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Pawana Dam Energy Dissipation – A Case Study

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Abstract: Energy dissipation is one of the important aspects in the life of dams, barrages and outlets. For the medium head projects, hydraulic jump type energy dissipator is a common choice. In case of ogee spillway when flood water is discharged, by the time it reaches the toe of spillway, the potential energy gets converted into kinetic energy. The flow turns into a supercritical flow having lot of erosion potential. The flow can be converted into a subcritical flow by forming a hydraulic jump on the downstream apron. Many dams are suffering and some of them are victims of the inadequate energy dissipation. Location of hydraulic jump plays a vital role to accomplish the task of energy dissipation. Present study is carried out to address this problem and a new technique is proposed to obtain appropriate location of hydraulic jump for variable discharge and corresponding tail water submergence conditions. The technique so developed is applied to the energy dissipator of one of the existing dams and the performance is confirmed by model study. The results and findings are presented herein.

Key Words: Hydraulic jump, ogee spillway, stepped weir, jump location, energy dissipation.

INTRODUCTION

Dams and reservoirs have become an integral part of modern civilizations. Apart from being a reliable source to fulfill various needs like drinking water and irrigation, they are becoming a part and parcel of nation's economies. Thus it is essential to safeguard and maintain such hydraulic structures. Many dams all over the world reported some sort of problems associated with the energy dissipating arrangements. The origin of the problem lies in the fact that the energy dissipators are conventionally designed for the design discharge of spillway but frequently they have to cater to discharges lower than the design discharge. Similar problems are encountered in case of Pawana dam in India. Pawana dam is a multipurpose irrigation project which is also supplying water for domestic purpose, industries and hydro power generation. It is situated at 40 km west of Pune and is an earthen dam with a masonry ogee spillway on the left flank. The catchment area is 113.36 km^2 and the command area is 74.68 km^2. Total length of dam is 1329 m which includes earthen portion of 905 m and masonry portion of 424 m. Gross capacity of dam is 305 Mm^3 which includes 31 Mm^3 of dead storage and 274 Mm^3 of live storage. The design discharge of spillway is 1250 m^3/s and there are 6 radial gates (each of size 12.19 X 4.27 m). Almost every year the spillway has to pass flood discharges which never exceeded even 50% of the design discharge (in last 40 years) since the commencement of the project. But some serious problems of erosion at the toe of spillway and in the tail channel are reported. Present study is undertaken to analyze the problems and propose a solution to improve the hydraulic efficiency of the stilling basin.

Problem Statement:

In case of earthen dams normally the spillways are located on either flank or entirely away from the dam depending upon the site suitability. In such situations to join the return flow to the river, a tail channel is constructed by excavating the rock. Various factors like apron level and slope of tail channel depend upon the geological conditions of the rock formations and the supercritical Froude numbers. To achieve economy, it is preferred to have apron level at higher elevation and provide considerable slope to the tail channel. This

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situation creates tail water deficiency due to which the tail water rating curve (TWRC) lies below the jump height curve (JHC) for all discharges. Due to this, the hydraulic jump may partially or fully sweep out of the basin. This may prove to be dangerous from the safety point of view of stilling basin, tail channel and other hydraulic structures (Tung and Mays 1982; Moharami *et al* 2000). Also due to inadequate energy dissipation, large amount of residual energy remains with the return flow which can cause erosion of the tail channel. Fig. 1 shows riprap provided in the tail channel of the pawana dam spillway to provide protection against tail channel erosion.

Fig. 1: Photograph showing riprap in the tail channel portion of Pawana dam spillway

Fig. 2. Photograph showing typical erosion at toe of Pawana dam spillway

Thus location of hydraulic jump should be restricted to a stipulated zone (i.e. on apron) to successfully accomplish the task of energy dissipation (Rouse et al 1958). For this purpose it is necessary to artificially increase the depth of water on apron and create favorable conditions for forced hydraulic jump formation (Rajaratnam and Murahari 1971). In present practice it is achieved either by depressing the apron or by constructing a rectangular broad crested weir at the end of the apron. The depth of depression or the height of weir is designed for design discharge of spillway. Thus at flood discharges lower than design discharge, a drowned or submerged jump is formed. The deleterious effects of drowned jump are well known. Firstly, high velocities near bed and bed shear stress sustain for longer distances with increased submergence (Argyropoulos 1963). Secondly, up to certain limit of submergence, the energy dissipation is satisfactory but after that it reduces (Govinda Rao and Rajaratnam 1963; Wu and Rajaratnam 1995). Fig. 2 shows typical erosion at toe of pawana dam spillway due to drowned hydraulic jump. Also the length of drowned jump exceeds that of the corresponding free jump requiring longer basin (Vittal and Al-Garni 1992). This further emphasizes the necessity of proper location of free hydraulic jump and the most appropriate location is the toe of spillway. Because at that location the pre jump depth (y_1) is minimum and the corresponding post jump depth (y_2) (i.e. sequent depth) is maximum. Also the supercritical Froude number F_{r1} is maximum and hence the energy dissipation is maximum.

Proposed Solution:

For appropriate location of hydraulic jump inside the apron for all operating conditions with reference to varying discharges and the corresponding tail water depths, it is proposed to design a stepped weir at the end of the apron. In past, few researchers worked in this regard and tried to give some justice to varying discharges (Vittal and Al-Garni 1992; Achour and Debabeche 2003). But without consideration of tail water submergences no design can be completed as such. In the present study average tail water submergence ratio (S_r) has been considered as one of the input parameters. A stepped weir design (where every step is designed to pass a particular discharge) mainly involves two aspects.

Condition 1 - With reference to Fig.3, for any cumulative discharge (Q_n) , $y_2 = y' + h$.

Condition 2 – The widths (b_1, b_2 etc. as shown in Fig. 4) of individual steps should be such that the summation of particular discharges contributed by individual steps under consideration should be equal to cumulative discharge in condition 1.

The proposed design would cater to a wide range of discharge from design discharge (Q_{max}) to a minimum discharge equal to 20 % of the design discharge (Q_{min}). ‘N’ intermediate discharges between Q_{min} and Q_{max} with an increment of (Q_{max} - Q_{min}) / (N+1) are considered resulting in (N+2) cumulative discharges corresponding to which there would be (N+2) steps in a stepped weir respectively. The equation for discharge Q over a rectangular sharp crested weir is given as

$$Q = \frac{2}{3} k C_d b \sqrt{2g}\, h^{3/2} \tag{1}$$

Where submerged flow coefficient k=1 for free flow over weir and k < 1 for the submerged weirs. By using equation (1), for (N+2) cumulative discharges, widths of (N+2) steps can be calculated as follows.

$$b = \frac{3}{2} \frac{Q}{k C_d \sqrt{2g}\, (y_2 - y')^{3/2}} \tag{2}$$

In equation (2), y_2 is calculated from the Belanger momentum equation as given below.

$$y_2 = \frac{y_1}{2} \left(-1 + \sqrt{1 + 8 F_{r1}^2} \right) \tag{3}$$

Where, $y_1 = \frac{Q}{B v_1}$, $v_1 = \sqrt{2gH}$, $F_{r1} = \frac{v_1}{\sqrt{g y_1}}$

The y’ is to be designed for Q_{min} and is given as, $y' = y_2/4$. Now, the only unknown in equation (2) is k. When tail water depth (y_t) rises above the crest, the crest of weir is said to be submerged. The effect of tail water submergence is discussed in USBR publications like Monograph 25 (1984), Manual of Water Measurement (2001) and Design of Small Dams (1974). Also Villemonte (1947) has studied effect of submergence on discharge over rectangular sharp crested weir and has given a relation between k and S_r. But a similar detailed analysis is not available for rectangular stepped weir. But as stepped weir can be considered to be made up of number of rectangular weirs, k can be obtained by Villemonte’s equation given below.

$$k = \frac{Q_s}{Q_f} = \left(1 - (S_r)^{3/2}\right)^{0.385} \tag{4}$$

Where, Q_s and Q_f are the submerged and free flow discharges respectively and

$$S_r = \frac{(y_t - y')}{(y_2 - y')}$$

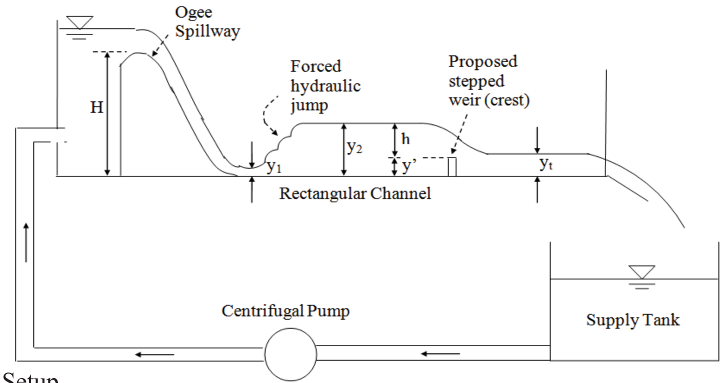

Fig. 3: Experimental Setup

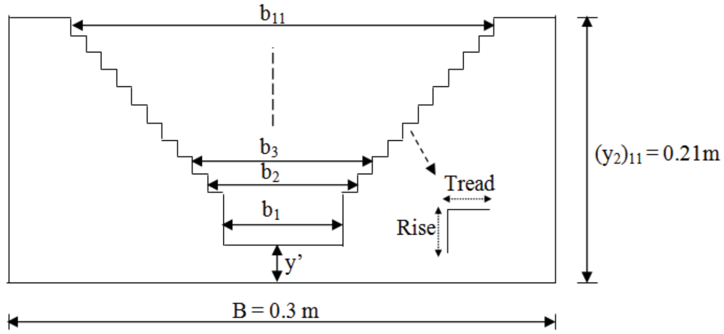

Fig. 4: Definition sketch of geometry of rectangular stepped weir

Design Philosophy:

Assumptions:

1. Channel is rectangular and prismatic with horizontal slope.
2. Flow is steady.
3. Head on upstream of ogee spillway (H) is constant (considering upstream and downstream floors at same elevation).
4. Discharge conditions are varying. (Variation up to 20% of design discharge).
5. Coefficient of discharge C_d remains constant.

Construction of Scale Model and experimentation:

A geometrically similar scale model (scale 1: 40) of Pawana dam spillway (single bay of single gate) is designed by using Froude's model law. Table 1 gives the design parameters of prototype and scale model. A spillway model is fabricated with 12 mm thick plywood and 5 mm thick perspex sheet. Fig. 3 shows the experimental setup – a rectangular prismatic channel and a spillway model placed in it. On the actual site the side walls are extended beyond the broad crested weir up to 80 m distance and after that there is a drop of 1.5 m. Efforts are made to simulate the site conditions in laboratory. Initially the tests are taken with the existing broad crested weir model for different discharges ranging from 0.004 to 0.02 m^3/s. It is observed that for all the discharges submerged jumps are formed. As in all the cases the tail water levels are well below the crest of weir, there is no effect of tail water submergence on the broad crested weir.

In the next trial new stepped weir with 11 steps is used instead of the existing broad crested weir. The widths of steps are determined by using equation 2. The other design parameters are C_d=0.623 and y'=0.024 m. A stepped weir is designed for average S_r = 0.15, for which k=0.98 is determined by using equation (4). But the experiments showed that stepped weir designed with k = 0.98 has formed little bit drowned jumps for all discharges. Thus k is slightly reduced and a revised k=0.96 is used while designing a stepped weir. Detailed geometry of stepped weir with k=0.96 (shown in Fig. 4) is given in Table 2. Table 2 shows rise and tread for all 11 steps. The stepped weir is then tested in laboratory for different discharges by maintaining the respective tail water levels. It is observed that for all the discharges the hydraulic jumps are formed near the toe of spillway. Also the residual energies of water in the tail channel are measured in both the trials.

Table 1. Dimensions of Pawana dam spillway model (single bay) by Froude's law (Scale 1 : 40)

Design Parameter	Designation	Prototype	Model
Stilling Basin width	B	12 m	0.3 m
Stilling Basin length	L	60 m	1.5 m
Height of spillway	H	25.3 m	0.6325 m
Upstream slope	-	Vertical	Vertical
Downstream slope	-	1 in 0.85	1 in 0.85
Height of broad crested weir	-	4.36 m	0.109 m
Top width of broad crested weir	-	2.95 m	0.074 m
Bottom width of broad crested weir	-	5.47 m	0.137 m
Upstream slope of broad crested weir	-	Vertical	Vertical
Downstream slope of broad crested weir	-	1 in 0.58	1 in 0.58
Design discharge of spillway	-	1250 m^3/s	0.124 m^3/s
Design discharge (single bay)	Q_{max}	208 m^3/s	0.020 m^3/s
Minimum discharge	Q_{min}	41.6 m^3/s	0.004 m^3/s
Supercritical Froude number (for Q_{max})	F_{r1}	8.18	8.18

Table 2. Geometry of stepped weir for k = 0.96

Step number	Rise (m)	Tread (m)	Step number	Rise (m)	Tread (m)
1	0.072	0.0585	7	0.010	0.0039
2	0.017	0.0107	8	0.010	0.0036
3	0.015	0.0070	9	0.009	0.0033
4	0.013	0.0054	10	0.008	0.0031
5	0.012	0.0048	11	0.008	0.0030
6	0.011	0.0042			

RESULTS AND DISCUSSION

Although the location of front of hydraulic jump, in all the cases is found to be slightly fluctuating near the toe of spillway, the mean location is found to be constant over a complete range of Q and corresponding S_r. Thus the performance of stepped weir section is verified experimentally. The experimental y_2 depths corresponding to both – broad crested weir (existing) and stepped weir (new) are compared with the ideal y_2 depths given by the Belanger momentum equation and entered in Table 3. These ideal y2 depths, experimental y_2 depths and the experimental y_t depths are plotted against Q in Fig. 5.

Table 3. Experimental results for the existing and new weir

Q (m^3/s)	y_2 (m) (ideal as per Belanger equation)	y_2 (m) for the existing broad crested weir	y_t (m) for the existing broad crested weir	y_2 (m) for the new stepped weir	y_t (m) for the new stepped weir	E_t (m) for the existing broad crested weir	E_t (m) for the new stepped weir
0.0200	0.21	0.208	0.05	0.195	0.054	0.140	0.131
0.0181	0.2	0.202	0.047	0.186	0.052	0.131	0.121
0.0160	0.188	0.195	0.044	0.175	0.049	0.119	0.109
0.0130	0.17	0.187	0.04	0.158	0.045	0.099	0.092
0.0095	0.146	0.175	0.035	0.135	0.04	0.077	0.072
0.0077	0.132	0.163	0.03	0.12	0.037	0.067	0.061
0.0065	0.122	0.15	0.025	0.11	0.035	0.063	0.055
0.0043	0.1	0.144	0.018	0.09	0.03	0.051	0.042

It shows that, over a range of Q and the given S_r, due to presence of weirs, all existing and new JHC are raised above respective TWRC. In case of stepped weir new JHC lie slightly below the ideal JHC. But for the existing broad crested weir up to around 75% of the design discharge (i.e. most frequent range of operation), the existing JHC lie above ideal JHC, after that, it approaches ideal JHC slowly and approximately matches with it at the design discharge. This shows that, in case of broad crested weir, for all the discharges up to 75% of design discharge, the y_2 depths are greater than the ideal y_2 depths and hence the jumps are submerged.

Even though new JHC lies slightly below ideal JHC it is forming free hydraulic jumps near toe of spillway for all the discharges. This is because the ideal JHC does not include frictional losses. Also Fig. 5 shows plots of existing and new TWRC. This indicates that the residual energy (E_t) of the tail water in case of stepped weir is relatively less as compared to the existing broad crested weir. Fig. 6 shows a plot of percentage energy dissipation (%ΔE) against F_{r1} for the existing and the new weir. It shows that the %ΔE in case of new weir is greater than that of existing weir.

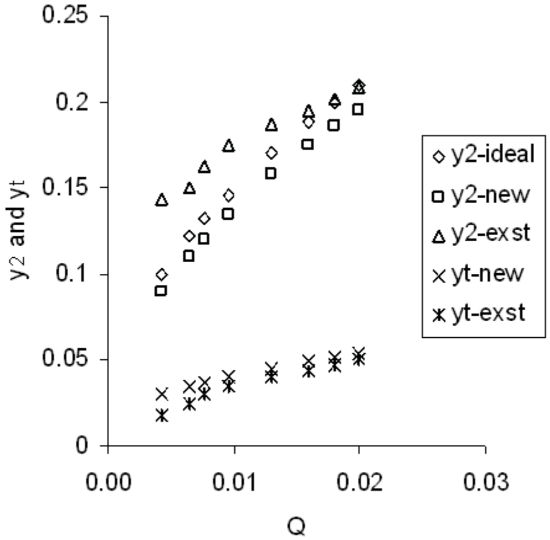

Fig. 5: Ideal-JHC, New-JHC, Exst-JHC, New-TWRC and Exst-TWRC

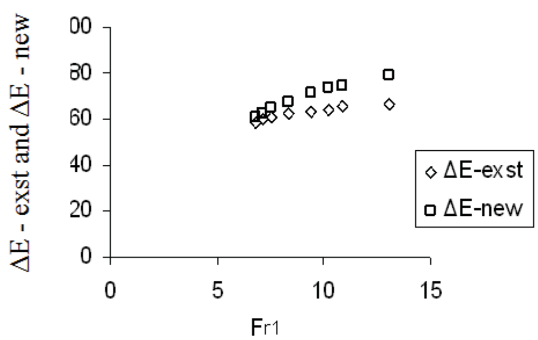

Fig. 6: Plot of ΔE Vs Fr1 for existing and new weirs

Conclusions:

A model case study of single bay of pawana dam spillway is carried out with 1:40 scale. The performance of stepped weir geometry is experimentally verified as the location of front of hydraulic jump is restricted near the toe of spillway for different discharges (ranging from Q_{min} to Q_{max} and $F_{r1} > 4.5$) and specific range of S_r. Parameters like percentage energy dissipation and residual energy of tail water are compared between the existing broad crested weir and new stepped weir. In case of stepped weir relatively the %ΔE is found to be greater and the E_t is found to be lesser (even lesser than 20% of E_t for the Q_{min}).

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Notations

B=Width of rectangular channel / stilling basin
b=Width of step of rectangular stepped weir
C_d=Coefficient of discharge for free weir
ΔE=Energy loss
E_t=Residual energy of tail water
Fr_1=Supercritical Froude number
H=Head on upstream of spillway

h=Head over weir crest
k=submerged flow coefficient
Q=Discharge
S_r=submergence ratio for the stepped weir crest
v_1=Supercritical velocity (neglecting losses)
y'=Height of weir crest from the channel bed
y_1=Pre jump depth
y_2=Post jump depth or sequent depth
y_t=Tail water depth

REFERENCES

Achour, B., M. Debabeche, 2003. "Control of hydraulic jump by sill in triangular channel" J. Hydr. Res., 41(3): 319-325.

Argyropoulos, P.A., 1963. Discussion of "The submerged hydraulic jump", by Govinda Rao, N.S., Rajaratnam, N., J.Hydr.Engg., ASCE, 89(3): 409.

Design of small dams, 1974. "Chapter IX-Spillways". A water resources technical publication, U.S. Bureau of Reclamation.

Govinda Rao N.S., N. Rajaratnam, 1963. "The submerged hydraulic jump". J.Hydr.Engg., ASCE, 89(1), 139-162.

Moharami, H., M. Ghodsian, S. Bahramian, 2000. "Optimal design of stilling basin". Proc. 4th International Conference on Hydro Science and Engineering, Seoul, 1-8.

Peterka, A.J., 1984. "Hydraulic design of stilling basins and energy dissipators". Eng. monograph no. 25, U.S. Bureau of Reclamation.

Rajaratnam, N., V. Murahari, 1971. "A contribution to forced hydraulic jumps". J. Hydr. Res., 9(2): 217-240.

Rouse, H., T.T. Siao, S. Nagaratnam, 1958. "Turbulence characteristics of the hydraulic jump". J. Hydr. Engg., ASCE, 84(1): 1528-1-31.

Tung, Y.K., L.W. Mays, 1982. "Optimal design of stilling basins for overflow spillway". J. Hydr. Engg., ASCE, 108(10): 1163-1178.

Vittal, N., A.M. Al-Garni, 1992. "Modified type III stilling basin – new method of design". J. Hydr. Res., 30(4): 485-498.

Water measurement manual., 2001. "Chapter 7-weirs". A water resources technical publication, U.S. Bureau of Reclamation.

Wu, S., N. Rajaratnam, 1995. "Free jumps, submerged jumps and wall jets". J. Hydr. Res., 33(2): 197-212.