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The Rehabilitation Solutions for the Hydraulic Jump Sweepout: A Case Study from India

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***Abstract*—**The tailwater requirements are essential criteria in the design of the stilling basins as energy dissipation of the spillways. The adequate tailwater level that ensures the hydraulic jump inside the basin should be fulfilled by the river's natural water level and the apron depth downstream of the chute. The requirements of the hydraulic jump should mainly be checked for the design flood; however, the drowned jump condition should not be critical in discharges less than the design flood. The tailwater requirement is not met in Almatti dam, which became operational in 2002 in India, and the jump sweeps out from the basin, resulting in significant scour in the apron and end sill. This paper discusses different hydraulic solutions as a sustainable remedy for dam rehabilitation. As the most cost-effective, sustainable solution, the deep apron alternative is proposed for the fewer spillway bays. The apron level of 15 out of 26 gates should decrease by 5.4 m compared to the existing design to ensure a safe hydraulic jump up to the discharge of 10,000 m^3/s, i.e., 30% of the updated Probable Maximum Flood (PMF).

***Keywords*—**Dam, spillway, stilling basin, Almatti.

I. Introduction

A spillway stilling basin is an essential component of hydraulic structures designed to dissipate the energy of flowing water to prevent erosion and maintain downstream stability. The design of spillway stilling basins is crucial for ensuring their effectiveness in controlling water flow and minimizing potential damage. Stilling basins are built downstream of structures such as chutes and gates to control the energy dissipation of hydraulic jump [1]-[3].

Several studies emphasize the importance of efficient energy dissipation within the stilling basin to minimize downstream erosion and turbulence. The hydraulic jump phenomenon has long been a subject of interest in fluid dynamics and hydraulic engineering. This natural occurrence, characterized by a sudden increase in water depth and decrease in velocity, has been studied extensively in open-channel flow systems. In the energy dissipater structures, baffle blocks and end sills are used to decrease the stilling basin length and prevent the jump sweepout from the basin, even if the tailwater depth is less than the conjugate depth of free hydraulic jump [4]. Forester and Skrinde [5] were the first ones to conduct studies about hydraulic jumps on an adverse-sloped surface. Harleman [6] was one of the first researchers who investigated the role of baffle blocks and their effects on flow characteristics at stilling basins. Armenio et al. [7] studied the pressure fluctuations using a negative step at the end of a hydraulic jump. Ohtsu and Yasuda [8] investigated the hydraulic jump on adverse steps with the effect of tailwater depth, Froude number, and the step heights on the type of hydraulic jump and divided the hydraulic jump into six categories. Various publications discuss using baffle blocks and deflectors to enhance hydraulic performance and reduce turbulence within the basin. Abdelazim and Yaser [9] studied the effect of stilling basin shapes on submerged hydraulic jump.

The results showed that stilling basins with end steps create the shortest submerged hydraulic jump in stilling basins. Tiwari [10] designed a stilling basin model with the effect of the wall and the end still and concluded that by a suitable design of the wall size, not only the efficiency of the stilling basin model increase, but also basin lengths decreased by 29% comparison to USBR IV stilling basin [11]. Gehlot and Tiwari [12] studied several models of the stilling basin at the pipe outlet with rectangular and circular sections. They used studies of previous researchers. Youngkyu et al. [13] and Hamedi & Fuentes [14] experimentally studied hydraulic jump, energy dissipation, and characteristics of downstream flow for different types of spillways with sluice gates. Pagliara and Palermo [15] compared two configurations of stilling basins and predicted the energy dissipation downstream of the stilling basin for them. Neveen [16] investigated the impact of channel slope on the characteristics of hydraulic jump and tested the attributes of hydraulic jump in the vertical valves located downstream of a rectangular channel. Gamal et al. [17] explored the impact of different shapes of stilling basins with different heights of the end steps on characteristics of submerged hydraulic jump and energy dissipation downstream of a sluice gate. Feimster [18] studied the impact of tailwater on designing several stilling basins in the USA. The design of the basin apron and its interaction with the flowing water is a significant focus in many studies, aiming to optimize flow patterns and energy dissipation. Literature often includes findings from scale model studies to validate design approaches and assess the hydraulic performance of different basin configurations.

II. Methodology

The following steps, criteria, and codes were used in the hydraulic calculation of the gated spillway with a sharp-crested ogee and stilling basin dissipater:

- The rating curve of the spillway was determined based on the current configuration of the spillway, e.g., the sharp-crested weir, ogee crest elevation (El), the approach channel water depth, etc.
- The downstream river natural rating curve was determined

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at the end of the stilling basin based on initial estimation. The curve needs to be verified based on observation or a river simulation model.

- The initial spillway discharge coefficient was determined from the configuration of the ogee and approach channel using the UBSR code. Then, the spillway discharge was calculated considering the existing spillway configuration and hydraulic contraction of the piers using the USBR code.
- The water surface (WS) profile was calculated on the ogee and the chute for three scenarios of the design discharge based on a water surface profile model.
- The hydraulic jump calculation was conducted based on the initial flow depth and Fr at the beginning of the stilling basin. The required conjugate depth was calculated, and the apron level was set so that the conjugate depth requirements were provided by the natural tailwater level and water depth of the apron.
- The apron effects and tailwater submerge factors were calculated, and they did not have any reduction effect on the spillway coefficient based on the USBR code. The chute WS profile and stilling basin parameters were updated based on the spillway discharge until the parameters were converged for the coefficient reduction factors and energy losses.
- The apron level was examined on different design discharges as per the USBR recommendation [11].
- The length and height of the type II stilling basin were determined based on the USBR code [11].
- The ogee nappe profile and cavitation indexes were estimated based on water surface profile calculation, and the cavitation damages were estimated based on the USBR code [11].
- The existing stilling basin of the Almatti dam is Type II. The water depth in the Type II basin should be about 5% greater than the computed conjugate depth because of the reduced margin of safety against sweepout as per the USBR code [11]. In this report, the marginal water depth of 5% has not been considered in the comparison of the alternatives. This modification should be considered for the confirmed alternative.

The following formulas and methodology were used in the simulation of the spillway release performance:

- The discharge over a spillway crest is limited by the same parameters as the weir, and determined by:

$$Q = CL_e H_e^{1.5} \tag{1}$$

where Q: rate of discharge, cubic feet per second (ft^3/sec); C: Coefficient of the discharge; Le: Effective length of the crest, feet; He: Total specific energy above the crest, feet.

- Effect of abutment and priers: The spillways include abutments of some type, and intermediate piers. The effect that the abutments and piers have on the discharge is accomplished by modifying the crest length using the following equation to determine the effective crest length Le:

$$Le = L - 2*(n*Kp + Ka)*He \tag{2}$$

where L = net length of crest; N = number of piers; Ka = pier contraction coefficient; Kp = abutment contraction coefficient.

- Effect of approach channel: Another factor influencing the discharge coefficient of a spillway crest is the depth in the approach channel relative to the design head defined as the ratio P/Hd, where P equals the crest elevation minus the approach channel invert elevation and Hd is the design head.
- Effect of Upstream Face Slope: The slope of the upstream spillway face also influences the coefficient of discharge.
- Effect of Heads Different from Design Head: When the ogee crest shape is different from the ideal shape or when the crest has been shaped for a head larger or smaller than the one under consideration, the discharge coefficient will differ. A wider shape will result in positive pressures along the crest contact surface, thereby reducing the discharge. With a narrower crest shape, negative pressures along the contact surface will occur, resulting in an increased discharge.
- Effect of Downstream Apron Interference and Downstream Submergence: When the water level below an overflow weir is high enough to affect the discharge, the weir is said to be submerged. The vertical distance from the crest of the overflow to the downstream apron and the depth of flow in the downstream channel, as it relates to the head pool level, are factors that alter the discharge coefficient.
- The basic principle used to analyze steady incompressible flow through a spillway is the law of conservation of energy expressed by the Bernoulli (energy) equation. The energy equation, generalized to apply to the entire cross-section of flow, expresses the energy at any point on the cross-section in feet of water by:

$$H = Z + \frac{P}{\gamma} + \alpha \frac{V^2}{2g} \tag{3}$$

where H = total energy head in feet of water above the datum plane; Z = height above a datum plane, feet; P = pressure at the point, pounds per square foot (lb/ft^2); ɣ = Specific weight of water, pounds per cubic foot (lb/ft^3); α = Energy correction coefficient; V = average flow velocity, feet per; g = acceleration due to gravity, ft/sec^2.

- The mean pressure at any location along a chute is determined using the principle of conservation of energy as expressed by the energy equation. The energy conservation requires that the energy at one location on the spillway be equal to the energy at any downstream location plus all intervening energy losses expressed in equation form and in units of feet of water:

$$Z_1 + \frac{P_1}{\gamma} + \alpha_1 \frac{V_1^2}{2g} = Z_2 + \frac{P_2}{\gamma} + \alpha_2 \frac{V_2^2}{2g} + H_L \tag{4}$$

where H_L = energy losses in ft. The energy loss is the direct result of three conditions: 1) boundary roughness (friction), 2) turbulence resulting from boundary alignment changes (form loss), and 3) boundary layer development.

- Methods for determining the energy loss related to boundary roughness (friction) have been developed by various investigators. The most notable and widely used methods are the Darcy-Weisbach equation, the Chezy equation, and the Manning equation.
- Turbulent boundary layer development energy loss is determined based on the surface roughness energy loss associated with free flow on an overflow crest spillway with a P/Hd ratio greater than one which is dependent upon the development of the turbulent boundary layer thickness.
- The turbulent boundary layer thickness is a function of the length, L, along the spillway from the start of the crest curve and the effective roughness, k, described as (all value in feet):

$$\frac{\delta}{L} = 0.08\left(\frac{L}{K}\right)^{-0.233} \tag{5}$$

The spillway energy loss, H_L, in terms of feet of head, is defined by:

$$H_L = \frac{\delta_3 u^3}{2gq} \tag{6}$$

where q = the unit discharge in cubic feet per second per foot (ft^3/sec/ft); u = potential flow velocity, ft/sec; δ_3= energy thickness (ft).

- Cavitation is defined as the formation of a gas and water vapor phase within a liquid resulting from excessively low localized pressures. The existence and extent of cavitation damage are dependent upon the boundary shape, the damage resistance characteristics of the boundary, the flow velocity, the flow depth, the elevation of the structure above sea level, and the length of time the cavitation occurs. The cavitation index, σ is derived from energy equations as follows:

$$\sigma = \frac{H_0 - H_V}{\frac{V_0^2}{2g}} \tag{7}$$

where H_0 = reference head, ft; H_V = vapor head of water, ft.

As σ decreases below the incipient cavitation level, the cavitation damage potential increases very rapidly.

- Hydraulic Jump Type Energy Dissipator, defined as a stilling basin, is used to dissipate kinetic energy by the formation of a hydraulic jump. The hydraulic jump involves the principle of conservation of momentum. This principle states that the pressure plus momentum of the entering flow must equal the pressure plus momentum of the exiting flow plus the sum of the applied external forces in the basin. The hydraulic jump will form when Froude number F_1, flow depth d_1 at the entrance, and the sequent flow depth d_2 satisfy the following equations:

$$\frac{d_2}{d_1} = 0.5\left[(1 + 8F_1^2)^{\frac{1}{2}} - 1\right] \tag{8}$$

$$F_1 = \frac{V_1}{(gd_1)^{\frac{1}{2}}} \tag{9}$$

- The energy loss in the hydraulic jump is equal to the difference in specific energies before, E_1, and after, E_2, the jump which can be estimated by:

$$\Delta E = E_1 - E_2 = \frac{(d_2 - d_1)^3}{4d_1 d_2} \tag{10}$$

- The length Lj of a hydraulic jump on a flat floor without baffles, end sill, or runout slope (not necessarily the stilling basin length) can be estimated by:

$$L_j = 8.0 d_1 F_1 \qquad for\ F_1 > 5$$

$$L_j = 3.5 d_1 F_1^{1.5} \qquad for\ 2 < F_1 < 5 \tag{11}$$

III. Case Study Feature

Almatti Dam was constructed across the Krishna River as one of the major multi-purpose reservoir projects in 2002 in India. The dam location is shown in Fig. 1. The downstream view of the Almatti dam site is shown in Fig. 2. The stilling basin scouring problem was identified as one of the rehabilitation's immediate action plans. It has been observed that energy is not being dissipated within the basin, resulting in scour pits due to high-velocity jets. The primary method of dissipating energy is to generate a hydraulic jump to convert flow from supercritical to subcritical and finally decrease the flow velocity.

The profile of the water surface shows that the jump runs away from the basin in the discharges of more than 50% of the maximum flood discharge. The reason for the damages is the high flow velocity in the apron because of the jump sweepout from the basin as shown in Fig. 3 [19], [20].

Due to inadequacy, a hydraulic jump is not formed inside the basin, and the water jet takes a ski jump with a parabolic jet profile impinging the d/s of the end sill. This impingement develops erosion or scour holes in the apron area and toe of the end sill and consequently results in structural damage subsidence.

The maximum tailwater level downstream of the Almatti dam is above Narayanapur reservoir's normal water level downstream. It seems that the assumptions of the tailwater requirements in the primary design of the spillway were not according to the real operation conditions of the project. Even if the stilling basin is repaired, the same damage will occur within one or two operating seasons due to inadequate tailwater elevation. Therefore, the remedy for stilling basin scouring is to provide the required tailwater of the hydraulic jump inside the basin for discharges up to the design flood.

Fig. 1 River catchment of the Almatti Dam [19]

Fig. 2 Overview of the Almatti Dam

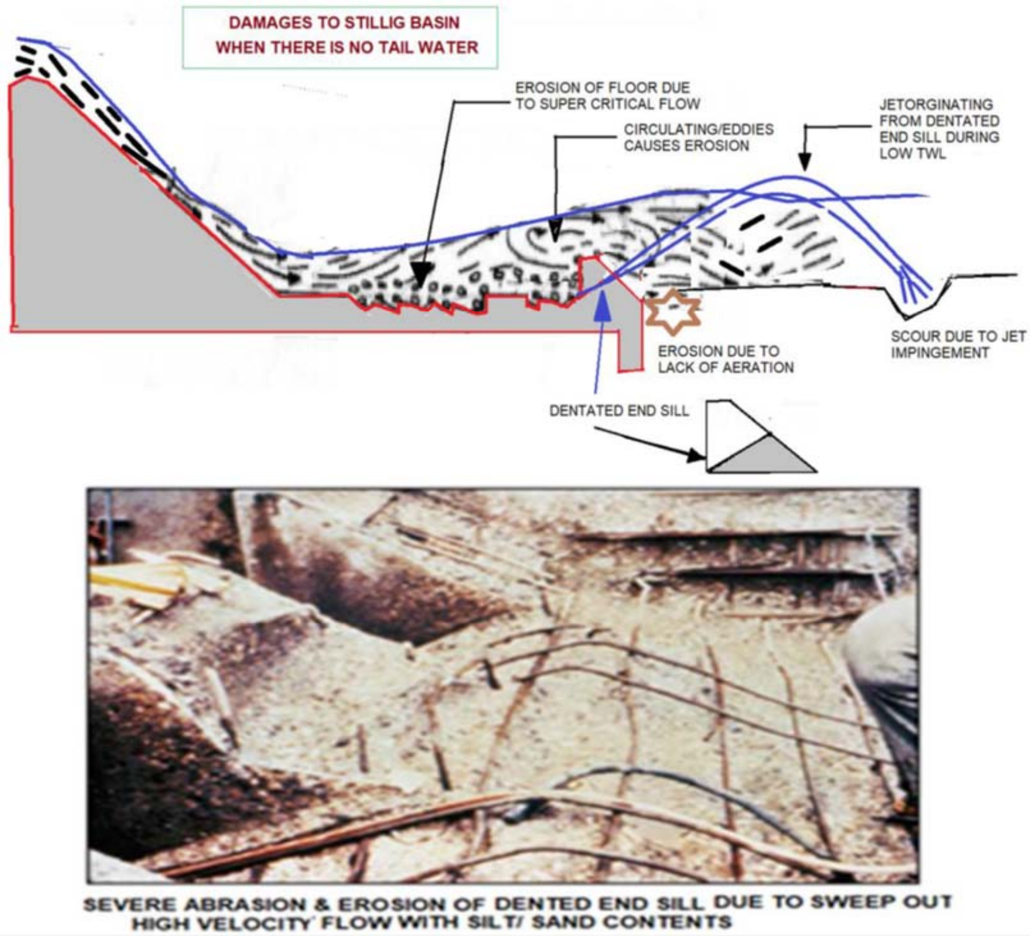

Fig. 3 The scouring problem of the stilling basin and downstream channel [19]

TABLE I
ALMATTI SPILLWAY HYDRAULIC CALCULATION FOR DIFFERENT DESIGN DISCHARGES

Parameter	Unit	Design Flood			Description
Flood Magnitude		PMF	1000 YR	~100 yr	
Q	m^3/s	31,000	25,000	20,000	The ratio of PMF to 1000 yr i.e., 1.24 seems very low
Tail Water	masl	497.7	496.6	495.7	The tailwater was calculated based on the width of the river and slope of 1:1000
C0	$m^{0.5}/S$	2.17	2.18	2.18	Sharp crested weir based on P/H0
Max. Head (H0)	m	11.24	9.70	8.33	The head on the spillway was set not to have any backwater in the basin
Max. El on the crest	masl	520.3	518.7	517.3	
Velocity at end of the Chute - V1	m/s	27.6	26.8	26.0	
Depth at end of the chute- D1	m	2.4	2.0	1.6	
Froude number - Fr		5.8	6.1	6.6	
The jump depth D2	m	18.0	16.0	14.1	
Stilling Basin Lengths Type II	m	72.0	64.9	58.2	
Depth of Apron to existing apron	m	5.8	4.8	4.0	

IV. RESULTS

A. Spillway Design Feature

The hydraulic calculation results are presented in Table I. As presented in the table, the jump depth is 18 m and 16 m for PMF and 1000 yr flood discharge, respectively, if the stilling basin apron El is selected on the tailwater requirement of the design flood. The flow velocity will be 27.6 m/s and 26.8 m/s in the design flood of PMF and 1000 yr, respectively at the end of the chute.

The water surface profile and cavitation index have also been calculated based on the standard step-backwater method along the spillway chute, and the results are presented in Fig. 4 for the design flood of PMF.

B. The Cavitation Index

The cavitation index (Flow Sigma, σ) — values less than 0.2 generally indicate a high potential for cavitation damage. For spillways with design cavitation index values of 0.1 to 0.2, cavitation damage has traditionally been mitigated through surface tolerance specifications and maintenance programs to ensure a smooth surface free of offsets and other anomalies. When cavitation index values drop below 0.1, USBR has typically employed aerators to add air to the flow and protect the spillway surface from damaging cavitation.

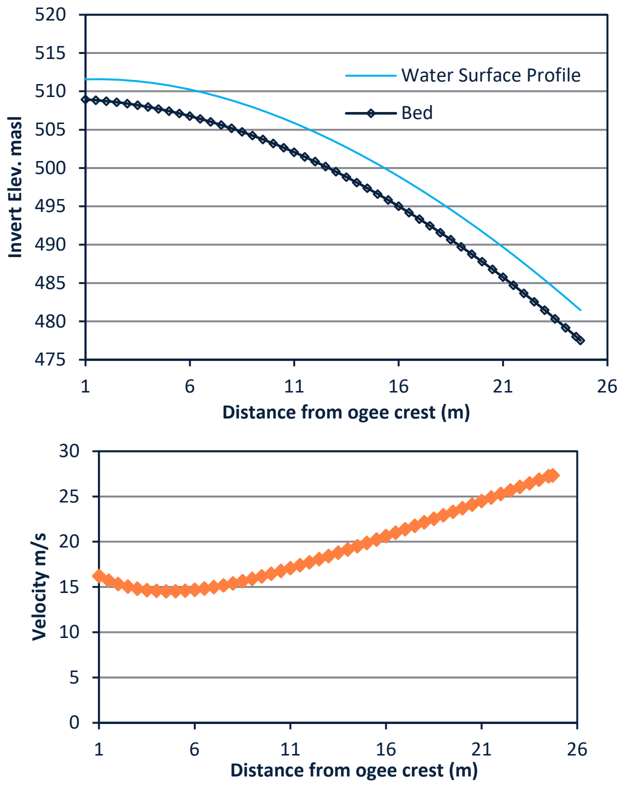

Fig. 4 Chute profile computation results

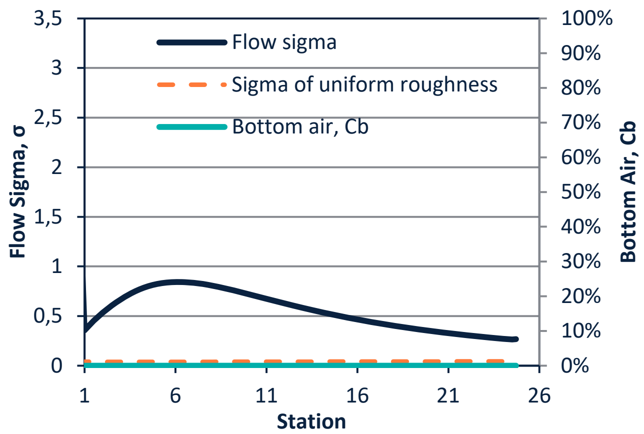

Fig. 5 Cavitation index along the chute

The cavitation index (Flow Sigma, σ) as shown in Fig. 5 is above 0.2 in the Almatti spillway for PMF design flood; therefore, it does not need any aerators or surface protection for cavitation.

C. The Tailwater Requirement

Tailwater rating curves for the regime of river below a dam are fixed by the natural conditions along the stream and ordinarily cannot be altered by the spillway design or by the release characteristics. The following statement is from the design of small dams [11]:

> For a jump-type stilling basin, downstream water levels for various discharges must conform to the tailwater rating curve. The basin floor level must therefore be selected to provide jump depths that most nearly agree with the tailwater depths. For a given basin design, the tailwater depth for each discharge seldom corresponds to the conjugate depth needed to form a perfect jump. Thus, the relative shapes and relationships of the tailwater curve to the depth curve will determine the required minimum depth to the basin floor.

This is shown in Fig. 6, where the tailwater rating curve is shown as curve 1, and a conjugate depth versus discharge curve for a basin of certain width is represented by curve 3. Because the basin must be deep enough to provide for full conjugate depth (or some greater depth to provide a safety factor) at the maximum spillway design discharge, the curves will intersect at point D. For lesser discharges the tailwater depth will be greater than the required conjugate depth, thus providing an excess of tailwater, which is conducive to the formation of a "drowned jump". If the basin floor is higher than indicated by the position of curve 3 on Fig. 6, the depth curve and tailwater rating curve will intersect to the left of point D. This indicates an excess of tailwater for smaller discharges and a deficiency of tailwater for higher discharges.

The Almatti stilling basin is a Type II USBR basin. The incoming flow velocity in the basin is higher than 60 ft/s, the type II basin has been adopted correctly. The chute blocks and dentated end sill effectively reduce the basin length; however, the water depth in the basin should be about 5% greater than the computed conjugate depth_because of the reduced margin of safety against sweepout as per the USBR code. In this paper, the marginal water depth of 5% has not been considered in the comparison of alternatives.

As per an initial river water depth calculation, the river's natural tailwater water depth is 8.5 and 7.5 m in the design flood of PMF and 1000 yr, respectively. If the required depth of jump is set on the downstream tailwater El) intersection of the conjugate depth curve and tailwater rating curve) as shown in Figs. 7 and 8, the apron El will be 479.7 masl and 480.7 masl for the PMF and 1000 yr design floods. The excavation depth will be 5.8 m and 4.8 m to ensure the hydraulic jump for the flood magnitudes of PMF and 1000 yr flood, respectively. If the design flood is considered 47,318 m^3/s based on conclusion of the DSRP conclusion, the depth of the apron level will be more than 9 m.

Where a tailwater rating curve is shaped, the level of the stilling basin floor is determined for some discharge other than the design capacity as shown in Fig. 9 [11].

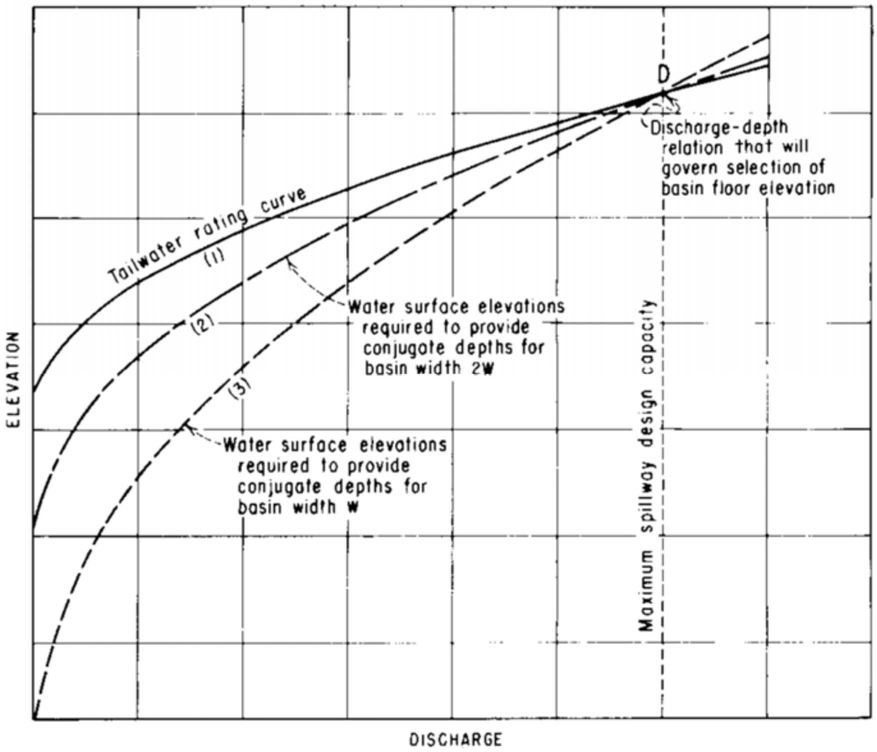

Fig. 6. Relationship of the conjugate depth curves to tailwater rating curve [11]

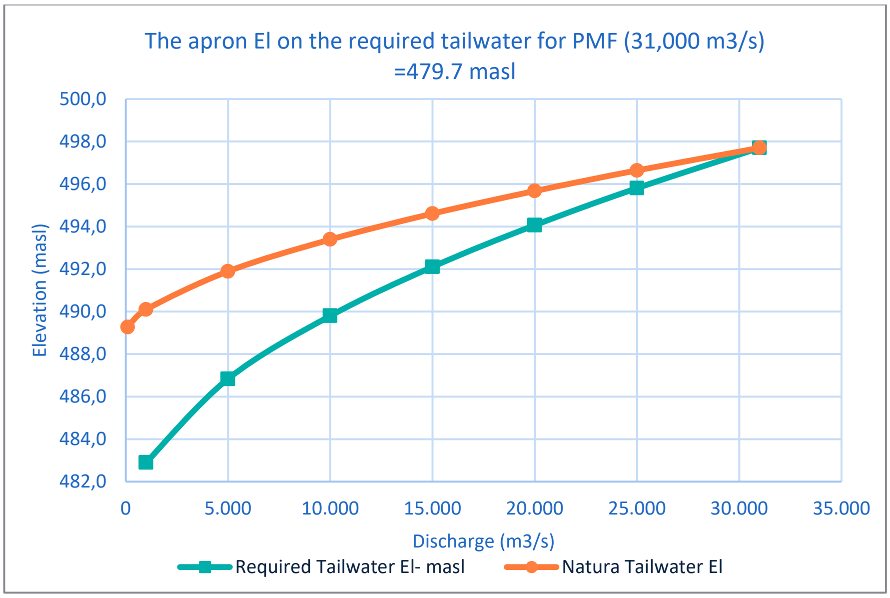

Fig. 7 Intersect of the conjugate depth curve and natural tailwater rating curve in the PMF design flood

In under-operation dams such as the Almatti dam, it is difficult to change the spillway configuration and set it based on the PMF design flood. Furthermore, the design flood of the spillway is lower than PMF as per the guidelines of India and most of the countries. Besides, the most frequent flood discharges are less than 10,000 m^3/s as per the observed floods from 2002 to 2013. Therefore, it is recommended to set the apron level based on the discharge of 10,000 m^3/s to 15,000 m^3/s for the Almatti spillway under operation. It is worth mentioning that the maximum discharge to set the apron level should be verified based on dam stability analysis results and the maximum possible excavation depth of the apron.

The conjugate depth of the hydraulic jump is 9.7 m in the design flood of 10,000 m^3/s. The apron level will be 483.65 masl for the entire gates in the discharge of 10,000 m^3/s based on the intersection of the jump depth curve and natural river rating curve as shown in Fig. 10. The apron excavation depth will be 1.85 m, which will provide a tailwater depth of 9.7 m, i.e., excavation depth plus the natural river water depth.

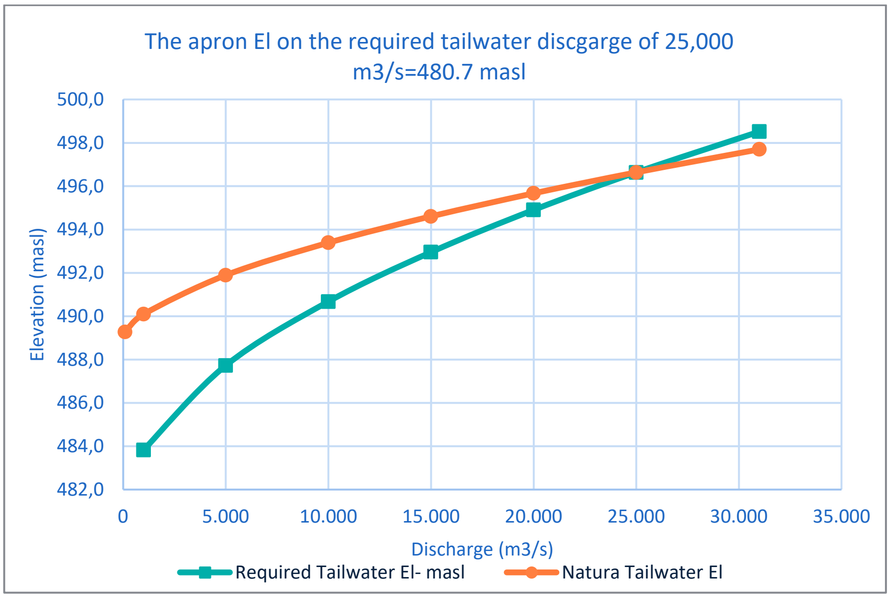

Fig. 8 Intersect of the conjugate depth curve and natural tailwater rating curve in the 1000 yr (25,000 m^3/s) design flood

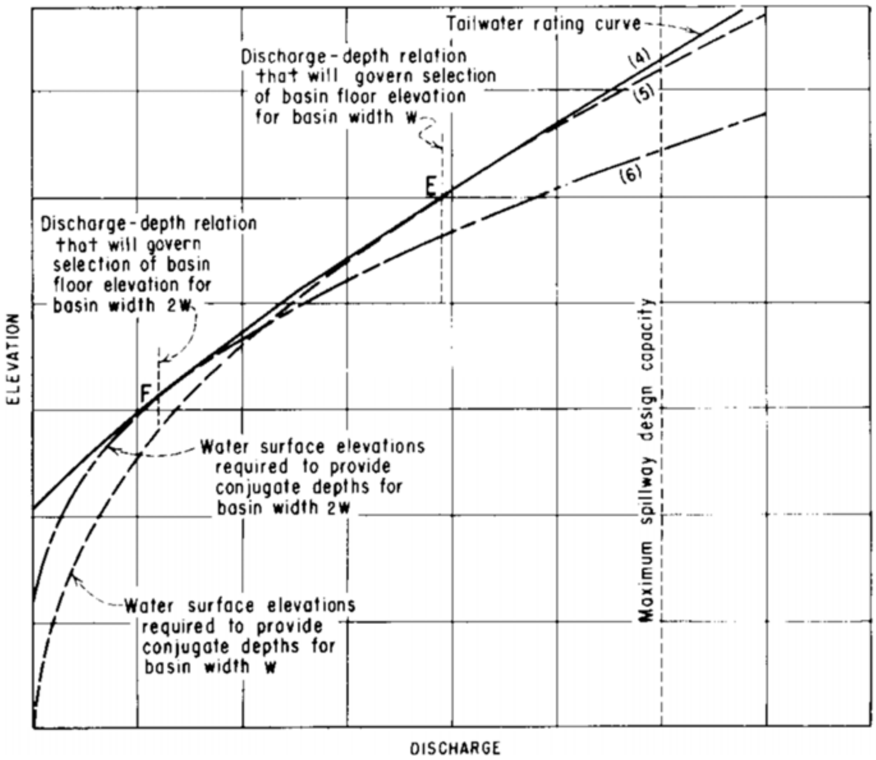

Fig. 9 The relationship of conjugate depth curves to tailwater rating curves

The dam foundation elevation of the concrete overflow section seems to be 481 masl based on the section of the profile of the dam [19], [20]. The current elevation of the apron is 485.5 masl and the modified apron El is 483.65 masl. The excavation depth with be 2.9 m considering ~1 m of the apron concrete thickness. Therefore, the excavation El will be 1.6 m higher than the dam foundation level. If the excavation El is set on the dam foundation El i.e., 481 masl, the apron level will be 482 masl, and the design discharge of the apron level will be ~ 17,500 m^3/s. The discharge to set the apron level is lower than PMF but it acceptable according to design codes and as the rehabilitation plan according to the most frequent flood magnitudes recorded during the operation of the dam.

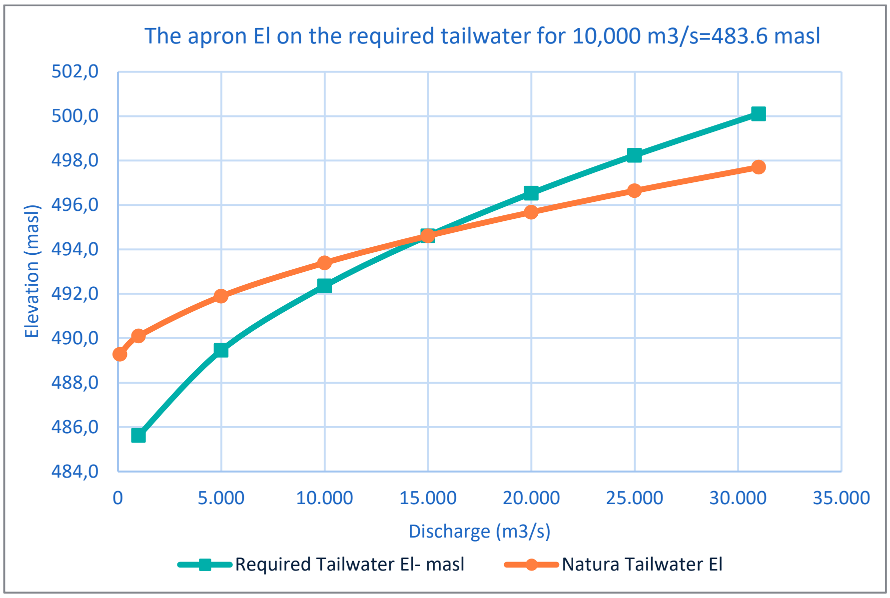

Fig. 10 Intersect of the conjugate depth curve and natural tailwater rating curve in 10,000 m^3/s discharge

D. Alternative Solutions

The solutions for the stilling basin scouring problem are generally categorized into hydraulic and structural solutions. The structural solution which includes strengthening the structure for the high flow velocity by high reinforcement, high strength, and anti-abrasion concrete, is not a sustainable solution, and the rehabilitation works should be repeated depending on the major flood frequency and damages of the basin and downstream channel. The hydraulic solution which is to provide the requirement of the hydraulic jump and prevent the jump sweepout from the basin, is a sustainable solution; however, it is more expensive than the structural solution in the existing dams. The structural solution was implemented in the Almatti dam in the previous rehabilitation plan; however, the basin was damaged again after the floods. Therefore, the second phase of the Dam Rehabilitation and Improvement Project (DRIP II) has focused on hydraulic solutions.

A waste weir has been proposed at the end of the stilling basin to provide the required tailwater for the hydraulic jump in the DRIP II. As a hydraulic solution, the weir height should be determined based on the necessary conjugate depth of the design flood. The river tailwater will not help to provide the required tailwater for the hydraulic jump in the waste weir alternative. In other words, the weir height should be set at the jump conjugate depth to provide the required tailwater, while in the deeper apron solution, the natural tailwater will effectively help to provide the downstream requirements for hydraulic jump inside the basin. The requirement of the conjugate depth should be considered 5% more in the Type II stilling basin as per the USBR recommendation [11].

The provision of the wall (weir) at the end of the stilling basin is shown in Fig. 11 [19]. The purpose of the weir is to ensure jump formation well within the stilling basin, avoiding severe cavitation damage. The height of the weir has been considered 6 m from the riverbed and 9.5 m from the stilling basin apron level i.e., 485.5 masl.

The proposed weir helps to provide artificial water depth for the hydraulic jump; however, the height of the weir should be determined based on the design flood and the required conjugate depth of the hydraulic jump. The natural tailwater will not help to provide the required tailwater for the hydraulic jump in the Project Screening Template (PST) proposed plan. Therefore, the weir height should be equal to the jump conjugate depth considering the confident limit for the Type II stilling basin. The weir may not be justifiable compared to the other alternatives such as a deeper apron. In other words, the weir will be submerged for the shortened alternatives and will not be efficient with the hydraulic jump in high flows. On the other hand, the stability of the weir against the high flow velocity of the spillway would be a severe issue if the weir height increases.

The proposed weir will be effective for floods less than 8,870 m^3/s as per the assessment for the tailwater requirements, while there are observed floods higher than this rate. The overflow from the weir will have high energy, and an energy dissipation system will likely be needed downstream of the weir.

Ansys Fluent software has been employed in order to study the physical properties of the flow field over the ogee spillway and stilling basin. The flow field of fluid (air and water) has been modeled using spillway and stilling basin geometry meshing for the Finite Volume Method.

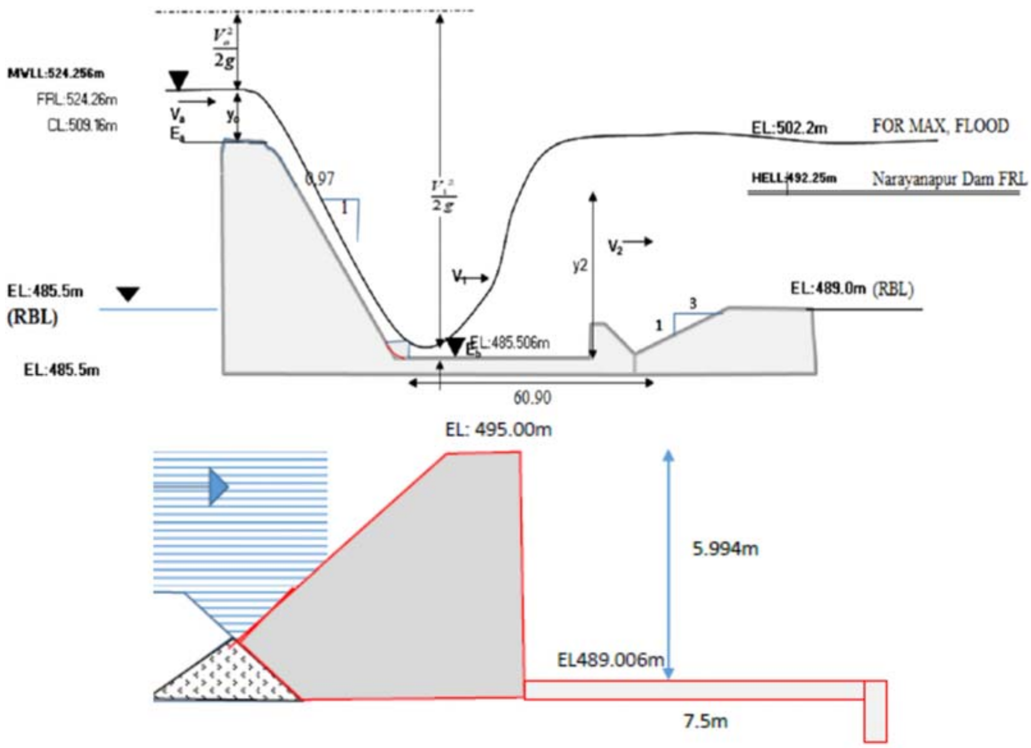

Fig. 11 The proposed profile of the waste weir at the end sill of the basin [19]

As per the result of the CFD model [21], the flow velocity at the end of the chute will be ~ 26 m/s and it will decrease to ~20 at the end of the basin in the PMF peak discharge i.e., 31,007 Cumec. Similarly, the flow velocity at the end of the basin is ~ 12, ~13, and ~17 m/s in the discharges of 15,574, 19,539, and 24,919 Cumec, respectively. The high flow velocity at the end of the basin indicates that the hydraulic jump is not formed properly in the basin for the modified layout for the above discharges as per the CFD results. Furthermore, the flow velocity at the toe of the waste weir is higher than ~13 m/s in all discharges. Considering the suspended sediment loads of the flood, the above-mentioned high velocity of the flow will scour the concrete in the basin for the modified layout of the waste weir. It means that the proposed layout of the DRIP II does not help for the formation of hydraulic jump in the investigated discharges i.e., higher than 15,000 Cumec. Moreover, when discharge over the spillway is sufficiently high, the hydraulic jump with lots of turbulence gradually moves towards the end sill.

The proposed remedy under the DRIP II is the provision of the wall (weir) at the end of the stilling basin to ensure jump formation well within the stilling basin, avoiding severe cavitation damages. The height of the weir has been considered 6 m from the riverbed and 9.5 m from the existing apron of the basin. The proposed weir has been simulated by a CFD model; however, the CFD results do not confirm the modified layout for forming the hydraulic jump in investigated discharges. The natural tailwater will also not help to provide the required tailwater of the hydraulic jump in the proposed plan. Therefore, the weir height should be equal to the jump conjugate depth. The weir is subjected to scouring in the toe because of the high flow velocity, and it will not be justifiable compared to the other alternatives from technical and economic points of view.

In the PST report, a waste weir and training wall has been proposed to be built at the end of the basin with a 6 m height of the riverbed El, i.e., 489 masl. Since the weir does not use the downstream river tailwater effectively to provide the requirement of the hydraulic jump, in the current report, a deeper apron solution was deemed a better solution compared to the waste weir. In the previous sections, the apron depth was discussed from different discharges for entire gates. However, the remedy for all gates is costly due to the high number of gates. Therefore, two other alternatives were added to the deeper apron solution by considering a limited number of gates, and the design configuration and BoQ are compared in Table II for different alternatives. The table includes the maximum design discharge of the alternatives along with the dimensions and the excavation and concrete volumes. As shown in the table, the design discharge of the waste weir is estimated as ~9800 m^3/s which is lower than the deep apron alternatives design discharges. As pointed out, the 5% marginal depth of tailwater of the Type II basin has not been considered for the alternative comparison. If the marginal depth is considered for tailwater, the discharge will decrease to 8,870 m^3/s for the DRIP II proposed waste weir solution.

Alternatively, making a deeper stilling basin is more justifiable if it does not influence the dam stability in the overflow section and is built in the dry season. The apron El should be determined based on the design flood considering the natural river tailwater elevation. A deeper apron would provide a higher tailwater and ensure the jump inside the stilling basin for the discharges up to the design flood. The river's natural tailwater will effectively help in this alternative to provide the requirements of the hydraulic jump. The apron bottom El should not be lower than the dam foundation in the overflow section; however, this criterion needs verifying based on the dam stability analysis.

A deeper apron has been proposed to provide the hydraulic requirements as an alternative solution to providing a weir at the end sill. Since a deeper apron needs excavation downstream of the dam, the requirements for the stability of the dam in this alternative should be assessed and verified based on the stability analysis results. The deeper apron solution feature for entire gates is shown in Fig. 12.

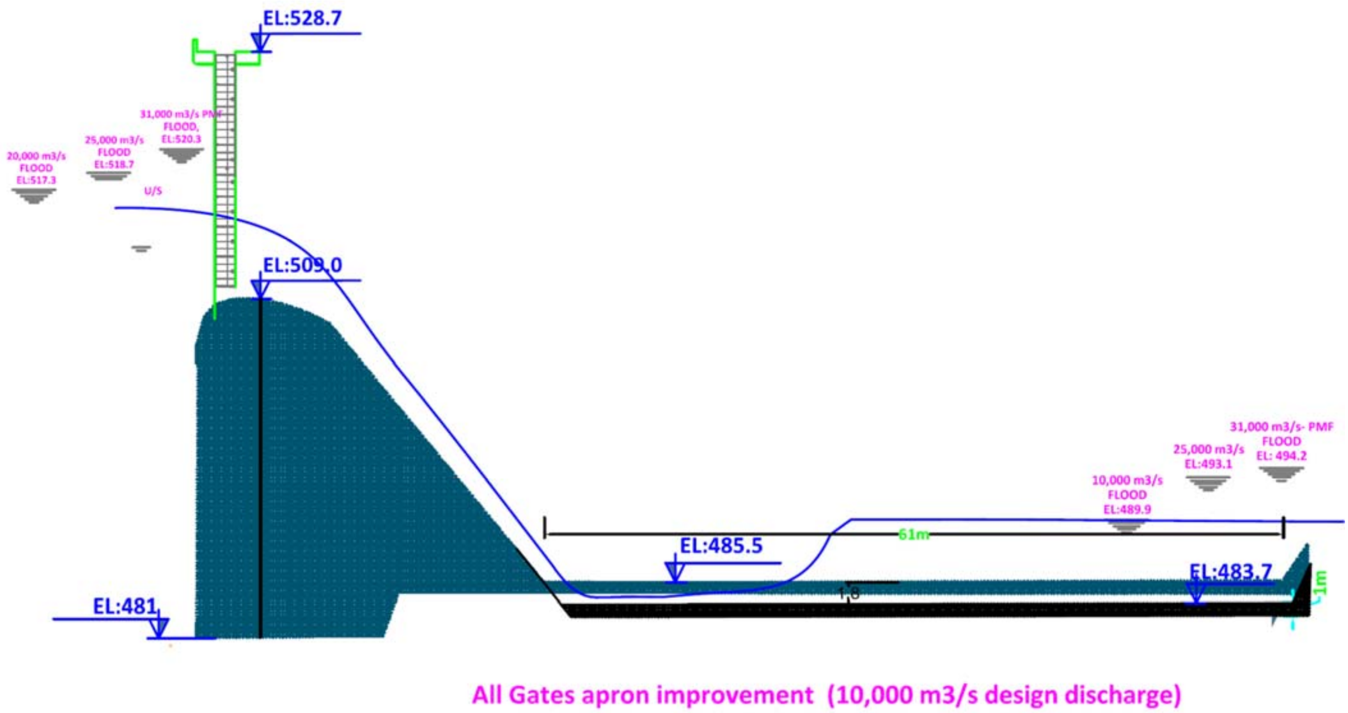

Fig. 12 Longitudinal profile of the all-gate deeper apron alternative

TABLE II
THE DESIGN CONFIGURATION FOR DIFFERENT HYDRAULIC SOLUTION ALTERNATIVES

Alternatives	unit	Alt 1-DRIP II	Alt 2	Alt 3	Alt 4	Alt 5	Description
		Waste Weir-for All Gates	Deeper Apron-for entire gates	Deeper Apron-for 15 Gates	Deeper Apron-for 15 Gates	Deeper Apron-for 10 Gates	
No of the gates	-	**26**	**26**	**15**	**15**	**10**	
Design discharge	m^3/s	**9869**	**10000**	**15000**	**10000**	**10000**	Selected design flood for hydraulic Jump requirement
Apron level	masl	485.5	483.7	477.9	480.1	476.5	Was not limited to dam foundation in Alt 3 & Alt 4 (the few gate alternatives)
Reservoir MWL	masl	514.2	514.2	518.9	516.5	519.0	MWL should not exceed the FRL (519 masl) for the few gate alternatives
Jump depth	m	9.5	9.7	16.7	13.3	16.9	
Apron depth to the existing apron	m	**0.0**	**1.8**	**7.6**	**5.4**	**9.0**	
Tailwater level	masl	493.4	493.4	494.6	493.4	493.4	
Basin Type II length	m	61.0	61.0	68.2	55.7	69.5	The existing basin length was considered as minimum length
Partition wall height	m	0.00	0.00	21.1	17.3	21.4	The jump freeboard was included in the wall height. Only needed in the few gates alternatives
Apron width	m	477.5	477.5	274.0	274.0	181.5	
Waste weir height	m	6.0	0.0	0.0	0.0	0.0	
The excavation volume	m^3	DRIP base case	82,902	160,097	98,196	126,358	
The reinforcement concrete volume	m^3	DRIP base case	29,128	23,019	18,156	17,088	

The minimum apron level has been set at 483.65 masl to provide the excavation El higher than the dam foundation El; however, this limitation has not been imposed for the fewer-gate alternatives. On the other hand, the limitation for the fewer-gate alternatives is the maximum allowable water level at the upstream. Since the other gates are closed in these alternatives, the maximum allowable water level should be no higher than FRL i.e., 519 masl. Considering the above limitations, the maximum design discharge is determined as 10,000, 15,000, 10,000, and 10,000 m^3/s in the alternatives of the deeper apron for entire-gate (Alt 2), 15-gate (Alt 3 and Alt 4), and 10-gate (Alt 5), respectively. As pointed out in the assumption, if the orifice function of the gates is considered for the reservoir water levels higher than 517.5 masl, the above discharges will slightly decrease for alternatives 3 and 5.

The excavation volume is lesser in alternative 2 (deeper apron of entire gates); however, the reinforced concrete volume is higher in this alternative. Therefore, the entire gate alternative seems to be rejected due to lesser design discharge and construction costs compared to the fewer-gate alternatives. The construction cost may increase in entire gates alternative by stage wise construction due to the construction period will likely be longer than 1 year (includes the flood season). Among the 15 and 10-gate alternatives, it is possible to increase the design to maximum ~ 15,000 m^3/s in 15 gates alternative (alternative 3); however, the maximum discharge is limited to ~10,000 m/s in 10 gates alternative (alternative 5) due to the limitation of water level upstream. Although the excavation and concrete volume is higher in 15-gate alternative for discharge of 15,000 m^3/s, the design discharge is ~50% of the PMF compared to the ~30% of the PMF in the 10-gate alternative. Therefore, the 10 and 15-gate alternatives are deemed a better solution from the flood risk point of view; however, an economic analysis should be conducted by including the construction costs and the flood risk to confirm the optimum alternative among the alternatives of 3, 4 and 5. The longitudinal profile of alternative 4 is shown in Fig. 13.

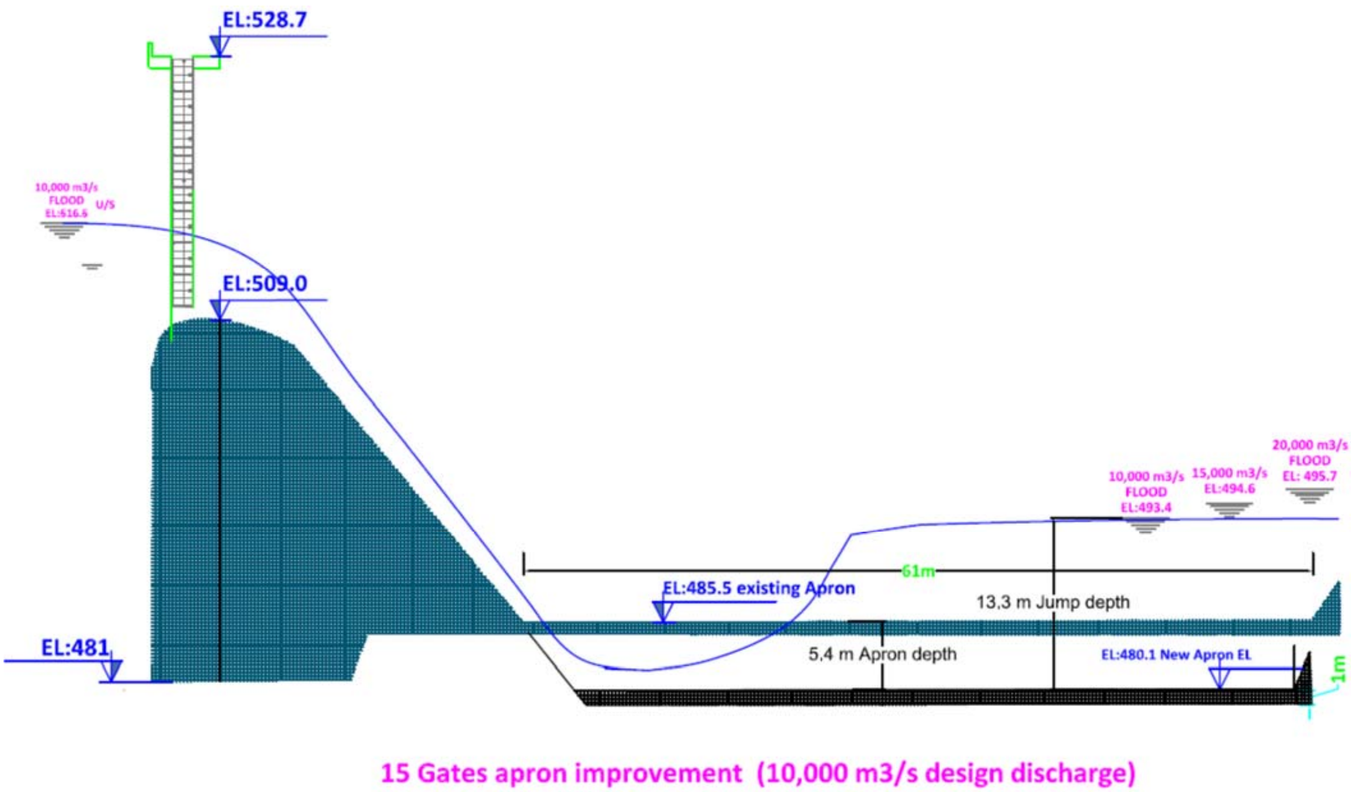

Fig. 13 Longitudinal profile of the 15-gate deeper apron alternative – the proposed alternative

V. Conclusion

The energy is not dissipated within the stilling basin in Almatti Dam, resulting in damages to the stilling basin due to high-velocity jets. The structural solution which includes strengthening the structure (implemented in DRIP I) for the high flow velocity was not sustainable, and the rehabilitation works should be repeated depending on the major flood frequency and damages of the basin and downstream channel.

The best and most sustainable solution is a hydraulic solution for the scouring problem in the stilling basin. Among the hydraulic solution alternatives, the deeper apron solution for a limited number of gates is deemed a more justifiable rehabilitation plan from economic and technical points of view.

In DRIP II, the study has been focused on a hydraulic solution, and a waste weir and training wall with a height of 6.5 m from the riverbed and 9.5 m from the stilling basin apron has been proposed in the rehabilitation plan. The maximum design discharge i.e., 9,870 m^3/s as per the results of this paper. The proposed waste weir compared to the deeper apron alternatives does not help the hydraulic jump to be formed inside the stilling basin for the investigated discharges because of the insufficient tailwater level in the high discharges. The natural river tailwater does not help the waste weir to provide the conjugate jump, while in the deeper apron solution, it effectively helps to provide the tailwater level for the hydraulic jump. The waste weir does not provide the requirement of the hydraulic jump for the discharge higher than 8,870 m^3/s considering the margin for the Type II basin. The required water depth in the basin Type II should be about 5% greater than the computed conjugate depth as per the USBR recommendation. This limitation has not been considered in the comparison of the different configurations in this paper. The erosion downstream of the proposed weir will most likely occur due to high flow velocity at the toe of the weir given the results of the DRIP II CFD model results. The height of the weir should increase for higher design discharges, and flow energy will be high downstream of the weir and the scouring problem of the spillway will endure.

The deeper apron alternative proposed in the current paper helps to provide the required tailwater level of the hydraulic jump. The apron depth needs to be determined depending on the maximum design discharge. The discharge to determine the apron level was discussed in this report. If the apron level is selected based on the discharge of 10,000 m^3/s which is higher than the observed major floods of the Almatti Dam after commissioning, the required depth of the apron will be 1.85 m for entire gates. The apron depth should increase for higher design discharge; however, in the fewer gates deeper apron alternatives, the apron level should be limited to a safe excavation depth of the apron. The dam foundation level of the overflow section is ~481 masl. The entire gates alternative (Alt 2) seems rejected due to lesser design discharge and high construction costs compared to the fewer-gate alternatives. In discharges lesser than the design discharge of the apron level, tailwater will be excess for the jump. With an excess of tailwater, the jump will be formed, and energy dissipation within the basin will be complete until the drowned jump phenomenon becomes critical. Existing chute blocks and end sills of the Almatti spillway will also assist in energy dissipation, even with a drowned jump. In the discharge higher than the design discharge of the apron level, the tailwater level will be insufficient for the hydraulic jump. With insufficient tailwater, the back pressure will be deficient and sweepout of the basin will occur; however, the probability of a flood with a higher frequency will be low and, consequently, the risk of damages will be acceptable.

Among the 15 and 10-gate deeper apron alternatives, although the concrete volume is higher in the 15-gate alternative, the maximum design discharge is ~15,000 m^3/s, i.e., 50% of the PMF compared to the ~30% of the PMF in the 10-gate alternative. The cost and flood risk analysis should be conducted to select the best alternative investigated in this paper. Nevertheless, the 15-gate alternative is deemed a better alternative from the flood risk and construction costs points of view. In this alternative, the apron should be lowered 5.4 m and 7.6 m from the current apron level for 10,000 m^3/s and 15,000 m^3/s design discharges, respectively. The basin Type II (chute blocks & dental end sill) length should increase from 61 m (the existing length) to 68 m for the design discharge of 15,000 m^3/s.

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