INNOVATIVE RESEARCH METHODOLOGY IN EVOLVING DESIGN OF SPILLWAYS FOR HIMALAYAN H. E. PROJECTS

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ABSTRACT

Design of hydraulic structures on mighty rivers flowing from Himalayas in India is one of the major challenges to the hydraulic engineers. Earlier, the spillways in Himalayan regions used to be designed using conventional methods i.e. high head storage dams. However, considering the experience of silting of dams and damages to power plants the trend is towards developing run-of-river schemes which utilize the stream flow as it comes, without any permanent storage being provided. The design of spillway in this area is challenging task to the engineer due to high sediment in river flows. Orifice spillway is the popular development in the construction of hydroelectric projects in mountainous regions due to their dual advantages of passing the flood and flushing of sediments from the reservoirs. Spillway profiles play an important role for attaining adequate performance in terms of various hydraulic aspects and making the structure economical and hydraulically efficient. Based on extensive studies on physical and numerical models, design of bottom and roof profile was optimized and equation for roof profile was developed considering various ranges of design heads and heights of orifice. This equation was initially tested to study the performance of spillway by varying the hydraulic parameters those remained same during development of its design. In order to validate the equation for global applicability, the case study for one of the real life orifice spillway projects is considered whose design was finalized by trial and error method using physical model studies. The suitability of equation was checked conducting numerical model studies using Computational Fluid Dynamics software. The performance of orifice spillway was assessed in terms of discharging capacity and pressure distribution on spillway surface for both the design i.e. specific design of the project and design of bottom and roof profiles proposed in the present study. The results obtained from physical and numerical models are presented in the paper. The proposed equation would be useful to the engineers in the preliminary design stage of an orifice spillway.

1. INTRODUCTION

Most of India's hydropower potential exists in the Himalayan and North Eastern regions. The perennial rivers with large hydraulic heads provide attractive possibilities for large scale power development. Due to the fragile geology of the hills and steep slope of the valley, the river carries a lot of sediment during monsoon. In case of hydropower plants, the main problems faced because of sediments are frequent chocking of strainers, damage of turbine blades and seals, sealing problems in hydro mechanical gates etc. Thus, removal of sediment in the vicinity of power intakes becomes essential to overcome the problems caused due to sediment (Khatsuria, 2004).

Traditional storage dams with overflow spillways do not function well as the reservoir gets filled up quickly and the spillway/small sluices are not able to remove this sediment. The orifice spillways in the form of breastwall/sluice have been widely recognized as the most appropriate spillways especially for run-of-river projects for handling both flood disposal and flushing of sediments from reservoir. Nevertheless, in spite of various advantages of orifice spillways, very few investigators have worked on orifice types of spillway (Bhajantri 2007, Bhosekar et al. (2012, 2014), Jothiprakash et al. (2015), Gadge et al. (2018 a). Over the past, the study of the spillway was mainly based on physical models (Mao et al. 2006, Hussain et al. 2014, Nguyen et al. 2015, WES, 1952). The recent development in computer software has advanced the use of computational fluid dynamics (CFD) in analyzing flow over the spillways (Jothiprakash et al. 2015, Nguyen and Wang 2015).

Gadge et al., (2018 b) carried out an extensive research especially on the design of orifice spillway using physical and numerical model studies. Based on the studies, an equation was developed for roof profile considering wide range of design heads and heights of orifice opening. The bottom profile was recommended in the form of $x^2 = 4h_d\, y$. In the present study, applicability of proposed roof design was initially checked by varying hydraulic parameters that kept constant while developing the equation. Orifice spillways are generally operated at gated condition. Hence, there may be possibility of occurrence of negative pressures on spillway bottom profile just downstream of gate for operation of small gate opening. In view of this, the design of roof and bottom profiles proposed in the paper was also tested mainly in terms of pressure distribution on spillway surface. The equation was also implemented on one case study of real life spillway project for which physical model results were available. Numerical model studies were carried out to assess the performance of spillway especially in respect of discharging capacity and pressure distribution along roof profile. The results obtained from the study are discussed in the subsequent sections.

2. HYDRAULIC DESIGN CRITERIA FOR SPILLWAYS CONSTRUCTED FOR HIMALAYAN PROJECTS

Orifice spillway in the form of breastwall is popular development in design of spillways especially in Himalayan region for passing the flood and removal of sediments. This type of spillway has advantage of lesser number of spillway spans, reduction in height of the spillway gates, the overall cost of gates and operating mechanism. A relatively smaller size of radial gate results in overall economy. An orifice spillway would allow the setting of its crest at significantly lower elevation, yet retaining the choice of a high dam for creating head for power generation. Greater depth of flow over the crest offers large margin for locating the power intake allowing large submergence for vortex free operation, at the same time keeping the intake as high above the river bed as possible to keep it free of sediments.

The hydraulics of this type of spillway changes with varying reservoir levels. The flow is free flow for reservoir water levels below the roof of the sluice, for higher water levels the flow is orifice flow. The range of design heads adopted on most of the project varies from 30-60 m. Due to large design discharges the orifice sizes varies between 8-20 m (w) x 12-22 m (d). This result in high velocities of the order of 20-30 m/s over the spillway crests corresponding to discharge intensity of the order of 200-340 $m^3/s/m$ (Bhosekar V. & Gadge P. 2018). The bottom profile of orifice spillway is flatter as compared to the overflow crest profile to avoid flow separation and negative pressures on the crest for small partial gate openings. Design of profiles plays a vital role in assessing the performance of spillway. These spillways have been adopted on most of the hydroelectric projects. Hence, systematic study is required to optimize its design with respect to various hydraulic parameters.

3. INNOVATION IN DESIGN OF ORIFICE SPILLWAY

The parameters such as design head or head over the crest (h_d), width (w) and height of orifice (d), design of bottom and roof profiles, height of spillway from upstream reservoir bed (P) are important to be considered while designing an orifice spillway. All the above parameters affect the performance especially in respect of discharging capacity and pressures on bottom and roof profiles. Figure 1 shows a typical cross section of orifice spillway.

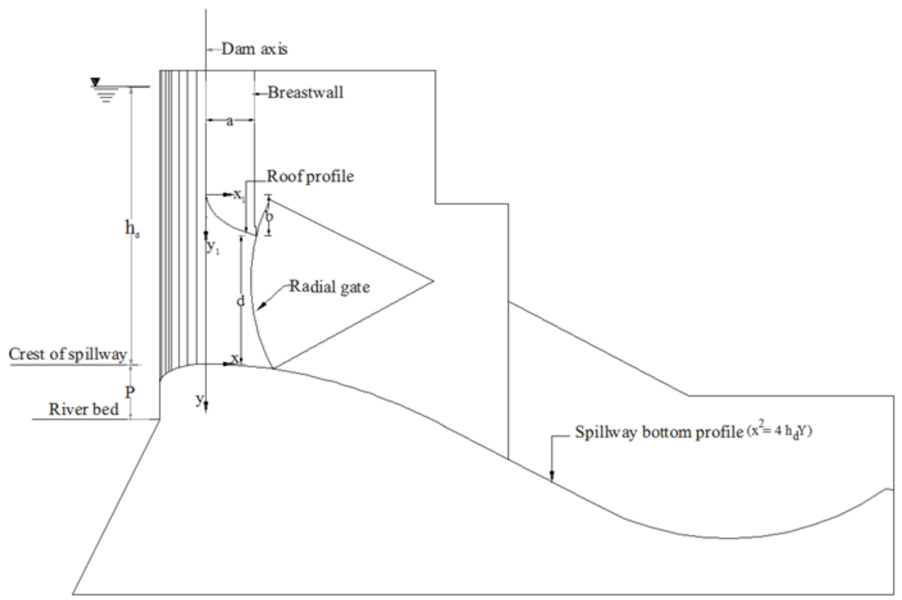


Figure 1 : Typical cross section of orifice spillway.

CWPRS has contributed for more than 25 projects in optimizing the design of orifice spillway by carrying out extensive studies on physical model. Each project is unique due to their site specific conditions. Studies revealed that design of spillway profiles is crucial in determining the efficacy of spillway. Especially, the design of roof profile was finalized by trial and error method with various alternatives. Gadge et al., (2018 b) carried out an extensive research on orifice spillway and developed the following equation for the design of roof profile using physical and numerical models. The numerical model developed for the study was validated by comparing data with physical model and the results were found in good agreement.

The following equation was developed considering all practical ranges of design heads i.e. 30 m to 70 m and heights of orifice ranging from 10 m to 20 m mostly adopted in most of the projects constructed worldwide.

$$x_1 = a\left(\frac{y_1}{b}\right)^{1.82} \tag{1}$$

$$a = 0.58 * (d) * \left(\frac{h_d}{d}\right)^{0.26} \tag{2}$$

Where 'x_1' and 'y_1' are the horizontal and vertical coordinates of the roof profile considering origin (0, 0) at the top of the orifice opening, 'h_d' is design head, 'd' is height of orifice opening, 'a' is length/thickness of roof profile and 'b' is height of curve. The equation is valid for height of roof profile designed for 0.4 times height of orifice opening. The height of crest of spillway (P) and width of spillway span was considered as 10 m and circular shaped pier nose of length 8 m was provided at the upstream of crest of spillway. The bottom profile was designed in the form of $x^2 = kh_d y$ (where k =4).

4. VERIFICATION OF PROPOSED EQUATION WITH VARYING HYDRAULIC PARAMETERS

The parameters that considered constant while deriving equations 1 & 2 may vary from project to project. In the present study, the proposed equation was verified with variation of 'P' and 'k' values using numerical model studies. The computational fluid dynamics (CFD) software FLUENT was used for numerical simulations. Studies were carried out for design head of 50 m and height of orifice 12 m. The results computed from the study are discussed below.

4.1 Performance of orifice spillway by varying height of crest of spillway (P)

Height of spillway crest was varied from 10 m to 20 m and 40 m. However, the other parameters such as width of spillway span, shape and size of pier nose were same as considered while developing equation. The bottom profile was designed in the form of $x^2 = kh_d y$ (where h_d = 50 m & k = 4) and roof profile was designed as per equation 1. The performance of spillway was assessed in terms of discharge passed through orifice and pressure distribution on the bottom and roof profiles.

The discharges through orifice were estimated 3199 m^3/s, 3182 m^3/s and 3164 m^3/s for P = 10 m, 20 m and 40 m respectively. There was a little change in discharge with variation of height of crest of spillway. The pressures were computed on the roof and bottom profile as shown in Figure 2a and 2b.

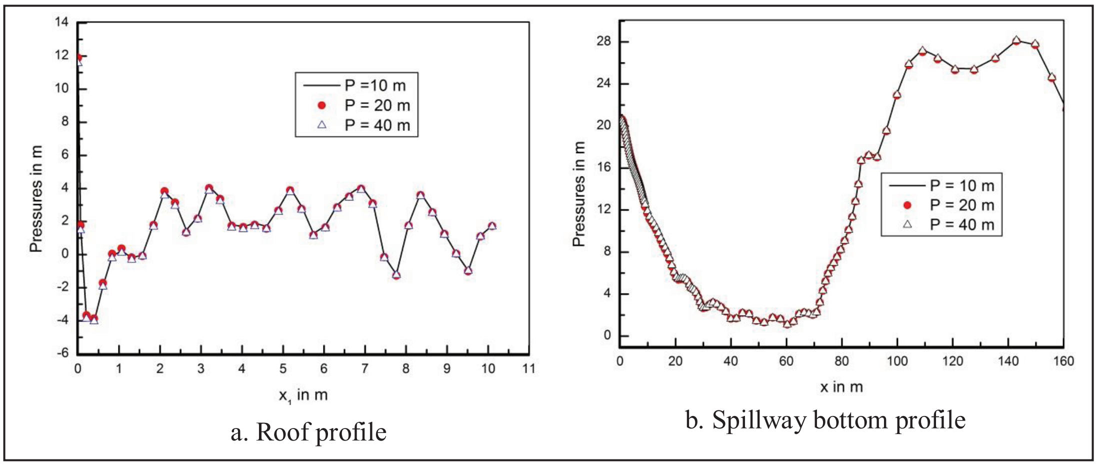


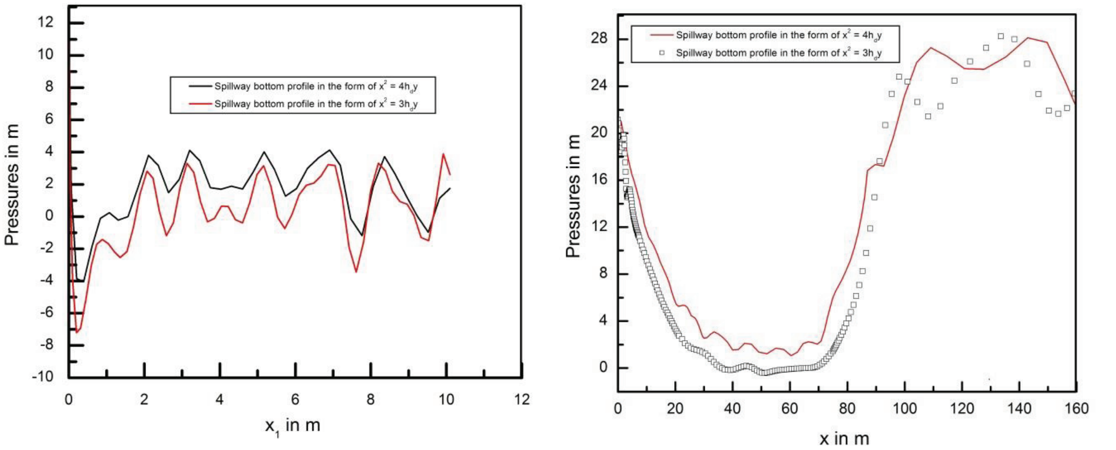
Figure 2 . Distribution of pressures on spillway profiles with variation in P values.

Figure 2a shows that there is sudden drop in pressure values in the initial region from 0 to 0.5. This may be due the abrupt change in the flow regime from subcritical to supercritical flow at the entrance of the orifice opening, signifying the presence of critical flow region generating transient flow instabilities. Afterwards, positive pressures were observed on the roof profile throughout its length. Also, acceptable pressure distribution was observed on spillway bottom profile as shown in Figure 2b. As the height of spillway changes, there is minor change in the pressure values. Hence, the proposed equations of roof and bottom profiles are found to be suitable for P values varies from 10 m to 40 m for satisfactory performance of orifice spillway for specific case.

4.2 Performance of orifice spillway by varying 'k' value in equation of spillway bottom profile

The spillway bottom profile is provided in the form of $x^2 = kh_d y$ (where $k = 3$, $h_d = 50$ m). The k value changed from 4 to 3 that change the slope of the profile. Other parameters such as width of spillway span, height of spillway and shape and size of pier were kept same as those considered while developing the equation. However, the roof profile was designed as per equations 1 & 2.

The studies indicated that the discharge through orifice was increased from 3199 m^3/s to 3323 m^3/s. Discharge was increased by about 3.8% due to the steep bottom profile that resulted in decrease in pressures on the bottom and roof profile as shown in Figure 3a and 3b.



a. Roof profile b. Spillway bottom profile

Figure 3 : Distribution of pressures on spillway profiles with variation in k value.

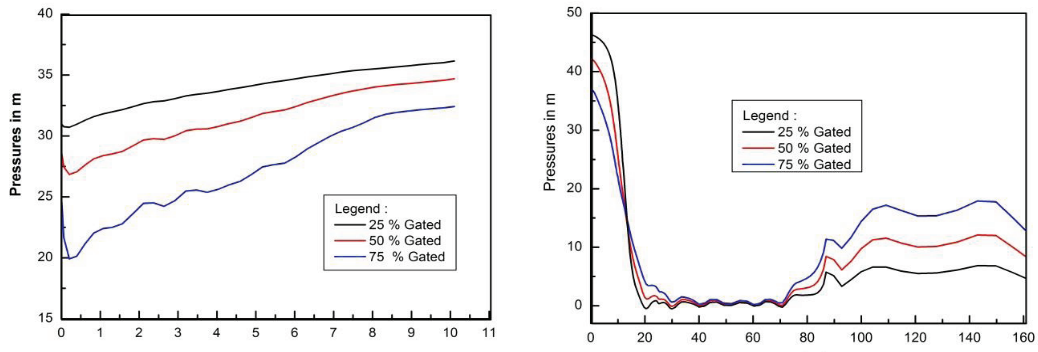
It can be seen from Figure 3a that there is a variation in pressures on roof profile with change in k value in equation of spillway bottom profile. Negative pressure of magnitude 4 and 7 were observed at the initial region for bottom profile designed with $k = 4$ and 3 respectively. Beyond this point, there are positive pressures on both the roof profiles throughout its length. The cavitation indices corresponding to pressures were found to be 0.3 and 0.14 respectively. Hence, there may the possibility of cavitation damage up to the distance of about 1m from crest of spillway for spillway bottom profile designed with equation $x^2 = 3h_d y$ as cavitation index value is less than critical cavitation index (Falvey, 1990). Figure 3b shows that due to steeper profile with $k = 3$, the velocity over the spillway surface increases that results in decrease in pressure values. However, the pressures were positive on both the spillway profiles.

Hence, it is stated that the proposed equation of roof profile with spillway bottom profile in the form of an equation $x^2 = 4h_d y$ is more suitable as profile becomes flat and possibility of occurrence of negative pressure for low gate opening may be minimized.

4.3 Performance of orifice spillway for gated operation

These spillways are generally operated at gated conditions to maintain high reservoir water level. Hence, numerical model studies were also carried out to simulate the flow through orifice at gated operation of spillway. Gate openings were considered as 25%, 50% and 75% of the full height of orifice opening. The corresponding opening sizes are 3 m, 6 m and 9 m respectively. Results were analysed in the form of pressures distribution on roof and bottom profiles as shown in Figures 4a and 4b respectively.

In gated condition, flow is pressurized below roof profile that results in positive pressures on the spillway bottom and roof profile. Hence, the design of roof profile is found to be safe in terms of cavitation damage for gated operation of spillway. When the flow passes through gate opening it suddenly changes from pressurized to free surface flow. The velocity of flow goes on decreasing with increase in gate opening for a particular head over the crest. This results in increase in pressures on the spillway bottom profile as shown in Figure 4b. Negative pressures were observed at few locations just downstream of gate lip for lower gate opening of 3 m (25% gate opening). However, corresponding cavitation index works out to be 0.2 which is equal to critical cavitation index. Hence, design of bottom profile can be considered as safe for small gate operation for this specific case.



a. Roof profile

b. Spillway bottom profile

Figure 4 : Distribution of pressures on spillway profiles for gated operation.

5. IMPLEMENTATION OF PROPOSED EQUATION ON REAL LIFE ORIFICE SPILLWAY PROJECT

Orifice spillways have been adopted on most of the hydroelectric projects in Himalayan region due to various advantages such as passing the flood, flushing of the sediments from reservoir, and keeping the power intake as high above the river bed as possible to keep it free of sediments. The design of each project is site specific and become complex due to vide variation of hydraulic and structural parameters. Physical model study is an indispensable tool to tackle this complex flow phenomenon and optimize the design of spillway. Literature reveals that the design of roof profile is crucial as it affects the discharging capacity of spillway. In most of the projects, the design was finalized by conducting physical model studies. The results obtained from physical model for one of the real life orifice spillway project have been presented in present paper. The studies were conducted for specific design of the project.

Also, the performance of spillway was studied by implementing the proposed equation of roof profile. Numerical model studies were carried out to check its suitability with respect to design head and height of orifice opening for specific project.

5.1 Project description

Punatsangchhu-I H. E. Project is located on Punatsangchhu River 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 original design of spillway consist of 7 sluices of dimensions 8 m (W) x 15 m (H) to pass probable maximum flood 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. Figure 5 shows cross section of spillway.

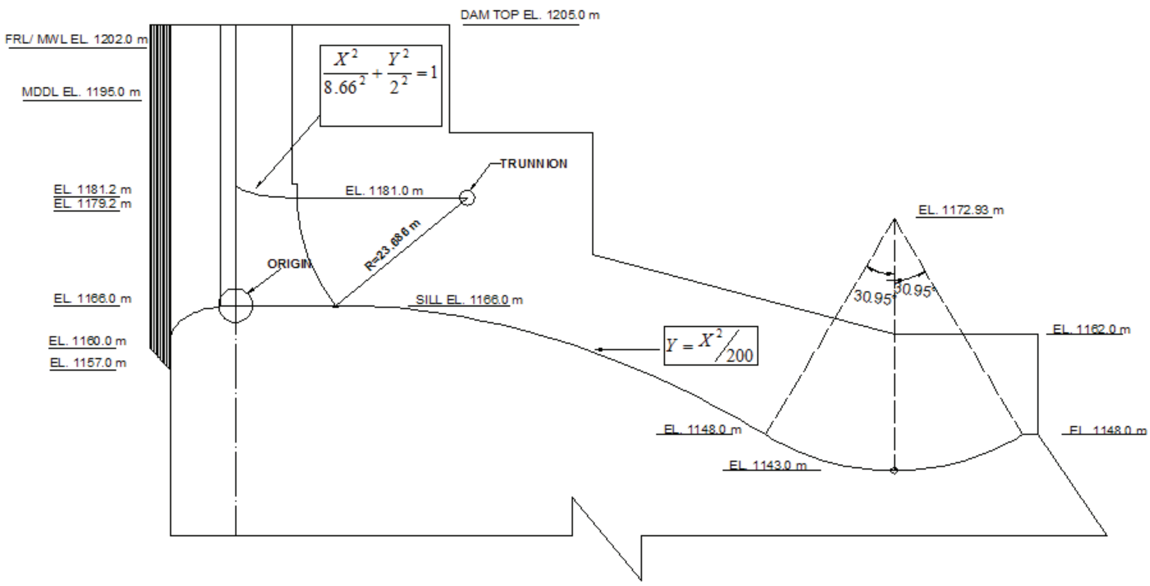


Figure 5 : Cross section of spillway

5.2 Results obtained by conducting the studies on 2-D sectional model

The physical model was constructed in a 1m wide and 10mlong flume at Central Water and Power Research Station (CWPRS), Pune, India. A 2-D sectional model was constructed to a geometrically similar scale of 1:50 with transparent Perspex sheets in a glass sided flume. One full span and two full piers with ski-jump bucket were incorporated in the model. The accepted equations for similitude, based on Froude number criteria were used to express the mathematical relationship between the dimensions and hydraulic parameters of the model and the prototype.

In the present problem, the length of breastwall 8.66 m and height of curve is in the ratio of 0.14. The bottom profile is in the form of $x^2 = 200y$ which is flatter as compared to $x^2 = 4h_d y$ (where h_d = 36 m). The roof profile/breastwall bottom profile is in the form of ellipse conforming to equation $\frac{x^2}{8.66^2} + \frac{y^2}{4.6^2} = 1$. Hydraulic model studies were carried out to determine discharging capacity of spillway for corresponding bottom and roof profiles for spillway operation at FRL condition i.e. 36 m above the spillway crest. The studies indicated that the discharge of 13,426 m³/s could be passed at FRL El. 1202 m with all 7 spans operating fully open. This is 15% less as compared to the design discharge of 15,800 m³/s (PMF+GLOF). Deficiency in discharging capacity was due the separation of flow in the vicinity of breastwall profile as shown in Figure 6.

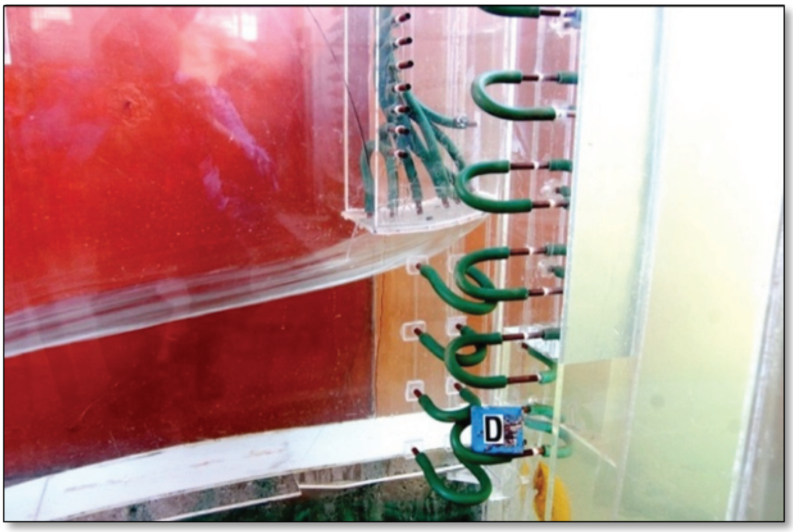


Figure 6 : Flow conditions in the vicinity of breastwall

The water surface was not following the breastwall bottom profile. It leaves the profile at upstream edge and thickness of jet at downstream edge is only about 13 m. Thus, the whole opening of orifice was not effective resulting in reduced capacity to pass the flow. Hence, design of breastwall/roof profile was modified and various alternative designs were tested on model as shown in Figure 7. The flow conditions in the vicinity of breastwall profile for original design are shown in Figure 6, whereas, Figure 8 shows the flow conditions for alternative 1 and 2 designs.

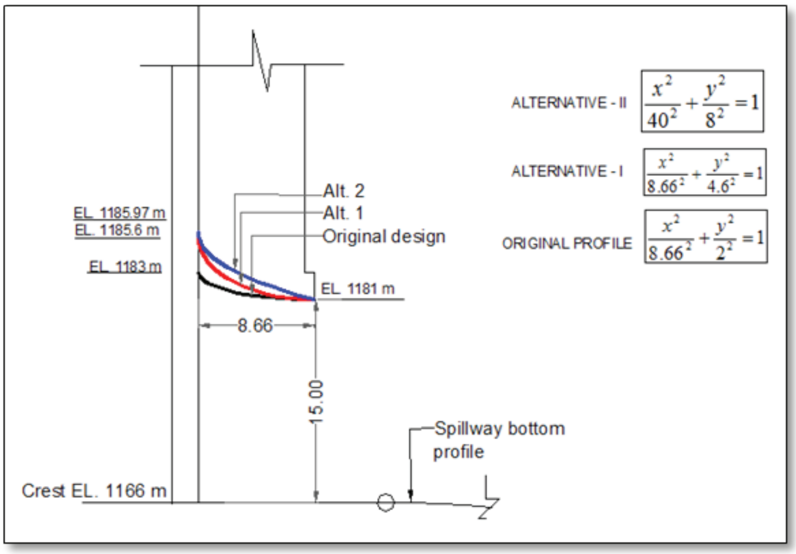
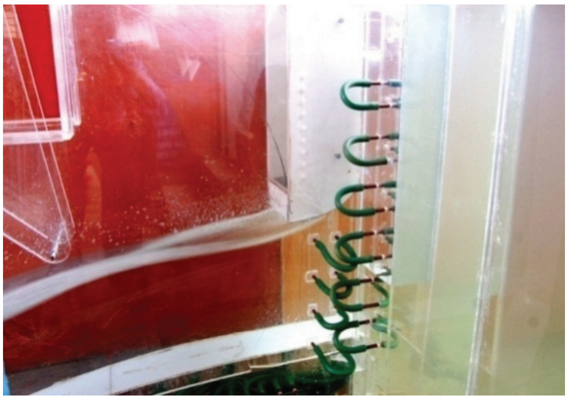


Figure 7 : Alternative designs of breastwall bottom profiles

Alternative 1



Alternative 2

Figure 8 : Flow conditions in the vicinity of breastwall bottom profile for various alternative designs.

In Alternative 1 design of breastwall, the spillway could able to pass the discharge of 15,169 m^3/s which is 4% less than the design discharge and the water surface follows only half upstream length of breastwall bottom profile. However, the spillway could able to pass the discharge of 15,554 m^3/s which is about 1.5 % less than the design discharge and the water surface follows the full length of breastwall profile for Alternative 2.The coefficient of discharge was increased from 0.68 to 0.78 by modifying the design of roof profile.

5.2.1 *Performance of orifice spillway with proposed equation of roof profile using numerical model studies*

For the present study, design of bottom profile, shape and size of pier, width of span was remained same as studied in physical model. However, roof profile was designed as per the proposed equation 1. Numerical model studies were carried out for spillway operating at FRL El. 1202 m same as studied in physical model. Initially, the validity of proposed roof profile was verified by comparing the design with original profile and profile finalized from physical model (Alternative 2) as shown in Figure 9. While plotting, the bottom coordinate of roof profile is kept constant to maintain height of orifice (d) same.

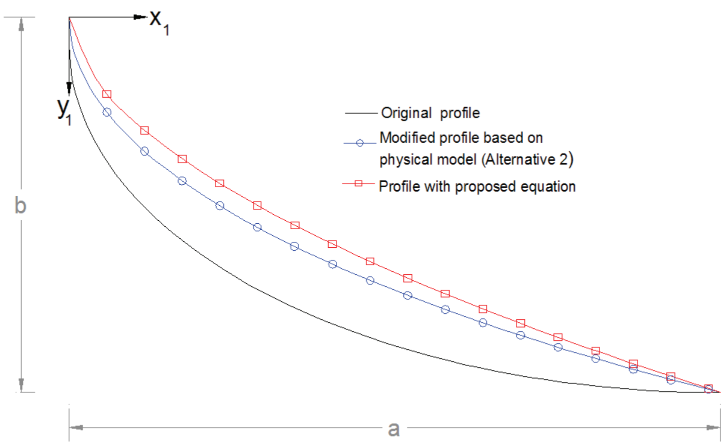


Figure 9 : Comparison of designs of roof profiles.

It is seen from Figure 9 that the roof profile designed with the proposed equation is far away from original profile and closer to the profile modified by number of trials taken on physical model. Studies indicated that the discharge of about 14,973 m^3/s could be passed through the orifice at FRL condition. The C_d value calculated was less by about 3.7% as compared to the C_d calculated for Alternative 2 design.

Literature reveals that the design of roof profile should be in such a way that it should have maximum discharging capacity and acceptable pressure distribution on the surface. Hence, the pressures were measured along the roof profile as shown in Figure 10. It was observed that pressures were negative with the same magnitude in the initial region for both the designs. Pressures were negative up to 4 m from crest of spillway on the profile finalized from physical model i.e. Alternative 2. However, pressures were positive on the roof profile designed using proposed Equation after a distance of 1m. The pressures were improved than observed on physical model for specific design of the project.

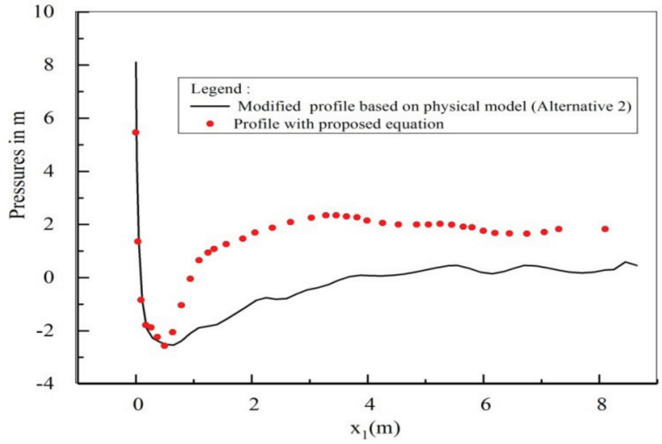


Figure 10 : Comparison of pressures on roof profiles with different designs.

5.2.2 *Performance of orifice spillway with proposed equation of roof and bottom profile*

Second attempt was made to assess the performance of orifice spillway with spillway bottom profile in the form of $x^2 = 144\,y$ ($x^2 = 4h_d y$). The other components such as upstream profile, width of span and shape of pier were kept same as per specific case study. However, the roof profile was designed as per the proposed equation 1.

The studies indicated that the discharge of about 17281 m^3/s could be passed through the orifice at FRL condition. The coefficient of discharge was increased from 0.78 to 0.87 by about 12%. This study proved the significance of design of bottom profile in improving the discharging capacity of spillway. Due to large increase in C_d value, there may be possibility of occurrence of negative pressures on spillway roof profile. Hence, the pressures on roof profile were also checked. The results are plotted as shown in Figure 11.

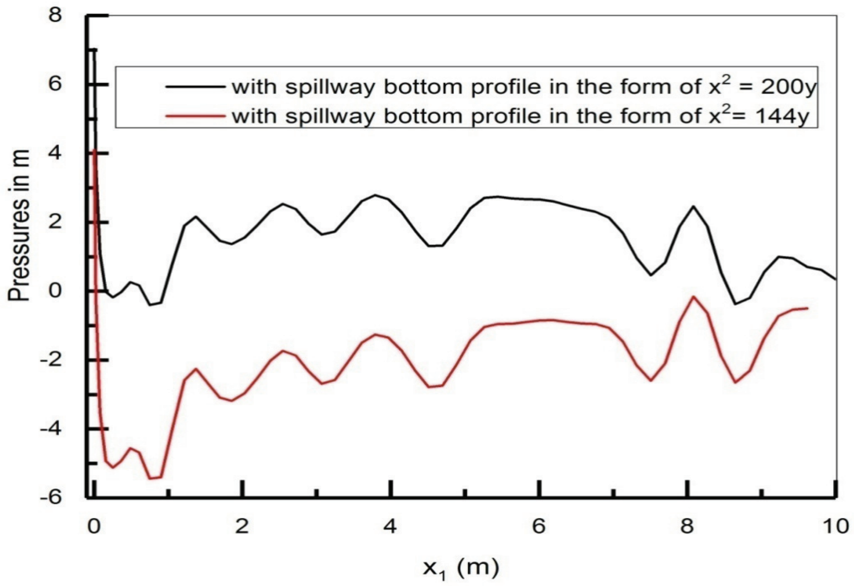


Figure 11 : Comparison of pressures on roof profiles with different designs of spillway bottom profiles.

It was observed that there is a large difference in pressures on the roof profile with change in design of spillway bottom profile. The pressures were found to be positive throughout the length of profile with spillway bottom designed for $x^2 = 200\,y$. However, the pressures on roof profile were decreased throughout its length for spillway bottom profile in the form of $x^2 = 144\,y$. Maximum negative pressure of 5 m was observed at a distance of about 1 m. However, the corresponding cavitation index works out to be 0.31 which is more than critical cavitation index of 0.2 (Falvey, 1990). Hence the proposed design of roof profile and bottom profile in the form of $x^2 = 4h_d y$ is found to be safe for this particular case study. However, it is needed to verify these equations for different case studies of orifice spillway projects to identify the limitations of proposed equations.

6. CONCLUSION

Orifice spillway is the popular choice for hydroelectric projects in Himalayan region due to its dual function of passing the flood and removal of sediments from the reservoir. The design of such spillways becomes challenging to the engineers due to wide range of hydraulic parameters, complex flow phenomena and complicated site conditions. Based on extensive experiments and simulations, equation was developed for design of roof profile considering wide range of design heads and heights of orifice. The bottom profile was recommended in the form of $x^2 = 4h_d y$. The proposed equation was initially verified by varying hydraulic parameters which remained same while developing the equation. Also, applicability of the proposed design was checked with one case study for real life orifice spillway project. The performance of orifice spillway with proposed design of bottom and roof profiles is found to be satisfactory in respect of discharging capacity and pressure distribution along spillway surface.

The design of roof profile of most of the orifice spillway projects in India has been finalized so far based on the trial and error method carried out on physical model studies. The proposed equation is a step forward in this regard and would be useful to the engineers at the initial stage of design. As the design of each project is unique and site specific, the performance of spillway with developed equations should be checked by conducting hydraulic model studies.

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