$ and $\bar{z}_2$ are the vertical distances of the centroids of the respective water areas A_1 and A_2 from the free surface. Now applying momentum equation between sections 1 and 2, we obtain

$$\frac{Q^2}{gA_1} + \bar{z}_1 A_1 = \frac{Q^2}{gA_2} + \bar{z}_2 A_2 \qquad (7.1)$$

Since for a rectangular channel $Q = A_1V_1 = A_2V_2$, $A_1 = By_1$, $A_2 = By_2$, $\bar{z}_1 = y_1/2$ and $\bar{z}_2 = y_2/2$, Eq.(7.1) gives

$$\frac{q^2}{g}\left(\frac{1}{y_1} - \frac{1}{y_2}\right) = \frac{1}{2}(y_2^2 - y_1^2) \qquad (7.2)$$

Where, q (= Q/B) is the discharge per unit width. Using $q = y_1V_1 = y_2V_2$, Eq. (7.2) may be recast as

$$\frac{V_1^2}{gy_1} = F_1^{\,2} = \frac{1}{2}\frac{y_2}{y_1}\left(\frac{y_2}{y_1} + 1\right) \qquad (7.3)$$

Equation (7.3) may be solved to yield

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right) \qquad (7.4)$$

Where, y_2/y_1 is known as the ratio between the sequent and the initial depths.

7.2.3 Length of the jump (L)

The length of a hydraulic jump is the horizontal distance from the front face of the jump to a point immediately downstream from the roller. This length cannot be determined by theory. Silvester (1964) demonstrated that for free hydraulic jumps in horizontal rectangular channels

$$\frac{L}{y_1} = 9.75(F_1 - 1)^{1.01} \qquad (7.5)$$

7.2.4 Energy loss in the jump

The total loss of energy in the jump is equal to the difference in specific energies before and after the jump. It can be shown that the total energy loss involved in a hydraulic jump in a horizontal rectangular channel is given by

$$\Delta H_{Total} = \Delta E_{Total} = E_1 - E_2 = \frac{(y_2 - y_1)^3}{4 y_1 y_2} \qquad (7.6)$$

Where, E_1 is the specific energy before the jump and E_2 is the specific energy after the jump.

The kinetic energy loss in the jump is given by the difference in velocity head before and after the jump. Thus

$$\Delta E_{K.E.} = \frac{1}{2g}(V_1^2 - V_2^2) \qquad (7.7)$$

Where, V_1 is the velocity before the jump and V_2 is the velocity after the jump.

7.2.5 Efficiency of the jump

The ratio of the specific energy after the jump to that before the jump (E_2/E_1) is known as the efficiency of the jump. It can be shown that the efficiency of the jump is given by

$$\frac{E_2}{E_1} = \frac{(8F_1^2+1)^{3/2} - 4F_1^2 + 1}{8F_1^2(2+F_1^2)} \qquad (7.8)$$

7.2.6 Height of the jump

The difference between the depths after and before the jump is known as the height of the jump. It is given by

$$h_j = h_2 - h_1 \qquad (7.9)$$

The ratio of the height of jump to the specific energy before jump is known as the relative height of the jump and is given by

$$\frac{h_j}{E_1} = \frac{\sqrt{1+8F_1^2} - 3}{F_1^2 + 2} \qquad (7.10)$$

7.3 Objectives of the experiment

i) To determine the type of the jump according to USBR classification.
ii) To measure the initial depth (y_1), sequent depth (y_2), length (L) and height (h_j) of the jump and compare them with the theoretical values.
iii) To determine the total energy loss, kinetic energy loss and efficiency of the jump and compare them with the theoretical values.
iv) To develop the theoretical characteristic curves of the hydraulic jump.

7.4 Experimental setup

In this experiment the hydraulic jump is produced by introducing a sluice gate in the flume. The experimental setup is given below.

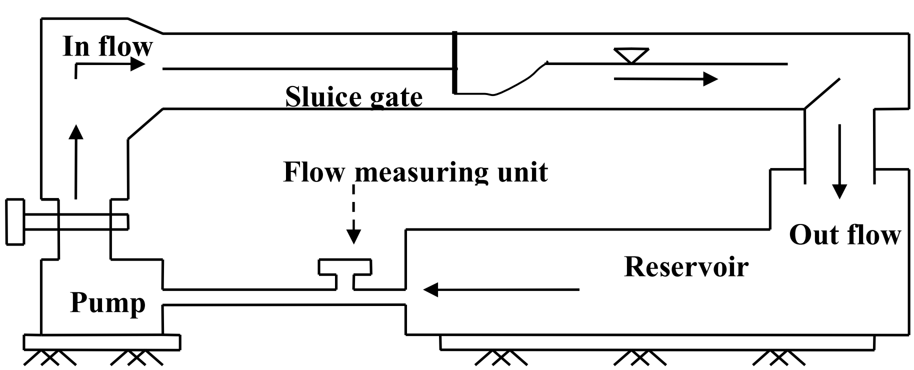


Fig. 7.2 Setup for hydraulic jump

7.5 Procedure

i) Measure the depth of flow before the jump at three points and average them to get the initial depth y_1.

ii) Measure the depth of flow after the jump at three points and average them to get the sequent depth y_2.

iii) Determine the velocity before the jump (V_1), then calculate F_1 and find the type of jump.

iv) Measure the length of the jump (L), then find L/y_1 and verify Eq.(7.5).

v) Compute the height of the jump (h_j) from Eq.(7.9), then find the value of h_j/E_1 and verify Eq.(7.10).

vi) Compute E_1 and E_2, then find the total energy loss E_1- E_2 and verify Eq.(7.6).

vii) Compute the kinetic energy loss in the jump using Eq.(7.7).

viii) Compute the efficiency of the jump E_2/E_1 and verify Eq.(7.8).

ix) Plot $\frac{E_2}{E_1}, \frac{h_j}{E_1}, \frac{y_1}{E_1}$ *and* $\frac{y_2}{E_1}$ vs F_1 to get the characteristic curves.

7.6 Typical shapes of graphs

Characteristic graphs of hydraulic jumps are a combination of four graphs shown in Fig. 7.3.

(i) $\frac{E_2}{E_1}$ vs F_1 graph: From Eq.(7.8), at F_1=1, $\frac{E_2}{E_1}$=1. So the curve starts from (1,1) and decreases with the increase in F_1.

(ii) $\frac{h_j}{E_1}$ vs F_1 graph: From Eq.(7.10), at F_1=1, $\frac{h_j}{E_1}$= 0. So the curve will start from (1,0) and increases up to a value of $\frac{h_j}{E_1}$= 0.507 at F_1= 2.77 and then decreases with F_1.

(iii) & (iv) $\frac{y_1}{E_1}$ and $\frac{y_2}{E_1}$ vs F_1 graphs: At critical state, i.e. at F_1 =1, $y_1 = y_2$ from Eq.(7.4) . So both curves will start from a common point at F_1=1. Now, at critical state, E =1.5 y. So, $\frac{y_1}{E_1} = \frac{y_2}{E_1} = \frac{2}{3}$ at this state. Hence, both the curves will start from (1, 2/3). The $\frac{y_1}{E_1}$ vs F_1 curve decreases with higher F_1 but $\frac{y_2}{E_1}$ vs F_1 curve increases up to a value of 0.8 at F_1 =1.73 and then decreases with higher F_1.

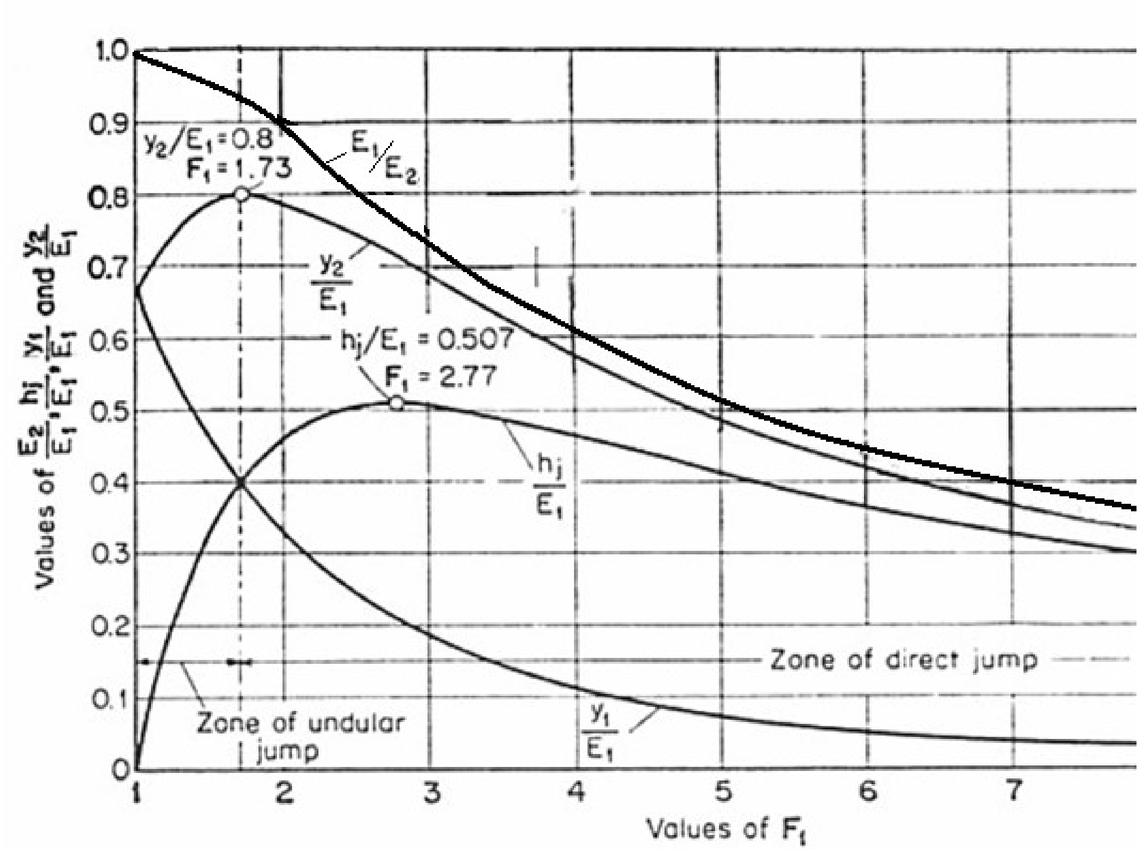


Fig. 7.3 Characteristics curves of hydraulic jumps in horizontal rectangular channels

7.7 Assignment

1. What are the different types of jumps according to USBR classification?
2. Why does the energy loss occur in hydraulic jumps? Is it really an energy loss?
3. What is tailwater depth? Explain why a hydraulic jump moves upstream when the tailwater depth is greater than the sequent depth and vice versa.

DATA SHEET

Experiment Name :
Experiment Date :

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

7.9 Data sheet

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

Depth				Velocity		F_1	Type of jump
y_1 (cm)		y_2 (cm)		V_1 (cm/s)	V_2 (cm/s)		

Verification of total energy loss					Kinetic energy loss (cm)	Verification of efficiency		
E_1 (cm)	E_2 (cm)	E_1-E_2 (cm)	RHS of Eq.(7.6) (cm)	Comment		E_2/E_1	RHS of Eq.(7.8)	Comment

Verification of length of jump				Verification of height of jump			
L (cm)	L/y_1	RHS of Eq.(7.5)	Comment	h_j (cm)	h_j/E_1	RHS of Eq.(7.10)	Comment

Characteristic curve

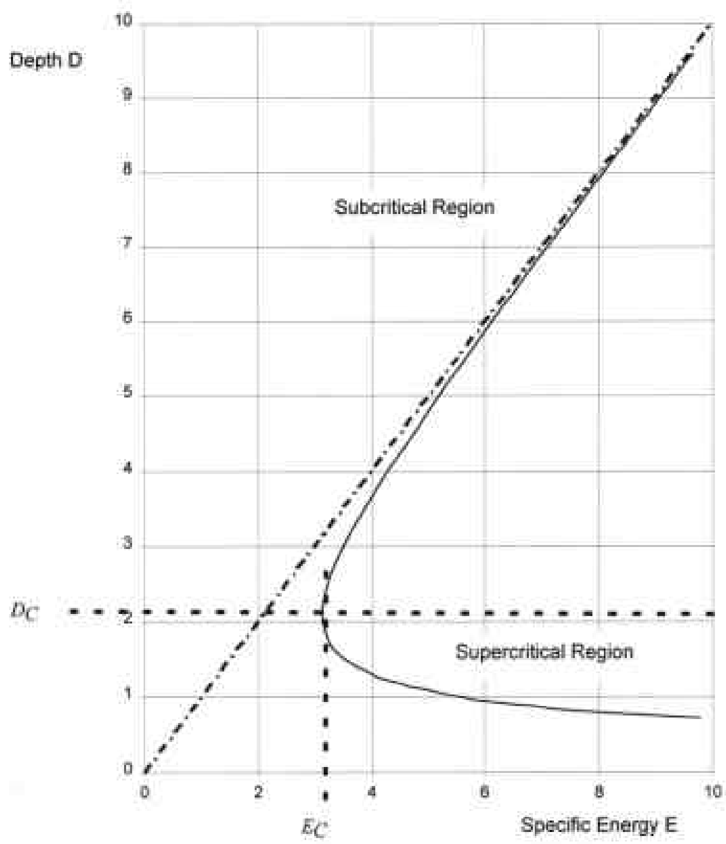
F_1	$\frac{E_2}{E_1}$	$\frac{h_j}{E_1}$	$\frac{y_1}{E_1}$	$\frac{y_2}{E_1}$

Course Teacher :
Designation :

Signature

Experiment No. 8

DEVELOPMENT OF GENERALIZED SPECIFIC ENERGY AND SPECIFIC FORCE CURVES



8.1 General

The concept of specific energy and specific force is extremely useful in the solution of many problems in open channel flow. This experiment deals with the development of generalized specific energy and specific force curves. These curves are useful in determining the state of flow in a channel, i.e. whether the flow is critical, subcritical or supercritical. Flow is critical when the Froude number is equal to unity. When the depth of flow is above the critical depth, the subcritical state of flow exists in the channel. When the depth of flow is below the critical depth, the supercritical state of flow exists. Also, the critical state of flow gives us several important conditions, such as, the specific energy and specific force are minimum for a given discharge, the discharge is maximum for a given specific energy and so on. All these conditions are used in designing the various types of transitions and in controlling the flow using different control structures, for example, in determining the height of a weir, the width of a flume, opening of sluice gate, etc.

8.2 Theory

8.2.1 Specific energy

Specific energy is defined as the energy per unit weight of water at any section of a channel measured with respect to the channel bottom. If the total energy at any section is given by

$$H = z + y + \frac{v^2}{2g} \qquad (8.1)$$

then the specific energy at any section of a channel is obtained by putting z = 0 as

$$E = y + \frac{v^2}{2g} \qquad (8.2)$$

Since Q = Av, Eq.(8.2) can be written as

$$E = y + \frac{Q^2}{2gA^2} \qquad (8.3)$$

For a rectangular channel, A = by. So Eq. (8.3) can be written as

$$E = y + \frac{Q^2}{2gb^2y^2} \qquad (8.4)$$

8.2.2 Specific energy curve

For a given channel section and discharge, the specific energy is a function of the depth of flow. When the depth of flow is plotted against the specific energy, a specific energy curve is obtained. This curve has two limbs CA and CB (Fig.8.1). From Eq. (8.4), when $y \rightarrow 0$, $E \rightarrow \propto$. So the limb CA approaches the horizontal axis asymptotically towards the right. Also, when $y \rightarrow \propto$, $A \rightarrow \propto$ so that $v^2/2g \rightarrow 0$ and $E \rightarrow y$, which implies that the limb CB approaches the E = y line (line OD) asymptotically. As the slope of the E = y line is 1, so it has an inclination of 45° with the horizontal axis and passes through the origin.

The specific energy curve (Fig. 8.1) shows that, there are two possible depths for a given value of E, the low stage y_1 and the high stage y_2, which are the called alternate depths.

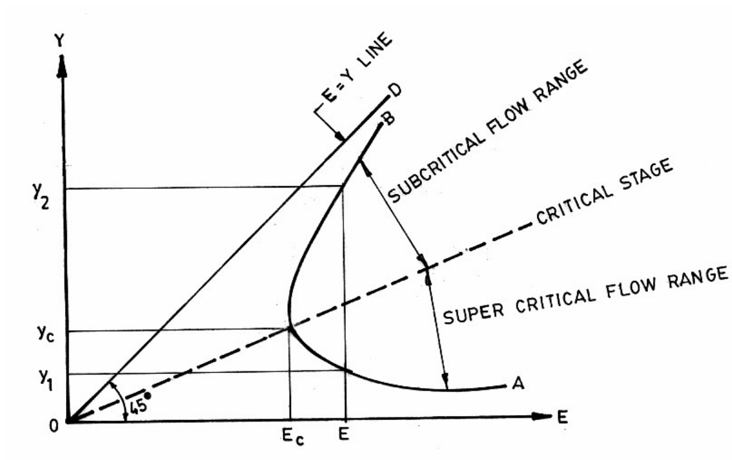


Fig. 8.1 Specific-energy curve

Differentiating Eq.(8.3) with respect to y and simplifying, we obtain

$$\frac{dE}{dy} = 1 - \frac{v^2}{gD}$$

Where, D is the hydraulic depth. At point C, the specific energy is minimum.

$$\therefore \frac{dE}{dy} = 0 \text{ so that } \frac{v^2}{gD} = 1 \quad \text{or,} \quad Fr^2 = 1 \quad \therefore Fr = 1$$

This condition represents the critical state of flow. At this condition, the two alternate depths apparently become one which is known as the critical depth y_c. When the depth of flow is greater than y_c, the velocity of flow is less than the critical velocity for the given discharge and hence the flow is subcritical. When the depth of flow is less than the critical depth, the flow is supercritical. Hence, y_1 is the depth of supercritical flow and y_2 is the depth of subcritical flow.

8.2.3 Generalized specific energy curve

If the discharge changes, the specific energy curve also changes, i.e. the curve moves to right if the discharge is increased and vice versa. In order to develop a generalized specific energy curve, i.e. to use one specific energy curve for different discharges, the curve is to be made dimensionless with respect to the critical depth, as the critical depth y_c is constant for a given discharge. So dividing both sides of Eq. (8.4) by y_c and after simplification, we obtain

$$\frac{E}{y_c} = \frac{y}{y_c} + \frac{1}{2}\left(\frac{y_c}{y}\right)^2 \qquad (8.5)$$

Equation (8.5) is the generalized form of the relationship between specific energy and depth of flow in which each term is dimensionless. The plot of this equation is shown in Fig. 8.2.

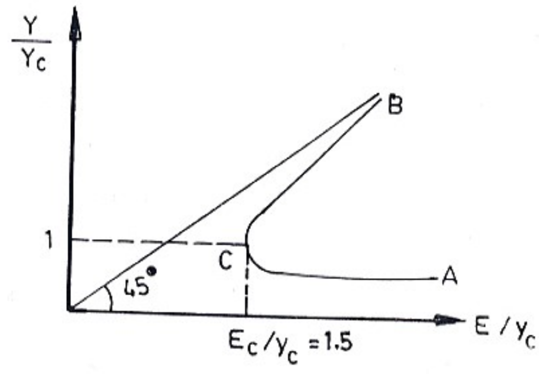


Fig. 8.2 Dimensionless specific energy curve

8.2.4 Specific force

Specific force is defined as the force at any channel section which is equal to the sum of the hydrostatic force and momentum of the flow passing the section per unit time. For a rectangular channel, the specific force is given by

$$F = \frac{1}{2}\rho g\, b y^2 + \rho \frac{Q^2}{by} \qquad (8.6)$$

8.2.5 Specific force curve

For a given discharge and section, the specific force F is a function of the depth of flow y only. Plotting the depth of flow y vs the specific force F produces the specific force curve (Fig.8.3). This curve has two limbs CA and CB. At $y \to 0$, $F \to \infty$. So the limb CA approaches the horizontal axis asymptotically towards the right. Now, at $y \to \infty$, $F \to \infty$, but at this condition F becomes proportional y^2. So the limb CB rises upward and extend infinitely towards the right.

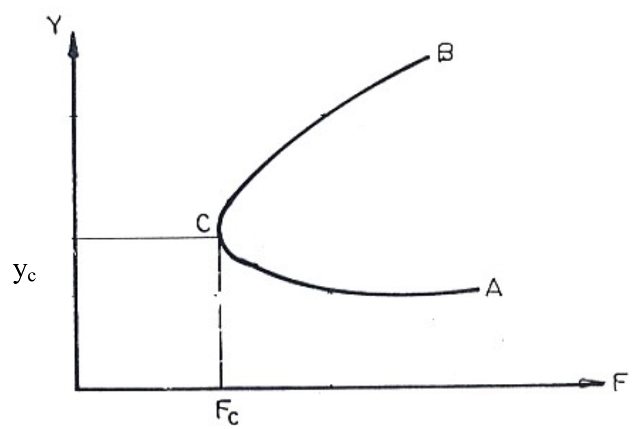


Fig. 8.3 Specific force curve

The specific force curve shows that, for a given specific force, there are two possible depths, y_1 and y2. These two depths constitute the initial and sequent depths of a hydraulic jump.

Differentiating Eq. (8.6) with respect to y and simplifying, we get

$$\frac{dF}{dy} = \rho g A - \rho \frac{Q^2}{AD}$$

At point C, the specific force is minimum. Therefore

$$\frac{dF}{dy} = 0 \text{ or, } \rho g A = \rho \frac{Q^2}{AD} \quad \text{or, } \frac{Q^2}{gDA^2} = 1 \quad or, \ \frac{V^2}{gD} = 1 \ \text{ so that } Fr^2 = 1 \ \text{ or, } Fr = 1$$

which is the same criteria developed for the minimum value of specific energy. Therefore, for a given discharge, minimum specific force occurs at minimum specific energy or at the critical state of flow.

8.2.6 Generalized specific force curve

If the discharge changes, the specific energy also changes accordingly, i.e. the specific force curve moves to right if the discharge is increased and vice versa. In order to develop a generalized specific force curve, i.e. to use one specific force curve for different discharges, the curve is to be made dimensionless with respect to the critical depth as the critical depth y_c is constant for a given discharge. So dividing both sides of Eq. (8.6) by $y_c^2 \rho g b$ and after simplification, we obtain

$$\frac{F}{y_c^2 \rho g b} = \frac{y_c}{y} + \frac{1}{2}\left(\frac{y}{y_c}\right)^2 \qquad (8.7)$$

Equation (8.7) is the generalized specific force equation and each term of this equation is dimensionless. The plot of this equation is shown in Fig. 8.4.

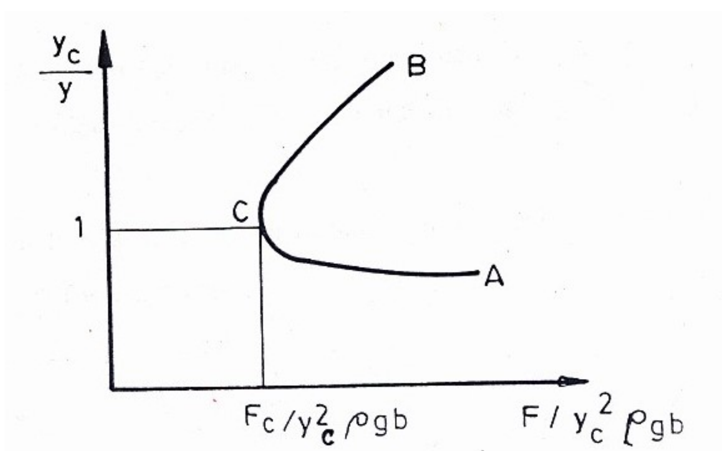


Fig. 8.4 Dimensionless specific force curve

8.3 Objectives of the experiment

i) To observe the flow profile in the experimental setup which depicts the variation of depth with change in energy?

ii) To plot the generalized specific energy and specific force curves from observed data.

8.4 Experimental setup

To plot the generalized specific energy and specific force curves, we have to observe the response of subcritical (slow) and supercritical (fast) flows to changes in the energy and force of a stream. For this, the setup as in Fig. 8.5 can be used.

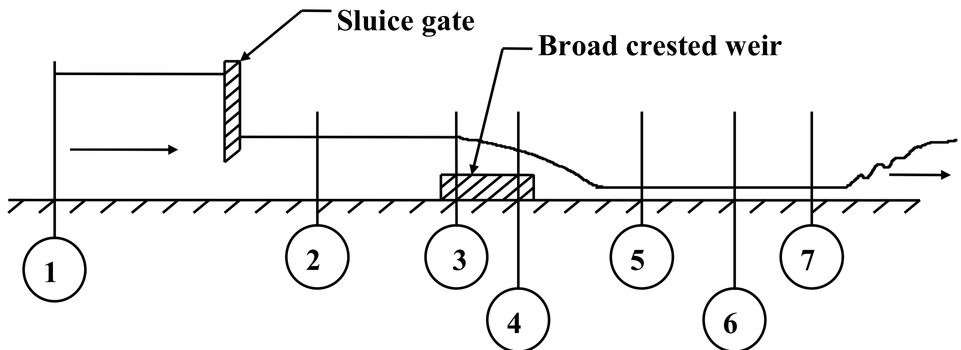


Fig. 8.5 Setup for development of generalized specific energy and specific force curves

8.5 Procedure

i) Determine depth of flow at three points as shown in Fig. 8.6 in each of the sections 1 to 7. Find the average depth for each section.

ii) Determine the actual discharge from the water meter and compute y_c.

iii) Compute E/y_c and $F/(y^2_c\rho gb)$ for each of the sections using Eqs.(8.5) and (8.7), respectively.

iv) Plot y/y_c vs E/y_c and y/y_c vs $F/(y^2_c\rho gb)$ on plain graph papers to get the generalized specific energy and specific force curves.

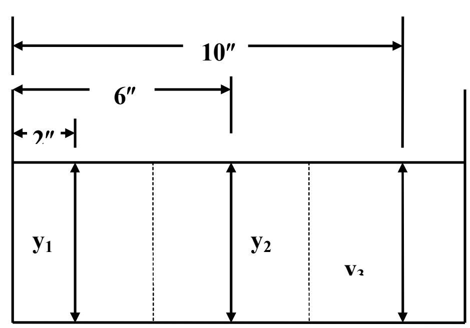


Fig. 8.6 Locations for measuring the depths

8.6 Assignment

1. How can you apply the dimensionless specific energy and specific force curves for computing specific energy and specific force for different discharges?

2. Can you use the dimensionless specific energy curve to find the specific force and vice versa? Explain.

DATA SHEET

Experiment Name :
Experiment Date :

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

8.8 Data sheet

Q = cm^3/s b = cm $y_c = (Q^2/gb^2)^{1/3}$ = cm

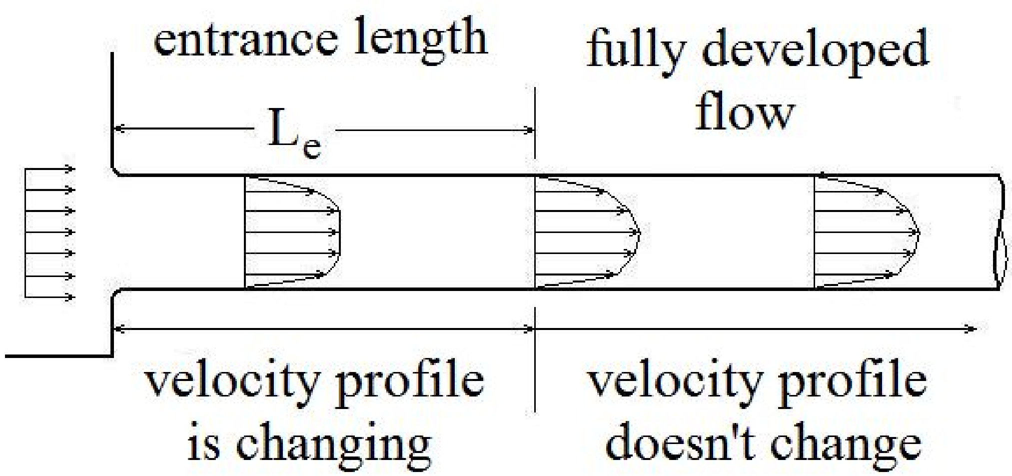
Section	y_1 (cm)	y_2 (m)	y_3 (cm)	$y = \frac{y_1 + y_2 + y_3}{3}$ (cm)	y/y_c	E/y_c	$\frac{F}{y_c^2 \rho g b}$
1							
2							
3							
4							
5							
6							
7							

Course Teacher:
Designation:

Signature

Experiment No. 9

VELOCITY DISTRIBUTION IN OPEN CHANNEL



9.1 General

Velocity of flow is an important parameter in open channel flow. In order to find out the channel discharge, the velocity distribution needs to be known. In an open channel the velocity is not uniform over the cross-section. The velocity is zero at the channel boundary and maximum at or near the free surface. This experiment deals with the velocity distribution in an open channel and determination of the energy and momentum coefficients. Practically the energy and momentum coefficients are very useful, as the application of energy and momentum equations requires these coefficients.

9.2 Theory

9.2.1 The boundary layer

When water enters a channel the velocity distribution across the channel section will vary with distance due to the presence of boundary roughness as shown in Fig. 9.1.

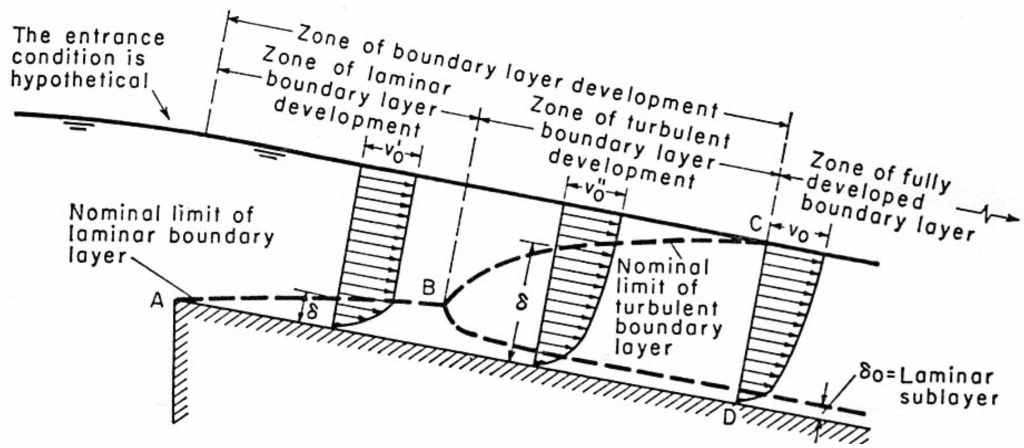


Fig.9.1 Development of boundary layer

The effect on the velocity distribution due to boundary roughness is indicated by the line ABC. Outside the surface represented by ABC, the velocity distribution is practically uniform. Near the channel surface and within the region ABC, velocity varies according to distance from channel surface. The region inside ABC is known as boundary layer. At the beginning of the flow in the channel, the flow is entirely laminar and a laminar boundary layer is developed along the channel surface, as shown by the curve AB. The velocity distribution in this layer is approximately parabolic. As water travels further along the channel, the flow in the boundary layer will eventually change to turbulent (at point B). Downstream from B a turbulent boundary layer is developed, as shown by the curve BC. The velocity distribution in this layer is approximately logarithmic. The turbulent boundary layer intersects the free water surface at a distance where the flow is assumed to be fully developed.

Even in a turbulent boundary layer, there is a very thin layer near the boundary in which the flow is laminar and is known as the laminar sublayer. The thickness of this layer is given by

$$\delta_0 = \frac{11.6\nu}{V_f} \qquad (9.1)$$

Where, V_f is the shear or friction velocity, given by

$$V_f = \sqrt{gRS} \qquad (9.2)$$

g is the acceleration due to gravity, R is the hydraulic radius, S is the slope of the energy line (taken to be equal to the slope of the channel bottom) and ν is the kinematic viscosity of water.

9.2.2 Surface roughness

The surface of a channel is composed of irregular peaks and valleys. The effective height of the irregularities is called the roughness height k. If the roughness height is less than the thickness of the laminar sublayer, the surface irregularities will be so small that all roughness elements will be submerged in the laminar sublayer and have no effect upon the flow outside the layer. Then the channel is said to be hydraulically smooth. For hydraulically smooth channel

$$0 \le \frac{kV_f}{\nu} \le 5 \quad \text{and } k < \delta_0 \qquad (9.3)$$

However, if the roughness height k is greater than the thickness of the laminar sublayer, the roughness elements extend their effects beyond the laminar sublayer and the channel is said to be hydraulically rough. For hydraulically rough channel

$$\frac{kV_f}{\nu} \ge 70 \qquad \text{and } k > \delta_0 \qquad (9.4)$$

There exists a transition zone in which the channel is neither smooth nor rough. For this zone

$$5 < \frac{kV_f}{\nu} < 70 \qquad (9.5)$$

9.2.3 Velocity distribution in turbulent flow

The flow of water in open channel is turbulent and the bed is normally rough. On the basis of Prandtl-von Karman logarithmic velocity distribution law, the velocity distribution in an open channel having hydraulically rough surface is given by

$$v = 5.75\, V_f \, log \frac{30\, y}{k} \qquad (9.6)$$

Where, v is the velocity at any point at a vertical distance of y from the channel bottom.

9.2.4 Cross-sectional mean velocity

The velocity varies in the vertical direction as well as in the lateral direction due to boundary friction. Cross-sectional mean velocity represents the average velocity over the cross-section. Using the velocity distribution given by Eq.(9.6), the cross-sectional mean velocity V is given by

$$V = V_f (6.25 + 5.75 \log \frac{R}{k}) \qquad (9.7)$$

9.2.5 Velocity distribution coefficients

As a result of non-uniform distribution of velocity over a channel section, the actual kinetic energy and momentum of flow passing a given cross-section are normally greater than those calculated on the basis of average velocity given by Eq.(9.7). So, the energy coefficient α and the momentum coefficient β are introduced in the energy and the momentum equations, respectively. The numerical value of the energy coefficient varies from 1.03 to 1.36 and the numerical value of the momentum coefficient varies from 1.01 to 1.12. for fairly straight prismatic channels. The ratio (α-1)/(β-1) varies from 2.8 to 3.

The energy and momentum coefficients are given respectively by

$$\alpha = \frac{\int v^3 dA}{V^3 A} = \frac{\sum v^3 \Delta A}{V^3 A} \qquad (9.8)$$

and

$$\beta = \frac{\int v^2 dA}{V^2 A} = \frac{\sum v^2 \Delta A}{V^2 A} \qquad (9.9)$$

Where, V is the velocity of flow in an elementary area ΔA.

For a rectangular channel we can write

$$\alpha = \frac{\sum v^3 \Delta y}{V^3 Y} \qquad (9.10)$$

$$\beta = \frac{\sum v^2 \Delta y}{V^2 Y} \qquad (9.11)$$

Where, Y is the total depth of flow and V is the cross-sectional mean velocity.

9.3 Objectives of the experiment

i) To determine the velocity distribution profile in the vertical.
ii) To calculate the channel roughness height k.
iii) To calculate the cross-sectional mean velocity V.
iv) To calculate the velocity distribution coefficients αand β.

9.4 Experimental setup

The setup for the experiment is given below. The longitudinal slope of the flume is 1 in 840.

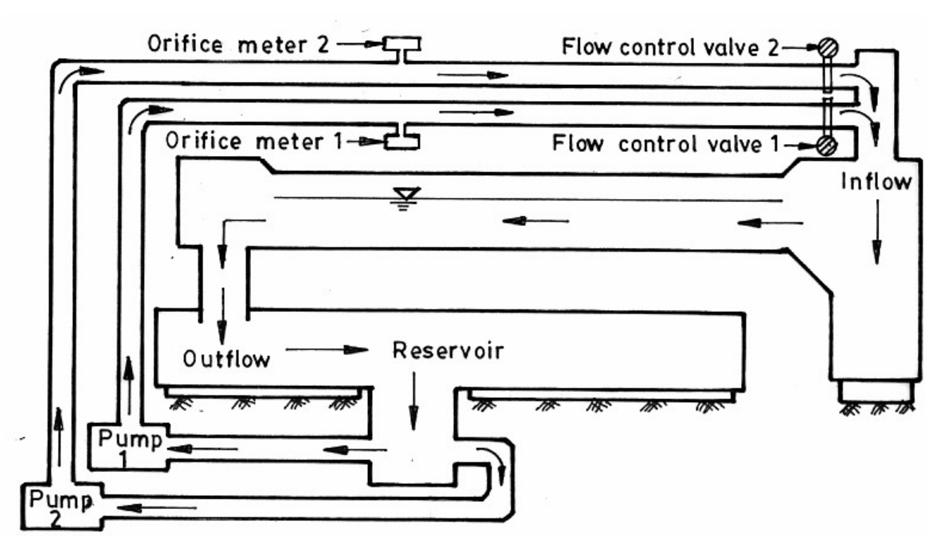


Fig. 9.1 Setup for velocity distribution in open channel

9.5 Procedure

To determine the velocity distribution profile in the vertical

i) Place the current meter at the middle of the flume.

ii) Measure the depth of flow Y.

iii) Place the current meter at water surface, at 0.2Y, 0.4Y, 0.6Y and 0.8Y from the water surface and near the bottom in the vertical plane and take the reading of revolution of the current meter (N) and corresponding time (t) at each depth.

iv) Calculate the point velocities at each depth by using the formula, $v = a(N/t)+b$, where a and b are the current meter constants.

v) Plot the point velocity (v) against the distance from the channel bottom (y).

To calculate the channel roughness height k

i) For every set of point velocity (v) and distance from bottom (y), roughness height (k) can be determined by using Eq.(9.6).

ii) Channel roughness height is obtained by averaging all values of k.

To calculate the cross-sectional mean velocity V

i) By using the average value of k, the cross-sectional mean velocity V is calculated using Eq.(9.7).

To calculate the velocity distribution coefficients α and β

i) Divide the width of the channel (flume) into 5 horizontal strips of equal width Δy.

ii) Find the average velocity (v) in each strip.

iii) Calculate $v^3\Delta y$ and $v^2\Delta y$ for each strip and sum them up.

iv) Using the cross-sectional mean velocity V, calculate α and β using Eqs.(9.10) and (9.11).

9.6 Shape of depth vs velocity graph

As the velocity distribution profile is logarithmic, the y vs v graph is logarithmic as shown in the following figure. For logarithmic velocity distribution, the maximum velocity occurs at the free surface. But in practice, the maximum velocity occurs below the free surface at a distance of 0.05 to 0.25 of the total depth.

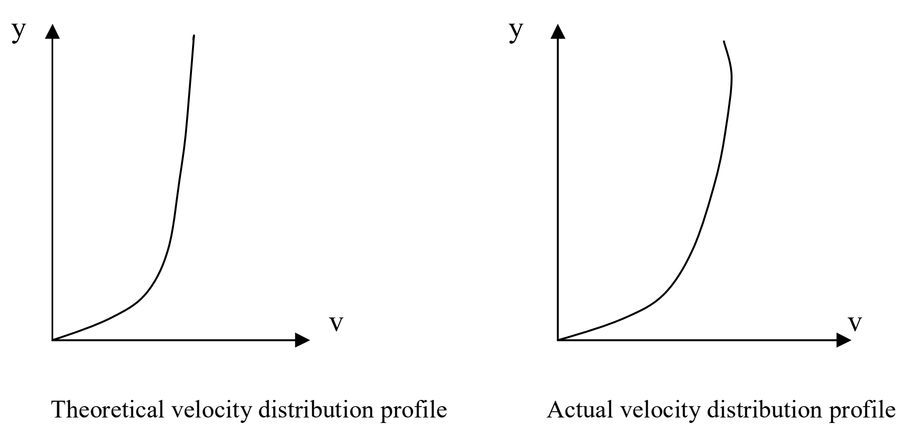


Fig. 9.2 Velocity distribution profile

9.7 Assignment

1. What do you mean by hydraulically smooth and rough channels? What is the criterion used to determine whether a surface is hydraulically smooth or rough?

2. Explain why the velocity distribution over a channel section is not uniform.

3. State whether the numerical values of the energy and momentum coefficients are higher or lower for turbulent flow than for laminar flow. Explain the reason.

DATA SHEET

Experiment Name :
Experiment Date :

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

9.9 Data sheet

Determination of point velocity and roughness height

Flume width, B = m   Total depth of flow, Y = m

Slope of channel, S =   Hydraulic radius, R = m

Shear velocity, V_f = m

Current meter constants, a =   b =

Vertical location of current meter			Current meter reading			Point velocity v (m/s)	Roughness height k (m)	Average k (m)
Location	Depth from water surface (m)	Depth from bottom y (m)	Total no. of revolution N (rev)	Time of observation t (sec)	Revolution per second n (rev/sec)			
At free surface								
At 0.2 Y								
At 0.4 Y								
At 0.6 Y								
At 0.8 Y								
Near bottom								

Determination of energy and momentum coefficients

Cross-sectional mean velocity, V=

No of strip	Average velocity in the strip ,v (m/s)	Thickness of the strip, Δy (m)	$v^2\Delta y$	$v^3\Delta y$	α	β	$(\alpha-1)/(\beta-1)$
1							
2							
3							
4							
5							
$\sum =$							

Course Teacher :
Designation :

Signature

Experiment No. 10

DETERMINATION OF DISCHARGE AND MEAN VELOCITY OF AN OPEN CHANNEL

10.1 General

Measurement of discharge is a principal work in hydrographic surveying. In order to design any river engineering work, the discharge and the mean velocity of the river is required. This experiment mainly deals with the measurement of discharge of a channel by the area-velocity method. Also, the values of Manning's n and Chezy's C are derived which are required to compute the discharge using a uniform flow formula. The relationship of n and C with the depth of flow is also developed. The experiment also deals with the development of depth-discharge relationship of the channel which is very useful to obtain the discharges of a channel for different depths of flow.

10.2 Theory

10.2.1 Depth-mean velocity

The velocity along a vertical varies from zero at the stream bed to maximum at or near the water surface (practically the maximum velocity occurs below the water surface at a distance of 0.05 to 0.25 of the total depth). The average velocity in the vertical is known as the depth-mean velocity. Generally, the average of the velocities at 0.2 and 0.8 depths below the water surface is approximately equal to the mean velocity in the vertical. The velocity at 0.6 depth below the water surface is also approximately equal to the mean velocity in the vertical. Normally, when the depth of flow is greater than 0.61 m (= 2 ft), the depth-mean velocity is determined by averaging the velocities at 0.2 and 0.8 depths; otherwise, the velocity at 0.6 depth is taken as the depth-mean velocity. So

Depth-mean velocity, $V = \frac{v_{0.2} + v_{0.8}}{2} = v_{0.6}$ (10.1)

10.2.2 Discharge

Measurement of discharge in irregular channels like rivers is a complex one. There are different methods and of them the area-velocity method is the familiar one. In this method a channel section is subdivided into a number of segments or strips or pockets by a number of successive verticals. The procedure for determining the mean velocity in a vertical is given in Art.10.2.1. If V_i be the mean velocity in a vertical, then discharge through a strip is given by

$Q_i = V_i \Delta A$ (10.2)

Where, ΔA is the area of the strip. The sum of the discharges through all the strips is the total discharge, i.e.

$Q = \sum Q_i$ (10.3)

10.2.3 Mean velocity

The mean velocity of the whole section is equal to the total discharge divided by the total area and is given by

$V = \frac{Q}{\sum \Delta A}$ (10.4)

10.2.4 Manning's n and Chezy's C

The velocity distribution in an open channel depends on the roughness height k which is related to Manning's n or Chezy's C. When velocities at 0.2 and 0.8 depths are known, Manning's n can be determined by the equation

$$n = \frac{(x-1)\,R^{1/6}}{6.78(x+0.95)} \qquad (10.5)$$

Where,

$$x = \frac{V_{0.2}}{V_{0.8}} \qquad (10.6)$$

With the value of n known, Chezy's C can be determined by the relation

$$C = \frac{1}{n} R^{1/6} \qquad (10.7)$$

This equation provides an important relationship between Chezy's C and Manning's n. The variation of n and C with the depth of flow is also significant. The values of n decreases with the increase in depth of flow. On the other hand, the value of C increases with the increase in depth of flow.

10.3 Objectives of the experiment

1. To determine the total discharge and mean velocity of the flow.
2. To calculate Manning's n and Chezy's C.
3. To plot n and C against depth of flow and observe the relationship between n and C.

10.4 Experimental setup

The setup for the experimental is given below. The longitudinal slope of the flume is 1 in 840.

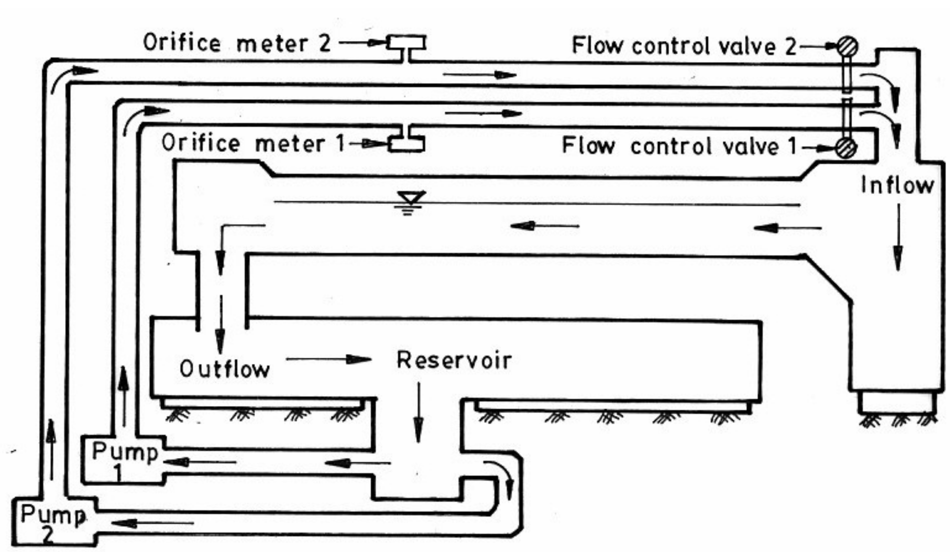


Fig.10.1 Setup for determination discharge and mean velocity of an open channel

10.5 Procedure

To determine the total discharge and the mean velocity of the flow

i) Divide the channel section into 3 vertical strips.
ii) Measure the depth of flow at the middle of each strip.
iii) Determine the mean velocity at each vertical using Eq.(10.1).
iv) Calculate the discharge through each strip using Eq.(10.2).

v) Calculate the total discharge using Eq.(10.3) and compare it with the actual discharge.
vi) Calculate the mean velocity using Eq.(10.4).

To calculate Manning's n and Chezy's C
ii) Compute the value of x at each vertical using Eq.(10.6) and find the average value of x.
iii) Using this average value of x, calculate n using Eq.(10.5).
iv) With the value of n, calculate C using Eq.(10.7).

To plot n and C against depth of flow
i) Plot n vs depth of flow in a plain graph paper.
ii) Plot C vs depth of flow in a plain graph paper.

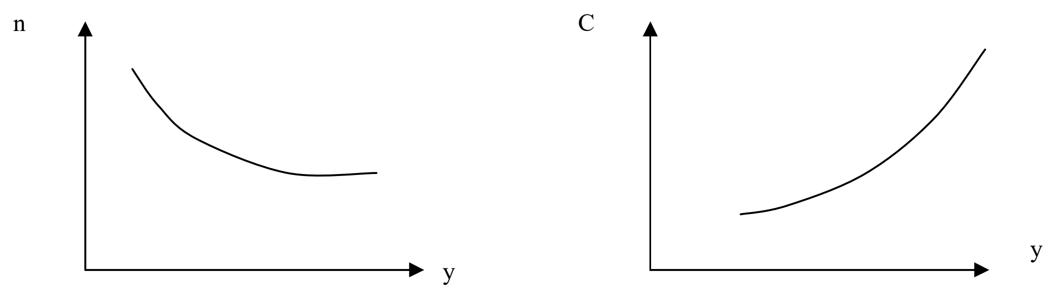
10.6 Shapes of the graphs

Manning's n vs depth of flow y

The exact shape of the curve can not be explained analytically. But experimental study shows that the value of n decreases with the increase in depth of flow.

Chezy's C vs depth of flow y

The exact shape of the curve can not be explained analytically. But experimental study shows that the value of C increases with the increase in depth of flow.



Variation of n with y   Variation of C with y

Fig. 10.2 Chezy's C vs depth of flow y graph

10.7 Assignment

1. State the use(s) of the Chezy and the Manning formulas. State the type(s) of flow for which these formulas are valid.

2. There is a limit to the number of strips or segments in determining the discharge of a river by the area-velocity method. What is the limit and why is this limit?

DATA SHEET

Experiment Name :
Experiment Date :

Student's Name :
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10.9 Data sheet

Determination of total discharge

Flume width, B = m Width of each strip = B/3 = m

Total depth of flow, Y= m Area of each strip, ΔA =(B/3)*Y= m^2

Current meter constants, a = b =

Location of current meter		Current meter reading			Point velocity v (m/s)	Depth-mean velocity V_i (m/s)	Discharge through the strip $Q_i=\Delta A*V_i$ (m^3/s)	Total discharge $Q=\sum Q_i$ (m^3/s)
Horizontal	Vertical	Total no. of revolution N (rev)	Time of observation t (sec)	Revolution per second n (rev/sec)				
At middle of first strip	At 0.2 Y							
	At 0.6 Y							
	At 0.8 Y							
At middle of second strip	At 0.2 Y							
	At 0.6 Y							
	At 0.8 Y							
At middle of third strip	At 0.2 Y							
	At 0.6 Y							
	At 0.8 Y							

So, mean velocity of the stream, $V = \frac{Q}{\sum \Delta A} =$

Calculation of n and C

Strip	Point velocity	X	Average x	Manning's n	Chezy's C
1	$v_{0.2}$				
	$v_{0.8}$				
2	$v_{0.2}$				
	$v_{0.8}$				
3	$v_{0.2}$				
	$v_{0.8}$				

Plotting n and C vs depth of flow

Depth of flow	Manning's n	Chezy's C

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Appendix
Lab Report Format

1. All students must have a same colored printed **cover page**. The design of cover page is provided with the lab manual. Students have to compose only the course teacher's name and designation ant their information.

2. An **index** is provided. It should be printed and set after the cover page. Table may be filling up by pen during each submission after test.

3. Each report must have a common printed **top page**. Only the experiment name and no. and the date may be filled up by pen. A top page design is provided.

4. **A4 papers** have to be used for preparing the lab report. Writing should be done with **pen**. Pencil may be used for any kind of sketch.

5. In each experiment of the lab report the following points must have to be present: **Objective, Equipment, Procedure, Data Table (signed), Sample Calculation, Result** and **Discussion**.

References

Chow, V. T (1957): Open Channel Hydraulics
Daugherty, R. L. and Franzini, J. B.: Fluid Mechanics with Engineering Applications
French, R.H (1980): Open channel Hydraulics
Henderson, F.M. :Open Channel Flow
Kraatz,D.B. and Mahajan,I.K.: Small Hydraulic Structures (FAO Irrigation and Drainage paper)
Michael, A.M.: Irrigation Theory and Practices
Sutradhar**,** S.C.: Principles of Design of Drainage Sluice

CE 374
Water Resources Engineering Sessional -I
(Lab Report)

Prepared For
Name of Course Teacher
Designation of Course Teacher
&
Name of Course Teacher
Designation of Course Teacher

Prepared By
Name of Student
Student's ID
Year/ Semester
Group

Name:
ID:

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Test no.	Test Name	Date of Performance	Date of Submission	Signature	Remarks