"Heaven's Light is Our Guide"

Rajshahi University of Engineering & Technology

RUET

Dept. of Civil Engineering

Course No: CE 3122

Course Name: Engineering Hydraulics Sessional

Submitted By:	**Submitted To:**
Name: Most. Afrin Sultana **Roll No:** 1700082 **Section:** B **Class:** 3rd year odd semester **Session:** 2017-18	**Dr. H.M. Rasel** Professor Dept. of Civil Engineering, RUET **Mahmud Hasan** Lecturer Dept. of Civil Engineering, RUET

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Heaven's Light Is Our Guide

RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 01

Name of Expt. Flow through a venture flume

SUBJECT: Engineering Hydraulics sessional	**SUBMITED BY :**
COURSE NO. : CE 3122	NAME: Most. Afroin Sultana
DATE OF EXPT. : 03-01-2021	GROUP :
DATE OF SUB. : 16-03-2021	ROLL NO: 1700082
	SESSION : 2017-18

Date: 03-01-21

Experiment No: 01

Experiment Name: Flow through a venture flume.

1. Introduction:

1.1 Flume:

- Flumes are open channel flow sections that force flow to accelerate. Acceleration is produced by converging the side walls, rising the bottom or a combination of both. Flumes are designed to force a transition from sub-critical to super-critical flow.

- Such a transition causes flow to pass through critical depth at flume throat. At the critical depth, energy is minimum and there is direct relationship between the water head and flow head. However, in super-critical flow the depth is small but velocity is large.

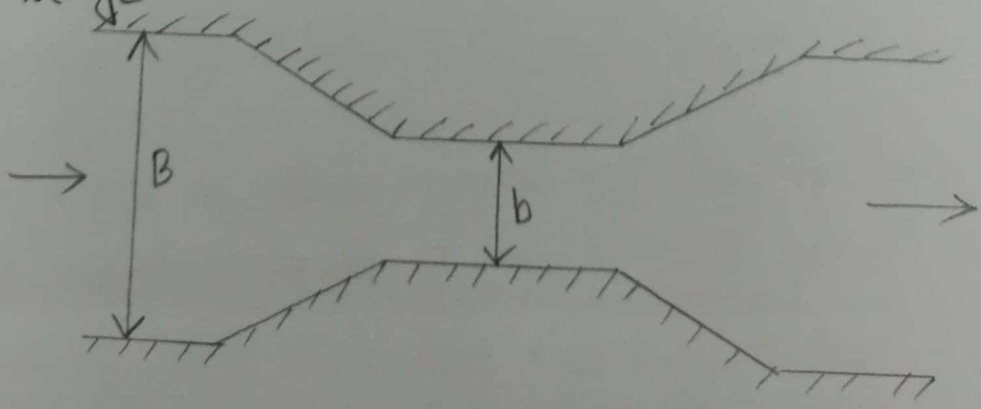

Fig. 1.a. Plan of the flume.

1.2 Venture-flume:

- Venture flume has a converging section, a throat section and a diverging section. A control flume that comprises a short constructed section followed by one expanding to normal width.
- The bed level is kept horizontal.
- The streamlines run parallel to each other at least over a short distance upstream of the flume.

2. Theory:

2.1. Dischange at Free Flow Condition:

- In case of free flow flume, the velocity of flow at the throat of the flume is greater than the critical velocity. Free flow is formed at the downstream end of the raised floor. Since the velocity and ~~flow~~ depth of flow at a section upstream of the entrance of the flume remains unaffected by downstream depth.
- It may be thus mentioned that as long as the downstream depth of flow is kept below the limiting values, the dischange passing through a free flow will be a function of only the depth of flow at

upstream to the entrance section.

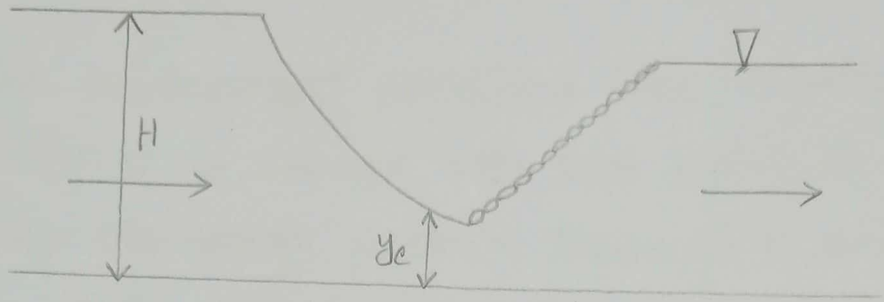

Fig. 1.b. Free flow condition.

2.1.1 Required formula for free flow condition:

Theoretical discharge:

$$Q_{tf} = \left(2/3\right)^{1.5} \sqrt{g}\, b\, H^{1.5} \quad \text{——— (i)}$$

Where, H is the head measured sufficiently upstream of the flume, b is the width of the venture flume at the ~~see~~ throat section.

Co-efficient of discharge:

$$C_{df} = \frac{Q_a}{Q_{tf}}$$

Where,

Q_a = Actual discharge

Q_{tf} = Theoretical discharge

2.2 Discharge at submerged flow Condition :

- For the submerged condition, the velocity of flow at the throat is always less than critical velocity and hence discharge passing through it will be a function of the difference between the depth of flow upstream of the entrance section and at the throat. Further since the velocity of flow at the throat is less than the critical velocity.

- The discharge Q passing through the channel can be calculated by measuring the depth of flow at the entrance and at the throat of the flume and applying the following formula.

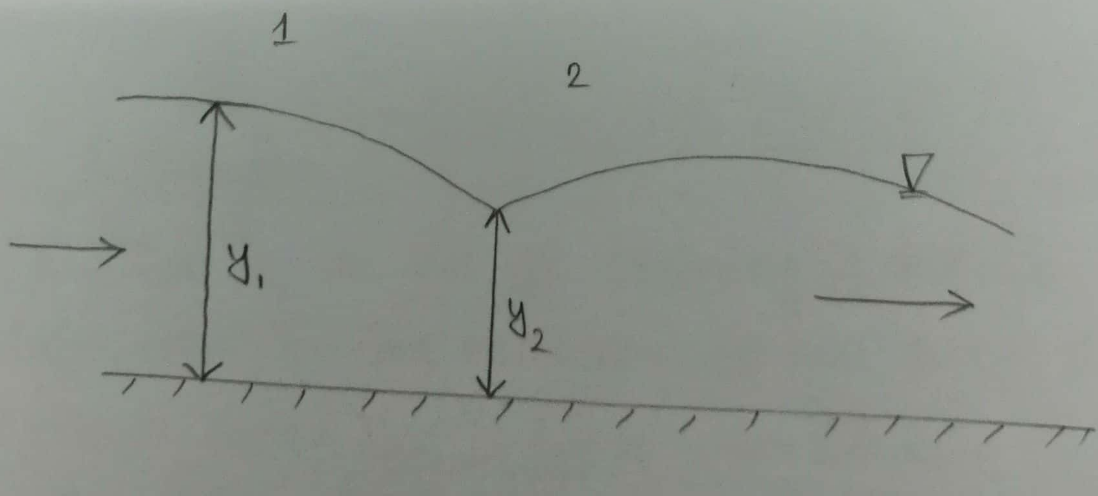

Fig. 1.c. Submerged flow condition.

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$$Q_{ts} = a\sqrt{2g(y_1 - y_2)} \quad \frac{1}{\sqrt{1-M^2}} \qquad \text{—————— (ii)}$$

Co-efficient of discharge,

$$C_{ds} = \frac{Q_a}{Q_{ts}}$$

Where, Q_a = Actual discharge

Q_{ts} = Theoretical discharge

a = Throat area = $y_2 \times b$

b = Width of the throat

h = Depth of flow at the throat

A = Upstream area = $y_1 \times B$

B = Width of entrance

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

2.3 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. $Q = ky^n$. This relation is known as the calibration equation for the device. So, calibration deals with determination of k and n and develop the equation $Q = ky^n$. The plotting of the calibration equation is known as calibration curve. There are two different ways to develop a calibration equation. These are

1) Plotting best fit line by eye estimation.
2) Developing best fil line by regression.

By eye estimation

As $\log Q = \log k + n \log y$, so if Q and y are plotted in a log log paper, the line will represent a straight line. Different sets of Q and y are plotted in a log log paper keeping y 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 from the equation $Q = ky^n$.

3. Objectives of the experiment :

1. Observation of free flow condition and submerged flow condition.

2. Determination of the co-efficient of discharge.

3. To plot actual discharge with respect to theoretical discharge in the plain graph paper.

4. To plot actual discharge with respect to upstream head in the log-log graph paper

5. Determination of exponent of y from the log-log paper.

4. Experimental setup:

The experimental setup for this experiment is given below.

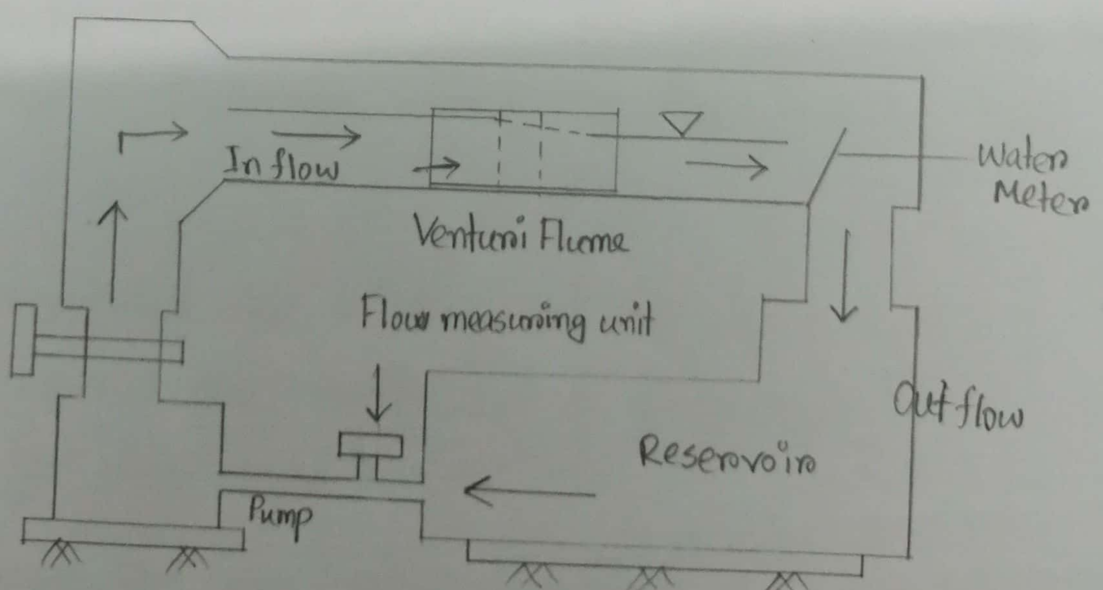

Fig. 1.d. Setup for flow through a venturi flume.

4.1: Required Apparatus:

a) Venturi flume
b) Pump
c) Flow measuring unit
d) Reservoir
e) Water meter

5. Procedure:

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

1) The depth of flow sufficiently upstream of the flume was measured and the theoretical discharge using equiation(i) was determined.

2) The reading of actual discharge was taken and hence the coefficient of discharge using Eq: $C_{df} = Q_a/Q_{tf}$ was found.

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

1) ~~Measure~~ The flow depths at sections 1 and 2 shown in fig. 1.c. ~~and det~~ are measured and the theoretical

discharge using Eq. (ii) was determined.

2) The reading of actual discharge was taken and hence the coefficient of discharge using Eq. $C_{ds} = Q_a/Q_{ts}$ was found.

To calibrate the flume (for free flow condition and submerged flow condition) by eye estimation

1) ~~Plot~~ The actual discharge against the corresponding upstream depth was plotted in a log log paper and the values of n and k as discussed in Art 2.3 were found.

2) The relationship $Q = ky_1^n$ was developed.

5.1 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 $y = mx + c$)

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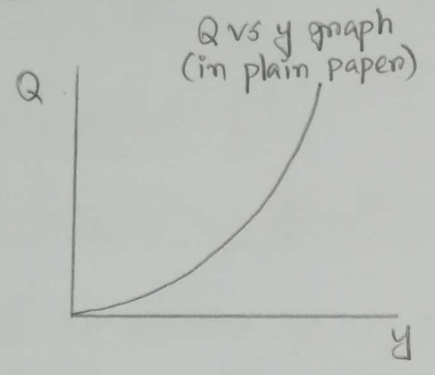

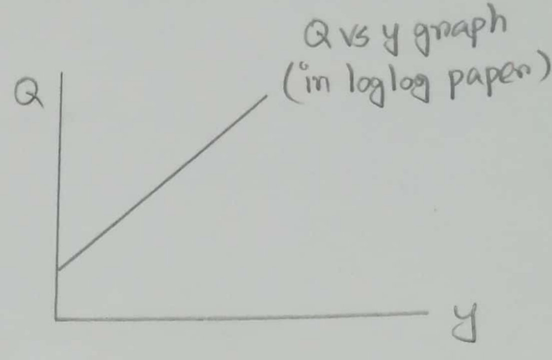

Fig. 1.e. Q (actual discharge) vs y (upstream depth of water) graph.

6. Data sheet:

Channel width, B = 7.75 cm Throat width, b = 1.82 cm

No. of Obs.	Actual Discharge Q_a (cm^3/s)	Free flow condition				Submerged flow condition					Mean C_{ds}
		H (cm)	Q_{tf} (cm^3/s)	C_{df}	Mean C_{df}	y_1 (cm)	y_2 (cm)	$M = a/A$	Q_{ts} (cm^3/s)	C_{ds}	
1.	500	8.00	702.11	0.71	0.75	8.20	5.80	0.17	735.34	0.68	0.69
2.	600	9.00	837.79	0.72		9.40	6.70	0.17	900.33	0.67	
3.	700	9.70	937.40	0.75		10.55	7.60	0.17	1067.70	0.66	
4.	800	10.40	1040.68	0.79		10.70	8.40	0.18	1044.17	0.77	
5.	900	11.10	1147.50	0.78		12.00	9.00	0.18	1277.54	0.70	
6.	1000	12.00	1289.85	0.78		13.30	9.50	0.17	1514.97	0.66	
7.	1100	13.35	1513.53	0.73		14.15	10.20	0.17	1658.04	0.66	

7. Calculation :

Observation 1:

For free flow condition:

Head of water at upstream, $H = 8.00\,cm$

Width of throat, $b = 1.82\,cm$

Actual discharge, $Q_a = 500\,cm^3/s$

We know,

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

$$= (2/3)^{1.5} \times \sqrt{981} \times 1.82 \times (8)^{1.5}$$

$$= 702.11\,cm^3/s$$

$$C_{df} = \frac{Q_a}{Q_{tf}}$$

$$= \frac{500}{702.11}$$

$$= 0.71$$

For submerged flow condition:

Head of water at upstream, $y_1 = 8.20$ cm

Depth of flow at throat, $y_2 = 5.80$ cm

Channel width, $B = 7.75$ cm

Throat area, $a = y_2 \times b$

$= 5.80 \times 1.82 = 10.56 \text{ cm}^2$

Upstream area, $A = y_1 \times B$

$= 8.20 \times 7.75 = 63.55 \text{ cm}^2$

$$M = \frac{a}{A} = \frac{10.56}{7.75} = 0.17$$

We know,

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

$$= 10.56 \times \sqrt{2 \times 981 (8.20 - 5.80)} \times \frac{1}{\sqrt{1-(0.17)^2}}$$

$$= 735.34 \text{ cm}^3/\text{s}$$

$$C_{ds} = \frac{Q_a}{Q_{ts}}$$

$$= \frac{500}{735.34}$$

$$= 0.68$$

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Observation 2:

For free flow condition:

$H = 9.00 cm$

$b = 1.82 cm$

$Q_a = 600 cm^3/s$

We know,

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

$$= (2/3)^{1.5} \sqrt{981} \times 1.82 \times (9)^{1.5}$$

$$= 837.79 \ cm^3/s$$

$$C_{df} = \frac{Q_a}{Q_{tf}}$$

$$= \frac{600}{837.79}$$

$$= 0.72$$

For submerged flow condition:

$y_1 = 9.40 \ cm$

$y_2 = 6.70 \ cm$

$B = 7.75 \ cm$

$a = y_2 \times b = (6.70 \times 1.82) \ cm^2 = 12.19 \ cm^2$

$A = y_1 \times B = (9.40 \times 7.75) \ cm^2 = 72.85 \ cm^2$

$M = a/A = \frac{12.19}{72.85} = 0.17$

We know,

$$Q_{t5} = a\sqrt{2g\,(y_1 - y_2)}\;\frac{1}{\sqrt{1-M^2}}$$

$$= 12.19 \times \sqrt{2\times981\times(9.40-6.70)} \times \frac{1}{\sqrt{1-(0.17)^2}}$$

$$= 900.33 \text{ cm}^3/s$$

$$C_{d5} = \frac{Q_a}{Q_{t5}}$$

$$= \frac{600}{900.33}$$

$$= 0.67$$

Observation 3:

For free flow condition:

$H = 9.70$ cm

$b = 1.82$ cm

$Q_a = 700 \text{ cm}^3/s$

We know,

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

$$= (2/3)^{1.5} \times \sqrt{981} \times 1.82 \times (9.70)^{1.5}$$

$$= 937.40 \text{ cm}^3/s$$

$$C_{df} = \frac{Q_a}{Q_{tf}}$$

$$= \frac{700}{937.40} = 0.75$$

For submerged flow condition:

$y_1 = 10.55$ cm

$y_2 = 7.60$ cm

$B = 7.75$ cm

$a = y_2 \times b = (7.60 \times 1.82)\ cm^2 = 13.83\ cm^2$

$A = y_1 \times B = (10.55 \times 7.75)\ cm^2 = 81.76\ cm^2$

$$M = a/A = \frac{13.83}{81.76} = 0.17$$

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

$$= 13.83 \times \sqrt{2 \times 981 \times (10.55 - 7.60)} \times \frac{1}{\sqrt{1-(0.17)^2}}$$

$$= 1067.70\ cm^3/s$$

$$C_{ds} = \frac{Q_a}{Q_{ts}}$$

$$= \frac{700}{1067.70} = 0.66$$

Observation 4:

For free flow condition:

$H = 10.40$ cm

$b = 1.82$ cm

$Q_a = 800\ cm^3/s$

We know,

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

$$= (2/3)^{1.5} \times \sqrt{981} \times 1.82 \times (10.40)^{1.5}$$

$$= 1040.68\ cm^3/s$$

$$C_{df} = \frac{Q_a}{Q_{tf}}$$

$$= \frac{800}{1040.68} = 0.79$$

For submerged flow condition:

$y_1 = 10.70$ cm

$y_2 = 8.40$ cm

$B = 7.75$ cm

$a = y_2 \times b = (8.40 \times 1.82)\ cm^2 = 15.29\ cm^2$

$A = y_1 \times B = (10.70 \times 7.75)\ cm^2 = 82.93\ cm^2$

$$M = \frac{a}{A} = \frac{15.29}{82.93} = 0.18$$

We know,

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

$$= 15.29 \times \sqrt{2 \times 981 \times (10.70 - 8.40)} \times \frac{1}{\sqrt{1-(0.18)^2}}$$

$$= 1044.17 \text{ cm}^3/s$$

$$C_{ds} = \frac{Q_a}{Q_{ts}}$$

$$= \frac{800}{1044.17} = 0.77$$

Observation 5:

For free flow condition:

$H = \cancel{11.5}$ 11.10 cm

$b = 1.82$ cm

$Q_a = 900$ cm³/s

We know,

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

$$= (2/3)^{1.5} \times \sqrt{981} \times 1.82 \times (11.10)^{1.5}$$

$$= 1147.50 \text{ cm}^3/s$$

$$C_{df} = \frac{Q_a}{Q_{tf}} = \frac{900}{1147.50} = 0.78$$

For submerged flow condition:

$y_1 = 12.00$ cm

$y_2 = 9.00$ cm

$B = 7.75$ cm

$a = y_2 \times b = (9 \times 1.82)\ cm^2 = 16.38\ cm^2$

$A = y_1 \times B = (12 \times 7.75)\ cm^2 = 93\ cm^2$

$M = \frac{a}{A} = \frac{16.38}{93} = 0.18$

We know,

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

$$= 16.38 \times \sqrt{2 \times 981 \times (12-9)} \times \frac{1}{\sqrt{1-(0.18)^2}}$$

$$= 1277.54\ cm^3/s$$

$$C_{ds} = \frac{Q_a}{Q_{ts}}$$

$$= \frac{900}{1277.54}$$

$$= 0.70$$

Observation 6:

For free flow condition:

$H = 12.00$ cm

$b = 1.82$ cm

$Q_a = 1000$ cm^3/s

We know,

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

$$= (2/3)^{1.5} \times \sqrt{981} \times 1.82 \times (12)^{1.5}$$

$$= 1289.85 \ cm^3/s$$

$$C_{df} = \frac{Q_a}{Q_{tf}}$$

$$= \frac{1000}{1289.85} = 0.78$$

For submerged flow condition:

$y_1 = 13.30$ cm

$y_2 = 9.50$ cm

$B = 7.75$ cm

$a = y_2 \times b = (9.50 \times 1.82)\ cm^2 = 17.29\ cm^2$

$A = y_1 \times B = (13.30 \times 7.75)\ cm^2 = 103.08\ cm^2$

$$M = \frac{a}{A} = \frac{17.29}{103.08} = 0.17$$

We know,

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

$$= 17.29 \times \sqrt{2 \times 981 \times (13.30 - 9.50)} \times \frac{1}{\sqrt{1-(0.17)^2}}$$

$$= 1514.97 \text{ cm}^3/s$$

$$C_{ds} = \frac{Q_a}{Q_{ts}}$$

$$= \frac{1000}{1514.97} = 0.66$$

Observation 7:

For free flow condition:

$H = 13.35$ cm

$b = 1.82$ cm

$Q_a = 1100 \text{ cm}^3/s$

We know,

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

$$= (2/3)^{1.5} \times \sqrt{981} \times 1.82 \times (13.35)^{1.5}$$

$$= 1513.53 \text{ cm}^3/s$$

$$C_{df} = \frac{Q_a}{Q_{tf}}$$

$$= \frac{1100}{1513.53} = 0.73$$

For submerged flow condition:

$y_1 = 14.15$ cm

$y_2 = 10.20$ cm

$B = 7.75$ cm

$a = y_2 \times b = (10.20 \times 1.82)\ cm^2 = 18.56\ cm^2$

$A = y_1 \times B = (14.15 \times 7.75)\ cm^2 = 109.66\ cm^2$

$M = \frac{a}{A} = \frac{18.56}{109.66} = 0.17$

We know,

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

$$= 18.56 \times \sqrt{2 \times 981\ (14.15 - 10.20)} \times \frac{1}{\sqrt{1-(0.17)^2}}$$

$$= 1658.04\ cm^3/s$$

$$C_{ds} = \frac{Q_a}{Q_{ts}}$$

$$= \frac{1100}{1658.04}$$

$$= 0.66$$

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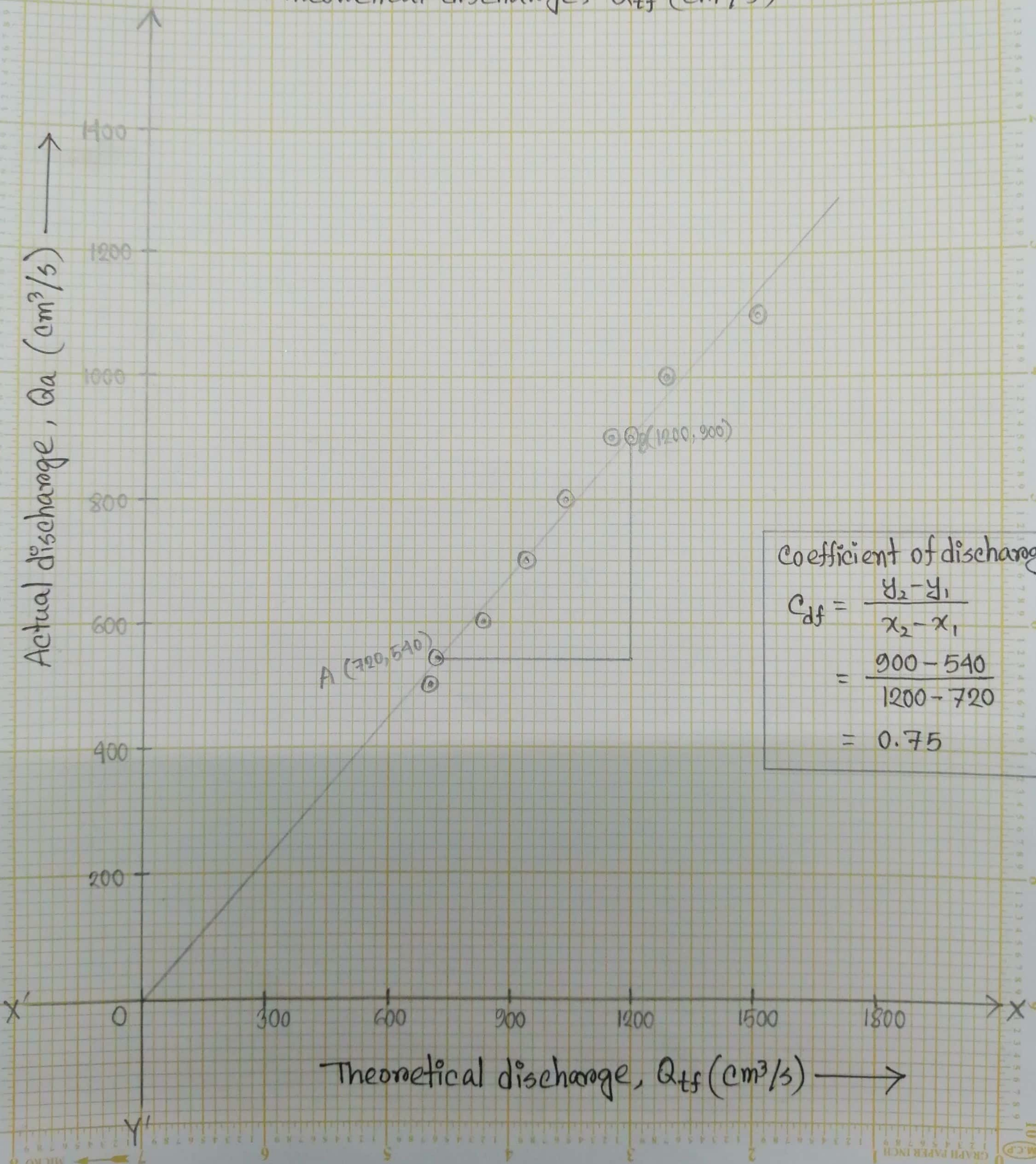
For free flow condition

Variation of Actual discharge, Qa (cm^3/s) with respect to Theoretical discharge, Q_{tf} (cm^3/s)



Coefficient of discharge,

$$C_{df} = \frac{y_2 - y_1}{x_2 - x_1}$$

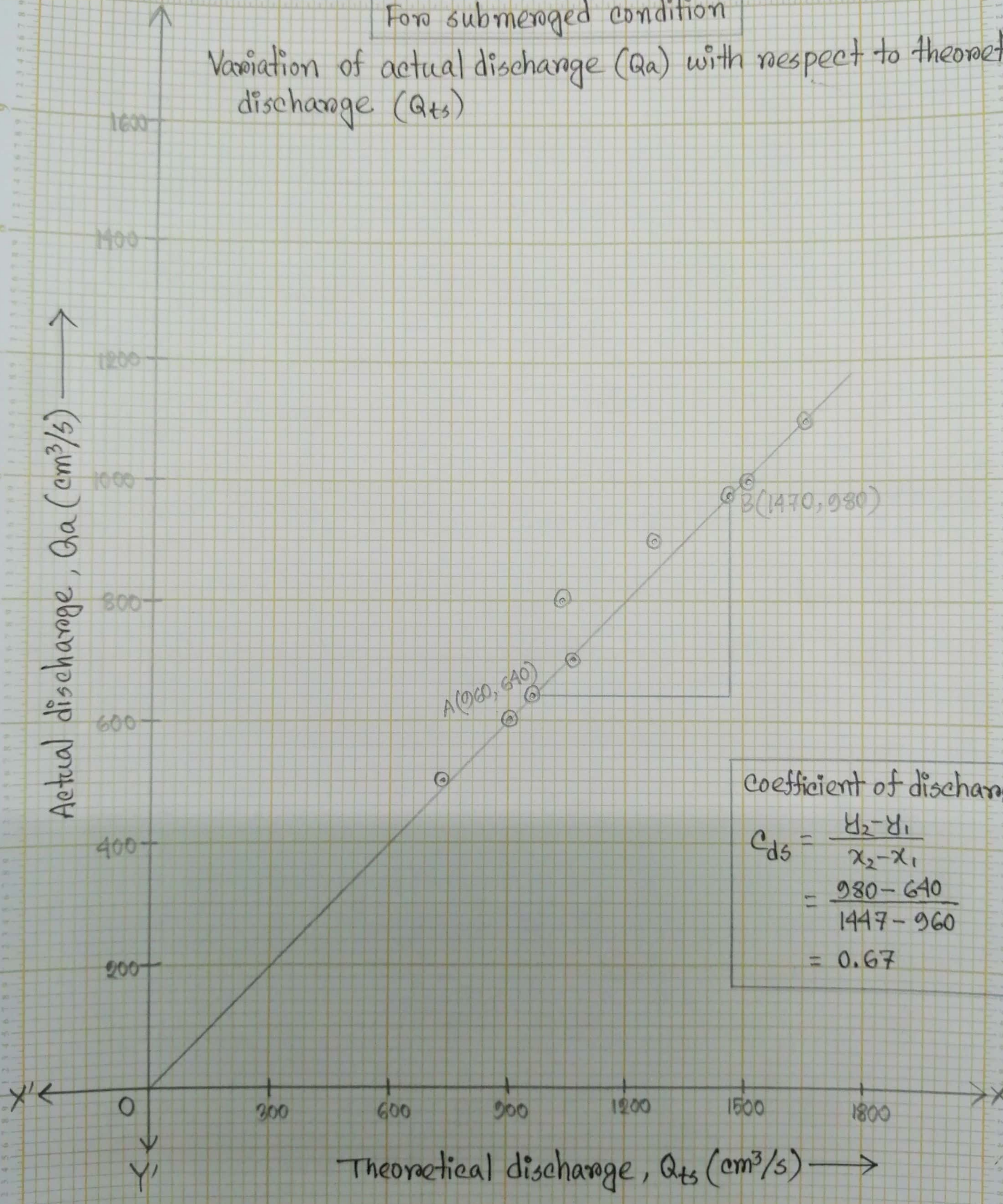
$$= \frac{900 - 540}{1200 - 720}$$

$$= 0.75$$

1700082

For submerged condition

Variation of actual discharge (Q_a) with respect to theoretical discharge (Q_{ts})



Coefficient of discharge,

$$C_{ds} = \frac{y_2 - y_1}{x_2 - x_1}$$

$$= \frac{980 - 640}{1447 - 960}$$

$$= 0.67$$

For Free flow condition

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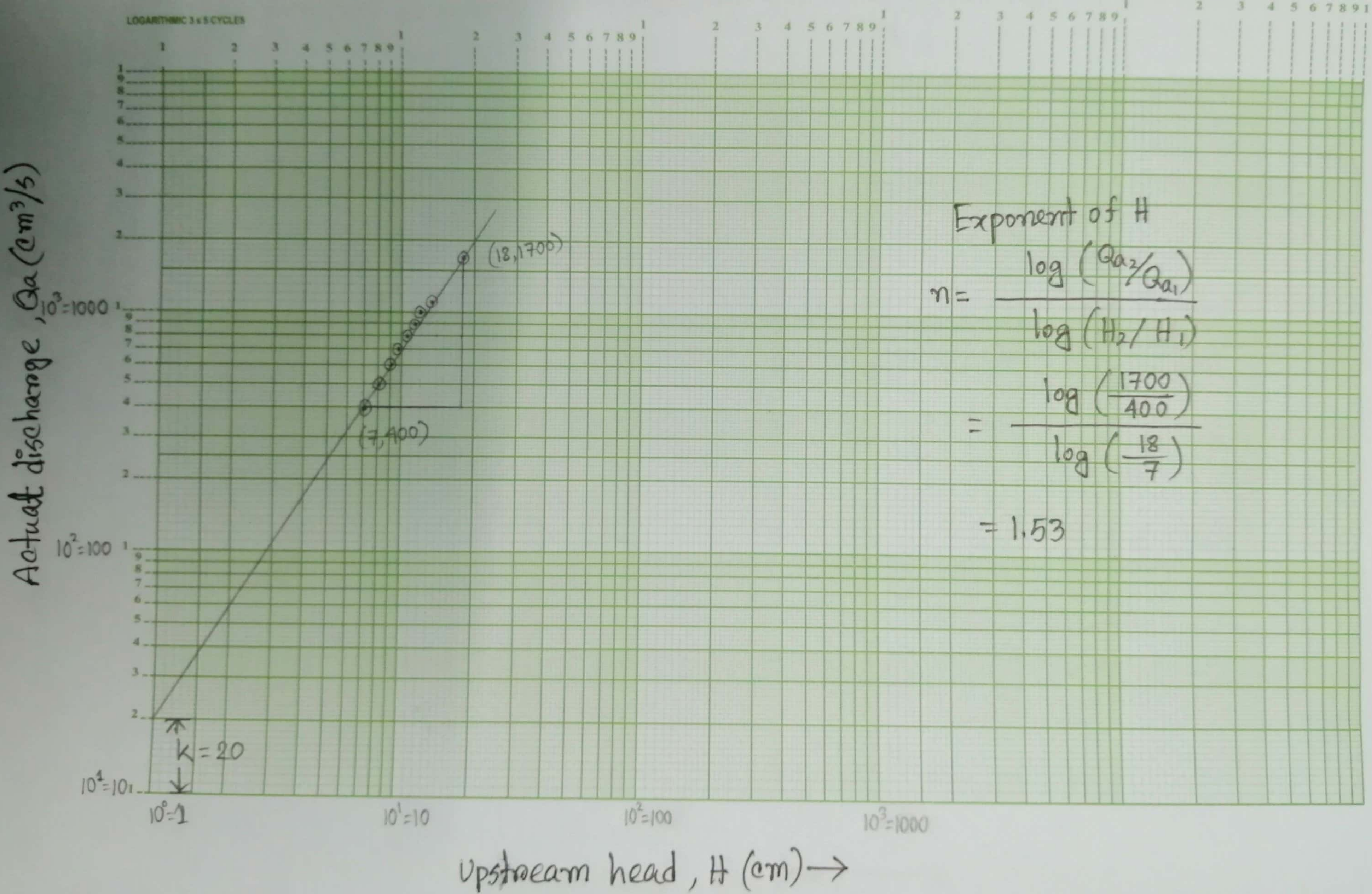
Variation of Actual discharge (Qa) with respect to upstream head (H)



Exponent of H

$$n = \frac{\log\left(Q_{a_2}/Q_{a_1}\right)}{\log\left(H_2/H_1\right)}$$

$$= \frac{\log\left(\frac{1700}{400}\right)}{\log\left(\frac{18}{7}\right)}$$

$$= 1.53$$

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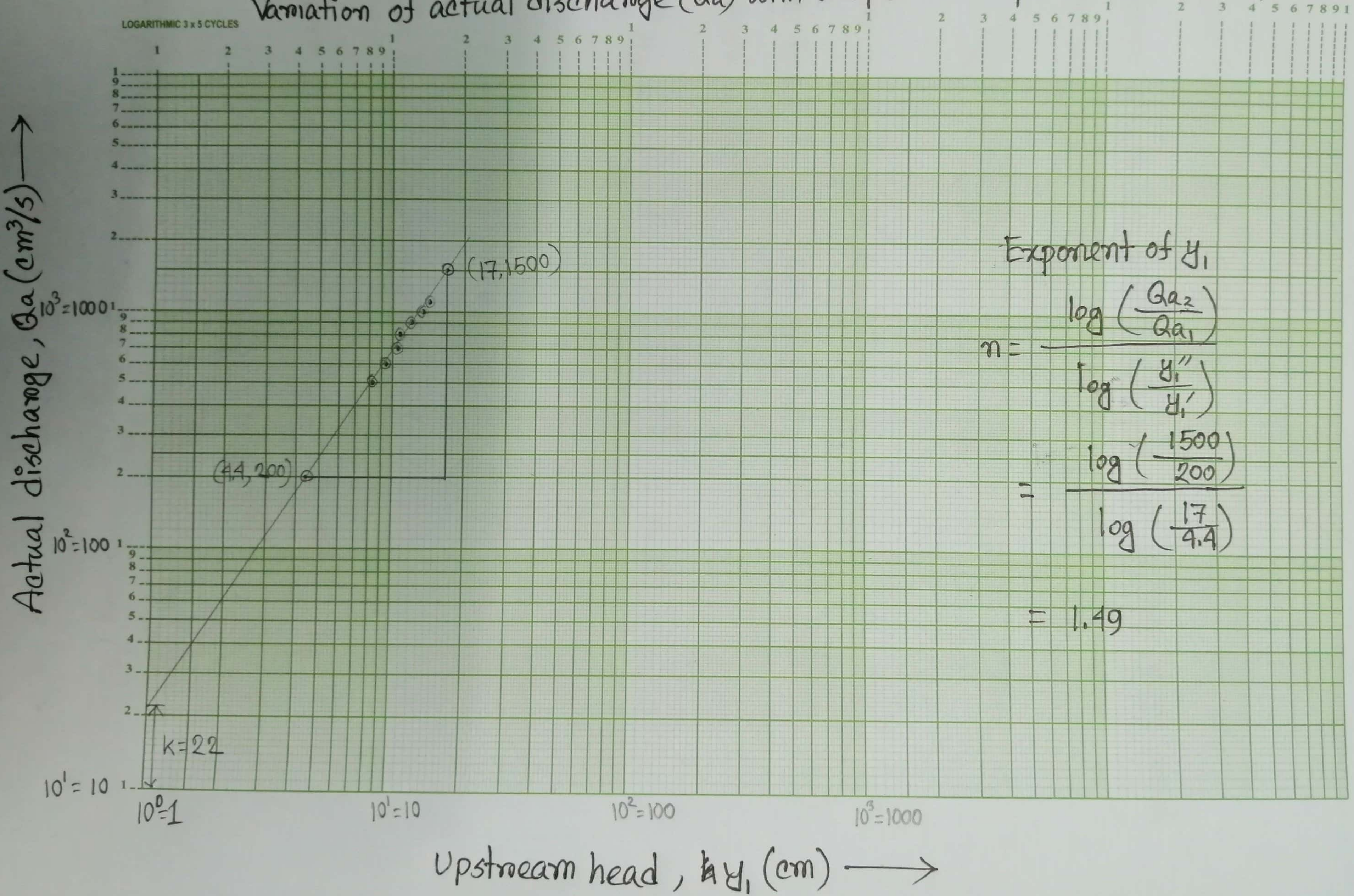
For submerged flow condition

Variation of actual discharge (Q_a) with respect to upstream head (y_1)



Exponent of y_1

$$n = \frac{\log\left(\frac{Q_{a_2}}{Q_{a_1}}\right)}{\log\left(\frac{y_1''}{y_1'}\right)}$$

$$= \frac{\log\left(\frac{1500}{200}\right)}{\log\left(\frac{17}{4.4}\right)}$$

$$= 1.49$$

7.1 Calculation from plain graph :

For free flow condition :

From graph we take two points A(720, 540) and B(1200, 900)

$$\therefore C_{df} = \frac{y_2 - y_1}{x_2 - x_1} = \frac{900-540}{1200-720} = 0.75$$

For submerged condition:

We take two points A(960, 640) and B(1470, 980) from graph.

$$\therefore C_{ds} = \frac{y_2 - y_1}{x_2 - x_1} = \frac{980-640}{1447-960} = 0.67$$

7.2 Calculation from log-log graph:

For free flow condition:

From graph we get, $Q_{a_1} = 400\ cm^3/s$, $H_1 = 7\ cm$

$Q_{a_2} = 1700\ cm^3/s$, $H_2 = 18\ cm$

$$\therefore \text{Exponent of } H_g\ n = \frac{\log\left(\frac{Q_{a_2}}{Q_{a_1}}\right)}{\log\left(\frac{H_2}{H_1}\right)} = \frac{\log\left(\frac{1700}{400}\right)}{\log\left(\frac{18}{7}\right)}$$

$$= 1.53$$

For submerged flow condition:

From graph we get,

$Qa_1 = 200\ cm^3/s$, $y_1' = 4.4\ cm$

$Qa_2 = 1500\ cm^3/s$, $y_1'' = 17\ cm$

$$\therefore \text{Exponent of } y_1,\ n = \frac{\log\left(\frac{Qa_2}{Qa_1}\right)}{\log\left(\frac{y_1''}{y_1'}\right)} = \frac{\log\left(\frac{1500}{200}\right)}{\log\left(\frac{17}{4.4}\right)}$$

$$= 1.49$$

8. Result:

For free flow condition:

a) Co-efficient of discharge C_{df} = 0.75 [From calculation]

b) Co-efficient of discharge, C_{df} = 0.75 [From plain graph]

c) Exponent of upstream head, n = 1.53 [From log-log graph]

For submerged flow condition:

a) Co-efficient of discharge, C_{ds} = 0.69 [From calculation]

b) Co-efficient of discharge, C_{ds} = 0.67 [From plain graph]

c) Exponent of upstream head, n = 1.49 [From log-log graph]

9. Discussion:

i) The co-efficient of discharge at free flow condition is larger than that of at submerged flow condition. This was due to the frictional head losses for hydraulic jump at submerged condition.

ii) The exponent of water head at upstream, H was deviated from theoretical value due to the experimental error and frictional losses.

Assignments

1. What are the advantages, disadvantages and uses of a venturi flume?

Answer:

a) Advantages of venturi flume:

I) It reduced relatively high head loss.

II) Most weirs create a dead water zone upstream of the installation which can serve as a setting basin for sediment and other debris present in the flow, But in venturi flume, no dead water is created.

III) The velocity of flow at the throat section is always less than critical velocity and hence the discharge passing through it will be a function of the difference between the depths of flow upstream of the entrance section and the throat.

IV) Since, the velocity at throat section is less than critical section velocity, hydraulic jump will not form at any section in the venturi flume.

b) Dis-advantages of venturi flume:

i) A venturi flume has a dis-advantages that, there is a relatively small head difference between the upstream section and critical section.

c) Uses of venturi flume:

i) Venturi flume is used widely in irrigation canal for measuring discharge.

ii) It is also used as an outlet of canal to supply water in the field or distributary canal.

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?

Answer: The difference between free flow and submerged flow is that, in case of free flow there exists a critical section and hydraulic jump may occur. But in case of submerged flow, there exists no critical section and hydraulic jump.

In laboratory flume, submerged flow can be produced by placing an obstacle on the channel section with certain contain, when there is no existance of critical section and hydraulic jump, submerged flow occurs in the flume.

The ratio H_2/H_1 where H_2 = downstream water surface elevation measured above the weir crest is called submergence ratio. In submerged flow, the dischange over the weir depends upon the sub-mergence ratio. Subcritical flow prevails all over the flume where the throat is submerged by the tail water. Submergence is larger than the modular limit and the co-efficient of discharge decreases with the submergence ratio H_2/H_1 at a rapid rate.

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Heaven's Light Is Our Guide

RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. ...02.........

Name of Expt. ...Flow through a Parshall flume...........

....

	SUBMITED BY :
SUBJECT: Engineering Hydraulics sessional	**NAME:** Most. Afrin Sultana
COURSE NO. : CE 3122	**GROUP :**....................
DATE OF EXPT. : 20-01-2021	**ROLL NO:** 1700082
DATE OF SUB. : 16-03-2021	**SESSION :** 2017-18

Date: 20-01-2021

Experiment No: 02

Experiment Name: Flow through a Parshall flume.

1. Introduction:

- 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 ~~a~~ 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 ~~b~~ 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, 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.

1.1 Parshall 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.

- The Parshall flume is fixed hydraulic structure originally developed to measure surface water and irrigation flows.
- The Parshall flume is now frequently used to measure industrial discharges, municipal sewer flows, and influent / effluent at wastewater treatment plants. Of all the flumes, the Parshall flume is the most recognized and commonly used.

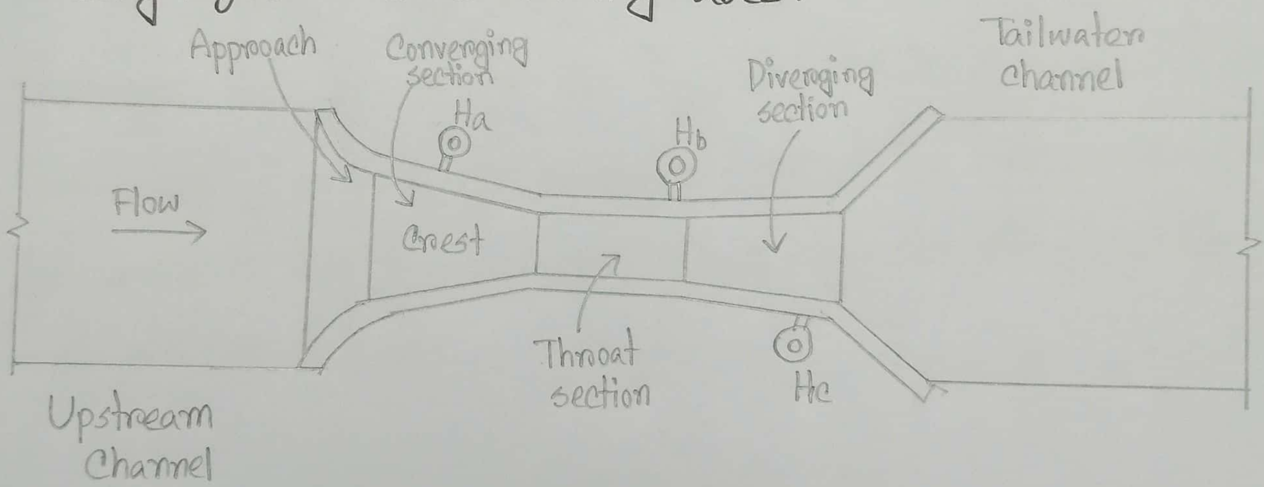

Fig. 2.a. Plan.

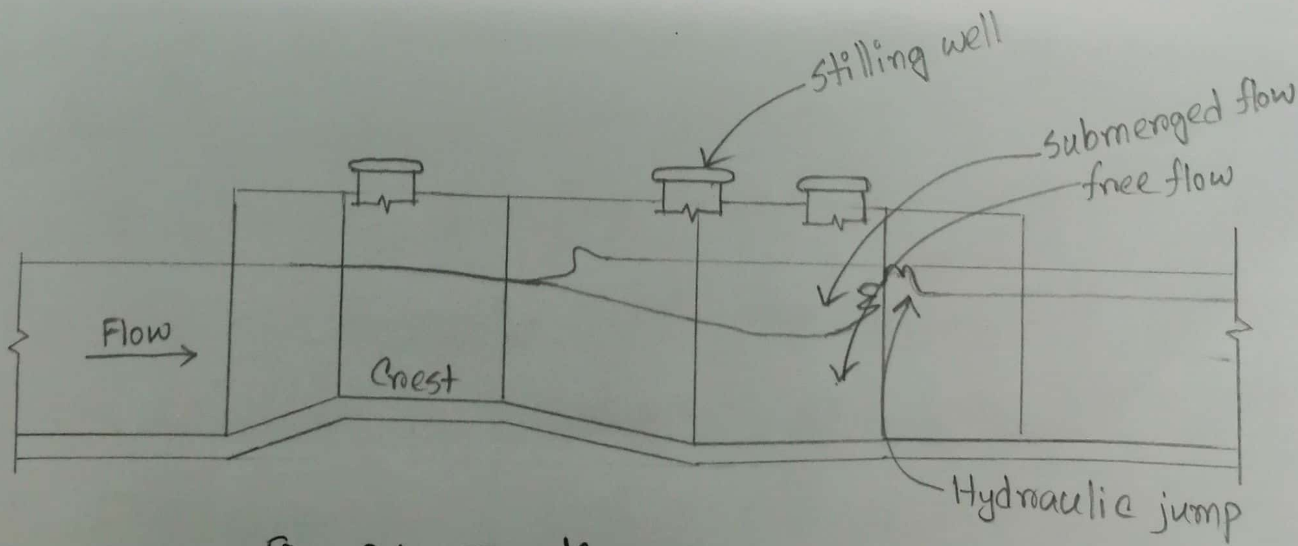

Fig. 2.b. Elevation.

1.2 Development of the Parshall Flume:

Development of the Parshall flume began in 1915 by Dr. R. L. Parshall of the U.S. Soil Conservation Service. Using the sub-critical venturi flume as his starting point, Dr. Parshall introduced several radical modifications with his Improved Venturi Flume – the most greatest of which was a drop in elevation through the throat of the flume. The drop produced supercritical flow through the throat of the flume. With supercritical flow, only one head measurement is necessary to determine the flow rate, greatly simplifying the use of the flume.

1.3 Design of the Parshall Flume:

The design of the Parshall flume consists of a uniformly converging upstream section, a short parallel throat section (the width of which determines the flume size), and a uniformly diverging downstream section. The floor of the flume is flat in the upstream section, slopes downward in the throat, and then rises in the downstream section; ending with a downstream elevation below that of the upstream elevation.

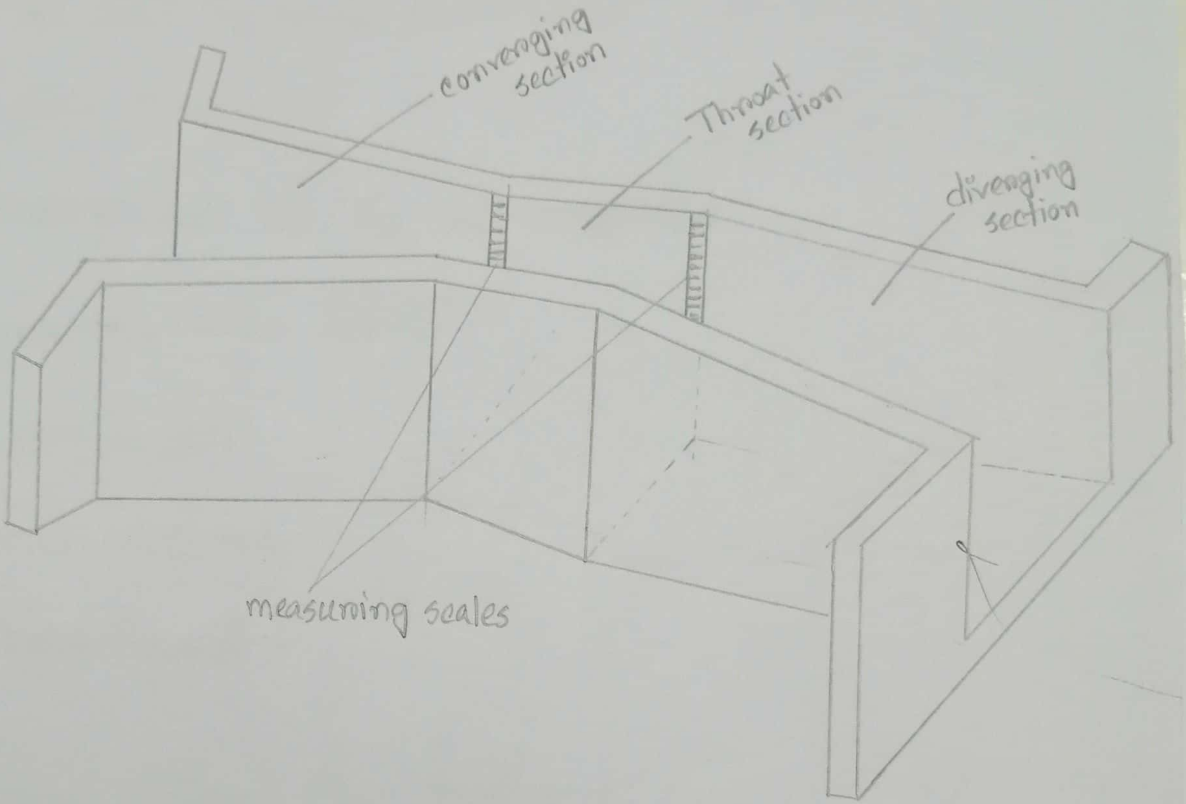

Fig. 2.c. Parshall Flume.

2. Working Procedure:

Free flow condition

1. The Ha was measured
2. Qa was recorded
3. Q_{tf} was computed
4. C_{df} was computed

For submerged condition

1. The H_a and H_b were measured
2. Q_{tf} was computed
3. Submergence H_b/H_a was computed
4. $Q_{correction}$ was found
5. Q_{ts} was found
6. Q_a was recorded
7. C_{ds} was found

2.1 In case of Free flow condition:

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.

The free flow discharge can be summarized as

$$Q = CH_a^n$$

where,

- Q is flow rate
- C is the free-flow coefficient for the flume.

- Ha is the head at the primary point of measurement
- n varies with flume size (e.g. 1.55 for a 1-inch flume)

The values of C and n depend on the throat width and are given in the following table

Table-1 Parshall flume discharg table for free flow conditions:

Throat Width	Coefficient (C)	Exponent (n)
1 in	0.338	1.55
2 in	0.676	1.55
3 in	0.992	1.55
6 in	2.06	1.58
9 in	3.07	1.53
1 ft	3.95	1.55
1.5 ft	6.00	1.54
2 ft	8.00	1.55
3 ft	12.00	1.57
4 ft	16.00	1.58
5 ft	20.00	1.59
6 ft	24.00	1.59
7 ft	28.00	1.60
8 ft	32.00	1.61
10 ft	39.38	1.60
12 ft	46.75	1.60
15 ft	57.81	1.60

Throat Width	Coefficient (C)	Exponent (n)
20 ft	76.25	1.60
25 ft	94.69	1.60
30 ft	113.13	1.60
40 ft	150.00	1.60
50 ft	186.88	1.60

$$Q_{th} = 4 W H_a^{1.522} W^{0.026}$$

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

Coefficient of discharge:

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))

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

2.2 In case of Submerged flow condition:

The percentage of submergence for the Parshall flume is given by $100\,H_b/H_a$. 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 (Q) can be found using the following:

Table 2

Flume size	St
1-3 in	0.5
6-9 in	0.6
1-8 ft	0.7
10-50 ft	0.8

$$Q_{net} = Q_{free\ flow} - Q_{correction}$$

$$Q_{correction} = M\left(0.000132\, H_a^{2.123}\, e^{9.2845t}\right)$$

Where Q is in ft^3/s, Ha is in feet.

For submerged flow, a depth of flow needs to be taken upstream (H_a) and downstream (H_b)

If H_b/H_a is greater or equal to S_t then it is a submerged flow. If there is submerged flow, adjustments need to be made in order for the Parshall Flume to work properly.

The discharge (Q) can be found using the following equations and table:

Table 3:

Size of flume	Multiplying factor, M
1 ft	1
1.5 ft	1.4
2 ft	1.8
3 ft	2.4
4 ft	3.1
5 ft	3.7
6 ft	4.3
7 ft	4.9
8 ft	5.4

- $Q_{net} = Q_{free\ flow} - Q_{correction}$
- $Q_{correction} = M(0.000132\ Ha^{2.123} e^{9.284\ St})$

Where:

- $S = Hb/Ha$
- M = multiplying factor

All various Q values are in ft^3/s, Ha is in feet, and M is varies in units.

3. Objectives of the experiment:

1. Observation of the flow characteristics for the increasing upstream level.
2. Observation of the effect of submergence
3. To plot actual discharge with respect to theoretical discharge and to determine the value of co-efficient of discharge.
4. Determination of the exponent of Ha

4. Experiment setup :

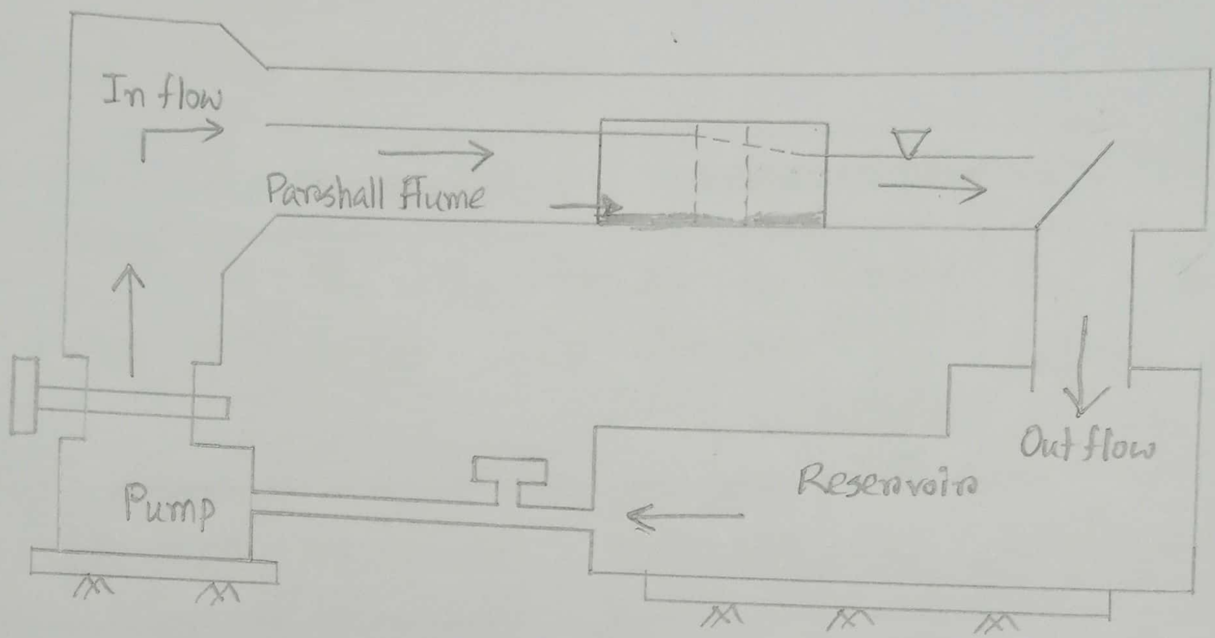

Fig. 2.d. Setup for ~~free~~ flow through a parshall flume.

4.1 Required apparatus:

a) Parshall flume
b) Flow measuring unit
c) Pump
d) Reservoir
e) Water meter

5. Data Sheet:

Channel width, b = 7.60 cm

Throat width, W = 3.10 cm

No. of Obs.	Actual Discharge Qa (cm^3/s)	Free flow condition				Submerged flow condition					
		Ha (cm)	Q_{tf} (cm^3/s)	C_{df}	Mean C_{df}	Ha (cm)	Hb (cm)	Submergence (%) = (Ha/Hb) × 100	Q_{ts} (cm^3/s)	C_{ds}	Mean C_{ds}
1.	500	7.0	1193.56	0.42	0.42	5.45	3.35	0.61	798.82	0.63	0.48
2.	600	8.0	1468.8	0.41		6.40	3.90	0.61	1024.5	0.59	
3.	700	8.7	1672.11	0.42		7.55	4.10	0.54	1342.5	0.52	
4.	800	9.4	1885.62	0.42		9.70	5.0	0.52	1980.2	0.40	
5.	900	10.0	2075.34	0.43		10.20	5.50	0.54	2140.47	0.42	
6.	1000	11.00	2406.08	0.42		11.30	5.90	0.52	2508.87	0.40	
7.	1100	1.75	2664.9	0.41		11.90	6.35	0.53	2717.57	0.40	

6. Calculation:

6.1 In case of free flow

Channel width, $b = 7.60\,cm$

Throat width, $w = 3.10\,cm$

$= 3.10 \times 0.3937$ inch

$= 1.22$ inch

For 1.22 inch throat width,

co-efficient, $C = 0.41236$

Exponent, $n = 1.55$

Observation 1:

Actual discharge, $Q_a = 500\,cm^3/s$

$= 500 \times 3.5315 \times 10^{-5}\,ft^3/s$

$= 0.01766\,ft^3/s$

$H_a = 7\,cm = 0.2296\,ft$

So, theoretical discharge, $Q_{th} = C H_a^n$

$= 0.41236 \times (0.2296)^{1.55}$

$= 0.04215\,ft^3/s$

Co-efficient of discharge, $C_{df} = \frac{Q_a}{Q_{th}}$

$= \frac{0.01766}{0.04215} = 0.42$

Observation 2:

$Q_a = 600 \, cm^3/s$

$= 600 \times 3.5315 \times 10^{-5} \, ft^3/s$

$= 0.02119 \, ft^3/s$

$H_a = 8 \, cm = 0.2625 \, ft$

So, $Q_{th} = C H_a^n = 0.41236 (0.2625)^{1.55}$

$= 0.05187 \, ft^3/s$

and $C_{df} = \frac{Q_a}{Q_{th}} = \frac{0.02119}{0.05187} = 0.41$

Observation 3:

$Q_a = 700 \, cm^3/s = 0.02472 \, ft^3/s$

$H_a = 8.7 \, cm = 0.2854 \, ft$

So, $Q_{th} = C H_a^n = 0.41236 (0.2854)^{1.55}$

$= 0.05905 \, ft^3/s$

and $C_{df} = \frac{Q_a}{Q_{th}} = \frac{0.02472}{0.05905} = 0.42$

Observation 4:

$Q_a = 800 \, cm^3/s = 0.02825 \, ft^3/s$

$H_a = 9.4 \, cm = 0.3084 \, ft$

So, $Q_{th} = CH_a^n = 0.41236\,(0.3084)^{1.55}$

$= 0.06659\ ft^3/s$

and $C_{df} = \dfrac{0.02825}{0.06659} = 0.42$

Observation 5:

$Q_a = 900\ cm^3/s = 0.03178\ ft^3/s$

$H_a = 10\,cm = 0.3281\ ft$

So, $Q_{th} = CH_a^n = 0.41236\,(0.3281)^{1.55}$

$= 0.07329\ ft^3/s$

and $C_{df} = \dfrac{0.03178}{0.07329} = 0.43$

Observation 6:

$Q_a = 1000\ cm^3/s = 0.03531\ ft^3/s$

$H_a = 11\,cm = 0.3609\ ft$

So, $Q_{th} = CH_a^n = 0.41236\,(0.3609)^{1.55}$

$= 0.08497\ ft^3/s$

and $C_{df} = \dfrac{0.03531}{0.08497} = 0.42$

Observation 7:

$Q_a = 1100\ cm^3/s = 0.03885\ ft^3/s$

$H_a = 11.75\ cm = 0.3855\ ft$

So, $Q_{th} = CH_a^n = 0.41236\,(0.3855)^{1.55}$

$= 0.09411\ ft^3/s$

and $C_{df} = \dfrac{0.03885}{0.09411} = 0.41$

6.2 In case of submerged flow:

For 1.22 inch throat width,

co-efficient, $C = 0.41236$

Exponent, $n = 1.55$

Observation 1:

$H_a = 5.45\ cm = 0.1788\ ft$

$H_b = 3.35\ cm = 0.1099\ ft$

$Q_a = 500\ cm^3/s = 0.01766\ ft^3/s$

Submergence, $H_b/H_a = \dfrac{3.35}{5.45} = 0.61 > 0.60$

$Q_{correction} = M\left(0.000132\ H_a^{2.123}\ e^{9.284\ S_t}\right)$

$= 1\left(0.000132 \times 0.1788^{2.123}\ e^{9.284 \times 0.5}\right)$ $\begin{bmatrix} M = 1 \\ S_t = 0.5 \end{bmatrix}$

$= 0.0004 \ ft^3/s$

$Q'_{ts} = CH_a^n = 0.41236 \times 0.1788^{1.55}$

$= 0.02861 \ ft^3/s$

$\therefore Q_{ts} = Q'_{ts} - Q_{correction}$

$= 0.02861 - 0.0004$

$= 0.02821 \ ft^3/s$

Co-efficient of discharge, $C_{ds} = \frac{Q_a}{Q_{ts}}$

$= \frac{0.01766}{0.02821}$

$= 0.63$

<u>Observation 2:</u>

$Q_a = 600 \ cm^3/s = 0.02119 \ ft^3/s$

$H_a = 6.40 \ cm = 0.2099 \ ft$

$H_b = 3.90 \ cm = 0.1279 \ ft$

Now, $\frac{H_b}{H_a} = \frac{3.90}{6.40} = 0.61 > 0.60$

$\therefore Q_{correction} = 1 \left(0.000132 \times 0.2099^{2.123} \times e^{9.284 \times 0.5}\right)$

$= 0.0005 \ ft^3/s$

and $Q'_{ts} = CH_a^n = 0.41236 \times (0.2099)^{1.55}$

$= 0.03668\ ft^3/s$

$\therefore Q_{ts} = Q'_{ts} - Q_{correction}$

$= 0.03668 - 0.0005$

$= 0.03618\ ft^3/s$

and $C_{ds} = \dfrac{0.02119}{0.03618} = 0.59$

Observation 3:

$Q_a = 700\ cm^3/s = 0.02472\ ft^3/s$

$H_a = 7.55\ cm = 0.2477\ ft$

$H_b = 4.10\ cm = 0.1345\ ft$

Now, $\dfrac{H_b}{H_a} = \dfrac{4.10}{7.55} = 0.54 < 0.60$

So, $Q_{correction}$ was not found.

$\therefore Q_{ts} = CH_a^n = 0.41236\,(0.2477)^{1.55} = 0.04741\ ft^3/s$

and $C_{ds} = \dfrac{Q_a}{Q_{ts}}$

$= \dfrac{0.02472}{0.04741} = 0.52$

Observation : 4

$Q_a = 800\ cm^3/s = 0.02825\ ft^3/s$

$H_a = 9.70\ cm = 0.3183\ ft$

$H_b = 5\ cm = 0.1641\ ft$

$\therefore$ Submergencer, $\frac{H_b}{H_a} = \frac{5}{9.70} = 0.52 < 0.60$

So, $Q_{correction}$ was not found.

$\therefore Q_{fs} = CH_a^n = 0.41236(0.3183)^{1.55} = 0.06993\ ft^3/s$

and $C_{ds} = \frac{0.02825}{0.06993} = 0.40$

Observation 5:

$Q_a = 900\ cm^3/s = 0.03178\ ft^3/s$

$H_a = 10.20\ cm = 0.3347\ ft$

$H_b = 5.50\ cm = 0.18046\ ft$

Now, $\frac{H_b}{H_a} = \frac{5.50}{10.20} = 0.54 < 0.6$

So, $Q_{correction}$ was not found.

$\therefore Q_{fs} = CH_a^n = 0.41236(0.3347)^{1.55}$

$= 0.07559\ ft^3/s$

and $C_{ds} = \frac{0.03178}{0.07559} = 0.42$

Observation 6:

$Q_a = 1000\ cm^3/s = 0.03531\ ft^3/s$

$H_a = 11.30\ cm = 0.3708\ ft$

$H_b = 5.90\ cm = 0.1936\ ft$

Now, $\frac{H_b}{H_a} = \frac{0.1936}{0.3708} = 0.52 < 0.60$

So, $Q_{correction}$ was not found

$\therefore Q_{th} = CH_a^n = 0.41236\,(0.3708)^{1.55} = 0.08860\ ft^3/s$

and $C_{ds} = \frac{0.03531}{0.08860} = 0.40$

Observation 7:

$Q_a = 1100\ cm^3/s = 0.03885\ ft^3/s$

$H_a = 11.90\ cm = 0.3904\ ft$ $\quad H_b = 6.35\,cm = 0.2083\ ft$

Now, $\frac{H_b}{H_a} = \frac{6.35}{11.90} = 0.53 < 0.60$

So, $Q_{correction}$ was not found

$\therefore Q_{ts} = CH_a^n = 0.41236\ (0.3904)^{1.55} = 0.09597\ ft^3/s$

and $C_{ds} = \frac{0.03885}{0.09597} = 0.40$

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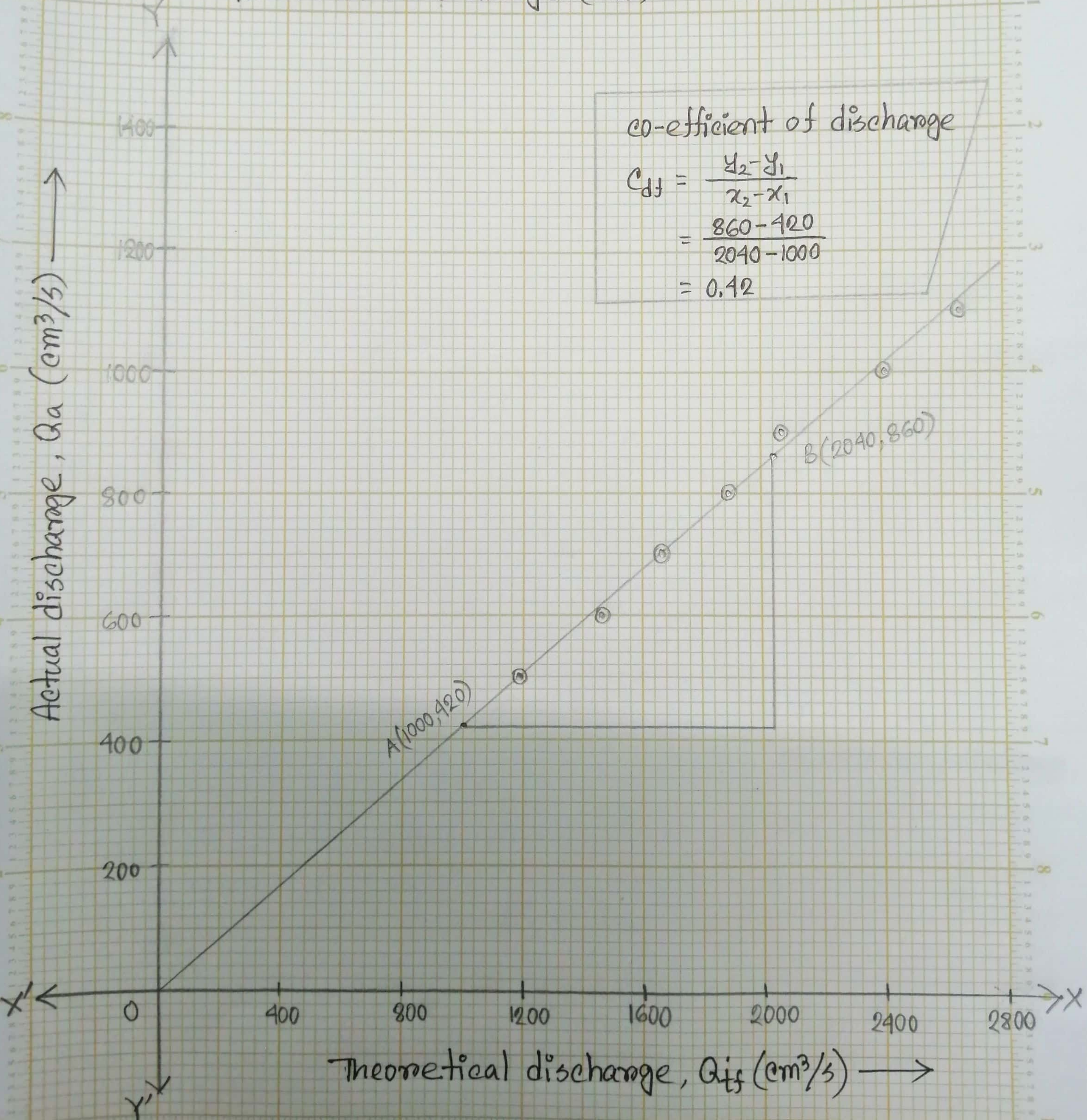
For free flow condition

Variation of actual discharge (Q_a) with respect to theoretical discharge (Q_{tf})



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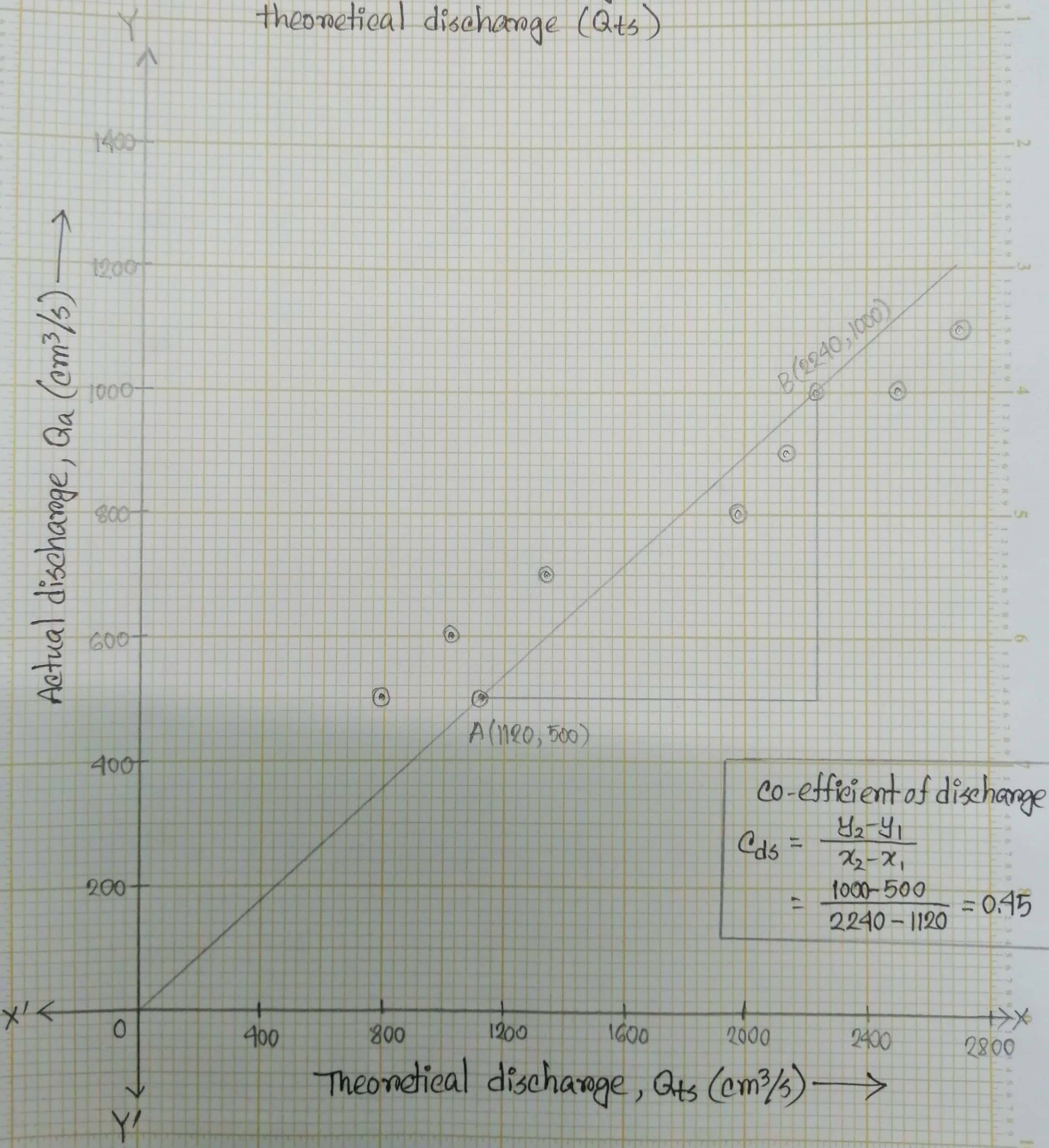
For submerged flow condition

Variation of actual discharge (Qa) with respect to theoretical discharge (Qts)



Co-efficient of discharge,

$$C_{ds} = \frac{y_2 - y_1}{x_2 - x_1}$$

$$= \frac{1000 - 500}{2240 - 1120} = 0.45$$

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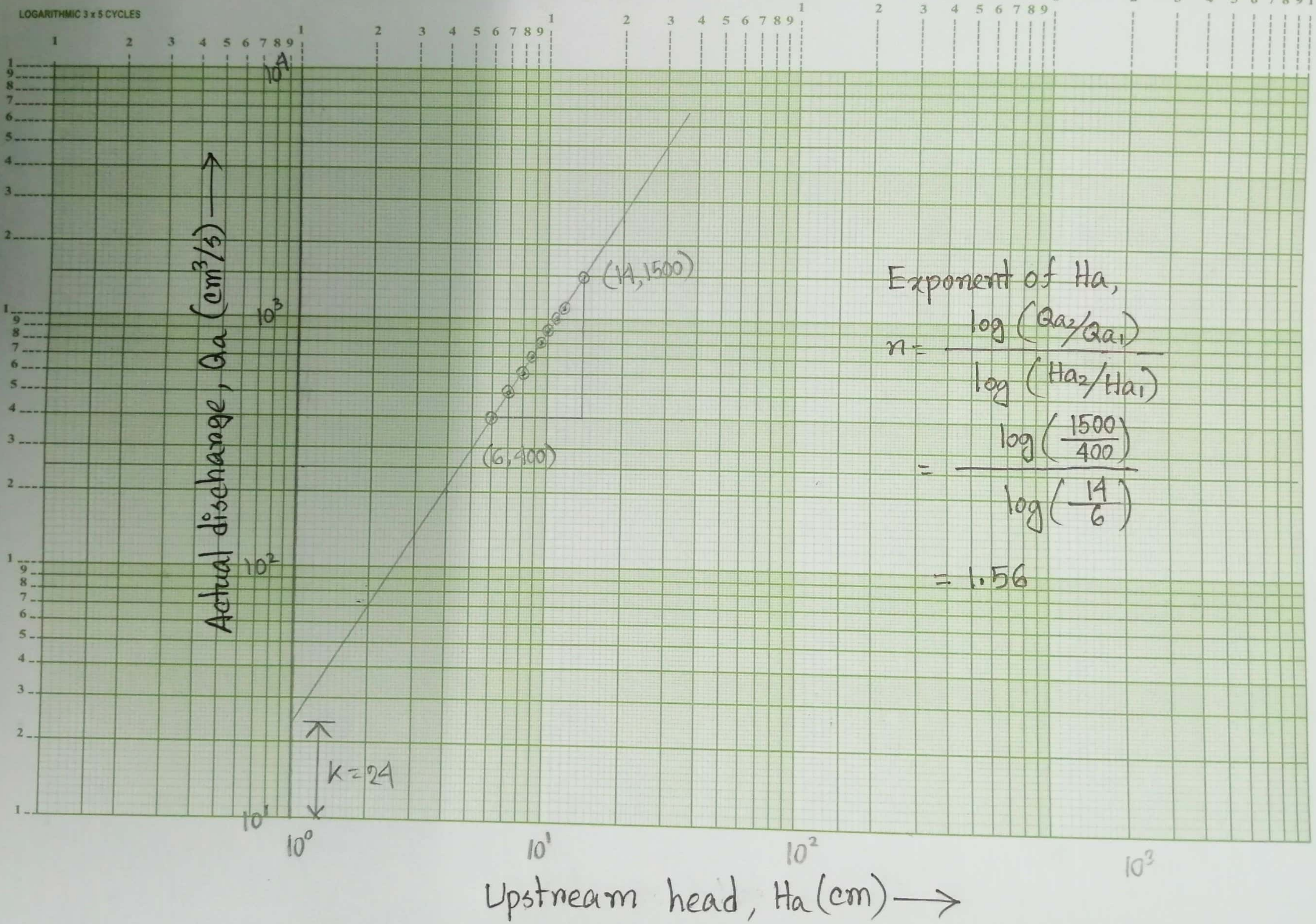
Free flow condition

Variation of actual discharge (Qa) with respect to upstream head (Ha)



Exponent of Ha,

$$n = \frac{\log\left(Qa_2/Qa_1\right)}{\log\left(Ha_2/Ha_1\right)}$$

$$= \frac{\log\left(\frac{1500}{400}\right)}{\log\left(\frac{14}{6}\right)}$$

$$= 1.56$$

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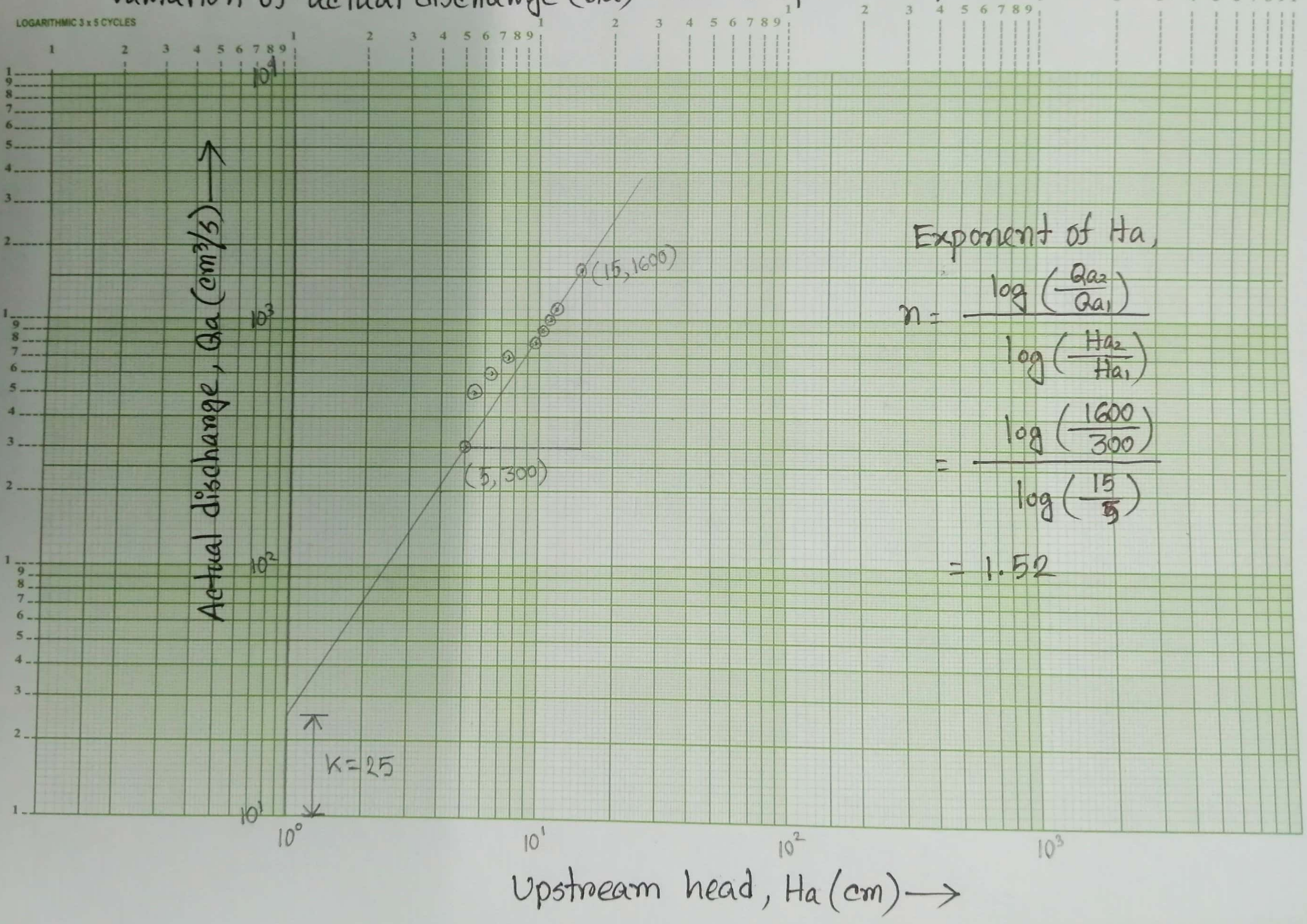
Submerged flow condition

Variation of actual discharge (Qa) with respect to upstream head (Ha)



Exponent of Ha,

$$n = \frac{\log\left(\frac{Q_{a2}}{Q_{a1}}\right)}{\log\left(\frac{H_{a2}}{H_{a1}}\right)}$$

$$= \frac{\log\left(\frac{1600}{300}\right)}{\log\left(\frac{15}{5}\right)}$$

$$= 1.52$$

6.3 Calculation from plain graph:

For free flow condition:

From graph we take two points A(1000, 420) and B(2040, 860)

$$\therefore C_{df} = \frac{y_2 - y_1}{x_2 - x_1} = \frac{860 - 420}{2040 - 1000} = 0.42$$

For submerged flow condition:

From graph we take two points A(1120, 500) and B(2240, 1000)

$$\therefore C_{ds} = \frac{y_2 - y_1}{x_2 - x_1} = \frac{1000 - 500}{2240 - 1120} = 0.45$$

6.4 Calculation from log-log graph:

For free flow condition:

From graph we get,

$Qa_1 = 400$, $Ha_1 = 6$

$Qa_2 = 1500$, $Ha_2 = 14$

$$\therefore \text{Exponent of Ha, } n = \frac{\log\left(Qa_2/Qa_1\right)}{\log\left(Ha_2/Ha_1\right)} = \frac{\log\left(\frac{1500}{400}\right)}{\log\left(\frac{14}{6}\right)}$$

$$= 1.56$$

For submerged flow condition:

From graph we get,

$Qa_1 = 300$, $Ha_1 = 5$

$Qa_2 = 1600$, $Ha_2 = 15$

$\therefore$ Exponent of Ha, $n = \dfrac{\log\left(\frac{Qa_2}{Qa_1}\right)}{\log(Ha_2/Ha_1)} = \dfrac{\log\left(\frac{1600}{300}\right)}{\log\left(\frac{15}{5}\right)}$

$= 1.52$

7. Result:

For free flow condition:

a) Co-efficient of discharge, $C_{df} = 0.42$ [From calculation]

b) Co-efficient of discharge, $C_{df} = 0.42$ [From plain graph]

c) Exponent of upstream head, $n = 1.56$ [From log-log graph]

For submerged flow condition:

a) Co-efficient of discharge, $C_{ds} = 0.48$ [From calculation]

b) Co-efficient of discharge, $C_{ds} = 0.45$ [From plain graph]

c) Exponent of upstream head, $n = 1.52$ [From log-log graph]

8. Discussion :

There is a difference between the co-efficient of discharge for free flow condition and ~~d~~ submerged condition due to frictional head loss. ~~e~~ The value of exponent of upstream head was also changed due to the frictional losses and experimental errors.

Assignment :

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

Answer:

The advantages of Parshall flume:

i) It is supported by all major open channel flow meter manufacturer. It is more accurate

ii) Correction can be made for submergence (up to 60%)

iii) It is suitable for portable or permanent installations.

iv) Modified layouts are available.

v) Head difference can be increased easily. Friction loss is decreased.

The disadvantages of Parshall flume:

i) It possesses complicated design.

ii) When heavy burden of debris is present, it becomes invalid.

III) If the submergence (%) exceeds 0.6, some corrections can be made for submergence by additional calculation.

The uses of Parshall flume:

i) It is commonly used in flow measurement of very large flow rates (volumetric flow rate).

ii) It is used in waste water treatment plant.

iii) Parshall flume accelerates flow through a contraction of both the parallel side walls and a drop in the floor at the flume throat.

iv) It is used as cleaning device due to high velocity. (debris cleaning

v) It is used in small irrigation canals.

2. What is the difference between free and submerged flow in laboratory flume? How can you produce submerged flow in a laboratory flume? What is the effect of submergence on the flow?

Answer:

Free flow condition	Submerged flow condition
i) It has a narrow throat in Parshall flume.	i) It has a large throat in Parshall flume.
ii) The velocity of flow of throat section is greater than the critical velocity.	ii) The velocity of flow of throat section is less than the critical velocity.
iii) It is also known as standing wave condition.	iii) It is also known as drawn flow condition.
iv) Hydraulic eddy on hydraulic jump is formed.	iv) Hydraulic eddy is not seen in this flow condition.

To produce submerged Flow condition, we have to submerge the throat section. We can submerge the throat section by using an obstacle at the downstream.

The effect of submergence on flow :

i) Effect of submergence flow on the Parshall flume is great and significant.

ii) Submergence flow can cause free surface vortices forced around in an intake pipe.

iii) It has great impact on flow rate of water & air.

iv) It has remarkable effect on flow pattern around short T-head super dike in a mild bend with rigid bead using numerical model.

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RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 03

Name of Expt. Observation of Hydraulic Jump

	SUBMITED BY :
SUBJECT: Engineering Hydraulics Sessional	**NAME:** Most. Afrin Sultana
COURSE NO. : CE 3122	**GROUP :**
DATE OF EXPT. : 31-01-2021	**ROLL NO:** 1700082
DATE OF SUB. : 16-03-2021	**SESSION :** 2017-18

Date: 31-01-2021

Experiment No: 03

Experiment Name: Observation of Hydraulic Jump

1. Introduction:

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. The 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 ~~d~~ 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. 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. This experiment is dealing with how the Hydraulic Jump is created and the characteristics.

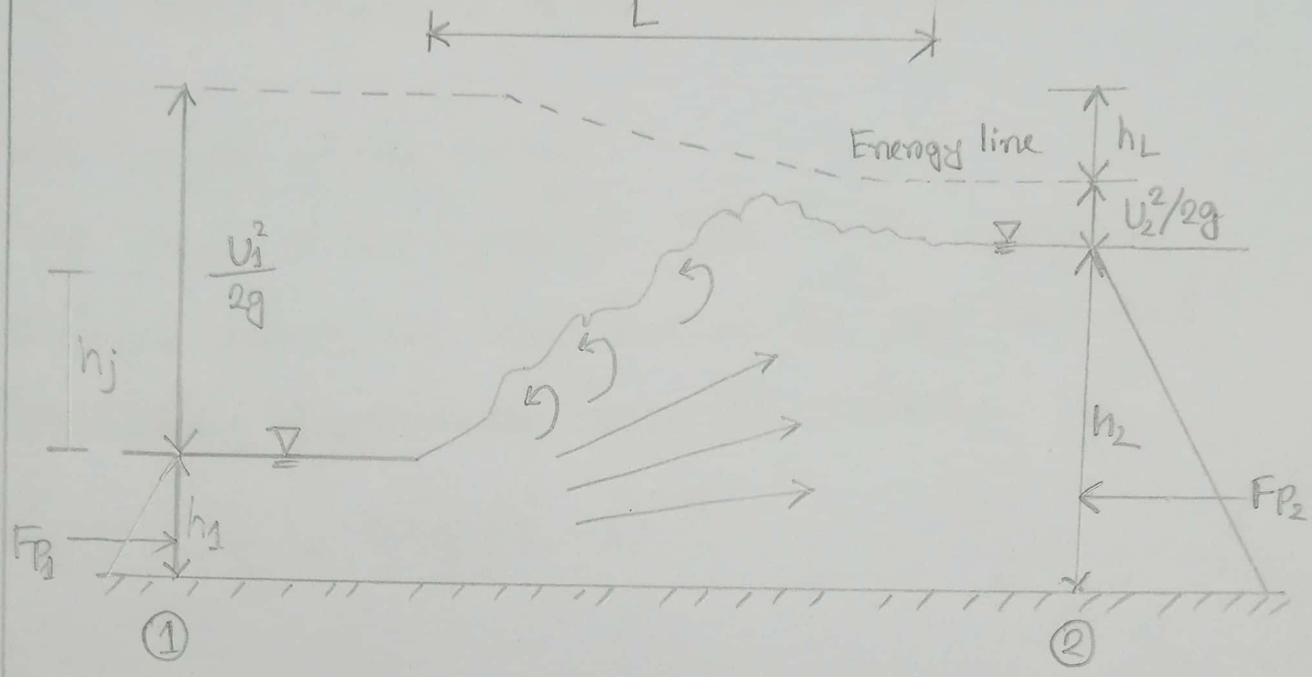

Fig. 3.a. Hydraulic Jump in a horizontal rectangular channel.

2. Working Formula:

2.1. Initial and sequent depths:

Consider a hydraulic jump on a horizontal floor in a rectangular channel as shown in figuger. 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). The relation between them is given below:

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

F_1 = Froude Number of the approaching flow.

2.2 Length of the jump:

The length of ~~the~~ a hydraulic jump is the horizontal distance from the front face of the jump to a point immediately downstream from the roller.

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

2.3 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.

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

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

2.4 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

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

2.5 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

$$H_j = y_2 - y_1$$

3. Objectives of the experiment:

1. To observe the movement of hydraulic jump for different downstream conditions.

2. To determine the height, length, type, energy losses and efficiency of the jump.

3. To plot E_2/E_1, H_j/E_1, y_1/E_1, y_2/E_1 vs F_1 in one plain graph paper.

4. To plot L/y_1, L/y_2, L/H_j vs F_1 in one plain graph paper.

5. To plot L vs H_j in a log-log paper.

6. To plot y_1 vs y_2 for different values of Q in a plain graph paper.

4. Experiment setup:

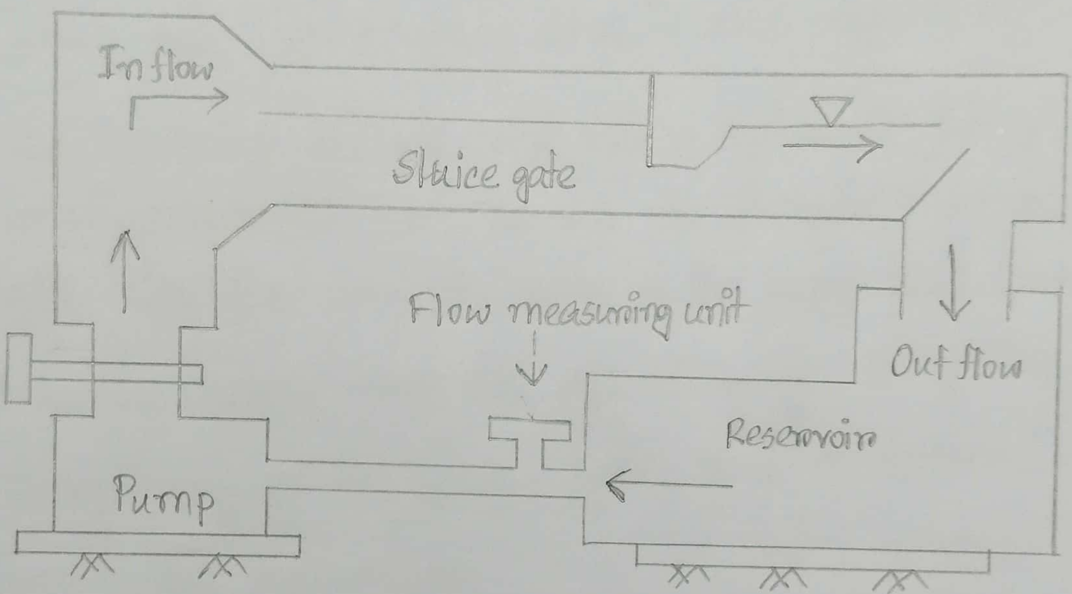

Fig. 3.b. Setup for a hydraulic jump

4.1. Required Apparatus:

a) Flume
b) Pump
c) Flow measuring unit
d) Reservoir
e) Water meter.

5. Procedure :

1) At first the depth of the flow was measured before the hydraulic jump and it was initial depth y_1.

2) Then, the depth of the flow after the hydraulic jump was measured and it was sequent depth y_2

3) The velocities before and after hydraulic jump were measured and hence the Froude Number was calculated and the type of the jump was found.

4) Kinetic energy from the equation was computed.

5) Efficiency of the jump, E_2/E_1 was computed from the equation

6) The energy before and the after jump were computed and energy loss $\Delta E = E_2 - E_1$ was found.

7) The length and height of the jump was recorded.

8) By plotting E_2/E_1, H_j/E_1, $\frac{y_1}{E_1}$, y_2/E_1 vs F_1, we can get the characteristic curve.

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6. Data Table :

Table -1 :

No of Obs	Q (cm^3/s)	L_j (cm)	H_j (cm)	y_1 (cm)	y_2 (cm)	F_1	$\frac{y_2}{y_1}$	E_1	E_2	ΔE	$\frac{E_2}{E_1}$	Type of Jump
1.	500	34.72	4.9	1.10	6.00	4.2	5.45	10.87	6.42	4.45	0.59	Oscillating Jump
2.	600	36.76	5.2	1.30	6.50	3.87	5	10.95	6.79	4.16	0.62	Oscillating Jump
3.	700	39.59	5.6	1.40	7.00	3.87	5	11.79	7.31	4.48	0.62	Oscillating Jump
4.	800	40.63	5.75	1.60	7.35	3.58	4.59	11.88	7.84	4.04	0.66	Oscillating Jump
5.	900	41.74	5.9	1.65	7.55	3.57	4.57	12.12	8	4.12	0.66	Oscillating Jump
6.	1000	44.27	6.25	1.75	8.00	3.57	4.57	12.82	8.46	4.36	0.66	Oscillating Jump
7.	1100	46.43	6.55	1.85	8.40	3.55	4.54	13.29	8.77	4.52	0.66	Oscillating Jump
8.	1200	47.01	6.65	2.00	8.65	3.39	4.33	13.71	9.46	4.25	0.69	Oscillating Jump

Table-2 Table for graph plotting

No. of Obs.	$\frac{E_2}{E_1}$	$\frac{H_j}{E_1}$	$\frac{y_1}{E_1}$	$\frac{y_2}{E_1}$	$\frac{L_j}{y_1}$	$\frac{L_j}{y_2}$	$\frac{L_j}{H_j}$	F_1	Q (cm^3/s)
1.	0.59	0.45	0.10	0.55	31.56	5.79	7.09	4.2	500
2.	0.62	0.47	0.12	0.59	28.28	5.66	7.07	3.87	600
3.	0.62	0.47	0.12	0.59	28.28	5.66	7.07	3.87	700
4.	0.66	0.48	0.13	0.62	25.39	5.53	7.066	3.58	800
5.	0.66	0.49	0.14	0.62	25.3	5.53	7.074	3.57	900
6.	0.66	0.487	0.136	0.624	25.3	5.53	7.083	3.57	1000
7.	0.66	0.49	0.14	0.63	25.09	5.52	7.089	3.55	1100
8.	0.69	0.5	0.15	0.63	23.51	5.43	7.069	3.39	1200

7. Calculation:

Observation 1:

Actual discharge, $Q_a = 500\ cm^3/s$

Upstream Depth, $y_1 = 1.10\ cm$

Sequent Depth, $y_2 = 6.00\ cm$

Height of the hydraulic jump, $H_j = y_2 - y_1$

$$= (6 - 1.1) = 4.9\ cm$$

$$F_r = \sqrt{\frac{(2\,y_2/y_1 + 1)^2 - 1}{8}} = \sqrt{\frac{(2\times\frac{6}{1.1} + 1)^2 - 1}{8}} = 4.2$$

Length of the hydraulic jump, $L_j = y_1 \times 9.75\,(F_r - 1)^{1.01}$

$$= 1.10 \times 9.75 \times (4.2 - 1)^{1.01}$$

$$= 34.72\ cm$$

$$E_1 - E_2 = \frac{(y_2 - y_1)^3}{4y_1y_2} = \frac{(6 - 1.10)^3}{4\times 6\times 1.10} = 4.46 \text{ ——— (i)}$$

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

$$= \frac{(8\times 4.2^2 + 1)^{3/2} - 4\times 4.2^2 + 1}{8\times 4.2^2(2 + 4.2^2)}$$

$$= 0.59 \text{ ——— (ii)}$$

From (i) and (ii) $\Rightarrow$

$E_1 = 10.87$ $E_2 = 6.42$

Observation 2:

$Qa = 600$

$y_1 = 1.30$

$y_2 = 6.50$

$H_j = y_2 - y_1 = 6.50 - 1.30 = 5.2$ cm

$$F_r = \sqrt{\frac{(2\cdot\frac{y_2}{y_1}+1)^2-1}{8}} = 3.87$$

$$E_1 - E_2 = \frac{(y_2-y_1)^3}{4y_1y_2} = \frac{(6.50-1.30)^3}{4\times6.50\times1.30} = 4.16 \text{ —— (i)}$$

$$\frac{E_2}{E_1} = \frac{(8F_r^2+1)^{3/2}-4F_r^2+1}{8F_r^2(2+F_r^2)} = \frac{(8\times3.87^2+1)^{3/2}-4\times3.87^2+1}{8\times3.87^2\times(2+3.87^2)}$$

$$= 0.62 \text{ —— (ii)}$$

From (i) and (ii) $\Rightarrow$

$E_1 = 10.95$ $E_2 = 6.79$

$$L_j = y_1\times9.75\times(F_r-1)^{1.01}$$

$$= 1.30\times9.75\times(3.87-1)^{1.01} = 36.76 \text{ cm}$$

Observation 3:

$Q_a = 700\ cm^3/s$

$y_1 = 1.40\ cm$

$y_2 = 7.00\ cm$

$H_j = y_2 - y_1 = 7 - 1.4 = 5.60\ cm$

$$F_r = \sqrt{\frac{\left(\frac{2y_2}{y_1}+1\right)^2 - 1}{8}} = 3.87$$

$$L_j = 1.4 \times 9.75 \times (3.87 - 1)^{1.01} = 39.59\ cm$$

$$E_1 - E_2 = \frac{(7-1.4)^3}{4 \times 1.4 \times 7} = 4.48 \quad \text{———— (i)}$$

$$\frac{E_2}{E_1} = \frac{(8F_r^2+1)^{3/2} - 4F_r^2 + 1}{8F_1^2(2+F_1^2)} = 0.62 \quad \text{———— (ii)}$$

From (i) and (ii) $\Rightarrow$

$E_1 = 11.79$ $\qquad$ $E_2 = 7.31$

Observation 4:

$Q_a = 800\ cm^3/s$

$y_1 = 1.60\ cm$

$y_2 = 7.35\ cm$

$H_j = 7.35 - 1.60 = 5.75\ cm$

$$F_r = \sqrt{\frac{(2y_2/y_1+1)^2-1}{8}} = 3.58$$

$$L_j = 1.60 \times 9.75 \times (3.58-1)^{1.01} = 40.63 \text{ cm}$$

$$E_1 - E_2 = \frac{(7.35-1.60)^3}{4 \times 1.60 \times 7.35} = 4.04 \quad \text{——— (i)}$$

$$\frac{E_2}{E_1} = \frac{(8F_r^2+1)^{3/2} - 4F_r^2 + 1}{8F_r^2(2+F_r^2)} = 0.66 \quad \text{——— (ii)}$$

From (i) and (ii)

$E_1 = 11.88$ $\quad$ $E_2 = 7.84$

Observation 5:

$Q_a = 900 \text{ cm}^3/s$

$y_1 = 1.65 \text{ cm}$

$y_2 = 7.55 \text{ cm}$

$H_j = 7.55 - 1.65 = 5.9 \text{ cm}$

$$F_r = \sqrt{\frac{(2y_2/y_1+1)^2-1}{8}} = 3.57$$

$$L_j = 1.65 \times 9.75 \times (3.57-1)^{1.01} = 41.74 \text{ cm}$$

$$E_1 - E_2 = \frac{(7.55-1.65)^3}{4 \times 1.65 \times 7.55} = 4.12 \quad \text{——— (i)}$$

$$\frac{E_2}{E_1} = \frac{(8Fr^2+1)^{3/2} - 4Fr^2 + 1}{8Fr^2(2+Fr^2)} = \cancel{7.02}\ 0.66 \quad \text{—— (ii)}$$

From (i) and (ii) ⇒

$E_1 = 12.12$ $E_2 = 8$

Observation 6:

$Q_a = 1000\ cm^3/s$

$y_1 = 1.75\ cm$

$y_2 = 8.00\ cm$

$H_j = 8 - 1.75 = 6.25\ cm$

$$Fr = \sqrt{\frac{(2y_2/y_1 + 1)^2 - 1}{8}} = 3.57$$

$$L_j = 1.75 \times 9.75 \times (3.57 - 1)^{1.01} = 44.27\ cm$$

$$E_1 - E_2 = \frac{(8-1.75)^3}{4 \times 1.75 \times 8} = 4.36 \quad \text{—— (i)}$$

$$\frac{E_2}{E_1} = \frac{(8Fr^2+1)^{3/2} - 4Fr^2 + 1}{8Fr^2(2+Fr^2)} = 0.66 \quad \text{—— (ii)}$$

From (i) and (ii) ⇒

$E_1 = 12.82$ $E_2 = 8.46$

Observation 7:

$Qa = 1100\ cm^3/s$

$y_1 = 1.85\ cm$

$y_2 = 8.40\ cm$

$H_j = 8.40 - 1.85 = 6.55\ cm$

$$F_r = \sqrt{\frac{(2y_2/y_1 + 1)^2 - 1}{8}} = 3.55$$

$$L_j = 1.85 \times 9.75 \times (3.55 - 1)^{1.01} = 46.43\ cm$$

$$E_1 - E_2 = \frac{(8.40 - 1.85)^3}{4 \times 1.85 \times 8.40} = 4.52 \quad \text{———— (i)}$$

$$\frac{E_2}{E_1} = \frac{(8F_r^2 + 1)^{3/2} - 4F_r^2 + 1}{8F_r^2(2 + F_r^2)} = 0.66 \quad \text{———— (ii)}$$

From (i) and (ii)

$E_1 = 13.29 \qquad E_2 = 8.77$

Observation 8:

$Qa = 1200\ cm^3/s$

$y_1 = 2.00\ cm$

$y_2 = 8.65\ cm$

$H_j = (8.65 - 2.00) = 6.65\ cm$

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$$F_r = \sqrt{\frac{(2y_2/y_1 + 1)^2 - 1}{8}} = 3.39$$

$$L_j = 2 \times 9.75 \times (3.39 - 1)^{1.01} = 47.01 \text{ cm}$$

$$E_1 - E_2 = \frac{(8.65 - 2)^3}{4 \times 2 \times 8.65} = 4.25 \text{ ———— (i)}$$

$$\frac{E_2}{E_1} = \frac{(8F_r^2 + 1)^{3/2} - 4F_r^2 + 1}{8F_r^2(2 + F_r^2)} = 0.69 \text{ ———— (ii)}$$

Froom (i) and (ii)

$E_1 = 13.71$ $E_2 = 9.46$

When $F_r = 2.5$ to 4.5 then the jump is known as oscillating jump

The F_r calculating from the experimental data table is between 2.5 to 4.5 so the types of jump is oscillating jump.

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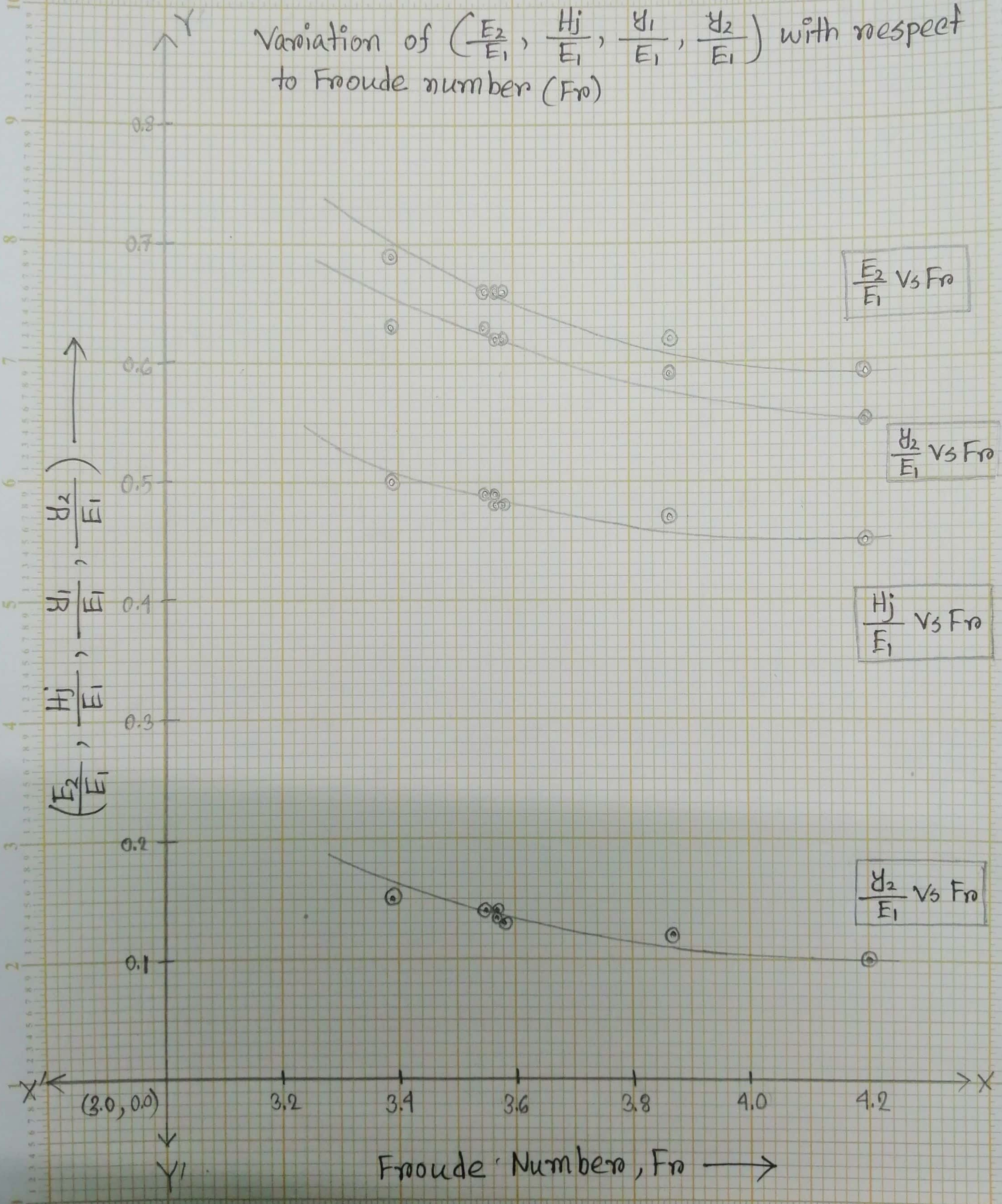
Variation of $\left(\frac{E_2}{E_1}, \frac{H_j}{E_1}, \frac{y_1}{E_1}, \frac{y_2}{E_1}\right)$ with respect to Froude number (Fr)
$\left(\frac{E_2}{E_1}, \frac{H_j}{E_1}, \frac{y_1}{E_1}, \frac{y_2}{E_1}\right)$
$\frac{E_2}{E_1}$ Vs Fr
$\frac{y_2}{E_1}$ Vs Fr
$\frac{H_j}{E_1}$ Vs Fr
$\frac{y_2}{E_1}$ Vs Fr
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
(3.0, 0.0)
3.2
3.4
3.6
3.8
4.0
4.2
X
X'
Y
Y'
Froude Number, Fr

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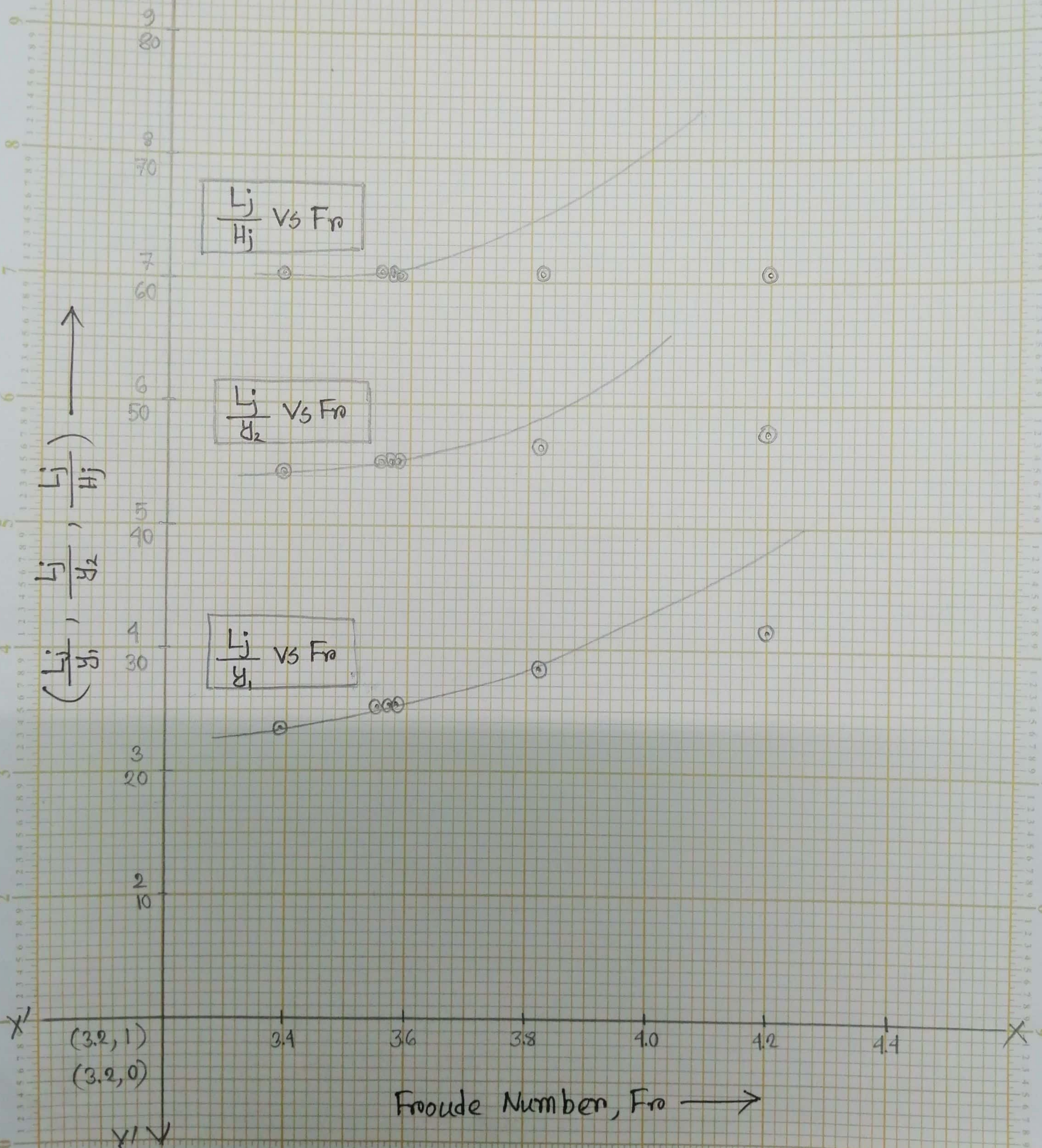
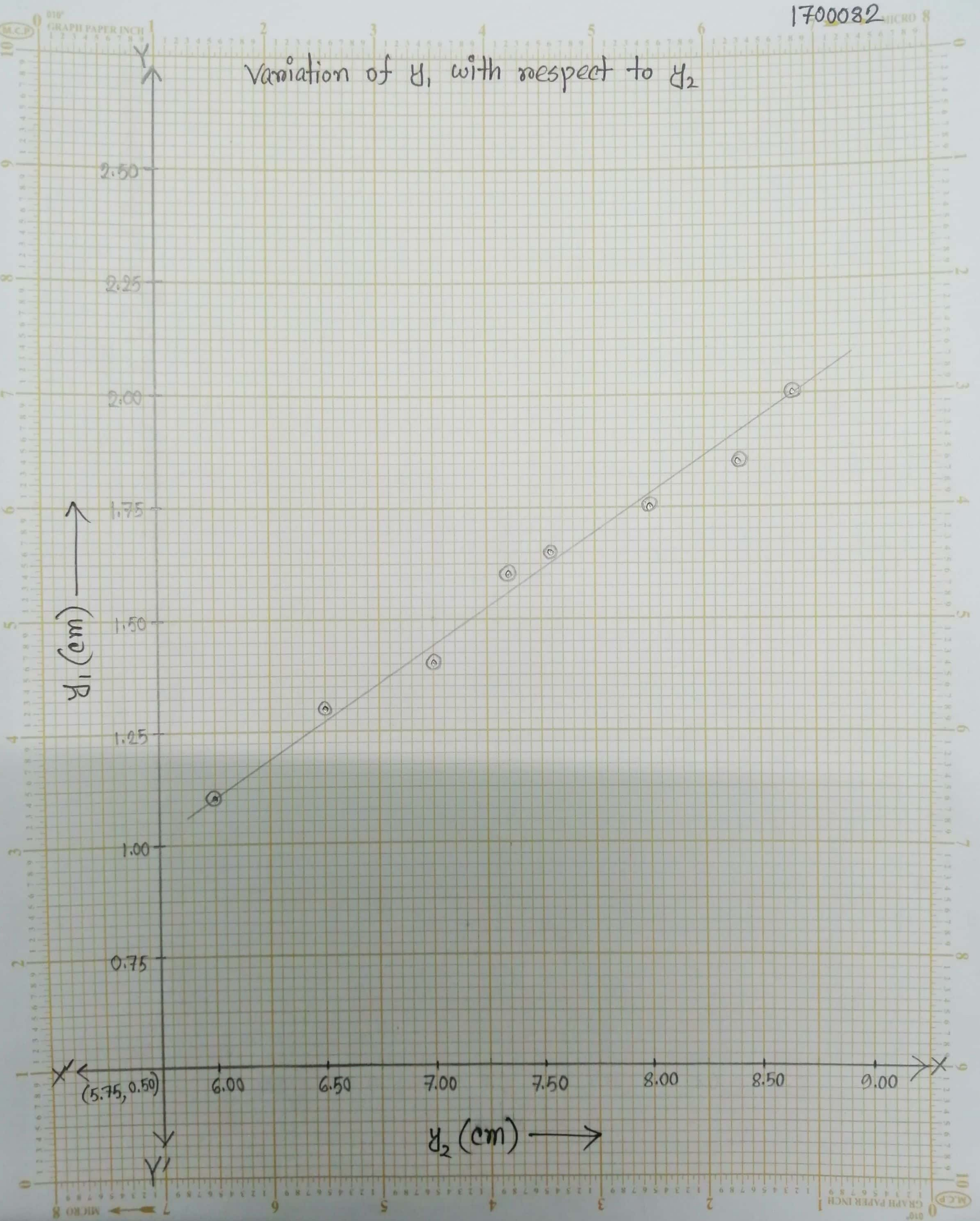
Variation of $\left(\frac{L_j}{y_1}, \frac{L_j}{y_2}, \frac{L_j}{H_j}\right)$ with respect to Froude number Fr

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Variation of y_1 with respect to y_2
Y
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
y_1 (cm)
X
(5.75, 0.50)
6.00
6.50
7.00
7.50
8.00
8.50
9.00
X
y_2 (cm)
Y'

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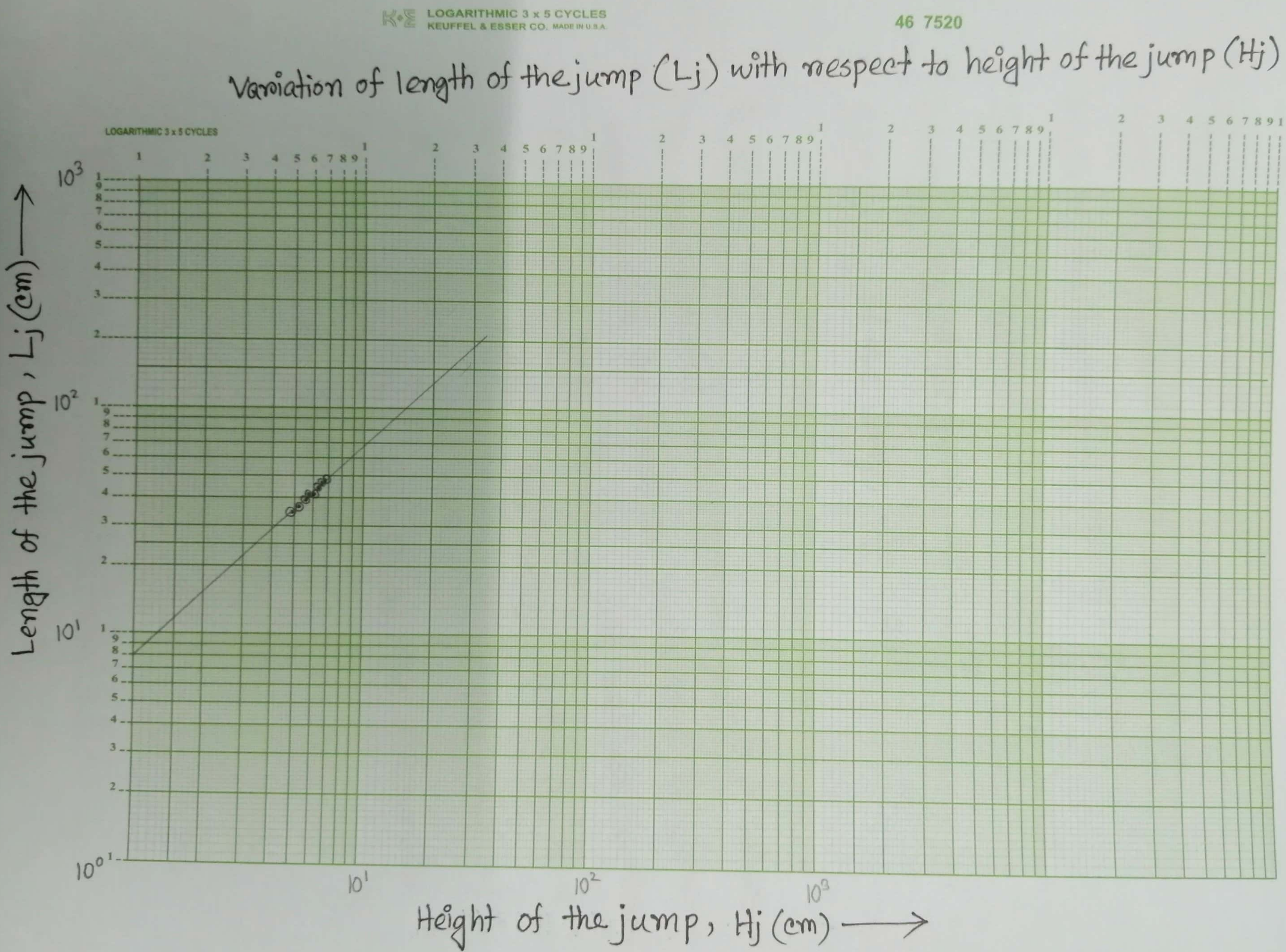
Variation of length of the jump (Lj) with respect to height of the jump (Hj)



8. Result:

Length of the hydraulic jump, L_j = 39.59 cm

Height of the hydraulic jump, H_j = 5.6 cm

Type of the hydraulic jump = Oscillating jump

Energy loss of the jump, ΔE = 4.48

Efficiency of the jump, $\frac{E_2}{E_1}$ = 0.62

9. Discussion:

The experimental values and the graphical values vary due to the frictional resistance between the channel bed. Besides, side wall contraction is another factor. The change in the position of the sluice gate can change the critical depth of the flow as well as the alternate and sequent depth.

Assignments

1. What are the different types of jumps according to USBR classification?

Answer: Depending upon the Froude number, there are five types of hydraulic jump:

i) Undular Jump: For $F_r = 1$ to 1.7, the water surface shows the undulation and the jump is called undulation jump.

ii) Weak Jump: For $F_r = 1.7$ to 2.5, the jump formed is called weak jump. As the velocity throughout is fairly uniform and only a small amount of energy is dissipated.

iii) Oscillating Jump: For $F_r = 2.5$ to 4.5, jump formed is known as an oscillating jump. In this case the entering jet of water oscillates back and forth from the bottom to the surface and back again.

iv) Steady jump: For Fr = 4.5 to 9, the jump is formed well stablished and stabilized and is called a steady jump. For this jump the energy dissipation ranges from 45 to 70 percent

v) Strong jump: For Fr = 9 and larger than Fr = 9, the jump formed is called a strong jump. In this case a rough surface prevails which contains downstream for a long distance.

2. Why does energy loss occur in hydraulic jumps? Is it really an energy loss?

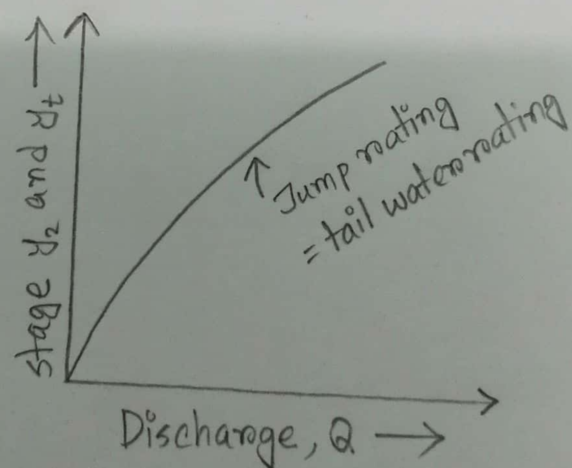
Answer: When a flow of liquid moves from high velocity to low velocity zone, hydraulic jumps occurs. At that tyme height turbulance and shear action are exhibited. Initial kinetic energy is converted into potential energy and some energy inversely lost through turbulance. This is why energy loss occurs in hydraulic jump.

3. What is tail water depth? Explain why a hydraulic jump moves upstream when the tail water depth is greater than the sequent depth and vice versa.

Answer: Tail water depth refers to the water located immediately downstream head from a hydraulic structure such as dam (excluding minimum release such as for fish water), spillway, bridge or culvert.

Case 1:

If $y_2 = y_t$



Here,
y_1 = tail water depth
y_2 = sequent depth

Fig. 3.1: y_2, y_t vs Q graph.

Case 2:

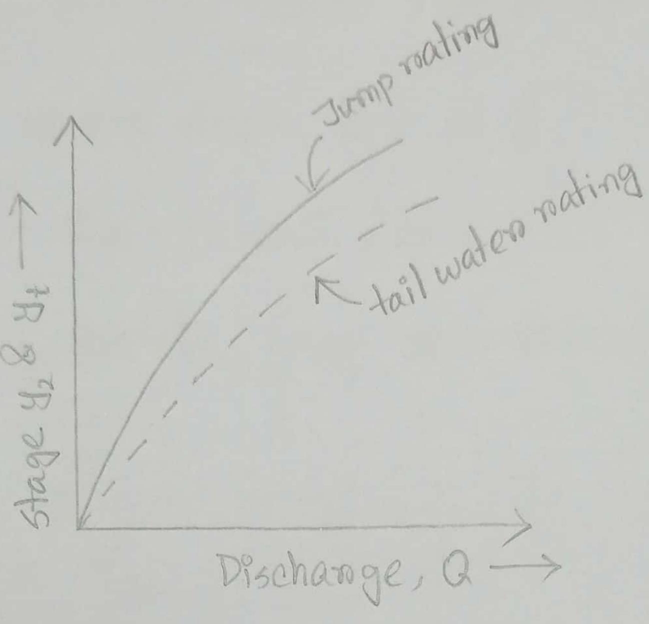


Fig. 3.2 : y_2, y_t vs Q graph

If $y_2 > y_t$ for the entire range of discharge, protected apron, sills are used to create jump within the basin. So it moves.

Case 3:

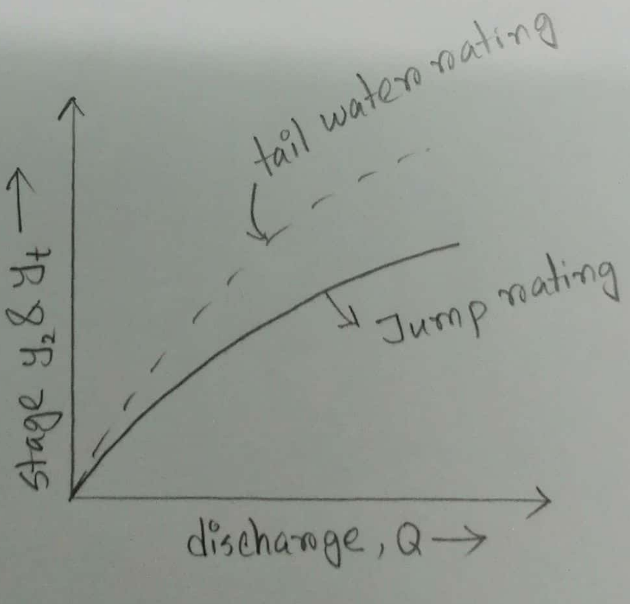


Fig. 3.3 : y_2, y_t vs Q graph

If $y_t > y_2$ for the entire range of discharge,

i) Sloping apron above bed level.

ii) Drop in the channel floor

So, the hydraulic jump moves upstream.

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RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 04

Name of Expt. Flow beneath a sluice gate.

SUBJECT: Engineering Hydraulics sessional	**SUBMITED BY :**
COURSE NO. : CE 3122	NAME: Most. Afrin Sultana
DATE OF EXPT. : 30-06-2021	GROUP :
DATE OF SUB. : 14-07-2021	ROLL NO: 1700082
	SESSION : 2017-18

Date: 30-06-2021

Experiment No: 04

Experiment Name: Flow beneath a sluice gate.

4.1 Introduction:

- 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 ~~b~~ for both urban areas and rural agricultural areas.
- The experiment deals with the measurement of discharge beneath a sluice gate.

4.2 Theory:

4.2.1 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.

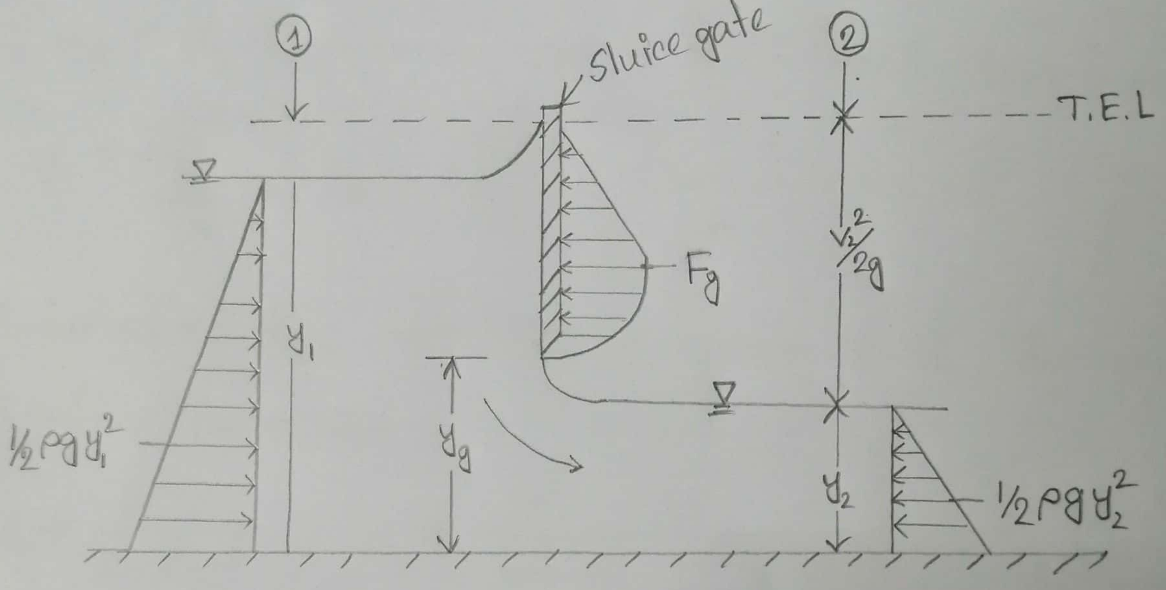

Fig. 4.a. Flow beneath a sluice gate.

4.2.2. 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 ~~1 enginee~~ energies at sections 1 and 2 as shown in Fig. 4.a. As the energy loss between the sections is negligible, we have

$$H_1 = H_2 \quad \text{———} \quad (4.1)$$ and therefore

$$y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} \quad \text{———} \quad (4.2)$$

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

$$y_1 + \frac{Q^2}{2gb^2y_1^2} = y_2 + \frac{Q^2}{2gb^2y_2^2} \quad \text{———} \quad (4.3)$$

Where, b is the width of the sluice gate.

Simplifying and rearranging the terms, we obtain

$$Q = by_1\sqrt{\frac{2gy_2}{(y_1/y_2 + 1)}} \quad \text{———} \quad (4.4)$$

Or alternatively

$$Q = by_2\sqrt{\frac{2gy_1}{(y_2/y_1 + 1)}} \quad \text{———} \quad (4.5)$$

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

$$Q = C_v b y_2 \sqrt{\frac{2gy_1}{(y_2/y_1 + 1)}} \quad \text{———} \quad (4.6)$$

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.

$$y_2 = C_c y_g \quad \text{———} \quad (4.7)$$

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. (4.6) becomes

$$Q = C_c C_v b y_g \sqrt{\frac{2gy_1}{(C_c y_g/y_1 + 1)}} \quad \text{———} \quad (4.8)$$

The above equation can also be written as

$$Q = C_d b y_g \sqrt{2 g y_1} \qquad (4.9)$$

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

Therefore,

$$C_d = \frac{C_c C_v}{\sqrt{\frac{C_c y_g}{y_1} + 1}} \qquad (4.10)$$

Equation (4.9) may also be written as

$$Q_a = C_d Q_t \qquad (4.11)$$

So that

$$Q_1 = b y_g \sqrt{2 g y_1} \qquad (4.12)$$

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

4.2.3. 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. ~~8~~4.a. 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

$$F_x = \left[(1/2)\rho g y_1^2 - (1/2)\rho g y_2^2 - F_g \right] b \quad \text{——} \quad (4.13)$$

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

$$F_x = \rho Q_a V_2 - \rho Q_a V_1 \quad \text{——} \quad (4.14)$$

Substituting for F_x and gathering terms, we obtain

$$F_g = \frac{1}{2}\rho g y_2^2\left[\left(y_1/y_2\right)^2 - 1\right] - \frac{\rho Q_a^2}{b^2 y_2}\left[1 - \frac{y_2}{y_1}\right] \quad \text{———} (4.15)$$

Simplifying and eliminating Q_a, we get

$$F_g = \frac{1}{2}\rho g \frac{(y_1 - y_2)^3}{y_1 + y_2} \quad \text{———} \quad (4.16)$$

- 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

$$F_H = \frac{1}{2}\rho g \left(y_1 - y_g\right)^2 \quad \text{———} \quad (4.17)$$

4.3 Objectives:

1. To determine the discharge beneath the sluice gate.
2. To determine C_v, C_c and C_d
3. To plot y_1 vs Q_a for different values of y_g in a plain graph paper.
4. To determine F_g and F_H and hence to find the ratio F_g / F_H

4.4 Experimental setup :

The experimental setup is given below :

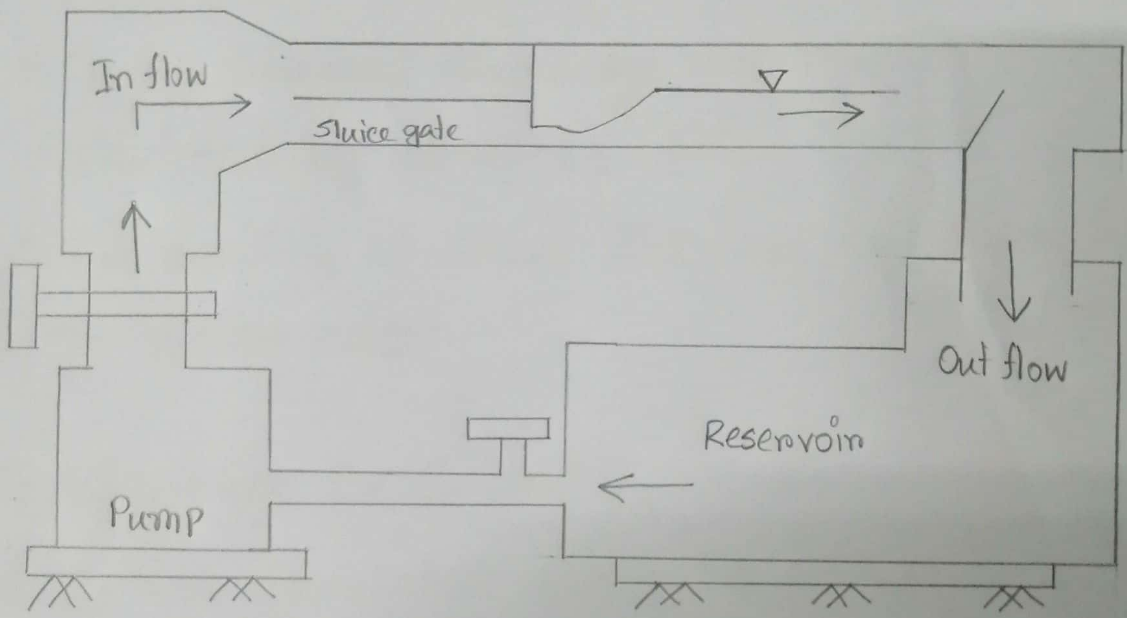

Fig: 4.b. Setup for flow beneath a sluice gate.

4.4.1: Required Apparatus :

a) Flume
b) Pump
c) Flow measuring unit
d) Reservoir
e) Water meter
f) Sluice gate.

4.5 Procedure:

To determine the discharge beneath the sluice gate

1. Measured y_1 and y_g
2. The Theoretical discharge was measured or calculated by using Eq (4.12)
3. The reading of actual discharge was taken from the water meter.

To determine C_v, C_c and C_d

1. C_c was calculated using Eq. (4.7)
2. C_v was calculated using the value of C_c and Eq (4.8).
3. C_d was determined using the values of y_g C_c, C_v and Eq (4.10)
4. y_1 vs Q_a for different values of y_g was plotted in a plain graph paper.

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

1. y_2 was determined

2. F_g was determined using Eq (4.15)
3. F_H was determined using Eq. (4.17) and the ratio F_g/F_H was calculated.

4.6 Shape of y_1 vs Q_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 vs Q_a graph for a higher value of y_g lies below the same graph for a lower value of y_g.

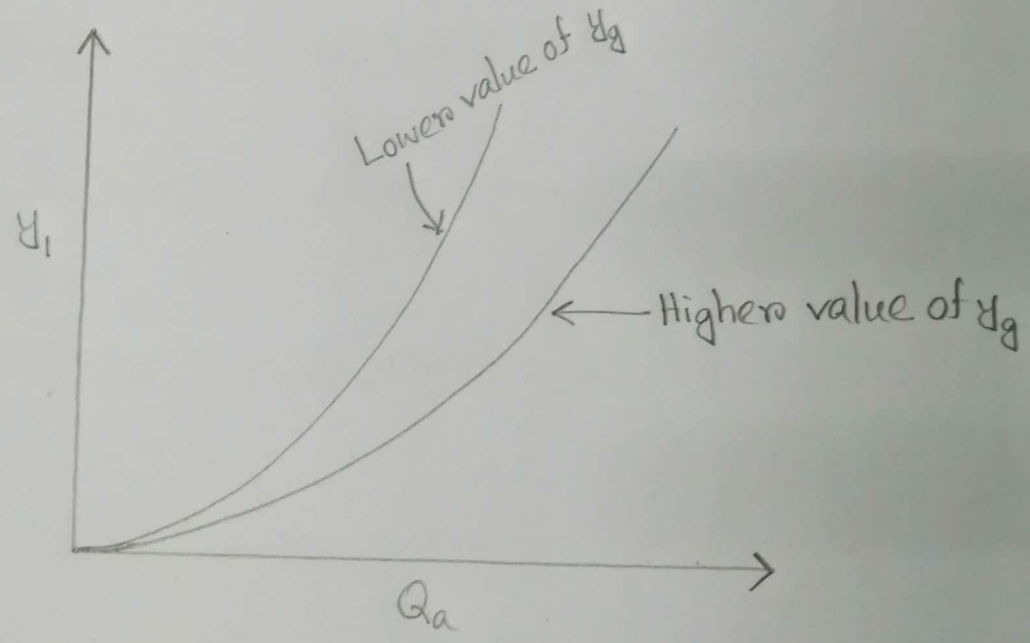

Fig. 4.c. Shape of y_1 vs Q_a graph.

4.7 Data Table:

Table-1:

For free flow condition:

Width of the sluice gate, b = 7.7cm

Sl No.	Y_g	Y_1	Y_2	Q_a	Q_{th}	C_v	C_c	C_d	F_g	F_h	Y_g/Y_1	F_g/F_h
01.	1.55	7.25	1.10	500	941.30	0.75	0.71	0.53	21936.76	15936.35	0.21	1.38
02.	1.55	9.60	1.10	600	1101.06	0.77	0.71	0.54	39723.60	31785.63	0.16	1.25
03.	1.55	12.10	1.15	700	1303.81	0.73	0.74	0.54	64661.94	54593.88	0.13	1.18
04.	1.55	15.00	1.15	800	1464.01	0.74	0.74	0.55	101047	88732.68	0.10	1.14
05.	2.00	9.80	1.60	900	1583.92	0.71	0.80	0.57	38707.45	29842.02	0.20	1.30
06.	2.00	11.20	1.60	1000	1708.34	0.73	0.80	0.58	51237.15	41515.92	0.18	1.23
07.	2.00	13.50	1.60	1100	1895.86	0.73	0.80	0.58	76894.56	64868.63	0.15	1.18
08.	2.00	17.50	1.85	1200	2510.21	0.52	0.93	0.48	136796.40	117842.60	0.11	1.16

Table: 2

For submerged flow condition:

Sl No.	Y_g	Y_1	Y_2	Q_a	Q_{th}	C_v	C_c	C_d	F_g	F_h	Y_g/Y_1	F_g/F_h
01.	1.55	8.0	6.90	500	4877.37	0.02	4.45	0.10	7955.27	20406.03	0.19	0.39
02.	1.55	11.0	7.70	600	6680.41	0.02	4.97	0.09	30032.19	43802.38	0.14	0.68
03.	1.55	13.90	8.40	700	8432.99	0.01	5.42	0.08	59770.53	74812.29	0.11	0.80
04.	1.55	16.10	9.00	800	9864.42	0.01	5.81	0.08	86883.09	103840.10	0.09	0.84
05.	2.00	14.4	8.80	900	8973.08	0.02	4.40	0.10	63122.02	75419.28	0.14	0.84
06.	2.00	12.5	9.20	1000	8419.93	0.02	4.60	0.12	34640.72	54077.63	0.16	0.64
07.	2.00	15.5	10.10	1100	10552.94	0.02	5.05	0.10	67102.77	89393.63	0.13	0.75
08.	2.80	19.4	11.18	1200	13377.19	0.02	4.00	0.09	122375.30	135162.20	0.14	0.90

4.8 Calculation:

For free flow condition:

Test-3:

Width of the sluice gate, $b = 7.7 cm$

$y_g = 1.55$ cm

$y_1 = 12.10$ cm

$y_2 = 1.15$ cm

$Q_a = 700$ cm^3/s

$\rho = 1$ gm/cm^3

$g = 981$ cm/s^2

Now,

Co-efficient of contraction, $C_c = \frac{y_2}{y_g}$

$= \frac{1.15}{1.55}$

$= 0.74$

Horizontal thrust, $F_H = \frac{1}{2}\rho g (y_1 - y_g)^2$

$= \frac{1}{2} \times 1 \times 981 \times (12.10 - 1.55)^2$

$= 54593.87$ $gm.cm/sec^2$

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

$$= \frac{1}{2}\times 1\times 981\times(1.15)^2\left[\left(\frac{12.10}{1.15}\right)^2 - 1\right] - \frac{1\times(700)^2}{(7.7)^2\times 1.15}\times\left[1 - \frac{1.15}{12.10}\right]$$

$$= 64661.94 \text{ gm cm/sec}^2$$

Theoretical discharge, $Q_{th} = b y_2\sqrt{\frac{2 g y_1}{(y_2/y_1)+1}}$

$$= 7.7\times 1.15\times\sqrt{\frac{2\times 981\times 12.10}{\left(\frac{1.15}{12.10}\right)+1}}$$

$$= 1303.81 \text{ cm}^3/\text{sec}$$

Co-efficient of discharge, $C_d = \frac{Q_a}{Q_{th}}$

$$= \frac{700}{1303.81}$$

$$= 0.54$$

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Co-efficient of velocity, $C_v = \frac{C_d}{C_c}$

$$= \frac{0.54}{0.74}$$

$$= 0.73$$

$$\frac{y_g}{y_1} = \frac{1.55}{12.10} = 0.13$$

$$\frac{F_g}{F_H} = \frac{64661.94}{54593.87} = 1.18$$

For submerged flow condition:

Test-3:

Channel width, $b = 7.7 cm$

$Y_g = 1.55 cm$

$Y_1 = 13.90 cm$

$Y_2 = 8.40 cm$

$Q_a = 700 \ cm^3/sec$

$\rho = 1 \ gm/cm^3$

$g = 981 \ cm/s^2$

Now,

Theoretical discharge, $Q_{th} = by_2 \sqrt{\frac{2gy_1}{(y_2/y_1)+1}}$

$$= 7.7 \times 8.40 \times \sqrt{\frac{2 \times 981 \times 13.90}{\left(\frac{8.40}{13.90}\right)+1}}$$

$$= 8433 \text{ cm}^3/\text{sec}$$

Co-efficient of discharge, $C_d = \frac{Q_a}{Q_{th}}$

$$= \frac{700}{8433}$$

$$= 0.08$$

Co-efficient of contraction, $C_c = \frac{y_2}{y_g}$

$$= \frac{8.40}{1.55}$$

$$= 5.42$$

Co-efficient of velocity, $C_v = \frac{C_d}{C_c}$

$$= \frac{0.08}{5.42}$$

$$= 0.01$$

Resisting thrust,

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

$$= \frac{1}{2}\times 1\times 981\times (8.40)^2\times\left[\left(\frac{13.90}{8.40}\right)^2 - 1\right] - \frac{1\times(700)^2}{(7.7)^2\times 8.40}\times\left[1 - \frac{8.40}{13.90}\right]$$

$$= 59770.53 \text{ gm cm/sec}^2$$

Horizontal thrust, $F_H = \frac{1}{2}\rho g\,(y_1 - y_g)^2$

$$= \frac{1}{2}\times 1\times 981\times (13.90 - 1.55)^2$$

$$= 74812.29 \text{ gm cm/sec}^2$$

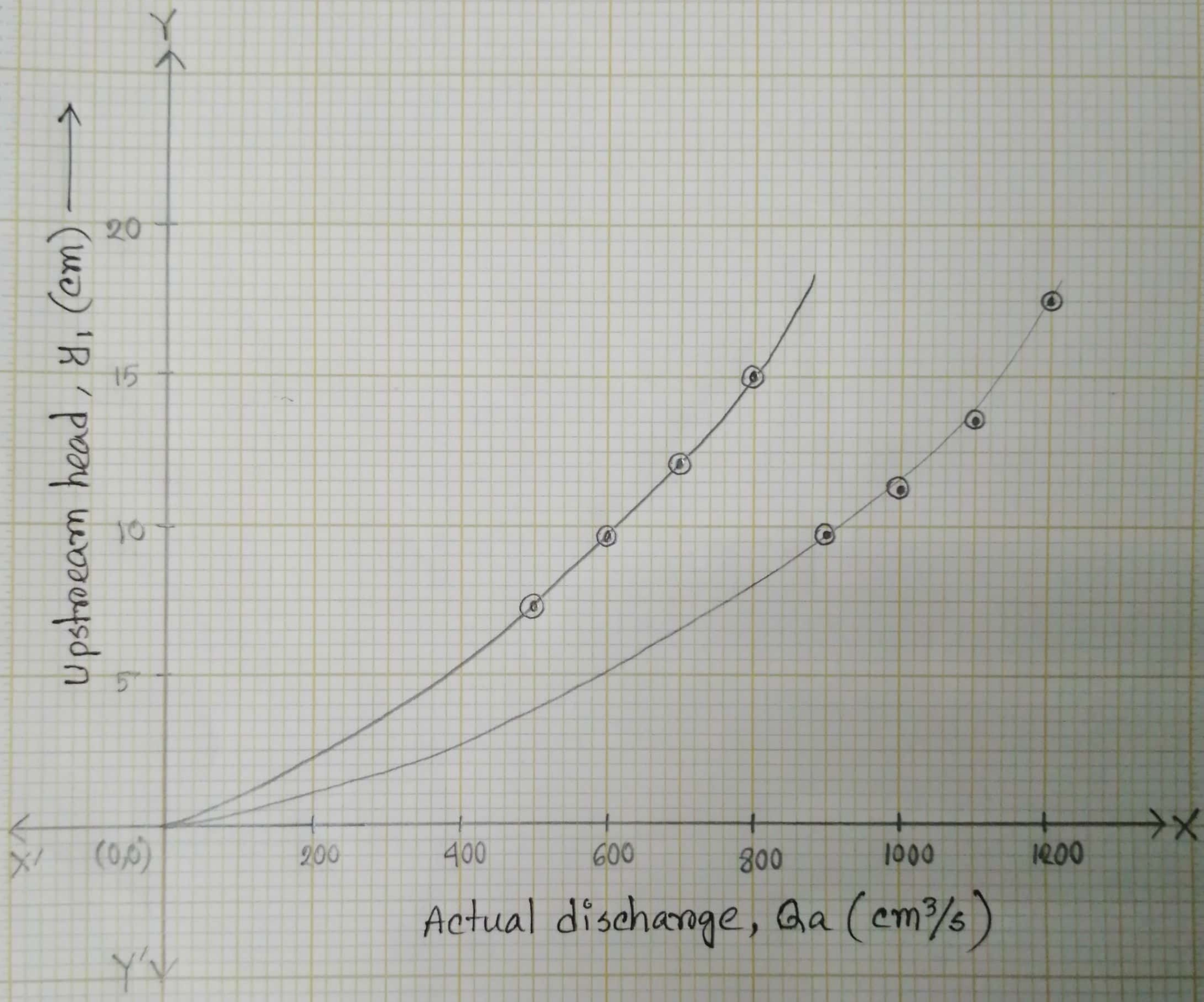
$$\frac{Y_g}{Y_1} = \frac{1.55}{13.90} = 0.11$$

$$\frac{F_g}{F_H} = \frac{59770.53}{74812.29} = 0.80$$

1700082

For free flow condition

Variation of upstream head (y_1) with respect to actual discharge (Q_a).



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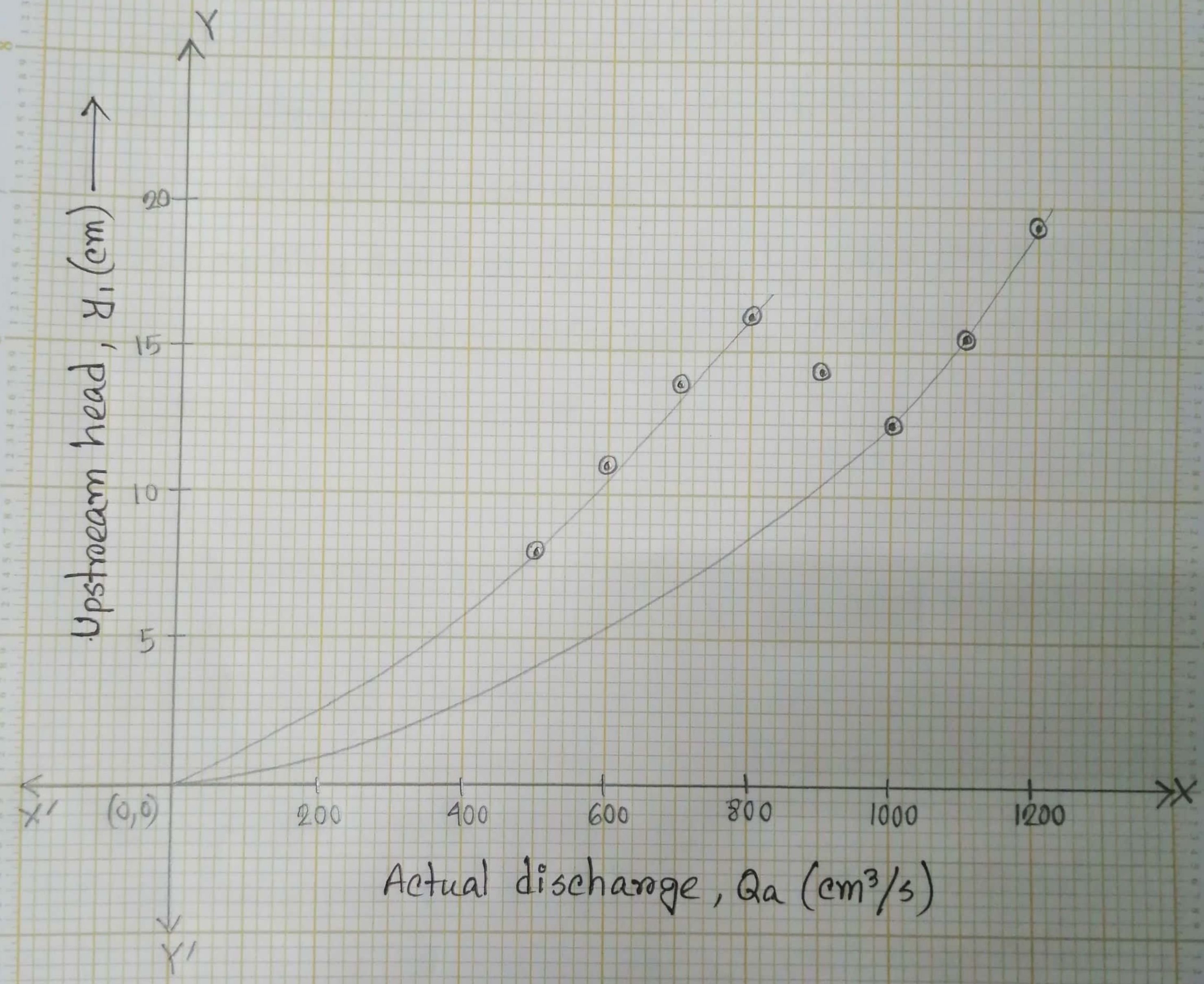
For submerged flow condition

Variation of upstream head (y_1) with respect to actual discharge (Q_a)



4.9 Results :

For free flow condition

a) Theoretical discharge of sluice gate, Q_{th} = 1303.81 cm^3/s

b) Co-efficient of velocity, C_v = 0.73

c) Co-efficient of contraction, C_c = 0.74

d) Co-efficient of discharge, C_d = 0.54

e) Resulting thrust of sluice gate, F_g = 64661.94 gm cm/sec^2

f) Horizontal thrust on sluice gate, F_H = 54593.88 gm cm/sec^2

g) The ratio F_g/F_H = 1.18

For submerged flow condition

a) Theoretical discharge of sluice gate, Q_{th} = 8432.99 cm^3/s

b) Co-efficient of velocity, C_v = 0.01

c) Co-efficient of contraction, C_c = 5.42

d) Co-efficient of discharge, C_d = 0.08

e) Resulting thrust on sluice gate, F_g = 59770.53 gm cm/sec^2

f) Horizontal thrust on sluice gate, F_H = 74812.29 gm cm/sec^2

g) The ratio F_g/F_H = 0.80

4.10 Discussion :

The result of this experiment varried due to the frictional resistance between the sheet of water and channel bottom. It was affected due to the effect of the rounding edge of the sluice gate.