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Jump Characteristics Over the Downstream of the Weir

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***Abstract*—Diversion barrages across rivers are an important class of hydraulic structures constructed for applying a portion of the river flow into off-taking canals. The recommendations and design procedures for barrages have evolved over time considering mostly the normal operation conditions. However, there have been instances where unusual or emergency situations have led to the formation of exceptional flow conditions, some of which have been observed and reported by engineers responsible for the operation and management of barrages. Weirs are the barriers constructed across the canal or any water body to alter its flow characteristics. In Weirs, after the maximum storage water will spill out from the weir to the downstream of the weir. The spilled out water from the weir will drop from the certain height on the depressed apron, which causes damage to the depressed apron, in turn damages the weir. In this research work, an attempt is made to observe the jump characteristics and the effect of different jumps for particular discharge in an open channel is studied. A model of rectangular weir is chosen in the laboratory. Flow characteristics over the depressed apron are studied, for the different jump types and their respective pressures are measured.**

Keywords—Depressed Apron; A-Jump; W-Jump; Erosion; tail water

I. INTRODUCTION

Providing weirs in channel will result in the different flow types on the channel bed in the downstream. The damage on the channel bed will take place due the impact of spilled water from the weir, the downstream component of the weir is considered as depressed apron.

In hydraulic structures with sluice gates over weirs like barrages, canal head regulators, cross regulators, etc. generally dissipate the surplus energy over a stilling basin followed by a secondary apron. Although stilling basins are designed to contain the jump the shape and the placement of the secondary apron governs the typical of the jump, particularly for low tail water levels. As such, when the tail water level is excessive, as during the passage of the flood through a barrage, the flow through a sluice bay is of the submerged typical. As the water level falls, there is a gradual transition to a free jump. For further reduction in the tail water level, the jump moves downstream, eventually forming over the secondary apron. Here, the shape of the jump depends a lot on the elevation of the apron and also on its shape, that is, whether it is horizontal or in the form of a depression.

It has been reported by field engineers that low tail water levels, caused occasionally due to low discharges in rivers, may cause unstable jump that may be violent enough to cause damage to the secondary aprons. In a few projects, therefore, a depressed secondary apron is provided which is expected to absorb the turbulent fluctuations and stabilize the jump, thereby tumbling the chances of it causing a structural failure.

In this research work, an effort has been made to investigate the jump characteristics over a depressed apron under very low tail water level conditions and estimation of pressure fluctuations over the depressed apron.

II. LITERATURE REVIEW

A. Studies on Secondary Apron

Mossa et al. [10] showed the different jump characteristics at an abrupt drop in secondary apron. This study shows us the different jump characteristics on secondary apron at different secondary apron depth.

Mishra [13], in his Doctor of Philosophy thesis worked on the secondary apron jump characteristics for depressed apron using small cement blocks. he found the formation of jump over the depressed secondary aprons, but no conclusive flow regime was identified.

B. Effect of low tail water level on the barrage stilling basin and depressed aprons stability

In spite of many researchers on the dissipation of energy of the hydraulic jump within the stilling basin and standardization of basin aprons by different agencies, the exact amount of deficiency of tail water depth is not considered [2]. In fact, it is perhaps practically not possible to estimate the water level on the downstream of a stilling basin at the time of design. Different factors, like bed retrogression or extraordinary conditions like damage of a gate during the time of a low discharge (with corresponding low tail water level) may cause the sweep out of the hydraulic jump from the stilling basin apron. This may manifest adversely by developing low pressure on the stilling basin floor or may cause excess scour of the unprotected riverbed on the downstream. An instance of the first if the stilling of the stilling basin of the Karnafuli project in Bangladesh [3]. However, the impact on downstream scour may be so severe as to threaten the stilling basin apron structure itself by undermining. Hager [1] describes a few prototype examples where there have been damages to stilling basin due to low tail water level. For examples, Lock and Dam number 1, Mississippi River (Minnesota) was originally equipped with a level concrete apron 10m below the spillway crest. The apron was too high for classical jump, except for high discharges. Low discharges caused excessive scour below the apron, which prompted the addition of a baffle wall within the basin to force a hydraulic jump under such conditions and dissipate a part of the energy.

The requirements for considering occurrence of unusual or abnormal conditions have not been included so far in the Indian standard for barrage design. However, the United States Army corps of Engineers [12] manual on design of navigation dams recommends designing the stilling basin and secondary apron for the following conditions, which take into account some of the extraordinary situations:

- Normal or above pool level
- Gate operation – one gate fully open, one gate partially open, or all gates opened equally, and
- Normal or minimum tail water level

Apparently, it has been established that the tail water level contributes significantly to the extent of downstream scour [11]. According to Hager [1] also reported the causes of erosion of the stilling basin and the tail water unprotected riverbed due to the following causes:

- Too short apron or shallow basin floor for the formation of an effective jump
- Poor structural shapes within the basin and resulting damages by cavitations
- Abnormal flow condition during the construction period
- Inadequate stilling action and misconception in hydraulic design, among others.

Noverk and Cabelka (1981) recommend using a factor (K) multiplied by the difference of the conjugate depths to estimate the lengths of the stilling basin. K is seen to vary between 4.5 and 5.5 with the lower value being applied for higher Froude number and the higher Froude number, encountered in barrages and low head dams. USBR [11], however, recommend the length of stilling basin as a factor of the tail water depth measured above the apron elevation. This will result in higher basin lengths because of the following reasons, as discussed by Hager [4]:

- The basin must be effective of the river even when damages have occurred to the appurtenances
- Due to degradation of the river channel, the tail water becomes lower over a period of years and reduces safety against sweep out.
- Unless the tail water curve is known to be correct, an extra safety should be added
- The actual tail water depth for increasing discharge lags the tail water curve for steady flow rapid discharge variations must be compatible with tail water if stable jump should occur, and
- Slightly submerged jumps (or sloping jumps) produce better efficiency

C. Study on Pressure Fluctuations

Severe pressure fluctuations associated with the energy dissipation in hydraulic jumps have been identified as a cause of wreckage in some stilling basins. Considering these fluctuations, Fiorotto and Rinaldo [6] developed a design criterion based on the following sequence of events.

- Pulsating pressures damage the joint seal between slabs.
- Through these joint seals, extreme pressure values may propagate from the upper to the lower surface of the slabs.
- Instantaneous pressure differentials between the upper and lower surfaces of the slabs can attain higher values.
- The resultant force stemming from the pressure differentials may exceed the weight of the slab and the anchor resistance.

The equivalent thickness of the slab (including the anchors' contribution, if present), is defined by Fiorotto and Rinaldo [6] as

$$s = \Omega\left(\frac{l_1}{y}, \frac{l_1}{I_1}, \frac{l_2}{I_2}\right)\left(c_p^+ + c_p^-\right)\frac{v^2}{2g}\frac{\gamma}{\gamma_c - \gamma}\,\bar{p} \qquad (1)$$

Where Ω = dimension less co-efficient related to the instantaneous spatial distribution of the pulsating pressure;

c_p^+ and c_p^- = positive and negative pressure coefficients;

$\frac{v^2}{2g}$ = kinetic energy head of the incident flow

(v= mean velocity of the incident flow); γ and γ_c = specific weights of water and concrete respectively. The Ω coefficient depends on the ratio of the slab dimensions, I_1 and I_2, the depth of the incident flow, and the integral scales in the longitudinal and transversal directions, I_1 and I_2. Because p(t) is a random stationary process, it is convenient to use the pressure fluctuation *p'(t) = p(t) –* $\bar{p}$ relative to the mean pressure value $\bar{p}$. The p'^{+}_{max} and p'^{-}_{max} are, respectively, the maximum and minimum measured pressure values. They are related to the c_p^+ and c_p^- coefficients; Fiorotto and Rinaldo [7] by

$$\frac{p'^{+}_{max}}{\gamma} = c_p^+\frac{v^2}{2g} \text{ and } \frac{p'^{-}_{max}}{\gamma} = c_p^-\frac{v^2}{2g} \qquad (2)$$

III. EXPERMENTAL PROGRAMME

Experimental investigations were carried out in the Hydraulics and Hydraulic Machinery Laboratory of Jawaharlal Nehru National College of Engineering, Shimoga, Karnataka

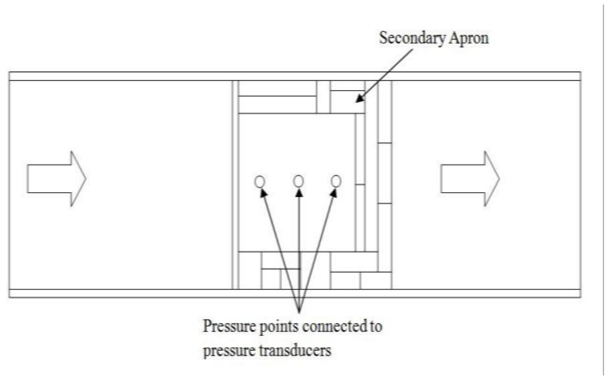

Fig. 1. Definition Sketch showing top view of the flume

Experiments were carried were performed till the maximum discharge is fed in the flume. For a particular discharge depth of water before the jump is measured y_1 and depth of water level after the jump is measured y_2. The length of the jump is measured as x_1. y' is the difference between y_1 and y_2.

Discharge is calculated rectangle weir discharge formula.

$$Q = \frac{2}{3} C_d B \sqrt{2g} y'^{1.5}$$

Experiments were carried out for different discharges with low tail water.

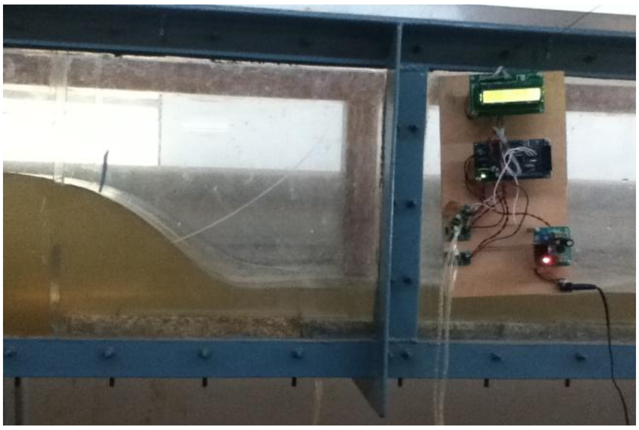

Fig. 2. Side View of the Flume

For the depressed aprons, pressure fluctuations under the jumps were obtained using pressure transducer which has pressure range 1 Pascal.

Three pressure transducers are used in the experiment at the three different positions in the depressed apron. The pressure transducers are connected to data acquisition/ switch unit & display unit for the data acquisition.

Pressure transducers are connected to pipes which are placed at three different locations on the surface of depressed apron as shown in figure below (Fig. 4), along the center line of the bay.

Pressure Transducers 'X', 'Y' & 'Z' are connected to points '1', '2' & '3' respectively on depressed apron.

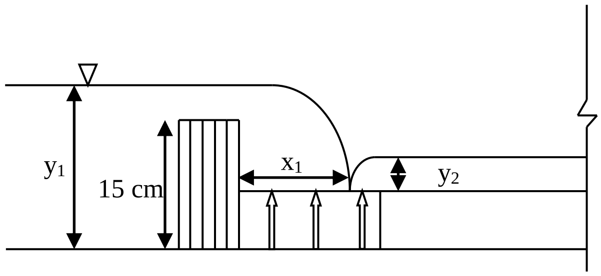

Fig. 3. Definition Sketch showing the sectional view of the flume

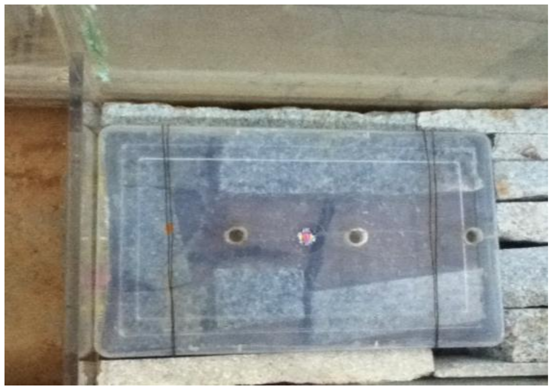

Fig. 4. Figure showing the points on depressed apron to which pressure transducers are fixed

Pressure Transducers and switch & display unit are fixed to the wooden board, which is fixed on the side of the flume.

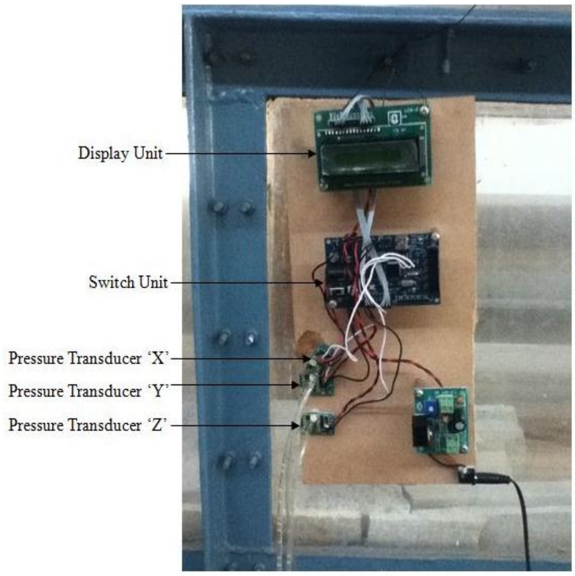

Fig. 5. Figure showing the pressure transducers and switch & display unit

A. *Calibration of Pressure Transducers*

Plastic Box, into which the three pipes connecting to respective Transducer are inserted up to the top surface, is immersed in a plastic bucket and water is filled to a height of 10 cm from the top surface of the box. Pressure readings from the switch unit is noted and compared with the calculated reading.

Pressure readings at:

- Pressure Transducer 'X' = 975 kg/ms2
- Pressure Transducer 'Y' = 971 kg/ms2
- Pressure Transducer 'Z' = 973 kg/ms2

Pressure calculated theoretically:

- Pressure at Point 1 = γ*g*h = 1000*9.81*0.010 = 981.00 kg/ms2
- Pressure at Point 2 = γ*g*h = 1000*9.81*0.099 = 971.19 kg/ms2
- Pressure at Point 3 = γ*g*h = 1000*9.81*0.099 = 971.19 kg/ms2

Theoretical readings almost match with the Transducer readings. Thus the pressure transducers readings are trustworthy.

B. Measurement Technique

For each experiment, measurements were carried out after the jump became stable. The water profiles of jump were taken by the point gauge and the tail water level recorded by using point gauge. Table -1 summarizes the experimental flow conditions, where for the particular discharge the experiment is carried out and readings were recorded.

The flow (jump) patterns are carefully analyzed for each run and results are reported in the results section.

For the pressure fluctuations, in each run of experiment pressure transducer are kept activated and readings are recorded through the switch unit.

TABLE I. PRESSURE AT TEST POINTS

Discharge (m^3/s)	Pressure (kg/ms^2)		
	Point '1'	***Point '2'***	***Point '3'***
6.432×10^{-3}	406	1070	913
7.534×10^{-3}	402	1085	905
8.299×10^{-3}	312	875	702
9.090×10^{-3}	299	854	672
10.319×10^{-3}	277	819	623
12.037×10^{-3}	285	866	640
13.154×10^{-3}	265	832	596
15.486×10^{-3}	283	913	636
17.942×10^{-3}	291	970	654

IV. RESULTS

Measurements are taken after Jump gets constant

V_1 - velocity at upstream side of the weir

V_2 - velocity at downstream side of the weir

F_{R1} - Froude number at upstream side of the weir

F_{R2} - Froude number at downstream side of the weir

TABLE II. JUMP CHARACTERISTICS FOR DIFFERENT DISCHARGES

Discharge No.	x_1 (cm)	y_1 (cm)	y_2 (cm)	y' (cm)	Q (m^3/s)
1	0.100	0.204	0.017	0.054	6.432×10^{-3}
2	0.115	0.210	0.020	0.060	7.534×10^{-3}
3	0.121	0.214	0.025	0.064	8.299×10^{-3}
4	0.135	0.218	0.028	0.068	9.090×10^{-3}
5	0.144	0.224	0.033	0.074	10.319×10^{-3}
6	0.153	0.232	0.038	0.082	12.037×10^{-3}
7	0.170	0.237	0.043	0.087	13.154×10^{-3}
8	0.194	0.247	0.049	0.097	15.486×10^{-3}
9	0.210	0.257	0.056	0.107	17.942×10^{-3}

TABLE III. JUMP CHARACTERISTICS FOR DIFFERENT DISCHARGES

Discharge No.	Q (m^3/s)	V_1 (m/s)	V_2 (m/s)	Froude No.	
				F_{R1}	F_{R2}
1	6.432×10^{-3}	0.112	1.351	0.079	3.308
2	7.534×10^{-3}	0.128	1.345	0.089	3.036
3	8.299×10^{-3}	0.138	1.185	0.095	2.390
4	9.090×10^{-3}	0.148	1.159	0.101	2.210
5	10.319×10^{-3}	0.164	1.116	0.110	1.961
6	12.037×10^{-3}	0.185	1.131	0.122	1.852
7	13.154×10^{-3}	0.198	1.092	0.129	1.681
8	15.486×10^{-3}	0.223	1.128	0.143	1.626
9	17.942×10^{-3}	0.249	1.144	0.156	1.540

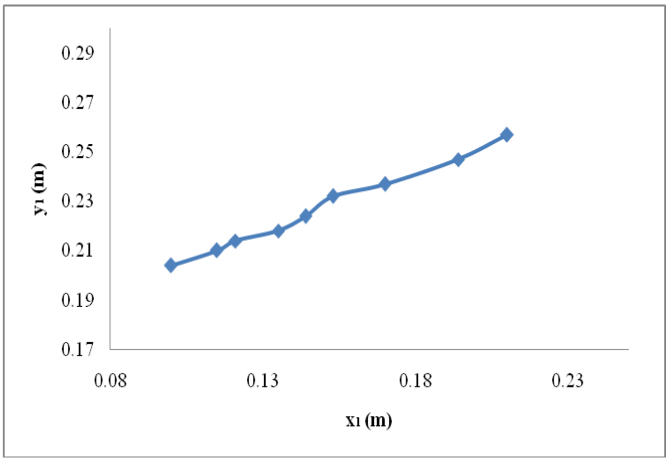

Fig. 6. Plot of x_1 v/s y_1

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As the depth of the flow in upstream side increases, there is a gradual increase in the length of the jump. Fig -6 shows the linear variation in the x_1 and y_1.

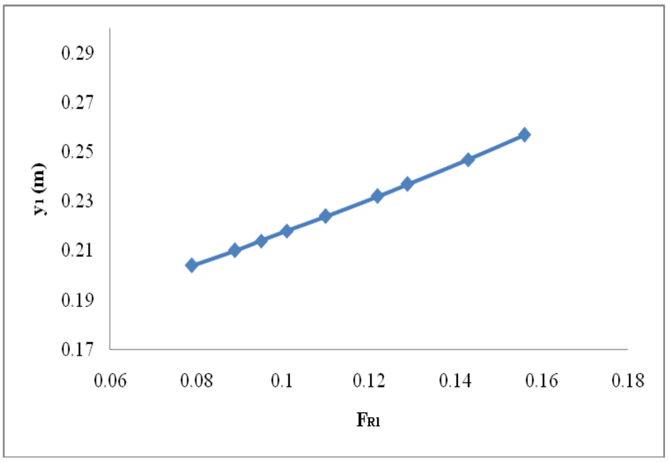

Fig. 7. Plot of F_{R1} v/s y_1

As the depth of the flow in upstream side increases, there is a gradual increase in the Froude number. Fig -7 shows the linear variation in the F_{R1} and y_1.

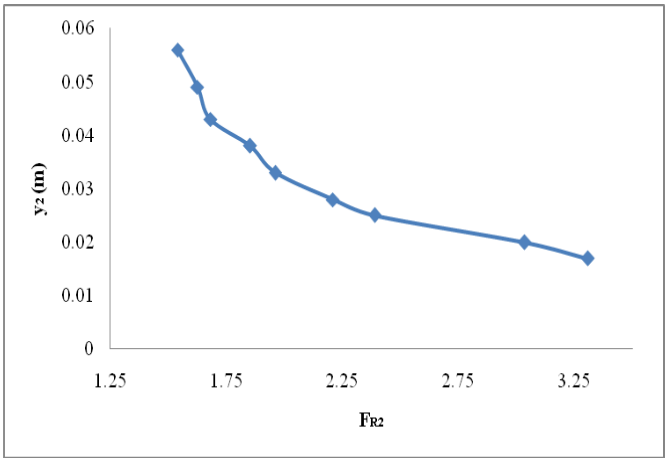

Fig. 8. Plot of F_{R2} v/s y_2

As the depth of the flow in downstream side increases, there is a rapid decrease in the Froude number. Fig -8 shows the linear variation in the y_2 and F_{R2}.

Fig. 9. Side View of the flume showing A-Jump for discharge No.4

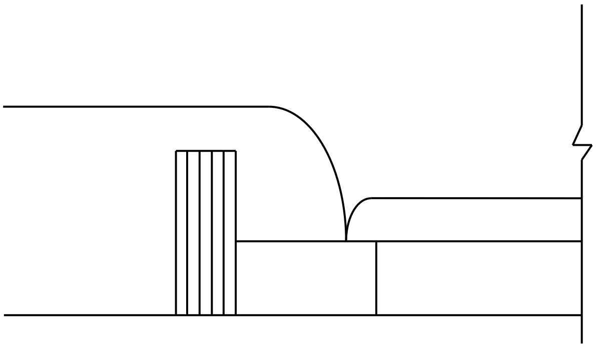

Fig. 10. Definition sketch of A-jump

Fig. 11. Side View of the flume showing W-Jump for discharge No.8

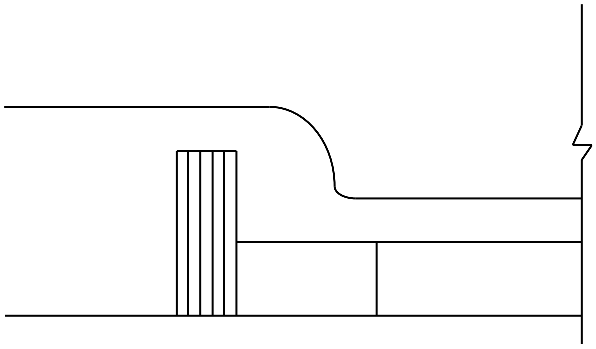

Fig. 12. Definition sketch of W-jump

TABLE IV. AVERAGE PRESSURE AT TEST POINTS FOR DIFFERENT JUMP TYPES

Type of Jump	Discharge No's	Average Pressure (kg/ms^2)		
		Point '1'	*Point '2'*	*Point '3'*
A-Jump	1, 2, 3, 4	354.75	971.00	798.00
W-Jump	5, 6, 7, 8, 9	280.20	880.00	629.80

V. CONCLUSIONS

It is observed from the experiments that the formation of hydraulic jump over the secondary apron depends upon the position of the secondary apron as well as the tail water elevation.

The following conclusions may be drawn from the experiments obtained:

- The flow type is of sub-critical nature in the upstream side of the weir as the value of Froude number falls below unity; whereas the flow type is of super-critical nature in the downstream side of the weir since Froude number value exceeds unity.
- A-type Jumps were formed for Discharge No's 1, 2, 3 and 4.
- W-type Jumps were formed for Discharge No's 5, 6, 7, 8 and 9.
- Jump transforms from A-Jump to W-Jump as the discharge increases.
- As flow is of super-critical nature n the downstream side, results in higher velocity which in turn leads to erosion in the downstream of the weir. So proper care should be taken at the downstream side, i.e., after the weir.
- The hydraulic jump is relatively more stable in lowered secondary aprons and the A and W types of hydraulic jump forms are observed to take place under low tail water levels.
- A-jump has the greater velocity than the W-jump. Therefore from analyzing the jump characteristics it is found that more care should be taken in downstream when A-jump is formed.
- Pressure is more at the point where the jump falls, and pressure is minimum at the point which located before the jump.
- The pressure decreases as the discharge increases, i.e., as the Jump is changing from A-type to W-type the pressure decreases, which is shown in Table -4.
- Hence we propose that W-type of Jump should be maintained to avoid erosion of downstream bed.

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