SECOND STAGE STILLING BASIN AS A SOLUTION FOR CHASHMA BARRAGE DUE TO DAMAGE CAUSED BY RETROGRESSION PHENOMENON

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Abstract- Chashma Barrage located on the Indus River in district Mianwali was constructed in 1971 as a part of Indus Basin Project. The Barrage is different in the sense that it has some storage capacity for regulation of water for irrigation releases, power generation and supply of cooling water for Chashma Nuclear Power Project. The riverbed below Chashma barrage has been degrading since its commissioning in 1971. This has resulted in low tail water levels for the whole range of discharges, and consequent deterioration in the performance of stilling basin of the barrage. For the maximum flood record of 1,038,873 cusecs observed in 2010, the tail water level was 5.3 ft. lower than design value. Similarly, for flood discharge of 636,000 cusecs in 2015, the tail water level was lower to the extent of 6 ft. from the designed figure. For discharges of 500,000 cusecs and below the tail water level is lower by 5 ft. on the average. Observations show that in general, tail water lowering has continued even after the record flood of2010. As a result of lowering tail water levels, the required conjugate depth for formation of hydraulic jump is not attained with the consequence of inadequate jump formation and the passage of undissipated energy downstream, causing scour and damages to stone apron, as a recurring feature. Solution for the formation of a stable jump on the protected area could be achieved from either of following options: Construction of a secondary weir and addition of an auxiliary stilling basin of appropriate length with a lower floor level, immediately below the existing stilling basin floor.

Keywords- Chashma Barrage; Stilling Basin; Retrogression.

1 INTRODUCTION

Chashma Barrage is located on the Indus River about 56 km downstream of Jinnah Barrage. The barrage supplies water to the Chashma Jhelum Link (CJ Link) Canal on the left bank and Chashma Right Bank Canal (CRBC) on the right bank. The cooling water supplies for the Chashma Nuclear Power Plant are also taken from the barrage through the CJ Link. A 184 MW hydropower plant was constructed subsequently on the right bank and was commissioned in the year 2001. Chashma Barrage, unlike other barrages in Pakistan has water storage capacity to regulate releases for irrigation and now for power generation also. The maximum and minimum designed reservoir levels are RL 649 ft. and 637 ft. respectively. The storage at the Barrage was designed to re-regulate the releases from Tarbela and floods of tributaries below Tarbela. The re-regulation and flood absorption capacity of the Barrage has, however, reduced significantly due to reduction in storage capacity as well as need for maintenance of pond for power generation. According to the survey of 2012, gross, live, and dead storage capacities have reduced from 0.87 to 0.348, 0.72 to 0.289 and 0.15 to 0.059 MAF. Chashma barrage received 82.0 MAF per annum on average basis, for period from 1998 to 2015, with minimum inflow of 62.9 MAF in year 2001, and a maximum inflow of 100.1 MAF in 2010. In last five-years period of 2011 to 2015, average inflow volume at Chashma barrage is computed as 82.7 MAF. As such during last five years barrage has received nearly average flows (82.7 vs. 82.0 MAF).

The maximum design discharge for the barrage is 950,000 cusecs through the gates whereas the downstream energy dissipation works are designed for 1,100,000 cusecs to account for 20% discharge concentration. Some 84,000 cusecs can be released through powerhouse since 2001. Theoretically this flow should not be considered in flood passing capacity of the project, as powerhouse is supposed to be closed if sedimentation concentration is high. Maximum historic peaks of flood passed through Chashma Barrage show that peak discharge of 1,038,873 cusecs that passed through barrage on 1st August 2010 was of 'Exceptionally High' category. Prior to 2010 the highest discharge of 786,600 cusecs passed on 3rd August 1976. 'Very High Flood' has also been observed in year 1992, and in year 2013. All other flood peaks, in the history of the barrage, remained as 'Low' or 'Medium' or 'High' flood stage/category.

The Chashma Barrage is 47 years old now and the riverbed below the barrage has been degrading since its commissioning, which resulted in low tail water levels for the whole range of discharges, and consequent deterioration in the performance of stilling basin of the barrage. Due to higher flow velocities now exiting at stilling basin has caused retrogression phenomenon which has shown a threat to the whole structure. It is necessary to point out that due to tail-water degradation, flow velocities exiting the stilling basin are now 17 to 22 % higher than those of the design and may reach 16 ft. per second in some standard bays and 18 ft. per second in certain under sluice bays. In addition to causing degradation of bed, such high velocities are instrumental in pushing the stone apron downstream which after displacement is heaped up as velocities fall in the wider river section below. In view of greater velocities now exiting at stilling basin, it may be necessary to review the existing conditions and propose new design specifications for the barrage which maybe second stage (or auxiliary) stilling basin as a solution for Chasma Barrage due to damage caused by retrogression phenomenon.

2 LITERATURE REVIEW

Valero et al. [1] investigated the performance of USBR Type-III stilling basin by using numerical simulations for eight different Froude numbers (F) ranging from 3.1 to 9.5. The steps cause an even higher decay of the maximum velocity within the basin. This decrease is more pronounced for smaller Froude numbers. Also, baffle blocks promote maximum velocity decay. Babaali et al. [2] performed computational modeling of the hydraulic jump in the stilling basin with convergence walls using CFD codes and concluded that the stilling basin has been accepted to be the most powerful hydraulic structure for the dissipation of the flow energy. The size and geometry of the stilling basin affect the formation of flow patterns, which can be influential for hydraulic performance of the whole system.

Jalut and El-Baaja [3] performed experimental studies for energy dissipation using stilling basins with one and two consecutive drops. This study presents the results of an experimental approach consisting of 1080 runs to achieve minimum length of hydraulic jump and maximum energy dissipation downstream of hydraulic structures using stilling basins with one drop and two consecutive drops. Rajaratnam and Hurtig [4] in the laboratory experiments have shown that screens or porous baffles with a porosity of about 40% could be used as effective energy dissipaters below hydraulic structures, either as a single wall or as a double wall. The experiments were carried out for a supercritical Froude Numbers from 4-13 and the relative energy dissipater was appreciably larger than produced by the corresponding classical hydraulic jump. In another study, a physical model was constructed to determine the size and placement of riprap downstream of Saint Anthony's Fall (SAF) stilling basins to ensure basin integrity [5]. The results show that the riprap size required for stability increases exponentially with the Froude Number and that larger riprap is required for stability if riprap is placed at the end sill level, compared to the placement at the basin floor level. Relationships are presented to determine the minimum size and length of riprap required to ensure basin integrity.

Moreover, Barjastehmaleki et al. [6] sought to reduce the severe pressure fluctuations in order to minimize damage to the stilling basin and the impact of Froude number, spillway length and width on the hydraulic characteristics and the water surface profile in different conditions. Higher value of The Froude number results in increased length of jump. (i.e. $Fr = 2$; $L = 10m$, $Fr = 4$; $L = 35m$, $Fr = 6$; $L = 50m$). With lower values of Froude number hydraulic jump will happen near the overflow toe which will cause higher energy dissipation and lower extent of erosion for the basin body. But if the hydraulic jump occurs in greater distance, lower amounts of energy will be dissipated, and basin body will be under greater risk of damage, which won't be economical and efficient. Type I stilling basin is more compatible with Froude numbers of $1 < Fr < 2.5$. Therefore, for optimal and economical design of type I stilling basins, we should avoid large Froude numbers because this type of basin is effective up to the Froude number of 3, and barely covers the larger values. Kim et al. [7] reported on the deterioration in the performance of the stilling basin of the Chasma barrage, as a result of lowering of tail water levels, the required depth conjugate depth for formation of hydraulic jump is not attained with the consequence of inadequate jump formation and the passage of un-dissipated energy downstream, causing scour and damages to baffle block, concrete block and stone apron. Solution for the formation of a stable jump on the protected area could be achieved from addition of an auxiliary stilling basin of appropriate length with lower floor level, immediately below the existing stilling basin floor. This will release the flood flows from barrage to riverbed at appropriate tail water depth. Construction of auxiliary basin could be easily managed during the present regime of barrage operation. Jüstrich et al. [8] the residual flow energy will produce scour in an alluvial riverbed if no mitigation structure (such as a dissipation basin) is installed downstream of a grade control structure. In the absence of a technical dissipation structure or a downstream apron, scour can occur in a loose riverbed downstream. Meftah et al. [9] investigated the effect of a W-weir (without upstream or downstream apron) on the downstream sediment transport processes, focusing on the scour-hole formation. The maximum scour depth was observed a short distance downstream of the weir, independent of the boundary conditions. Carvalho et al. [10] confirmed that the case of fully filled floor gave the smaller values of scour parameters. The experimental works were carried out by Abdelhaleem [11] indicated that the floor blocks should occupy between 40% and 55% of the floor width and the most favorable conditions result when the baffles are placed perpendicular to the incoming flow.

3 METHODOLOGY

3.1 Data collection

To collect and review all the available data concerning the weir such as discharge design, cross section and long section of the weir is provided by WAPDA, Lahore. The barrage has 52 bays of 60 ft. width each including seven (7) under-sluice bays on the left and four (4) under-sluice bays on the right. In addition, a fish ladder and navigation lock form part of the barrage. It is a glacis type weir, fitted with radial gates, with crest level in standard bays at El. 622ft. and in under-sluices at El.617 ft. The normal pond level is El.642 ft. and storage pond level at El. 649ft. The barrage was designed for a 100-year flood equivalent to 950,000 cusecs but it successfully passed the exceptional high flood of 10,38,873 cusecs in year2010.

The stilling basin of the main weir is 140.6 ft. long with its floor level at El.604 ft. It has two rows of impact blocks downstream of glacis and two rows of baffle blocks at the end of concrete floor, followed by concrete blocks apron and flexible stone apron. The stilling basin of the under-sluices is comparatively longer (by about 20 ft.) and set at 5 ft. below the floor level of the main weir. The stilling basin design has been developed based on model studies and does not conform to any standard USBR type basin. It also allows for flow concentration of 20 %. The off-taking CJ Link canal has a regulator with 8 bays of 40 ft. width each, while the Chashma Right Bank Canal has its regulator with 2 bays of 40 ft. width each.

3.2 Damages to Concrete Block Apron and Flexible Stone Apron

Below the concrete stilling basin, the barrage is provided with 40 ft. long concrete blocks apron, consisting of concrete blocks of size 5 ft. x 5 ft. x4 ft. laid over 2 ft. thick inverted filter. It is followed by a flexible stone apron which is 5 ft. thick and 104 ft. long. In the recent event, nearly 40 to 60 blocks were settled or dislodged, and a few were reported to have been carried downstream by the river current for a distance of 100 ft. The extent of settlement of blocks between bays 50/51 and in front of bay 49 ranged between 0.4 to 4.5 ft over a length of 60 to 75 ft. The filter material filled in the gaps between the blocks has been totally washed out. Considering the severity of the event, the filter material beneath the blocks might have been also washed out. The relief wells in the affected part were also damaged. In addition, a scour pit of considerable size developed below dislodged concrete blocks. Stone apron downstream of concrete blocks has also been disturbed and pushed downstream by the river current as shown in Figures 1 and 2.

It is necessary to point out that due to tail-water degradation, flow velocities exiting the stilling basin are now 17 to 22 % higher than those of the design, and may reach 16 ft. per second in some standard bays and 18 ft. per second in certain under sluice bays. In addition to causing degradation of bed, such high velocities are instrumental in pushing the stone apron downstream which after displacement is heaped up as velocities fall in the wider river section below. The flexible stone apron is the first line of defense against riverbed degradation below the barrage. The 104 ft. long and 5 ft. thick stone apron provided at Chashma Barrage is expected to launch at 30°angle and provide protection from scour. The stone specified by designers is of weight 40-250 lbs., 80 percent of which is to be heavier than 80 lbs. and not more than 5 % to be less than 40 lbs. In view of greater velocities now exiting at stilling basin, it may be necessary to revise the specifications for stone apron to minimize quantity of stone used to carry out annual O & M of flexible stone apron.

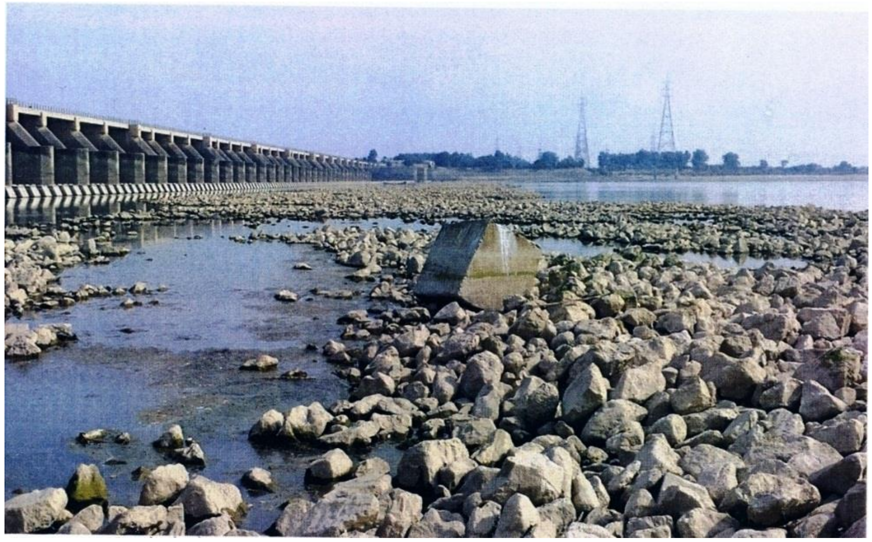

Figure 1: Dislodged concrete block carried away by river current in Front of Bay 51

Figure 2: View of settled concrete blocks right under-sluice

4 RESULTS AND ANALYSIS

3.3 Degradation of Tail Water Levels

The tail water rating curves provides estimated tail water levels for normal river state, accreted state, and retrogressed state for the range of river discharges at site. It may be observed that a retrogression of 5 ft. was allowed at higher discharges and 10 ft. at low discharges. The retrogressed levels are however used to check the design for proper submergence of hydraulic jump and adequate energy dissipation.

The data of minimum tail water levels observed against various discharges for the period from 2002-2016. This information has been used along with tail water levels reported for flood peaks of various years to estimate the present tail water rating curve in retrogressed state. It may be observed that existing retrogressed levels are 5 to 8 ft. lower than the design retrogressed levels over the whole range of discharges. For the flood of discharge 1038873 cusecs observed in 2010, the reported tail water was 5.3 ft. lower than the design retrogressed level. Similarly, for discharge of 636,000 cusecs observed in 2015, the tail water was 6 feet lower than design retrogressed level. The recent tail water (minimum) data for various discharges in 2013 and 2015 is given in Table 1. It may be observed that for lower discharges up to 200,000 cusecs, tail water level may be improving or rebounding, but this is not the case for higher discharges, for which tail water degradation is continuing.

Table 1: Tail Water Levels for 2013 & 2015

Years	Minimum Tail Water Levels for Various Discharges							
	20,000	50,000	100,000	200,000	300,000	400,000	500,000	600,000
2013	607 ft.	610.9 ft.	613.2 ft.	615.7 ft.	618.4 ft.	619.7 ft.	620.8 ft.	621.8 ft.
2015	609 ft.	612.1 ft.	613.7 ft.	616.2 ft.	616.8 ft.	617.8 ft.	618.5 ft.	619.3 ft.

The long term trends of degradation of tail water levels (minimum) for discharge of 300,000, 400,000, 500,000 and 600,000 cusecs are shown in Figure 3 (a,b,c,d). It may be observed that there was spectacular degradation in the initial years, but it seems to have been slowed down now.

It is recommended that river cross section may be periodically observed at 1 km interval up to 25 km distance downstream of barrage to understand the state of riverbed degradation which is resulting in tail water lowering. In addition, a mathematical modeling study for riverbed changes below Chashma Barrage should also be carried out to predict future river behavior.

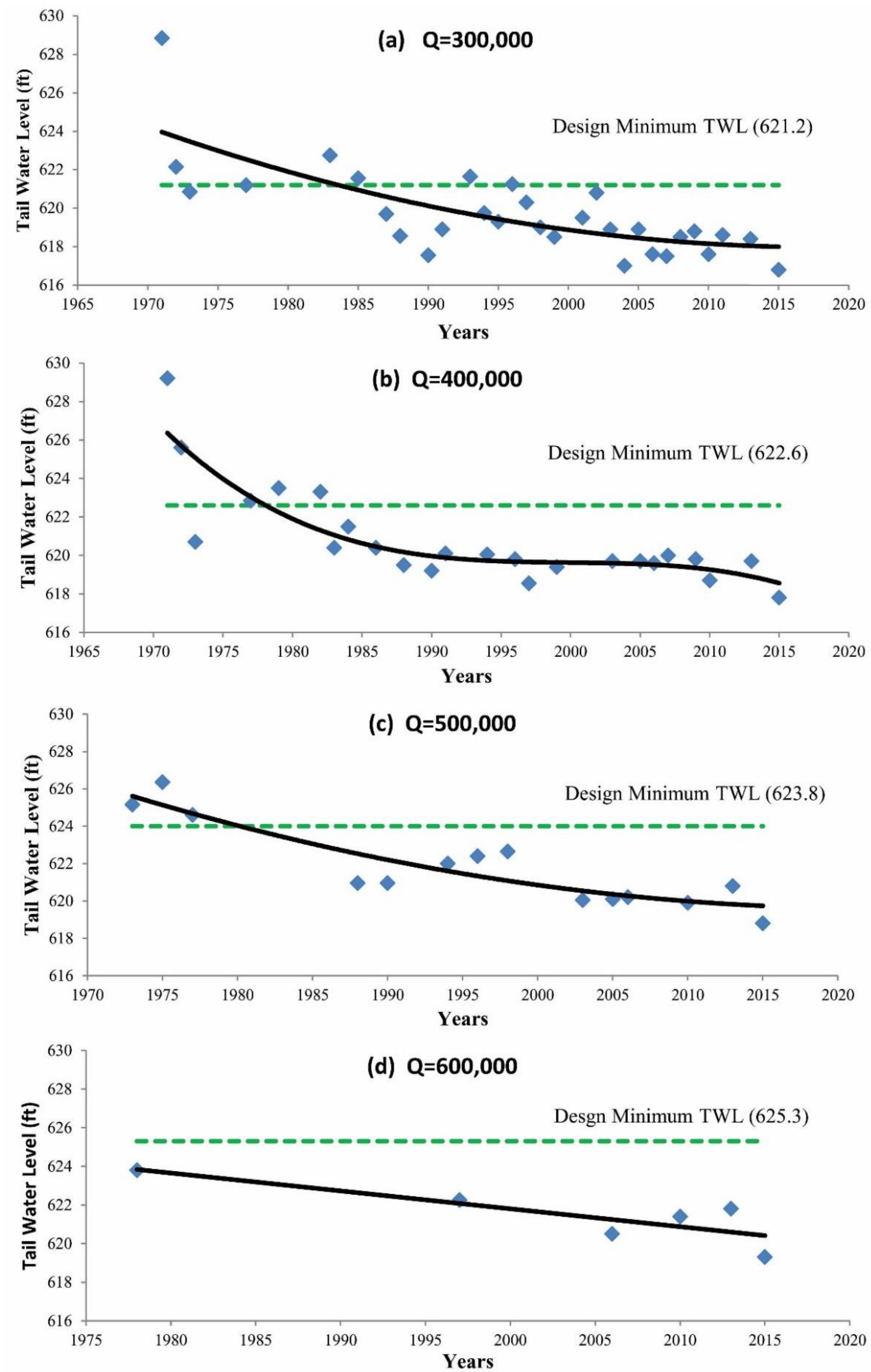

Figure 3: (a,b,c&d) Degradation Trends of Tail Water Levels at various Discharges

4.1 Jump Formation and Energy Dissipation in the Stilling Basin

The high rate of retrogression was a serious concern even during the early years of barrage operation, as it affected the performance of stilling basin and its safety. These two cases have been analyzed, using upstream energy levels with existing retrogressed water levels. For the 950,000 cusecs discharge, the existing tail water level has been found short by 6.23 ft. for standard bays, and 6.27 ft. short in under sluice bays, considering both pier effect and 20 % flow concentration. For 500,000 cusecs discharge, the tail water level is short by 4.07 ft. in standard bays and 4.58 ft. for under sluice bays as indicated in Table 2. These values represent 15 to 23 percent of the required depth for jump formation and are no more small values to be ignored. In fact, the existing tail water depths are insufficient to form proper jump and one could, at the best, expect only partial or incomplete jump formation with inadequate energy dissipation. A strong tendency of sweep-out of jump from the stilling basin is indicated. High velocity currents emerging out of stilling basin are likely to continue eroding the riverbed and tail water lowering. No wonder the degradation trends of tail water are continuing, though at decreased rate, even after 45 years of commissioning of barrage.

Table 2: Adequacy of Exiting Tail Water Levels for Jump Formation

Description	**Discharge 950,000 cusec**				**Discharge 500,000 cusec**	
	Standard Bays		Under-sluice Bays		Standard Bays	Under sluice Bays
Upstream Energy Level (ft.)	641.4		641.4		634.2	634.2
Discharge Distribution (cusec)	690,000		260,000		346,000	154,000
Pier Effect	Yes	Yes	Yes	Yes	Yes	Yes
20% Flow Concentration	No	Yes	No	Yes	Yes	Yes
Discharge Intensity at glacis $ft^2/sec.$	280.5	336.6	393.9	472.7	168.78	280
Required tail water depth, ft.	24.97	26.63	29.88	31.67	18.77	24.28
Existing Tail water depth, ft.	20.40	20.40	25.40	25,40	14.70	19.70
Deficiency (-) of tail water depth ft.	-4.57	-6.23	-4.48	-6.27	-4.07	-4.58
Deficiency of tail water depth (%)	18.3	23.4	15.0	19.8	21.7	18.9

It should be pointed out that if water is passed at higher upstream pond levels with gated operation, the problem of insufficiency of tail water will be severer, and will be felt even at lower discharges, as shown in Figure 4 (a & b). Therefore, it is stressed that flood flows should be discharged through the barrage at the lowest possible pond level.

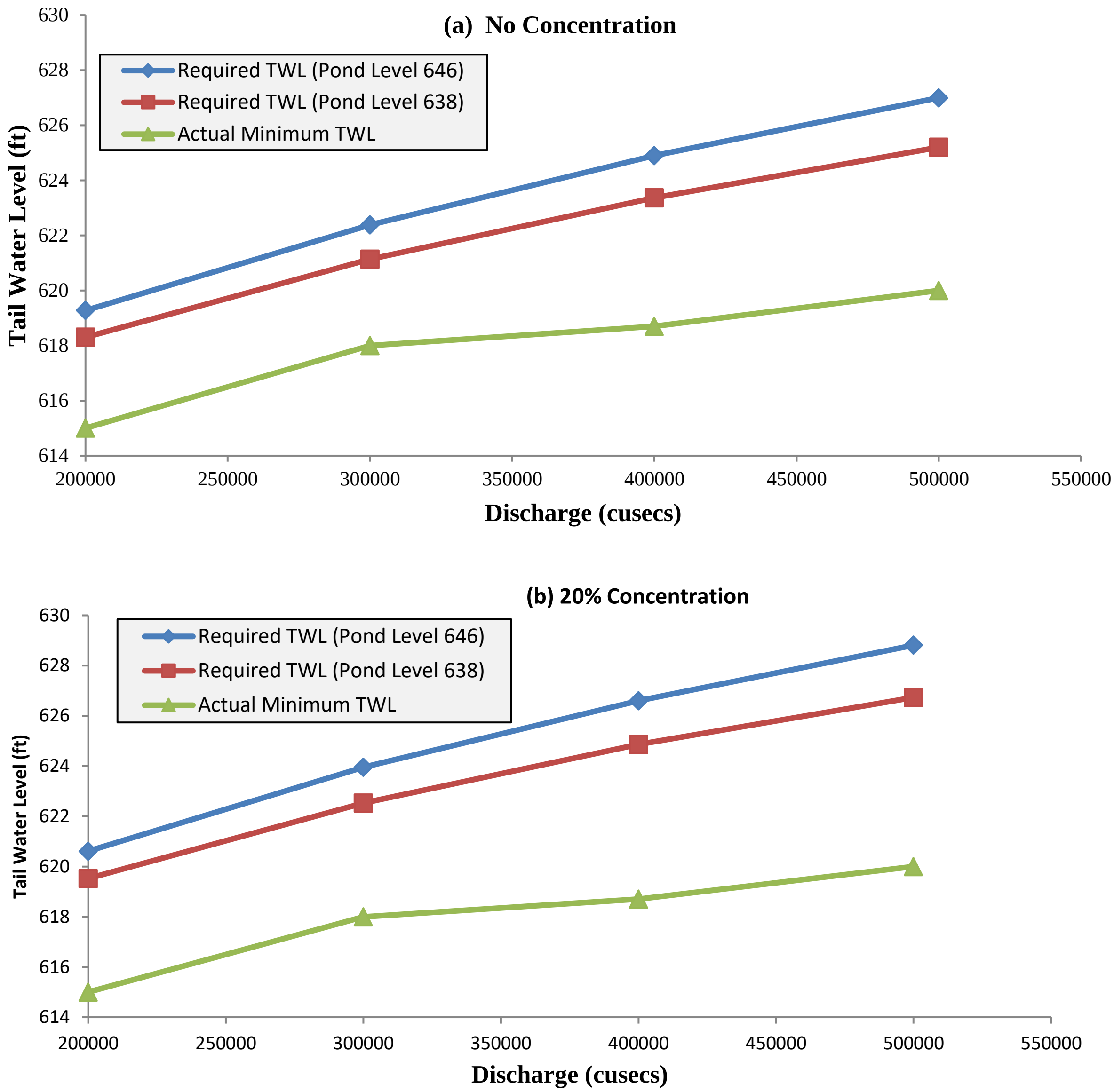

Figure 4: Comparison of Required and Existing Tail Water Levels (TWL) for various Discharges Passed at Different Pond Levels

5 CONCLUSIONS

It is now necessary to find a permanent solution to the unsatisfactory performance of stilling basin, to minimize the recurring damages to apron areas, and to neutralize the riverbed degradation due to release of un-dissipated energy and high velocity currents below the barrage. The available options are:

(i) *Deepening the Stilling Basin:* If the stilling basin was to be re-designed, its floor will have to be set at considerably lower level to provide the required conjugate depth in accordance with existing tail water conditions. This will require dismantling of existing floor, impact blocks, baffle blocks and concrete blocks over the inverted filter and relaying the same. This option will not be cost effective, difficult to implement and does not appear to be feasible, hence not recommended.

(ii) *Construction of a secondary weir:* This option with appropriate crest level and distance from existing Barrage center line tested on computer and thereafter suitable scale hydraulic model, would enable the required tail water level to be maintained for jump formation and energy dissipation, reduce damages to impact blocks, and concrete blocks and stone apron areas. It will also relax the restriction of adherence to 40 ft. head across limit and thus permit relatively higher pond levels to be maintained for increased energy generation.

(iii) *Construction of a second stage (or auxiliary) stilling basin:* Under this option a second stage stilling basin would be constructed below the existing stilling basin; the two stilling basins will jointly kill the energy of water and release it in tranquil manner to the river below. Its crest would be set at relatively lower level as it is not intended to raise the tail water levels because the function of killing the remaining energy (undissipated energy) will be performed by the second stage stilling basin.

To find an appropriate solution to the problem, it is recommended that a detailed study for remedial measures to improve performance of stilling basin, for options (ii) and (iii) may be initiated.

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