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Flow over a Hump in an Open Channel: An Experimental Investigation of Discharge and Coefficient of Discharge

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***Abstract*—Flow over a hump is an open-channel hydraulic phenomenon in which a raised portion of the channel bed changes local water depth and velocity. This laboratory study investigates flow over a bed hump and estimates the coefficient of discharge by comparing the actual discharge, measured volumetrically, with the theoretical discharge calculated from the continuity equation and Bernoulli's energy equation. The experiment used a rectangular tilting flume of width 0.076 m, a pointer gauge, a bed hump, and a collecting tank of plan dimensions 0.4 m × 0.4 m. A tank-level rise of 0.1 m corresponds to a collected volume of 0.016 m^3. Upstream depth, depth over the hump, and collection time were recorded for seven runs. Recalculation from the transcribed readings gives an average coefficient of discharge of approximately 0.707; this result is provisional because some handwritten table entries differ from the recalculated values and should be checked against the original notebook. The measured depth over the hump was lower than the upstream depth in all seven runs. The study demonstrates the practical use of energy and continuity principles and highlights the influence of measurement uncertainty and idealizing assumptions on experimental discharge estimates.**

***Index Terms*—flow over hump, open-channel hydraulics, discharge, coefficient of discharge, specific energy, laboratory experiment.**

I. Introduction

Open-channel flow is the movement of water through a channel whose free surface is exposed to atmospheric pressure. Rivers, canals, drainage channels, spillways, and laboratory flumes are common examples. The flow depends on discharge, channel geometry, water depth, velocity, bed slope, and energy conditions [1].

A hump is a raised section of the channel bed that changes the available flow depth and the local hydraulic conditions. As water passes over the raised section, depth and velocity adjust according to the continuity and energy relationships, so a hump provides a useful laboratory arrangement for examining the relationship between water depth, velocity head, and discharge.

In an idealized analysis, the energy equation is applied between an upstream section and a section at the hump crest. For a horizontal bed and negligible energy loss, the difference in flow depths is related to the difference in velocity heads. The continuity equation expresses velocity in terms of discharge and cross-sectional area, allowing the theoretical discharge to be calculated from measured depths. Actual laboratory flow may differ from this prediction because of friction, turbulence, local disturbances, non-uniform velocity distribution, and measurement uncertainty. The coefficient of discharge, defined as the actual discharge divided by the theoretical discharge, provides a convenient comparison between the two.

The aim of this work is to investigate experimentally the flow of water over a hump in an open channel and to determine the coefficient of discharge. The specific objectives are (i) to understand the hydraulic behavior of water flowing over a raised channel bed; (ii) to measure the upstream depth and the depth over the hump under different flow conditions; (iii) to calculate the upstream and crest cross-sectional areas; (iv) to determine the actual discharge by the volumetric method; (v) to calculate the theoretical discharge using the continuity and Bernoulli energy equations; (vi) to determine the coefficient of discharge for each run; and (vii) to compare measured and theoretical discharge and identify possible sources of uncertainty.

II. Literature Review

The theoretical basis of flow over a hump comes from established open-channel hydraulics. Chow [1] presents the fundamental relationships among flow depth, velocity, energy, and channel geometry. These principles explain how a change in bed elevation can alter the available specific energy and the flow depth associated with a given discharge.

Subramanya [2] provides a further treatment of continuity, energy relationships, and flow regimes relevant to laboratory channel experiments. The continuity equation links discharge with cross-sectional area and mean velocity, while the energy equation relates depth and velocity head between selected sections.

The laboratory manual supplied for this experiment specifies a horizontal flume, depth measurements upstream and over the hump, volumetric discharge measurement, and calculation of the coefficient of discharge. This study follows that procedure and applies it to the seven completed runs in the supplied handwritten observation sheet. The discussion above is a concise theoretical review rather than a systematic review of individual journal articles; a final publication-oriented paper should add verified peer-reviewed studies on flow over bed humps, critical flow, and discharge measurement.

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III. Theoretical Background

A. Continuity Equation

For steady flow with no significant lateral inflow or outflow, the discharge is constant between the upstream section (1) and the hump crest (2):

$$Q = A_1V_1 = A_2V_2 \quad (1)$$

where Q is the discharge (m^3/s), A is the cross-sectional area (m^2), and V is the mean velocity (m/s). Hence $V_1 = Q/A_1$ and $V_2 = Q/A_2$.

B. Bernoulli's Energy Equation

For the idealized case of a horizontal bed and negligible energy losses, the energy equation between the two sections is

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

Substituting $V_1 = Q/A_1$ and $V_2 = Q/A_2$ from (1) and rearranging gives the theoretical discharge

$$Q_{th} = \frac{A_1A_2}{\sqrt{A_1^2 - A_2^2}}\sqrt{2g(y_1 - y_2)} \quad (3)$$

where y_1 is the upstream depth, y_2 is the depth over the hump, $g = 9.81$ m/s^2, and A_1 and A_2 are the respective flow areas. Equation (3) is appropriate only when the stated assumptions are reasonably satisfied.

C. Cross-Sectional Areas

For a rectangular channel of width B, $A_1 = By_1$ and $A_2 = By_2$. The flume width recorded in the manual is $B = 0.076$ m.

D. Actual Discharge by the Volumetric Method

The collecting tank has plan dimensions 0.4 m × 0.4 m, and the measured water-level rise is 0.1 m. The collected volume is therefore $0.4 \times 0.4 \times 0.1 = 0.016$ m^3, and the actual discharge is

$$Q_{act} = \frac{0.016}{t} \quad (4)$$

where t is the collection time in seconds.

E. Coefficient of Discharge

$$C_d = \frac{Q_{act}}{Q_{th}} \quad (5)$$

The coefficient of discharge is dimensionless and compares the measured actual discharge with the idealized theoretical estimate.

IV. Experimental Setup and Methodology

A. Apparatus

The apparatus listed in the supplied laboratory manual comprises a tilting flume, a bed hump, an adjustable tailgate or sluice gate, a pointer gauge mounted on a trolley, a discharge-measuring tank, and a stopwatch. The channel width is 0.076 m. The measuring tank has plan dimensions 0.4 m × 0.4 m, and the collection time is measured for a 0.1 m rise in water level. The experimental parameters are summarized in Table I.

TABLE I
Experimental Parameters

Parameter	Value
Flume width, B	0.076 m
Tank length	0.4 m
Tank width	0.4 m
Water-level rise in tank	0.1 m
Collected volume	0.016 m^3
Completed runs in supplied sheet	7

B. Procedure

The hump was fixed securely on the floor of the tilting flume, and the flume bed slope was set to zero to provide a horizontal bed. The water supply was started, and a steady flow was established using the control valve. The pointer gauge was positioned upstream, away from the local drawdown region, and the upstream depth y_1 was measured. The pointer gauge was then positioned at the centre of the hump, and the depth y_2 was measured. Water was collected in the measuring tank, and the time for a 0.1 m rise in water level was recorded. The flow rate was then varied and the measurements were repeated. Areas, actual discharge, theoretical discharge, and coefficient of discharge were calculated for each run.

The manual recommends 8–10 flow conditions; however, only seven completed runs were visible in the supplied handwritten record, so the present analysis uses those seven runs.

V. Results and Discussion

A. Observations

Table II is a transcription of the supplied handwritten observation sheet. Because handwriting is not perfectly clear, the values should be checked against the original laboratory notebook before final submission.

TABLE II
Transcribed Experimental Observations

Run	y_1 (m)	y_2 (m)	t (s)
1	0.0285	0.0164	44.99
2	0.0275	0.0168	54.27
3	0.0264	0.0162	27.40
4	0.0256	0.0148	27.80
5	0.0270	0.0179	25.47
6	0.0273	0.0169	27.47
7	0.0257	0.0170	31.09

B. Sample Calculation (Run 1)

For Run 1, $B = 0.076$ m, $y_1 = 0.0285$ m, $y_2 = 0.0164$ m, $t = 44.99$ s, and $g = 9.81$ m/s^2. The upstream area is $A_1 = 0.076 \times 0.0285 = 2.166 \times 10^{-3}$ m^2, and the area over the hump is $A_2 = 0.076 \times 0.0164 = 1.2464 \times 10^{-3}$ m^2. The actual discharge is $Q_{act} = 0.016/44.99 = 3.556 \times 10^{-4}$ m^3/s. From (3), the theoretical discharge is $Q_{th} = 7.426 \times 10^{-4}$ m^3/s, so $C_d = (3.556 \times 10^{-4})/(7.426 \times 10^{-4}) = 0.479$. The handwritten sample reports $C_d = 0.486$; the recalculation from the transcribed time and depths gives approximately 0.479. The discrepancy may arise

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from rounding or from a different original input and should be verified.

C. Tabulated Calculations

All entries in Table III were recalculated from the transcribed depths, collection times, flume width, and tank volume. The average coefficient of discharge is $C_{d,avg} = (\Sigma\, C_d)/7 \approx 0.707$.

TABLE III
RECALCULATED DISCHARGE RESULTS

Run	A_1 (10^{-3} m^2)	A_2 (10^{-3} m^2)	Q_{act} (10^{-4} m^3/s)	Q_{th} (10^{-4} m^3/s)	C_d
1	2.166	1.246	3.556	7.426	0.479
2	2.090	1.277	2.948	7.389	0.399
3	2.006	1.231	5.839	6.976	0.837
4	1.946	1.125	5.755	6.346	0.907
5	2.052	1.360	6.282	7.678	0.818
6	2.075	1.284	5.825	7.388	0.788
7	1.953	1.292	5.146	7.118	0.723

D. Variation in Depth

The upstream depths range from 0.0256 m to 0.0285 m, while the depths measured over the hump range from 0.0148 m to 0.0179 m. In all seven runs, the depth over the hump is lower than the upstream depth. This is consistent with the local influence of the raised bed, although depth data alone do not establish whether critical flow occurred.

E. Actual and Theoretical Discharge

The actual discharge was calculated by the volumetric method, whereas the theoretical discharge was calculated from the energy and continuity relationships. The actual discharge ranges from approximately 5.146×10^{-4} to 6.282×10^{-4} m^3/s for Runs 3–7, with lower values of approximately 3.556×10^{-4} and 2.948×10^{-4} m^3/s for Runs 1 and 2, respectively. The theoretical discharge ranges from approximately 6.346×10^{-4} to 7.678×10^{-4} m^3/s.

For all seven runs, the theoretical discharge is higher than the actual discharge. This is consistent with the fact that the theoretical equation represents an idealized flow condition, while actual laboratory flow may be affected by energy losses and measurement uncertainty.

F. Coefficient of Discharge

The recalculated coefficient ranges from approximately 0.399 (Run 2) to 0.907 (Run 4), with a mean of approximately 0.707. The variation between runs indicates that the ratio of measured to theoretical discharge was not constant across the recorded conditions. These values should be treated as provisional until the handwritten readings and calculations have been verified.

G. Measurement Quality

Depth measurement errors affect both cross-sectional areas and the depth difference in the theoretical discharge equation, while timing errors directly affect the actual discharge. The mismatch between several handwritten values and the recalculated table reinforces the need to verify the raw data and intermediate arithmetic before reporting final results. The current dataset does not independently establish critical flow or a hydraulic jump; additional velocity and depth analysis, or direct observation, would be needed to support such claims.

VI. LIMITATIONS AND FUTURE SCOPE

A. Limitations

The study has the following limitations: (i) only seven completed runs were available, although the manual recommends 8–10 conditions; (ii) the exact hump geometry is not fully documented in the supplied observation sheet; (iii) the theoretical equation assumes negligible energy losses and idealized flow between sections; (iv) pointer-gauge readings and collection timing are subject to measurement uncertainty; (v) some handwritten calculations differ from values recalculated from the transcribed observations; and (vi) no direct measurements of energy loss, velocity distribution, or critical-flow conditions were supplied.

B. Future Scope

Future work could (i) repeat the experiment for additional discharge conditions and replicate each measurement; (ii) test different hump heights and geometries; (iii) measure the water surface at additional positions along the flume; (iv) calculate the Froude number and specific energy at the upstream and crest sections where sufficient data are available; (v) investigate energy losses and the conditions for critical flow; and (vi) compare verified laboratory measurements with a numerical or theoretical model.

VII. CONCLUSION

This laboratory investigation examined water flow over a hump in a rectangular open channel using depth measurements and volumetric discharge measurements. The flume width was 0.076 m, and the collecting-tank volume corresponding to a 0.1 m water-level rise was 0.016 m^3. Seven completed runs were transcribed from the supplied observation sheet.

For each run, cross-sectional areas were calculated from the measured depths, the actual discharge was obtained from the collected volume and time, and the theoretical discharge was estimated using the continuity and Bernoulli energy relationships. The measured depth over the hump was lower than the upstream depth in all seven runs, and the recalculated theoretical discharge exceeded the actual discharge in every run.

The mean coefficient of discharge calculated from the transcribed observations was approximately 0.707, with individual values ranging from approximately 0.399 to 0.907. These results are provisional because some values in the handwritten calculation table differ from the recalculated values, and verification against the original notebook is necessary before the figures are treated as final. The experiment demonstrates the practical use of energy and continuity principles and the importance of careful measurement in open-channel hydraulics.

REFERENCES

[1] V. T. Chow, *Open-Channel Hydraulics*. New York, NY, USA: McGraw-Hill, 1959.

[2] K. Subramanya, *Flow in Open Channels*. Tata McGraw-Hill Education.

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