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Hydraulic Model Studies for Optimization of Spillway and Energy Dissipator of Hydroelectric Dam Project - A Case study

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ABSTRACT
Dams along with the hydroelectric projects are critical infrastructure constructed with large investments and recurring expenditure of maintenance. The hydraulics of the dam should be optimized in all aspects before the execution of construction work. Spillway plays very important part in any dam project, therefore it should be analyzed and design safely to avoid damage on downstream side. The most reliable method of investigation of flow over spillways is performing experiments on scaled models. For number of decades the art of hydraulic modeling has been an important tool in solving hydraulic problems. This paper explains a case study of a hydroelectric project which is located in Western Bhutan. Hydraulic model studies is conducted on a 1:70 scale 3-D Physical model to optimize the design of stilling basin. Various studies/parameters such as pressure distribution along the spillway profile, water surface profiles over the spillway, and performance of the spillway is carried out in the physical model studies. After performing experimentation on physical model, different alternatives has been suggested and has been carried out successfully on dam site.

Keywords: Spillway, Energy Dissipator, Cavitation Index, Orifice Spillway, Ski-Jump Bucket, etc.

I. INTRODUCTION

The spillway is a structure constructed over a dam to discharge surplus water from upstream to downstream end without endangering safety of dam. The main function of spillway is to pass surplus water at downstream side. The Spillway should be structurally and hydraulically stable and it should be provided with proper energy dissipation. Therefore, hydraulic modeling plays an important role in designing and analyzing the complex physical processes. The design of dam is based on some factors such as location of spillway, the topography of area, purpose of dam and location of the spillway.

The design of recent project involving new upgraded spillway haves have benefited from the complementary use of physical scale modelling. As well as providing significant benefits to the design process for the projects, the design process recently adopted for complex spillway designs involved the initial development of a concept of preliminary design using theoretical and empirical design methods. These arrangements were then analysed and optimised through “Hydraulic Modelling” and where required, Physical scale modelling is used for final verification and refinement.

The present study on physical model is to assessment of performance of spillway in disposing probable maximum flood (PMF) within permissible water level and energy dissipator in disposing of flood safely to downstream, assessment of maximum scour depth and formation of plunge pool in scour studies and to suggest any modification necessary in improving the performance.

Theoretical Background

Orifice Spillway

Spillways are devices provided in conjunction with dams to pass surplus water for reservoir regulation and safety. They are provided in various forms according to their functions such as Ogee, Chute, shaft or morning glory, siphon, chute, side channel, tunnel spillway, sluice spillway, breast wall spillway etc. The latter two forms are also called as orifice spillway. Fig 1 shows orifice spillway.

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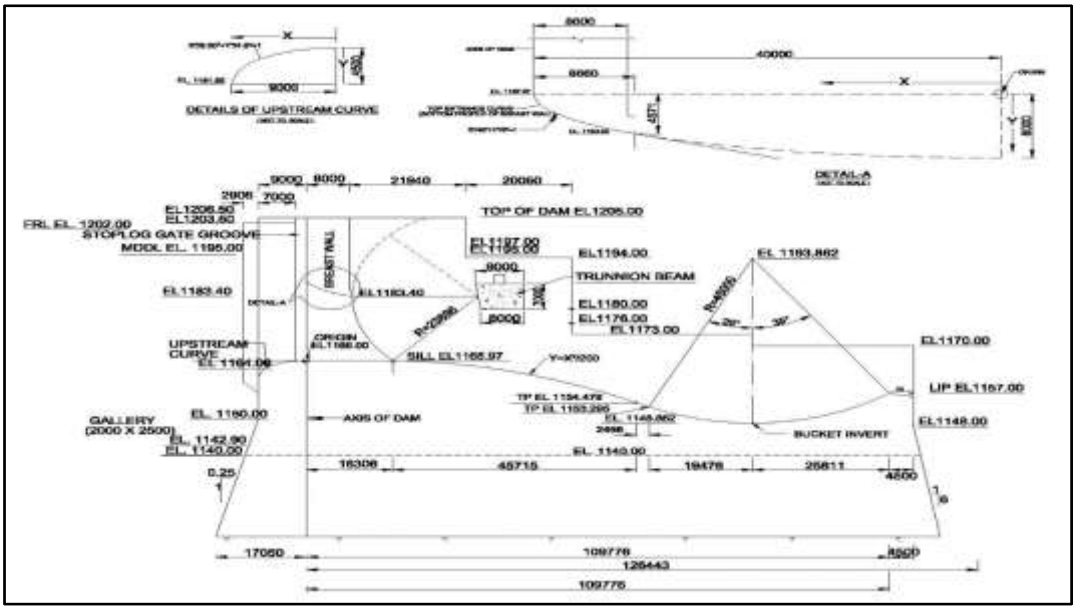

Figure 1: A typical breast wall / Orifice Spillway

Advantages and Disadvantages of Orifice Spillways

Several large dams have been constructed with orifice spillways having breast walls or sluices. The main advantages are:

- Can be accommodated in a narrow valley
- Reduction in height of spillway gates
- Reduction in number of spillway spans

The possible disadvantages of an orifice spillway are as follows:

- Decreased operational reliability of the structure, which would normally necessitate the addition of an emergency gate an alternate means of closing a spillway span if the main gate was in operative due to trash wedging resulting in a decrease in the flow capacity. The decreased flow capacity could result in overtopping of the dam.
- Although trash wedging can occur with an overflow spillway as well, gaining access to the blocked flow passage of an orifice spillway would be more difficult.

Hydraulic Design of Orifice Spillway

Though several large dams have been constructed in India and abroad with orifice spillways, the hydraulic considerations design of orifice spillway do not have fixed guidelines. The information available for orifice spillways indicate that water head of the order of 30-60 m over the crest is a common feature. The width and height of the spillway openings vary between 6-10 m and 10-14 m respectively with discharge intensities of 100 m^3/s per m and above. Thus, huge breast walls of the height ranging from 20-40 m are required to be supported on 6-7 m thick piers. Also, larger velocities associated with the high heads of an orifice spillway may increase the cavitation and erosion damage to the structure.

Hydraulic Design Considerations of Orifice Spillway

The following conditions arise while designing a system to serve the dual purpose of flood and sediment disposal:

- Discharge characteristics of spillway
- Waterway of the structure in relation to the width of the natural water course
- Size and dimensions of the orifice and its disposition with respect of the power intakes
- Protection of the spillway surface from abrasion
- Choice of energy dissipator and its design

Hydraulics of Orifice Spillways

Design of orifice spillway involves following parameters:

- Design of Crest Profile
- Discharging capacity of spillway
- Design of Roof of Breast Wall / Sluice
- Structural design Considerations

Design of crest profile

The hydraulics of orifice spillway changes with the varying reservoir levels. The flow is free flow for reservoir water levels below the top of the

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roof of the sluice. Orifice flow is for higher water level flow. The crest profile is need to be designed for pressurized flow. The spillway crest profile is flat as compared to the standard WES crest profile to avoid separation and negative pressures on the crest for small partial gate openings. The crest profile follows the equation

$$x^2 = 4 * H * y \quad (1)$$

Where, H is the head over the centreline of the orifice.
The bottom profile of the breast wall is generally an elliptical profile.

Discharging Capacity of Spillway

Orifice type spillways are provided in the dam at lower level for smootheing the flushing of the sediment from the reservoir also for spilling the flood water. However, in most of the hydraulic structures, particularly in the earthen and rock fill dams, the spillway is not a part of the dam and it is provided on either of the banks of the river. An experimental study for discharge characteristics of orifice type spillway is carried out under straight and oblique approach flow. Past analysis of data indicates that discharge through the spillway decreases with increase of obliquity of the flow.
Generally, the orifice flow condition sets in for heads over crest in excess of about 1.5 to1.7 H_m, where $H_m$is the height of the orifice opening.
For free flow conditions the discharge is given by,

$$Q = \frac{2}{3}\sqrt{2g}Cd\ LH^{3/2} \quad (2)$$

Where,
Q = Discharge in m^3/s
Cd =Coefficient of discharge
L = Length of spillway in m
H = Head over the crest in m

For Orifice flow conditions the discharge is given by,

$$Q = Cd.\,n\,.\,A\,\sqrt{2g\,(H - Hm}/2) \quad (3)$$

Where,
Q = Discharge in m^3/s
Cd = Coefficient of discharge
n = Number of span
A = Area of orifice in m^2 = L x D
H - Hm= head over the centre line of orifice

The coefficient of discharge is influenced by the entrance profile - composed by roof profile or the bottom profile of the breast wall, spillway crest profile, side wall profiles if provided. The coefficient of discharge for the orifice flow is ranges from 0.75 to 0.95. Figures 2 definition sketch for calculation of discharging capacity. Like Ogee Spillways, the design of breast wall spillways has not been standardized. Therefore, recourse is taken to study the existing structures, while designing a new project.

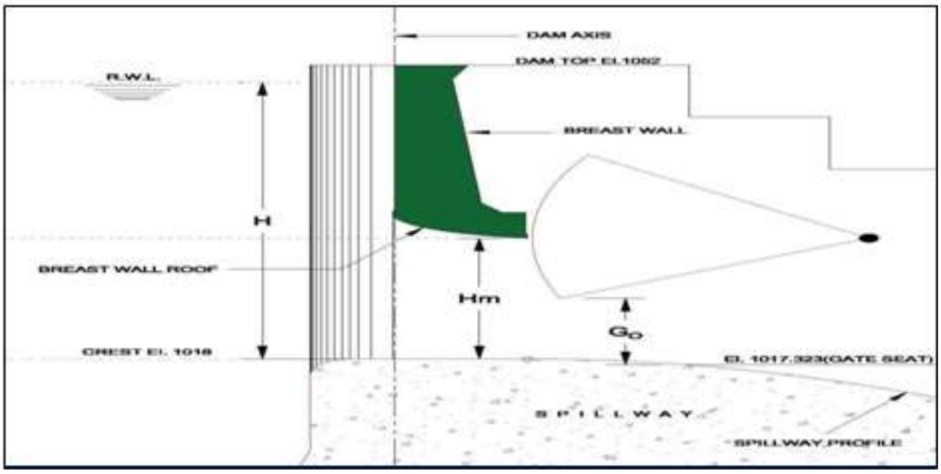

Figure 2: Definition sketch for calculation of discharging capacity

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Design of Roof Profile of Breast Wall / Sluice

Hydraulic design of roof profile of breast wall is very important, because the bottom profile of the breast wall guides the flow smoothly. This controls the coefficient of discharge of the spillway. This profile should be simple to construct and the pressures on the profile should not be negative. Usually, a profile in the form of quarter of an ellipse and it is given by equation.

$$\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1 \quad (4)$$

Where,

'a' is width of semi-major axis i.e. the width of breast wall and

.'b' is width of semi-minor axis which governs the steepness of the profile.

Figure 3 show a typical breast wall bottom profile. Usually steep profiles define increased coefficient of discharge, whereas flat profiles help to reduce the discharging capacity. However, as the profile becomes steep negative pressures increases. High negative pressures with cavitation index below 0.2 are unacceptable. The bottom profiles of breast walls are usually steel lined to avoid cavitation damage.

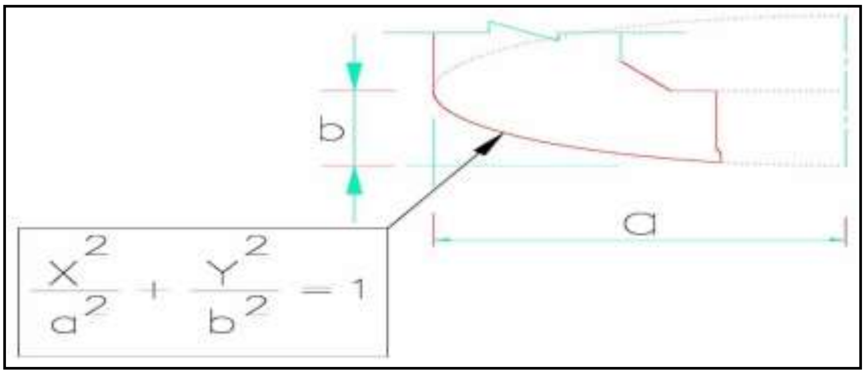

Figure 3: A typical Breast wall/sluice bottom profile

Methodology

Physical modelling

A lack of understanding of physical processes or complex boundary conditions in many fluid mechanics/ hydraulics problems which are not amenable to numerical or analytical techniques are investigated by physical models. Physical model studies are indispensable tools to optimize various components of reservoir and appurtenant structures. The hydraulic design of various components of a river valley project involves two types of problems viz. site specific problems and problems connected with complex hydraulic flow phenomena.

The site-specific problems are due to topography at the site, availability of foundation, nature of soil and rock strata etc. The problems associated with complex flow phenomena are many viz. non uniform flow in the approach portion creating vortices, rapidly varied flow because of complex geometry, high velocities due to high heads leading to cavitations damages, high turbulence causing hydrodynamic forces on the structure and erosion of the river bed and banks downstream, flow induced vibration for wide range of operating conditions.

Modelling Techniques

Following are important parameters which are considered while designing any model,

- Terms of reference for model studies
- Method of solution

a. Physical model
b. Combination of physical and math model
c. Desk study

- Number and types of models

a. Models with fixed/movable bed
b. Three/two dimensional models

- Scale of model
- Factors influencing scale of model

a. Space
b. Head
c. Discharge

Spillway models are built geometrically similar to prototypes. The force of gravitation causes flow of water in open channel and hence dynamic similitude is closely approximated according to the Froude's law. With the same fluid (water) in the prototype and model complete similarity of all forces is not possible resulting in scale effects. Following are the important phenomena for which scale effects are encountered in modelling of spillways: friction, air entrainment, turbulence, cavitation, fluid-structure interaction and local scour downstream of spillway.

In a Froudian model where viscous and surface tension forces are ignored, scale effects may influence the results if a very small model is used. This is because the effects of viscous and surface tension forces become increasingly important as the scale of a phenomenon reduces. Small Froudian models should be avoided to ensure that viscous and surface tension forces do not distort the Froudian similarity. For example, a model should not be so small in size that a flow, which is turbulent in the prototype, becomes laminar in the model. The Froude number and the Reynolds number each define unique relationships between the scale ratios Lr, Trand Ur. They cannot be simultaneously satisfied without manipulating fluid properties, which at best is a difficult proposition. In Froudian models, Reynolds number is always smaller than the prototype value.

It is established by many investigators that if a model is big enough to simulate large eddies (inertial eddies) so as to ensure turbulent flow conditions in the model, many hydraulic parameters are independent of Reynolds number if Re > $5x10^5$. It is believed that a model Reynolds number of at least $5x10^5$ and above will minimize the scale effects. This basically requires that the ratio of inertia to gravity forces be the same in model and prototype. It also may be viewed as a ratio of water velocity, U, to shallow-water wave velocity, $(gY)^{1/2}$, in a channel of depth Y. The Froude-number similarity criterion prescribes

$$Fr = \frac{Frp}{Frm} = \frac{Ur}{\sqrt{Yr}} = 1 \quad (5)$$

Note that, as most models are subject to the same gravitational field that prevails at full scale, g_r= 1. The Froude-number criterion sets the scale ratios, other than geometric scale.The resultant scales consequent to Froude number criterion (Eq.v) are summarized in the following table 1.

Table 1 Scale Relationship Based on Froude Number Similitude with ρ_r= 1

Variable	**Relationship**	**Scale**
Length	L=Length	$L_r= X_r=Y_r$
Slope	$S = \frac{\text{horizontal length}}{\text{Vertical length}}$	$S=\frac{lr}{lr}=1$
Velocity	$U=\frac{Lenth}{Time}$	$t_r = \frac{Lr}{Ur} = Lr½$
Acceleration	$t = \frac{Velosity}{Time}$	$a_r = \frac{Ur}{lr} = \frac{lr½}{lr½} = 1$
discharge	Q = Velocity X Area	$Q_r = U_r A_r = Lr½$
Force	F= Mass X Acceleration	$F_r = \rho_r Lr^3 1 = Lr^3$
Pressure	Pressure in meter of water head	$\rho_r = L_r$
Reynolds Number	$Re= \frac{Ul}{v}$	$(Re)_r=L_r^{1/2}L_r = L_r^{3/2}$
Manning's n	$n = \frac{R2/3XS1/2}{v}$	$n_r=L_r^{1/4}$

Data Requirements

For conducting model experiments, it is necessary to obtain correct information from the prototype. The entire operation of the model depends on the equality of the prototype data. The data would help in establishing the model prototype conformity pattern and to enhance the predictability of the model. Generally, the following prototype data would be required for planning, construction of spillway models and conducting model studies.

1) Maximum design outflow discharge for spillway and energy dissipator.
2) Gauge-discharge (Tail Water Rating) curve at about 200 to 300 m downstream of the spillway up to the maximum outflow discharge.
3) Index plan showing location of dam and course of river for about 1 km upstream and 1 km downstream, water spread, tributaries upstream and downstream of the site, important structures etc.

4) Cross sections of the river at about 50 m interval for a distance of 1000 m upstream and downstream of the dam axis. If the approach is curved immediately upstream, the cross section should extend at least 150 m beyond the curve.
5) A plan showing river course, dam complex, power intake, position of river cross sections and base line.
6) Layout plan: Dam layout plan showing the changes along the dam axis for the important structures such as left and right end of the spillway with reference to a baseline connected to the dam axis and location and orientation of the power intake.
7) Spillway section with details such as upstream and downstream crest profiles giving equations and radii of curves, tangent points, slopes and dimensioned details of energy dissipator. Cross sections of the non-overflow section of the dam.
8) Details of spillway gates and piers in plan and sections including distance of trunnion axis of radial gate with reference to dam axis/crest axis, gate seat elevation, geometric profile of breast wall and details of stop log groove.
9) Details of power intake including plan and sections of bell mouth entrance, transition, trash rack piers and rib beams, dimensions of gate grooves. Surface-tension effects start to become important if wear of order 100 or less. This occurs when the radius of surface curvature is small in comparison to liquid thickness or depth, for instance, for liquid drops, bubbles, capillary flow, ripple waves, and very shallow flows in small hydraulic models. The air water flow is a function of Weber number.

The scale of the model is chosen depending upon availability of space, discharge and head. Spillway models are scaled to provide flow depth over the crest of at least 75 mm for the design normal operating head to reduce the effect of viscosity and surface tension. In general, large models rather than small models should be built, as permitted by available space, operating head and water supply. Sometimes, cost and operational difficulties dictate the selection of model scale. The model scale for medium sized spillway would be around 1:50 to 1:60.

Construction Methodology of Model

After determining the scale ratio, construction of the model requires following considerations:

- Materials of construction
- Construction accuracy and other requirements
- River topography to be reproduced in the model including nearby structures

A model need not be made of the same materials as the prototype. If surfaces over which water flows are reproduced in shape and the roughness of the surfaces is approximately to scale (in fact smoother in the model than corresponding to prototype roughness), the model will usually be satisfactory. Generally, the riverbed is made up of smooth cement plaster; spillway, non-overflow section of the dam etc in masonry with neat plaster, spillway piers in teakwood, radial gates in sheet metal and outlets are fabricated in transparent Perspex.

Close tolerances, particularly in critical areas such as spillway crests, tangent points, energy dissipating appurtenances, model dimensions etc are essential. Greatest accuracy should be maintained where there will be rapid changes in direction of flow and very high velocities occur. The profiles of spillways and their allied structures are finished to their final shape with the help of metallic templates fixed in alignment and elevation.

Piezometers are generally welded to the templates so that their alignments are secured. The finishing of piezometers in models should be done carefully to prevent measurement errors that would result from improper installation. Complicated curves for bell mouths of sluice spillways, breast walls, bends and transitions can be made from Perspex which has been heated in oven and reshaped by pressing between the male and female concrete moulds.

Operation of Model

Once the model is ready for experimentation, the operating programme of the model should be carefully planned to evaluate the performance of the proposed design. The operating programme can be divided into two phases:

- Adjustment phase
- Experimental phase

The adjustment phase includes preliminary trials to identify model defects and inadequacies. The need for partial redesign, revision or shifting of measuring instrument is often indicated by these trial runs. The experimental phase includes regular model studies after removing all the defects observed during the adjustment phase. Generally, the approach flow conditions, water surface profiles, pressure profiles, Cavitations Index etc.

Approach Flow Condition

To observe if the flow approaching the spillway is generally uniform and if no, to find out its effect on functioning of spillway, dam and differential pressures on spillway piers etc.

Water Surface Profiles

Water surface profiles in downstream of spillway along the centre line, left training wall and right training wall are observed for various conditions. It is checked weather water profile touch the trunnion axis .If it touches the trunnion axis shaped of breast wall should be changed. Also behaviour of root jet is noted. If the rooster tail is over topping the divide walls and training walls in the bucket region they should be raised suitably to accommodate water profiles. If they are eroding banks of river it should be protected adequately to with stand against swaying of rooster jet..

Following are the steps involved in the Experiment,

1. First gauge zero (GZ) is measured.
2. Steady state condition is obtained for required discharge and gate condition.
3. Pointer gauge reading are noted for different chainages.
4. Water level (WL) is calculated from this observed pointer gauge reading (PGR) using following formula:

   WL= PGR xLr+ GZ

   where, Lr is Scale of model

Pressure Distribution

Pressure on crest profile and other appurtenant structures to ascertain that no dangerous sub-atmospheric pressures leading to cavitation damage exist. Pressure profile along the centre line of spillway, along of the pier and on the breast wall for the entire range of discharges for gated and ungated operation conditions are observed. Pressure profiles are needed while construction of spillway to know point of higher pressure where joint should not be provided. From the observed reading we can calculate cavitation index.

Cavitation Index

Cavitation is a process of passing from the liquid to vapour state by changing the local pressure while the temperature remains constant. The local pressure reduction associated with cavitation can be caused by separation of the flow from the boundary. However once the cavitation starts, the cavitation bubbles grow and travel with the flow to an area where the pressure field will cause them to collapse. When the cavitation bubbles collapse or implode close to or against a solid boundary, extremely high-pressure shock waves are generated with a pressure intensity of about 200 times the ambient pressure (Falvey, 1990), which causes damages (pitting). Cavitation damages on many spillway surfaces have been well documented by Falvey (1990) and many others. A parameter called as cavitation index (σ) can be used to define various occurrences of cavitation due to a critical combination of the flow velocity, flow pressure and vapour pressure of water.

$$\Sigma = \frac{PO - PV}{\frac{PV^2}{2}}$$

(5)

Where,

P_0= Reference Pressure

P_V = Vapour Pressure

V_0 = Reference Velocity

A criterion for assessing cavitation damage has been proposed by Falvey (1990) as follows,

C.I. should be more than 0.2 and constant if it is less than 0.2 major damages may occur to the structure. If C.I. is greater than 0.2 is considered that no damage will occur. If it is between 0.2 to 0.12 it is considered as minor damage and less than 0.12 is considered as major damage.

Design of Training Walls and Divide Walls

Water surface profiles along the training walls and divide walls to finalize their profiles.In order to avoid spreading of the jet, divide walls were provided along the rear slope. The provision of divide walls would enable to pass the normal flow through central sluices and keep ski jump trajectory away from banks.

Measurement Techniques

The discharges on the hydraulic models of spillway are measured on the standing wave flume or Rehbock weir using hook gauge of 0.1 mm least count in a stilling well. The accuracy of discharge measurement would be around ±2%. Water levels are measured using pointer gauges fitted with a vernier scale having a least count of 0.1 mm. Reservoir water surface elevations are measured at a location far enough to be free from drawdown and other effects. Tail water levels are measured by a hook gauge having a graduation of 0.1 mm mounted in a stilling well at a distance of about 4 to 5 m downstream of dam axis.

Tail water adjustments are made at the downstream end of the model using wooden strips of varying widths or adjustable tailgate. Piezometers (copper tubes) of 3 to 5 mm diameter are provided on the spillway surface along the center of the span for measurement of pressures. Pressures are measured by connecting rubber tubes

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to the piezometers and to open tube manometers with vertical water columns and could be directly converted to prototype pressure head in meters of water using scaled water manometer board placed by the side of the model.

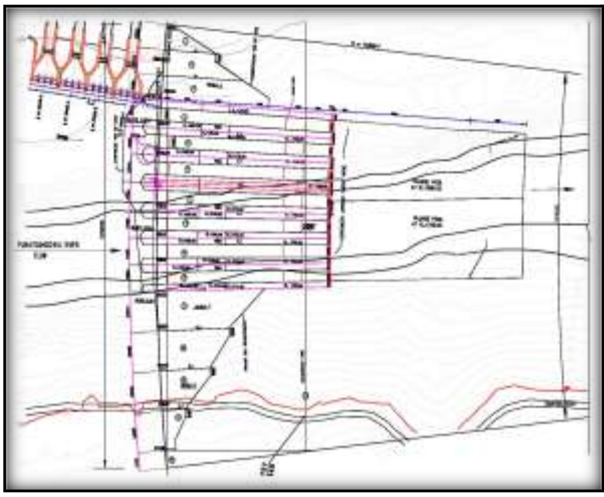
Experimental Studies:
Punatsangchhu (Stage I) Hydroelectric Project, Bhutan – Case Study I

Punatsangchhu-I H.E. Project is located on Punatsangchhu River in Wangdue Phodrang disctirct in Western Bhutan. The project envisages construction of a 136 m high concrete gravity diversion dam to generate 1200 MW of power utilizing a net head of 343 m at an underground power house. The main spillway is in the form of sluices to pass Probable Maximum Flood (PMF) of 11,500 m^3/s and Glacial Lake Outburst Flood (GLOF) of 4,300 m^3/s. The MWL / FRL are at El.1202 m and the MDDL is at El. 1195 m. The crest of the spillway is at El. 1166 m. Ski -jump bucket has been provided as energy dissipator. Fig. 4 sows the project layout plan.

Fig. 4: Project Layout Plan

- **Original Design of Spillway**

The original design of spillway consisted of sluice spillway of size 8 m x 13.2 m with 7 span having equation of crest profile as $x^2 = 300y$.
Expermental Studies has been carried out for assessing the coefficient of discharge which in turn revealed that the value of coefficient of discharge would be around 0.78 and not 0.89 as assumed in the design.
The equation of crest profile was changed from $x^2 = 300y$ to $x^2 = 200 y$. Fig. 5 & 6 show plan and cross section of spillway for the original design respectively.



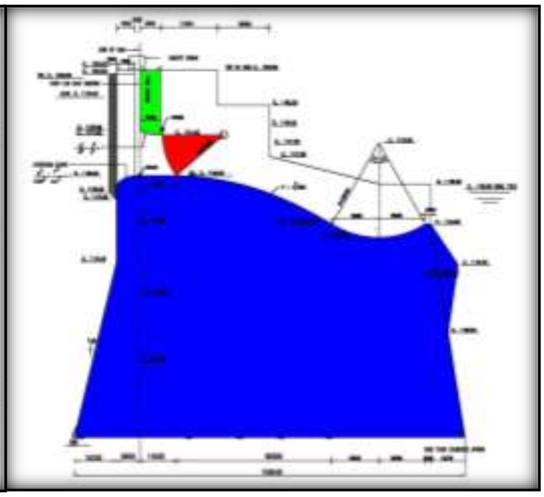


Fig. 5: Plan of Spillway **Fig 6: Cross Section of spillway**

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➢ **Performance of Ski Jump Bucket – Original Design**

Due to constriction at about 500 m downstream of dam axis, a pool formed at downstream which extended upto the spillway results in submerged ski action even for the discharge of 3,950 m^3/s (25%) and there was no formation of ski action for the gated operation of spillway. Tail water level observed at 300 m d/s of dam axis was found to be higher than the tail water rating curve supplied by project authorities. Hence modifications suggested in the design of bucket in the form of bucket radius as 45 m and increase in exit angle to 350. Consequently, the bucket invert and the lip were raised to El. 1148.862 m and 1157 m respectively. Photo 7 and 8 show the performance of ski jump bucket for discharges of 3,950 and 15,800 m^3/s.

Photo 7: Flow Conditions for 3,950 m^3/s

Photo 8: Flow Conditions for 15,800 m^3/s

➢ **Performance of Ski Jump Bucket – Modified Design of bucket and revised C/S**

The performance of the ski jump bucket was found to be unsatisfactory as boil formed downstream of the bucket and the ski jump was submerged for the entire range of discharges for the gated and ungated operation of spillway. Due to boosting of tail water levels attributed to a constriction in the valley at about Ch. 400 m to 500 m downstream of dam axis, dressing the banks and bed of river suggested and the further studies were carried out with the modified dressed river cross sections (up to rock-line) with the revised tail water rating curve.

➢ **Performance of Ski Jump Bucket with dressing of river cross sections upto rockline**

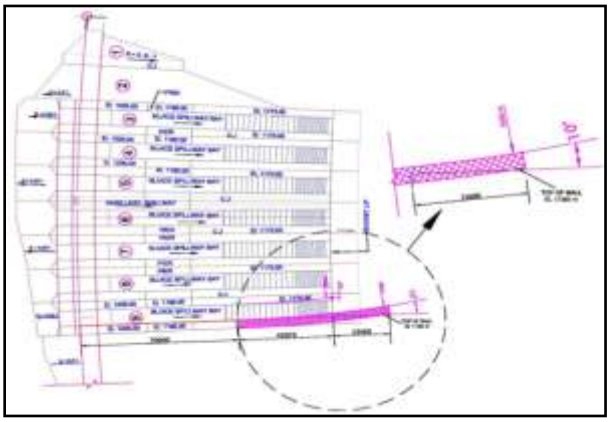
Performance of ski-jump bucket improved substantially with dressing of river overburden and consequently lowered tail water levels clear ski-jump action was forming for all the discharges for gated operation of spillway. However, the bucket lip was seen getting submerged due to high tail water levels for all conditions. With ungated operation of spillway, the ski-jump jet is not fully ventilated and submerged ski-jump action was noticed for the discharges corresponding to 6,900 m3/s (60% of PMF) and above. However, the performance of ski-jump bucket was not hampered due to submergence. Clear ski-action was observed for 2,875 m^3/s (25% of PMF) and below. Photo 7 and 8 show the performance of ski jump bucket for 15,800 m^3/s.

Photo 9 & 10: Flow Conditions for 15,800 m^3/s

- **Modified Design Of Spillway and Energy Dissipator – With Deflector Wall**
- **Performance of deflector wall**

To protect right bank downstream of spillway due to weak geology, model studies conducted with deflector wall replacing right training wall for various operating conditions to assess the performance of spillway. The studies indicated that the jet is deflected by about 8 to 9 m while passing the discharges mentioned above with ungated and gated operation respectively. The jet was seen mixing with the adjacent jet of span No. 6, without affecting the throw distance. Hence, the 10°deflector wall with radius 500 m and top elevation at El. 1180 m and 25 m long downstream of bucket lip (Fig. 11) is satisfactory, since it could deflect the jet by about 7 to 8 m to the left, at the point of impingement.

Photos 13 to 14 show the flow conditions with training wall and deflector wall for discharge of 15,800 m^3/s respectively. Fig. 5 shows performance of deflector wall for 15,800 m^3/s.



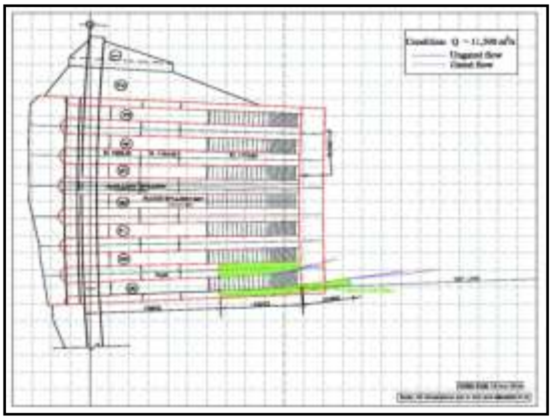


Fig. 11: Deflector wall details **Fig. 12: Performance for Q=11,500 m^3/s**

Photo 13: Flow Conditions with training wall **Photo 14: Flow Conditions with deflector wall**

- **Additional Hydraulic Model Studies for Revised Design - With Deflector Wall**

Due to right bank stability constraint and considering the flaring of jet from ski jump bucket, the option of removing one rightmost bay of spillway and to pass the design flood through remaining six bays by changing gate sizes from 8 m x 15 m to 8 m x 17.4 m was explored. Accordingly detailed hydraulic model studies were conducted for the revised design of spillway and energy dissipator in the existing model.

- **Discharging Capacity of spillway**

Discharging capacity of the spillway is considered to be adequate as maximum discharge of 15,368 m^3/s could be passed at FRL El. 1202 m. With one gate closed, the discharging capacity of

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12,933 m^3/s (12.4% above PMF) could be passed at FRL El. 1202 m.

➢ **Performance of Ski Jump Bucket and Deflector Wall**

For ungated operation of spillway, the ski-jump jet is marginally ventilated and slightly submerged ski-jump action was noticed at the lip for the discharges corresponding to 6,900 m^3/s (60% of PMF) and above. However, the performance of ski-jump bucket was not susceptible due to the submergence. For gated operation of spillway, ski-jump action was forming for all the discharges. However, the bucket lip was seen getting submerged slightly due to high tail water levels for higher discharges. Elevation of ski jump bucket lip of auxiliary spillway is found to be adequate. Photos 15 and 16 shows the flow conditions downstream of spillway for various discharges.

Studies were conducted for assessment of performance of deflector wall for 2,875 m^3/s (25% of PMF), 6,900 m^3/s (60% of PMF) and 11,500 m^3/s (PMF) for both ungated and gated operation of spillway. The studies indicated that the jet was mixing with the adjacent jet of Span No. 5 without affecting the throw distance and deflection of jet of Span No. 6 due provision of deflector wall is about 8 to 9 m (at the point of impingement) for various operating conditions as mentioned above.

Photo 15 & 16: Flow Conditions for 6900 m^3/s and 15,800 m^3/s

➢ **Flow Conditions Downstream of Spillway**

Flow conditions after jet impingement in the plunge pool are predominantly straight and forward for all the operating conditions. Elevation of ski jump bucket lip of auxiliary spillway is found to be adequate. For gated operation of spillway ski-jump action was forming for all the discharges. However, the bucket lip was seen getting submerged slightly due to high tail water levels for higher discharges. For ungated operation of spillway, the performance of ski-jump bucket was not susceptible due to submergence. Hence, the performance of ski jump bucket is satisfactory for entire range of discharges for both gated and ungated operation of spillway.

The performance of 100 deflector wall with curvature radius as 500 m and top elevation at El. 1180 m downstream of bucket lip was found to be satisfactory as it deflects the flow from rightmost span by 8 to 9 m towards left side. Due to recessed portion at the right of deflector wall, mild intensity return flows were observed. Forward velocities of the order of 3 to 7 m/s were observed downstream of plunge pool indicating satisfactory performance of ski jump bucket.

II. CONCLUSIONS

Performance of ski jump bucket was observed for entire range of discharges and reservoir water levels for ungated and gated operation of spillway.For ungated operation of spillway, the ski-jump jet is marginally ventilated and slightly submerged ski-jump action was noticed at the lip for the discharges corresponding to 6,900 m^3/s (60% of PMF) and above.

However, the performance of ski-jump bucket was not susceptible due to the submergence. Formation of hydraulic jump in the bucket and cascading of flow over bucket lip was observed for the discharges up to 4,800 m^3/s (RWL El. 1179.8 m) with ungated operation of spillway while gradually increasing the discharge. However, while decreasing the discharge gradually, ski-action was observed for the discharge up to 1000 m^3/s (RWL El. 1170.8 m) and further reduction in discharge resulted in hydraulic jump in the bucket. Clear ski action forms for span No. 1 to 2 and span No. 1 to 4 from left for the discharges corresponding to

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2,875 m^3/s (25% of PMF) and 6,900 m^3/s (60% of PMF) respectively for ungated operation of spillway. At the same time, hydraulic jump was observed for span 3 to 6 and span 5 to 6 for the discharges corresponding to 2,875 m^3/s (25% of PMF) and 6,900 m^3/s (60% of PMF) respectively for ungated operation of spillway. Intense turbulent flow conditions were observed during formation of cascading flow. It is suggested that an apron of 20 m length may be provided downstream of ski jump bucket anchored to the sound rock to avoid undermining of bucket for the cascading flows.

It was observed that for gated operation of spillway ski-jump action was forming for all the discharges. However, the bucket lip was seen getting submerged slightly due to high tail water levels for higher discharges. The maximum throw distance of the ski-jump jet (for 11,500 m^3/s) is about 70 m from bucket lip pushing the tail water downstream of dam axis. Thus, the performance of ski-jump bucket is found to be satisfactory for entire range of discharges.

For higher discharges and for gated operation of spillway, the ski jump of auxiliary spillway is able to push the tail water downstream without causing submergence of bucket upto discharge of 15,800 m^3/s (PMF+GLOF). Hence, elevation of ski jump bucket lip of auxiliary spillway is found to be adequate.

Deflector wall of 100 with radius 500 m, 25 m length from bucket lip and top elevation at El. 1180 m (same as adopted for original design of spillway with 7 spans) is provided along the right training wall downstream of bucket lip. Studies were conducted for assessment of performance of deflector wall for 2,875 m^3/s (25% of PMF), 6,900 m^3/s (60% of PMF) and 11,500 m^3/s (PMF) for both ungated and gated operation of spillway. The studies indicated that the jet was mixing with the adjacent jet of Span No. 5 without affecting the throw distance and deflection of jet of Span No. 6 due provision of deflector wall is about 8 to 9 m (at the point of impingement) for various operating conditions as mentioned above.

Further Studies – With Curved Stilling Basin as EDA

Considering protection of right bank at downstream of spillway, model studies with revised design of energy dissipator (Curved stilling basin) is in progress. Photo 13 to 16 show model work is in progress for curved stilling basin type of EDA.

Photo 17 to 20 : Curved stilling basin EDA (work in progress)

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