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OPTIMIZING THE DESIGN AND LAYOUT OF SPILLWAYS AND ENERGY DISSIPATORS FOR CASCADE HYDROPOWER PROJECTS IN RAVI BASIN, INDIA

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ABSTRACT

A large hydropower potential lies in the Himalaya. There is an advantage to develop the hydropower projects in cascade by utilizing the head and perennial discharge available in the Himalayan rivers. Flood and Sediment management are the governing criteria for the design and operation of these projects. Since the design and operation of the projects are unique and posing challenges like narrow valleys, high sediment load, flash floods, weak geology etc., hydraulic models can be effectively used for simulating the flow conditions and optimizing the project design and layout. Chamera Hydro-Electric Project, Stage-I, II and III with gross installed capacity of 1071 MW across River Ravi in India were designed as a cascade hydroelectric projects. Extensive model studies were carried out on geometrically similar 3-D comprehensive models to optimize the layout and design of the major components of the dam viz. Spillway, Energy dissipator and power intake. The findings obtained from the studies have been adopted in finalizing the design and layout of project components and flushing operation for sustaining the reservoir capacity. All the three projects are in operation and performing efficiently.

1. INTRODUCTION

Himalayan rivers provide some of the greatest challenges as well as opportunities in hydro power development. One of the major advantages of hydro projects out of a numerous others is the capability to provide the peaking power. To ensure the same, some diurnal storage (live capacity) requires to be provided in the reservoir. Sediment spells doom for this diurnal storage as it occupies the precious little space meant for peaking purposes. Mountainous terrain, narrow gorges, steep gradients, young geology, high level of seismicity and excessive silt load are a few factors which influence the river systems of the Himalayan region. Floods in catchment of rivers originating from the Himalayas result from rainfall, snow-melt, glacier-melt, flash floods due to cloud burst etc. Harnessing these rivers to generate the much needed electricity by constructing hydraulic structures in such inclement conditions is a real challenge to the engineers. The sediment in the river increases due to young and fragile geology of the mountains, glacier silt in snow-melt, and steep river slopes. Thus, the storage space provided in the storage dams is lost, in the process making such reservoirs a non-viable option. Another major concern is the siltation in front of the power intakes and the consequent damage to the equipments or the generating units of the hydropower plants. In one of the hydropower project the reservoir behind 100 m high dam silted up in four years and the power plant also sustained damages due to sediments. Projects such as Salal, Baira Siul, Chilla are facing regular damage due to silt. In view of this, attention is focused on developing run-of-the-river scheme wherever possible along with suitable sediment disposal arrangement. In order to ensure, efficient flood disposal along with flushing of sediments from the reservoir, innovations in spillway design like provision of orifice spillways with crest very near to the river bed is envisaged. The energy dissipators are designed giving special consideration to sediment flushed through the spillway (CWPRS, 2008). Although no design procedures are readily available for designing the energy dissipators for orifice spillways, experience from the hydraulic model studies and prototype provides useful guidelines. Physical modelling is widely used to investigate the design and operational issues in hydraulic engineering. An advantage of physical models is its potential capacity to replicate many features of a complicated flow situation. The present paper describes the case studies for Chamera H.E. Projects, Stage – I, II and III cascade hydropower projects on river Ravi, Himachal Pradesh for which the hydraulic model studies were carried out on 3-D comprehensive and 2-D sectional models for evolving the optimum design of spillway, energy dissipator and appurtenant structures.

2. HYDRAULIC MODEL STUDIES FOR CHAMERA-I H. E. PROJECT

The Chamera-I H.E. Project, the lower-most project in cascade, envisages construction of a 125 m high concrete gravity dam across river Ravi in Chamba district of Himachal Pradesh. The original design of breast wall spillway has9 spans of size 10 m x 12.5 m, separated by divide walls. The maximum head over the crest would be 32.5 m. The spillway has been designed to pass a maximum discharge of 22,000 m^3/s at design MWL El. 762.50 m. The energy dissipation arrangement consists of a flip bucket with 15^0 lip angle and a pre-excavated plunge pool. This layout as governed by topographical and geological conditions resulted inthe dam axis inclined by about 65° to the general direction of the river. Figure 1 shows the original layout of spillway and plunge pool. Figure 2 shows typical cross sections of the spillway, breast wall and plungepool.

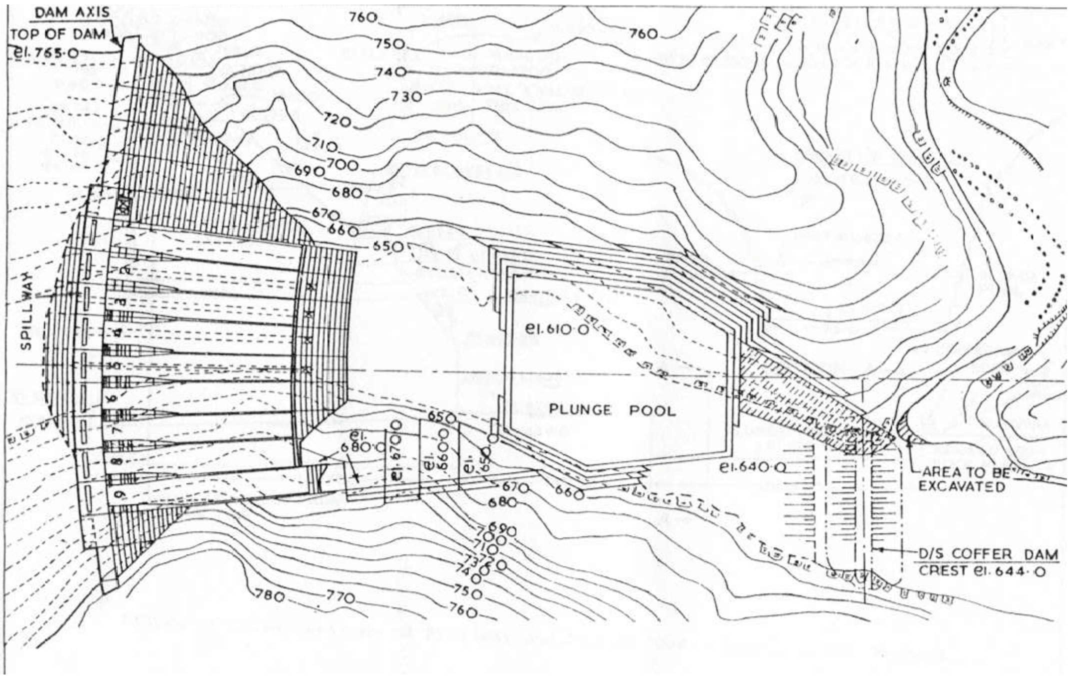


Figure 1 : General layout plan of the spillway – original design

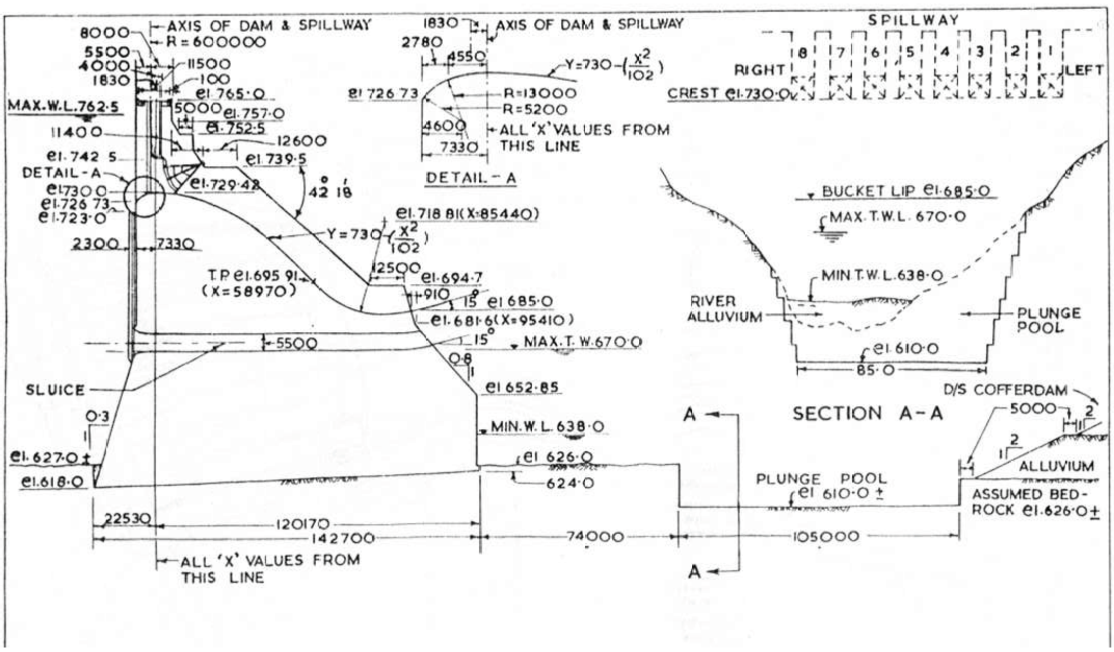


Figure 2 : Cross sections of the spillway and plunge pool – original design (Khatsuria et. al., 1991)

The layout of the spillway and plunge pool was finalized after extensive hydraulic model studies on two models viz. 1:100 scale 3-D composite model and 1:60 scale 2-D sectional model.

2.1 The modelstudies

For studying the general flow conditions on upstream and downstream of spillway and efficacy of the layout, particularly that of the plunge pool, a 1:100 scale composite model was constructed incorporating the entire spillway and river reaches 1500 m upstream and 2000 m downstream of the dam axis. The river course and the plunge pool downstream of the spillway were reproduced in rigid form as well as semi-erodible using sand-bentonite mixture. The studies for the crest profile, discharging capacity and pressure distribution were conducted on the 1:60 scale sectional model incorporating one typical span along with breast wall, radial gate, divide wall etc. This model was fabricated in transparent perspex.

2.1.1 *Studies for the original design of the spillway*

The studies were first conducted on 1:100 scale composite model for the original layout of spillway. The studies for a discharge of 18,600 m^3/s corresponding to 8 spans freely overflowing at full reservoir level (FRL) El. 760.0 m indicated that there would be a severe attack of high velocity flow (10 to 12 m/s) on the left bank. The flow appeared to bifurcate downstream of the plunge pool and resulted in the formation of return currents of the order of 8 m/s along the right bank. Studies with the semi-rigid reproduction of the plunge pool indicated that with the erosion of the banks, the intensity of return flow would increase. The flow issuing from the right extreme span was falling near the right excavated face of the plunge pool which together with the return current there, accelerated the erosion of the right bank. Under this condition the maximum depth of scour in the plunge pool reached around El. 585.0 m. The above flow conditions were considered to be unsatisfactory and not acceptable.

It was apparent that satisfactory flow conditions could be ensured only if the flow issuing from the flip bucket was generally in line with the alignment of the river. It was also felt that the spillway should have only 8 spans centrally located so that the river banks on both the sides of the spillway remain free from attack by the flow from the extreme spans.

The alternative layout was developed further from the qualitative studies on the composite model. The right extreme span No.9 was treated as abandoned. An approximate representation of curved chute deflecting towards right, varying from 7^0 to 10^0 was represented by incorporating skew divide walls in continuation of spillway piers. The divide walls with 10° deflection were found to perform reasonably well although some piling of flow along the pier occurred. It was considered that more gradual curvature of the bend in combination with super elevation of the spillway surface would help to reduce the piling of water along the well. It was also seen that if alternate spans were provided with the flip bucket having lip angle of 35°, instead of 15°, more equitable distribution of discharge would result in the plunge pool area. These studies also enabled optimum positioning of 8 spillway spans along the dam alignment without any shift of the dam axis. On the basis of the above observations an alternative layout comprising with curved divide walls was prepared. This layout designated as alternative layout-I was first studied on the sectional model to assess the discharging capacity and pressures along the curved chute and divide walls.

2.1.2 *Studies on Alternative-I*

The 1:60 scale sectional model was constructed incorporating one full span and two divide walls along with breast wall, radial gate etc. fabricated fully in perspex. Arrangements were made for measurement of pressures, water profiles anddischarges.Studies were first conducted to assess the discharging capacity of 8 spans to see whether by varying parameters such as orifice opening, breast wall profile and maximum reservoir level, the required discharging capacity could be obtained with 8 spans. Figure 3 gives the results of these studies for three different orifice openings of 10 m x 12.5 m, 10 mx 13.0 m and 10 m x 13.5 m and for reservoir water levels up to top of the dam El. 765.0 m. The bottom profile of the breast well as proposed originally was also modified to give higher co-efficient of discharge. These profiles and the corresponding variation of the coefficients of discharges are also shown in Figure 3. On the basis of these studies it was determined that with the increase in MWL from El. 762.50 m to El. 765.0 m, an orifice opening of 10.0 m x 12.8 m and the modified profile of the breast wall, the required discharge capacity with 6 spillway spans could be ensured. Hence, further studies were conducted with the above configurations.

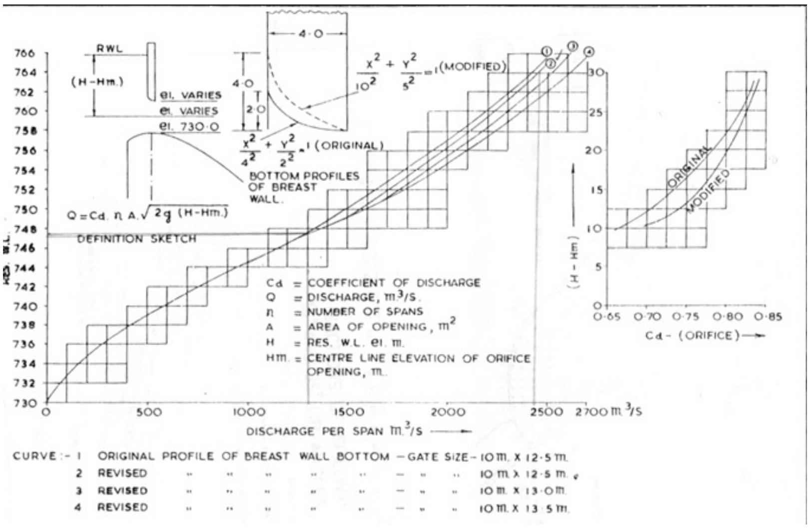


Figure 3 : Discharging capacity of spillway for different sizes of spillway spans

The hydraulic performance of the curved chute along with the divide walls was then studied for a typical spillway span for different reservoir levels and gate openings.Piezometric pressures were obtained on the spillway profile as well as on the divide walls along the entire length, on 22 cross sections. Each cross section contained 8 piezometers, 3 each on the divide walls and 2 on the spillway profile.It was observed that while the pressures on the crest profile were only slightly negative (up to -1.5 m of water), those on the left side of the divide wall in the portion just downstream of the pier, in the region of change of curvature, were highly negative. They varied from -2.0 m to -4.0 m of water. The pressures on the bucket region were of the order of 33.0 m of water. The maximum difference in the depths of flow on the two sides of the divide walls was about 9 m of water. The excessive negative pressure could be attributed to somewhat sharper transition of the deflection and could be controlled by providing larger radius and subtended angle. The high pressures in the bucket region necessitated increase in the thickness of the divide wall in that portion, from 2 m to 3 m, from structural design consideration. Thus, an alternative layout II of the curved spillway chute was worked out as shown in Figure 4. In this layout the spillway profile was kept the same, whereas the alignment of the divide walls was modified by providing large radii transitions. This was designated as alternative layout II.

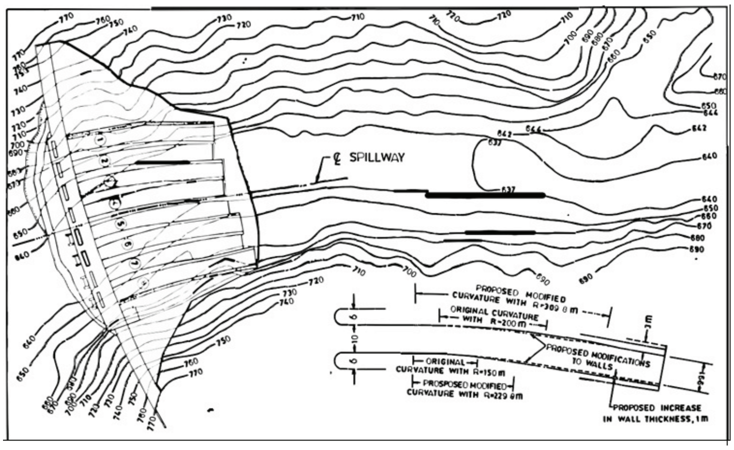


Figure 4 : Alternative layout of spillway and curved divide wall (Khatsuria et.al., 1991)

2.1.3 *Studies on Alternative layout – II*

Studies were done for the pressure distribution on the spillway profile and divide walls for various conditions of operation of the spillway. The modified transitions improved pressure distribution and brought down the negative pressures to -1.5 m of water, considered permissible as the cavitation index is above 0.2. The pressures in the bucket region remained almost the same as with alternative I layout. The maximum difference in the depths of flow on the two sides of the divide wall was 4 m of water. The alternative layout II evolved after conducting extensive model studies ensures satisfactory hydraulic conditions in the spillway chute as well as in the plunge pool. The reduction in the cost due to elimination of one spillway span is considerably more than the extra cost due to curved divide walls. Also, the modifications proposed would not adversely affect the time schedule of the project.

3. HYDRAULIC MODEL STUDIES FOR CHAMERA H. E. PROJECT – II

The Chamera H.E. Project, Stage- II is a diversion dam on river Ravi. The dam is located 30 km upstream of the existing Chamera H.E. Project, Stage- I. It is a run-of-the-river scheme to generate 300 MW of power utilizing a gross head of 267 m. The project comprises of a 43 m high concrete gravity dam and spillway, power intakes, inlet tunnels, 2 Nos. of desilting chambers, 7.83 km long head race tunnel, 3 Nos. of pressures shafts, 104.5 m high, 70.2 m dia restricted orifice type surge shaft and 3.48 km long tail race tunnel.The spillway is a 78 m long overflow structure divided into four bays using 6 m wide piers. They are equipped with 21.5 m high radial gates. The spillway is designed to pass a standard project flood of 9,000 m^3/s. The FRL is at El. 1162 m, the MWL at El. 1164.85 m and MDDL is at El. 1152 m. In the original design of the spillway, the crest was at El. 1141 m with the crest profile conforming to equation $x^2 = 100y$ followed by a hydraulic jump stilling basin at El. 1136 m. The crest of the spillway is barely 3 m above the river bed which is at an average level of El. 1138 m. This is to enable flushing of the sediments deposited in the reservoir. The power intake was aligned at 100^o to the dam axis in the original design. Figure 5 shows the general layout plan and the section of the spillway for the original design.

Hydraulic model studies were conducted for assessing the following aspects:

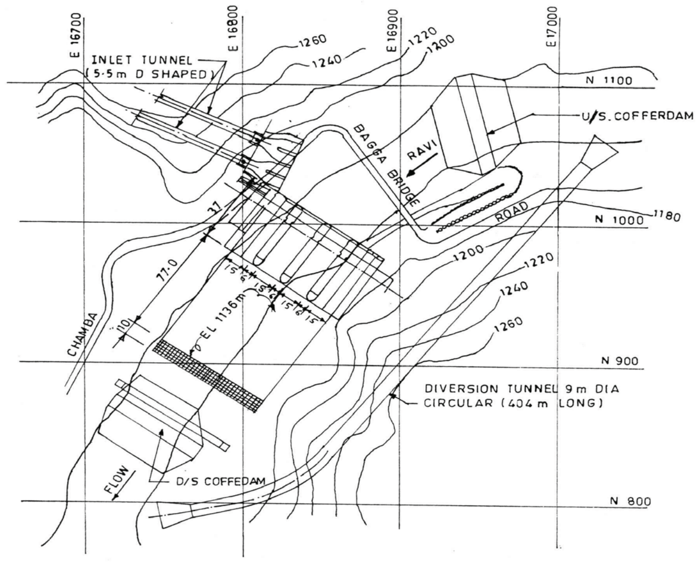
- Flow conditions upstream and downstream of spillway
- Adequacy of the spillway crest profile in respect of discharging capacity and pressures
- Water surface profiles along the spillway
- Performance of the hydraulic jump stilling basin
- Location and alignment of the power intake

The studies were carried out on a 1:40 scale 2-D sectional model and 1:55 scale 3-D comprehensive model. The studies on the sectional model were conducted in a glass sided flume reproducing two spans of the spillway along with the energy dissipator. Studies were conducted for investigating discharging capacity of the spillway, suitability of geometry of the crest profile in terms of piezometric pressures and performance of the stilling basin for the entire range of discharges.The 1:55 scale 3-D comprehensive model incorporated the non-overflow and overflow sections of the dam with the recommended design of energy dissipator, the river reach of 611 m upstream and 734 m downstream and power intake at 100^o m to the dam axis.

3.1 Studies on the original design of the spillway

3.1.1 *Flow conditions upstream of spillway*

Studies were conducted for the entire range of discharges up to the design maximum discharge of 9000 m^3/s with free flow and partial gate operation of the spillway to observe the flow conditions upstream of the spillway and in the vicinity of power intake. Studies indicated that flow conditions upstream of the spillway for free overflow were not satisfactory



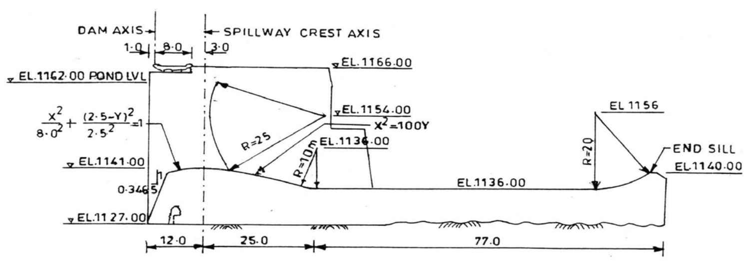


Figure 5 : General layout plan and cross section of spillway – original design

as the flow was supercritical and seen riding on the right bank near the intake and the left end spans drew very less discharge. This is caused by the bend and steep slope of the order of 1:60 of the river channel just upstream of the spillway. For the discharge of 6750 m^3/s the flow conditions were more or less the same. For the discharges of 4500 and 2250 m^3/s, the flow conditions were relatively mild.

Studies were also conducted for discharges up to 6750 m^3/s keeping the reservoir water level at El. 1162 m by partial gate operation. For the discharge of 6750 m^3/s, the flow conditions were turbulent near the spillway and power intake area and strong vortices were observed near the gate grooves in right spans. The approach flow was at about 45° to the piers which generated vortices upstream of the piers in right spans. For the discharges of 4500 m^3/s and below, the flow conditions were relatively mild and weak vortices were forming intermittently. These vortices had no apparent visible adverse effect on functioning of thespillway.The original alignment of the power intake was at 100° to the dam axis.The studies were conducted with partial gate operation of the spillway passing the discharges of 6750; 4500; 2250; 1000 and 500 m^3/s by keeping the reservoir at El. 1162 m and power intake passing a discharge of 177.5 m^3/s. Return flows were observed in front of the power intake and vortices were generated in front of the right spans of the spillway. In order to improve the flow conditions, it was decided to modify the alignment of the power intake by tilting it towards left so that it made an angle of 90^o with the dam axis.With the modification, return flows were not observed in front of the intake. Intensity of vortices was reduced considerably and flow conditions improved substantially. For very low discharges the conditions were very tranquil. The average velocity in front of the power intake varied from 2.6 m/s to 0.15 m/s for the discharges of 6750 to 500 m^3/s passing over the spillway. As such, the modified alignment of the power intake at 90° to the dam axis was found to be satisfactory and was recommended.

3.1.2 *Optimizationof the spillway section*

It was observed that the design discharge of 9,000 $m^{3/}s$ could be passed at a reservoir water level of El. 1160.90 m as against the design RWL of El. 1160.82 m which might be acceptable. However, the coefficient of discharge obtained from this was about 0.572 which was considered to be too small. The short fall in the discharging capacity could be attributed to two factors: the geometry of the crest profile in the form of equation $X^2 = 100Y$ and submergence of the crest by the tail water levels.The profile conforming to $X^2 = 100Y$ was rather too flat considering that the maximum head under such conditions would be around 21 m. A profile conforming to $X^2 = 84\ Y$ was found to be an appropriate profile and was incorporated in the model.

For a spillway which is only 3 m above the normal river bed, submergence of the crest by tail water would be inevitable. It is seen that the crest would be submerged by the tail water level for discharges exceeding 1500 m^3/s. It is well known that submergence of the crest does affect its discharging capacity. With the modified profile it was observed that the design discharge of 9000 m^3/s could then be passed with the reservoir level of El 1159.8 m which offered a coefficient of discharge of0.62. Studies with passing of small discharge of 500 m^3/s with a small gate opening indicated that pressures on the crest were positive everywhere. The studies for higher discharges up to maximum outflow flood of 9000 m^3/s also indicated positive hydrostatic pressures. This profile was therefore adopted for further studies on the comprehensive model.

3.1.3 *Optimization of Stilling basin type Energy dissipator*

Studies were conducted on 1:40 scale 2-D sectional model for the discharges corresponding to 25%, 50%, 75% and 100% of the maximum discharge of 9000 m^3/s. It was observed that for the free overflow condition, oscillating hydraulic jump was formed in the stilling basin. However, for the discharges of 50% and above the energy dissipation was inadequate and turbulence persisted beyond the end sill. The high velocity flow downstream of the end sill continued with bottom layers of the flow running along the bed downstream of the spillway. This observation is indicative of imperfect hydraulic jump which otherwise would have caused this flow to be lifted up by the end sill.When the discharges were passed through partial gate openings, maintaining FRL El. 1162 m, the hydraulic jump was sweeping out of the basin like a ski-jump jet for 25% discharge, viz. 2250 m^3/s. For higher discharges, the jump was oscillating and on the verge of sweeping out. These observations pertain to the tail water levels as per the original curve. Subsequently, a revised tail water curve was supplied which indicated tail water levels lower by around 1m. The performance with this would still be inferior. In addition to this, the length of the stilling basin was also found to be inadequate.

In view of this, the design was further modified by lowering the apron level to El. 1131 m which would ensure satisfactory conditions up to discharges of about 6000 m^3/s, both for free overflow as well as partial gate operating conditions. This design is designated as Alternative II design. The length of the apron was 94 m i.e. $4.25*Y_2$. The end sill top was at El. 1136m. The alternative II design of the stilling basin aimed at ensuring good energy dissipation for the discharges up to about 6000 m^3/s, which corresponds to 100 year return period. It was however, seen that hydraulic jump corresponding to very small discharges of the order of 500 to 2000 m^3/s was submerged. Studies were conducted to assess the self-cleansing properties of the adopted design of the stilling basin viz. alternative II design, particularly, for smaller discharges of frequent occurrence. These studies could however be conducted only in a qualitative manner. The stilling basin was filled with fine silt and smaller discharges were passed down the spillway with partial gate opening at FRL El. 1162 m. It was indicated that the silt deposited in the stilling basin could be flushed out of the basin for discharges of the order of 1800 m^3/s to 2000 m^3/s with reservoir at FRL El. 1162 m.

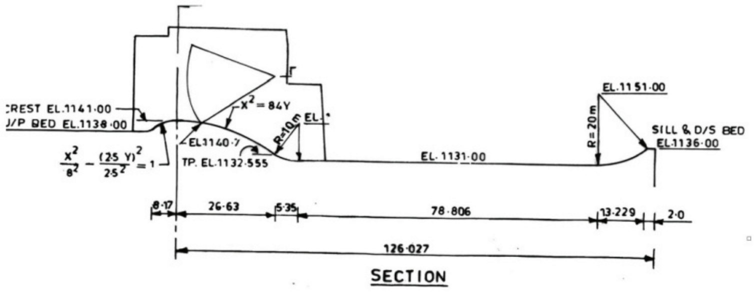


Figure 6 : Cross section of modified spillway profile

The provision of suitable protection works downstream of the stilling basin would prove to be advantageous in the long run. In view of this, it was recommended that the key of the stilling basin end sill be taken adequately below and anchored to the fresh rock to guard against any scour approaching the end sill and possible undermining of the same. In addition to this an apron of length 15 m may be provided downstream of the stilling basin at El. 1136 m for additional protection.

4. HYDRAULIC MODEL STUDIES FOR CHAMERA-III H. E. PROJECT

Chamera H.E. Project, Stage III is the uppermost project in cascade located in Chamba District of Himachal Pradesh. It is a run-of-the-river scheme to generate 231 MW of power utilizing a gross head of the order of 230 m. The project comprises of a 68 m high and 78 m long concrete gravity dam. The main spillway is 55.5 m long and equipped with breast walls. There are three spans 12.5 m wide separated by 5 m thick piers. The spillway radial gates have dimensions of 12.5 m (W) x 16.5 m (H). The spillway has been designed for passing the PMF of 11,400 m^3/s at FRL. The energy dissipation arrangement was in the form of 50 m long stilling basin. An additional spillway in the form of 11 m diameter Tunnel envisaged in the original design. This was provided to comply with the requirement of one extra gate during PMF.

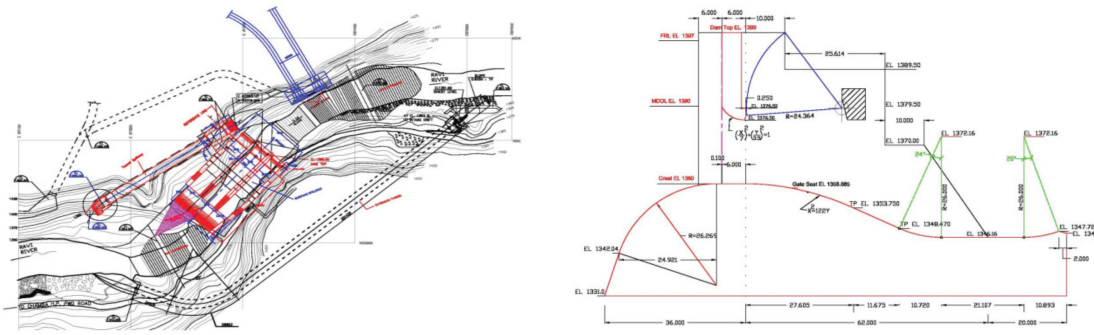


Figure 7 : Layout plan and cross section of spillway – original design

A 3-D comprehensive model was constructed to a geometrically similar scale of 1:60 incorporating river reach of about 800 m upstream and 650 m downstream of dam axis. The model was reproduced in 12 m wide and 30 m long rectangular tray. The river bed and banks were reproduced in smooth cement plaster. The spillway profiles, piers and radial gates were reproduced in PVC foam sheets and Perspex sheet and were painted with P.U. paint so as to have a very smooth surface. Figure 8 shows the upstream view and downstream view of model.

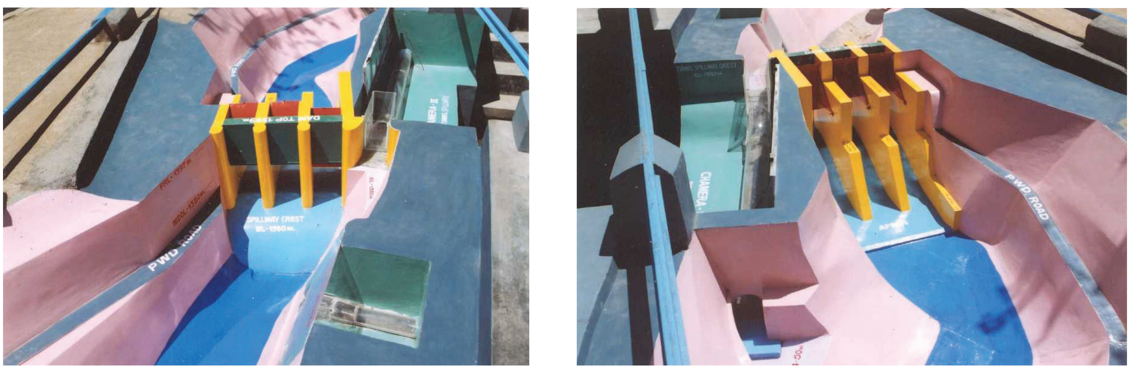


Figure 8 : Upstream and Downstream view of the Model

Hydraulic model studies were conducted for the following aspects for entire range of discharge and reservoir water levels (CWPRS, 2005):

- Discharging capacity of the orifice and tunnel spillways
- Water surface profiles and Pressures on the orifice spillway
- Performance of energy dissipator

4.1 Hydraulic model studies for the orifice spillway

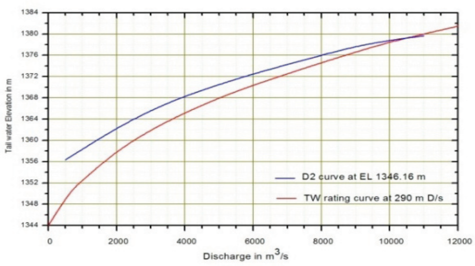
Discharging capacity of the spillway has the paramount importance in assessing the performance of spillway. Studies were conducted for discharging capacity of orifice spillway and tunnel spillway with full gate opening and various reservoir water levels. The design maximum discharge of 11,400 m^3/s could be passed through the surface and tunnel spillway with Reservoir water level of El. 1394 m and only through surface spillway with FRL El. 1397 m. A discharge of 12,560 m^3/s could be passed at FRL El. 1397 m from both surface and tunnel spillway operating simultaneously. Therefore, the combined discharging capacity of the spillways was found to be adequate.

Water surface profiles were observed at the centre of the surface spillway, along left and right training walls for various conditions. The trunnion of the surface spillway was getting submerged intermittently by the water surface profile corresponding to the maximum discharge of 11,400 m^3/s. Hence, it is required to raise the elevation of the trunnion considering the water profile, free board and bulking of flow due to air entrainment. The training walls were getting overtopped for almost all the discharges beyond 6000 m^3/s. It may be mentioned here that the high contours on the right side in front of the stilling basin need to be dressed down.

Pressures were measured for the entire range of discharges up to the PMF of 11,400 m^3/s for the ungated as well as partial gate operation conditions. The maximum negative pressure of 0.5 m was observed for a discharge of 2850 m^3/s with partial gate operation. The corresponding cavitation index for this condition is much above the critical cavitation index of 0.2 and as such the same is acceptable. In view of this, the performance of the surface spillway profile in respect of pressures is considered to be satisfactory.

4.2 Optimization of the hydraulic performance of energy dissipator

The performance of energy dissipator was observed for the entire range of discharges by maintaining the tail water level at ch. 290 m downstream of dam axis. It was seen that hydraulic jump was forming in the stilling basin for the entire range of discharges except for lower discharges of the order of 1000 m^3/s and below. However, the hydraulic jump was unstable and very sensitive to the tail water level and was very violent for the higher discharges. This is a typical case of low Froude number stilling basin where the Froude number varies from 2 to 5 with very high discharge intensity of the order of 200-300 m^3/s for PMF. The jump height curve has also been calculated and plotted on the Figure 9 with the tail water rating curve.



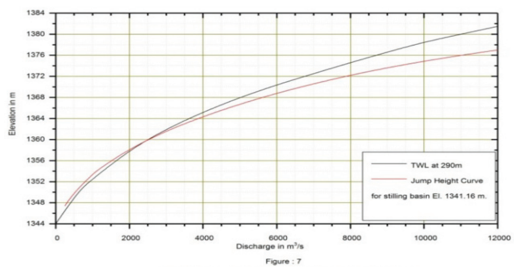


Figure 9 : TWL and Jump Height Curves for stilling basin El 1346.16 m and 1341.16 m

It can be seen that there is a deficiency of about 6 m for the discharge of 1000 m^3/s which reduces to about 3-4 m up to a discharge of 6000 m^3/s. The tail water rating curve and jump height curve cross at 10,000 m^3/s. The tail water is more than jump height curve by about 1.0 m for the design maximum discharge of 11,400 m^3/s. Therefore it is expected that the jump may sweep out and ski-action may prevail up to 6000 m^3/s and an unstable jump may form for higher discharges. In view of the unsatisfactory performance of the stilling basin, it is proposed to lower the stilling basin by about 5 m and extend the length of the stilling basin by 30 m and provide an apron of about 30 m immediately downstream. The left training wall in the stilling basin is splayed by 6^0. This constriction in the width of stilling basin has resulted in return eddies on the left side. Therefore, it is suggested that the training walls on both the sides should be provided straight and the top level be raised to EL 1370 m above the tail water level corresponding to a discharge of 5700 m^3/s (50 % of PMF).

The performance of the revised stilling basin with El.1341.16 m was observed for the entire range of discharges up to the design maximum discharge of 11,400 m^3/s. The tail water level at 290 m downstream of dam axis was maintained as shown in Figure 9. From the figure it is seen that the jump height curve and the tail water curve are almost the same for discharges upto 3,000 m^3/s. Thereafter, the tail water-rating curve is higher than the jump height curve indicating submergence of the hydraulic jump for higher discharges. It was observed that the hydraulic jump which formed in the basin was contained in it for all the discharges. The jump was steady for discharges of 5,700 m^3/s and below. Thus, lowering the stilling basin and increasing the length of the basin have resulted in improving the hydraulic performance of the stilling basin. The basin is deep enough to provide for full conjugate depth for discharges up to 5,700 m^3/s. The training walls contain the jump for discharges up to 5,700 m^3/s and get overtopped for higher discharges. Figure10 show the performance of the energy dissipator for discharge of 4010 (2850+1160) m^3/s at FRL El. 1397 m.

Figure 10 : Performance of Energy dissipator for Q= 4010 (2850+1160) m^3/s at FRL El 1397 m.

The stilling basin has dual responsibility of energy dissipation and flushing of sediments. Experiments were carried out to ascertain the self-cleansing capacity of the stilling basin. The stilling basin was filled with unseived sand up to the lip level. With the upstream reservoir level at FRL El.1397 m and the tail water level as per the rating curve the performance of the stilling basin was observed for discharges of 25% and 10% of maximum discharge. In the former case the sand got flushed in no time and travelled downstream. When aggregates of 20 mm size were introduced, it was found that even that got swept out of the basin. For a discharge of 1000 m^3/s, the material was flushed out of the stilling basin, but it remained just downstream of the sill as enough energy was not available to push it further downstream. This is also an indication of the high degree of energy dissipation in the stilling basin for 1000 m^3/s discharge. Thus, the performance of the stilling basin was found to be acceptable in respect of self-cleansing capacity. Further, when the right end span was closed, eddies were observed in the stilling basin which were bringing in the flushed material back into the basin and churning. The same was repeated by closing the other two spans one after the other and return eddies were observed in the basin along with the churning of the flushed material. Therefore, susceptibility of the basin to abrasion damage cannot be ruled out with unequal operation of the spillway bays. Hence, it is recommended that all the spans of the spillway be operated equally and partially.

5. CONCLUSIONS

- Hydraulic model studies played significant role in optimizing the spillway and energy dissipators for all the three cascade hydropower projects.
- In Chamera-I H.E. Project, the alternative layout evolved after conducting extensive model studies ensured satisfactory hydraulic conditions in the spillway chute as well as in the plunge pool. The reduction in the cost due to elimination of one spillway span is considerably more than the extra cost due to curved divide walls. Also, the modifications proposed would not adversely affect the time schedule of the project.
- In almost all the cases, the upstream flow conditions are generally sub-critical. However, in the Chamera-II H.E. Project due to steep slope of the river the river flow conditions upstream of the spillway were supercritical, due to low crest there was a bearing on discharging capacity as well as flow conditions in the stilling basin. The high depth of overflow of 21 m led to a high discharge intensity of 150 $m^3/s/m$ and low Froude number of 2-4. This had direct implications on the design of energy dissipator. The studies for energy dissipator indicated the need for designing the same for dual function of disposal of floods and flushing of sediment. The studies were conducted for alternative design to evolve suitable dimensions of the stilling basin.The studies were also helpful in assessing excavation of left bank and alignment of power intake.
- Due to deficient tail water level, the performance of the stilling basin was not satisfactory for Chamera-III H.E. Project. From the extensive model studies, the stilling basin was lowered by 5 m to El.1341.16 m and extended by 30 m to a total length of 50 m. The performance of the stilling basin improved with these modifications for the entire range of discharges and was recommended.
- All the three projects have been executed based on modifications suggested by extensive model studies and functioning satisfactorily.

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