SURFACE FLOW HYDRAULICS OF TAUNSA BARRAGE: BEFORE AND AFTER REHABILITATION

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ABSTRACT*:* Taunsa barrage is one of the important diversion structures, built across Indus River. The barrage experienced ripping of downstream floor and damages to impact/friction blocks in 1959, just after one year of its completion. Similar damages were reported and subsequently got repaired in years 1962, 1986, 1994, 1998 and 2003. Excessive retrogression and consequently the sweeping of hydraulic jump were noted as the main reasons for the damages. Feasibility study on "Rehabilitation and Modernization of Taunsa Barrage" completed in Year 2005, proposed subsidiary weir with crest at EL424, to be constructed downstream of the barrage. The subsidiary weir location and crest level was finally fixed on the basis of physical model study carried by Irrigation Research Institute (IRI), Lahore, under the technical supervision of Punjab Barrages Consultants (Joint Venture of NDC-NESPAK in association with ATKINS Consulting Engineers, UK). Arguments have emerged regarding technical rationality of the provision of subsidiary weir downstream of Taunsa Barrage as rehabilitation structure. The surface flow analyses were carried out using computer software HEC-RAS to establish the location of hydraulic jump for various tail water level scenarios. This paper discusses the calibration of HEC-RAS model and the surface flow hydraulics of Taunsa barrage before and after its rehabilitation. The RAS model results indicate that under the existing condition (without subsidiary) the tail water depth was sufficient to develop hydraulic jump over the glacis.

Key Words: Jinnah barrage, rehabilitation and modernization project, subsidiary weir, two-step weir, HEC-RAS

INTRODUCTION

Recently billions of rupees have been spent on rehabilitation and modernization of Taunsa barrage. A subsidiary weir at about 900ft downstream of the barrage was constructed to raise tail water level. The existing concrete floor was overlaid by RCC slab and replenishment of loose stone apron was carried out.

Hydraulic performance of barrage, silt exclusion system and the subsidiary weir are yet not tested at higher discharges. Furthermore placing of hydropower complex within the barrage becomes difficult with the construction of subsidiary weir.

Feasibility Report Taunsa Barrage (2005) recommended subsidiary weir; whereas Chaudhry, 2008 noted that the hydraulic design of Taunsa barrage stilling basin was adequate. Surface flow analysis helps to establish whether the hydraulic deficiencies were existed at the Taunsa barrage.

Due to the advancement in computer software, studies of existing hydraulic structures with appurtenants such as gates, baffle and friction blocks become possible. The computer software HEC-RAS (Hydrological Engineering Center-River Analysis System) can be used to model gradually and rapidly varied flows. HEC-RAS is one dimensional model whereas the flow in stilling basins in such cases is three dimensional. The RAS model results give good comparison but the modular should have thorough understanding of the hydraulics of real flow problems.

Barrage Details: Taunsa barrage consisted of 53 weir bays; with clear span of 60 ft. The left and right undersluices are having 7 and 4 bays, respectively with the clear span of 60 ft. Barrage width between abutments is 4346 ft, whereas the waterway for weir and undersluices sections is 3862 ft. Crest and floor levels for the weir and undersluices are at EL428, EL416 and EL425, EL413, respectively.

Two divide walls bifurcate weir and undersluices sections of the barrage. In left and right undersluices, two fish ladders are provided alongwith the divide walls. Taunsa barrage has 22 ft wide navigation bay and silt exclusion system in its right and left undersluices, respectively.

Trimmu-Panjnad Link canal and Muzaffar Garh canal off-take from the right flanks of the barrage, withdrawing 12000 cusec and 8300 cusec, respectively. From right flank the D.G Khan canal having capacity of 8900 cusec, off-takes. Maximum pond level and the highest tail water level at the barrage were taken at EL446 and EL444, respectively.

Modeling Flow using Computer Software HEC-RAS: Real challenge in this modeling was to develop flow conditions replica of the corresponding flow on prototype structure. The emphasis was to develop water surface profile, with special reference to jump location.

For this purpose one complete bay and two half bays (134 ft) along with piers, gates and other structural arrangements, were modeled using HEC-RAS software.

Barrage upstream water level at gated control flow was maintained at about EL446 by controlling gate opening. For ungated flow the upstream water level were maintained at observed/projected level by adjusting river bed slope and Manning's roughness coefficient. The tail water level was established by changing river bed slope/elevation. Computer model so developed was modified, to model subsidiary weir.

Energy Dissipation System at Taunsa Barrage: Taunsa barrage energy dissipation system consisted of 77 ft long stilling basin, two rows of baffle/impact blocks and two rows of friction blocks (Chaudhry 2009). The baffle/impact blocks were placed at 10 ft from the toe of glacis, whereas the friction blocks were provided instead of the end sill wall. The depth between crest and downstream floor was 12 ft before rehabilitation and becomes 11 ft after the barrage rehabilitated.

During rehabilitation the 2 ft top layer of existing concrete floor on downstream glacis and stilling basin was removed and 3 ft thick reinforced concrete floor of 4000 psi was overlaid. The baffle/impact blocks were replaced with chute blocks and end sill. A subsidiary weir was constructed at 900 ft from the barrage crest as the main rehabilitation structure.

Retrogression and Water Level Variation: Feasibility Report Taunsa Barrage (2005) noted that the tail water level retrogressed by about 4 ft and 7 ft, under gated and ungated flow, respectively. Taunsa Barrage (2005) also noted that the tail water level at the design discharge (1000000 cusec) was EL444; consequently the prevailing water level, considering retrogression of 7 ft shall be EL437, whereas the tail water level maintained in the physical model study was EL433, (Hydraulic Model Study 2005).

Chaudhry (2009) reviewed the tail water level variation at Taunsa Barrage. Tail water levels at the Taunsa barrage were computed using tail water level data at the Jinnah barrage. The same tail water levels were used in RAS modeling to establish the location of hydraulic jump.

Surface Flow Hydraulics before Rehabilitation: Surface flow analysis was carried out at weir section of the barrage. RAS model showed that the jump remained on glacis, which ascertained that RAS precisely modeled the location of hydraulic jump. Model results showed that the jump remained well on glacis under gated and ungated flow (Figures 1-4), for the tail water levels computed using data at Jinnah barrage (Chaudhry 2009). The location of hydraulic jump was also studied for maximum retrogressed level EL 437 at the discharge of 1000000 cusec. The hydraulic jump still remained on the glacis (Figure 5) and jump terminated over the paved floor.

The downstream velocity increased with the increase in discharge (Table 1) but the velocity remained well within acceptable limits. The Froude number remained less than 0.40, indicating subcritical flow even at the design discharge.

Hydraulics of the stilling basin was also studied developing tail water level EL 433.40 (Figure 7). Result showed that the super critical flow remained up to a distance of about 450 ft from the barrage crest. The results indicated that such low tail water levels at higher discharges were practically not feasible at the prototype under any condition and may be mistakenly used in the physical model study. The provision of rehabilitation structure (Subsidiary weir) on the basis of such model study in which unrealistic tail water levels were considered is not having any hydraulic justification.

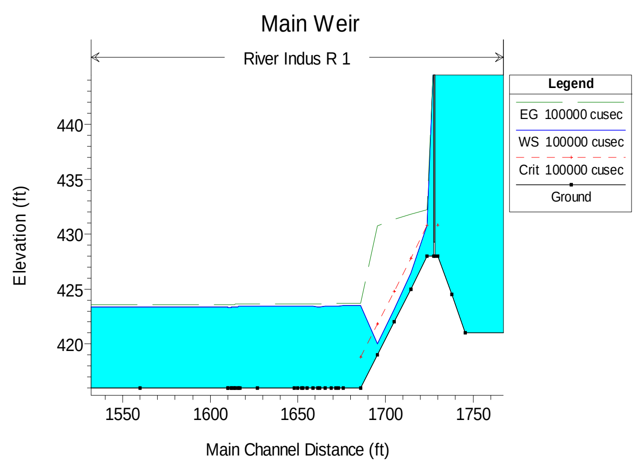

Fig. 1 Water surface profiles at weir section, for the discharge of 100000 cusec.

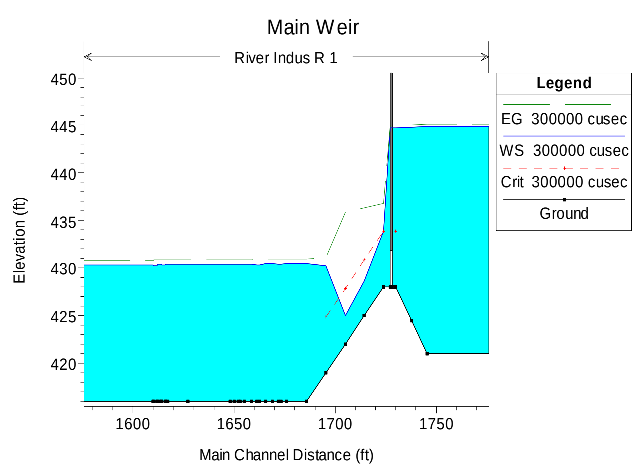

Fig. 2 Profiles at weir section, for the discharge of 300000 cusec.

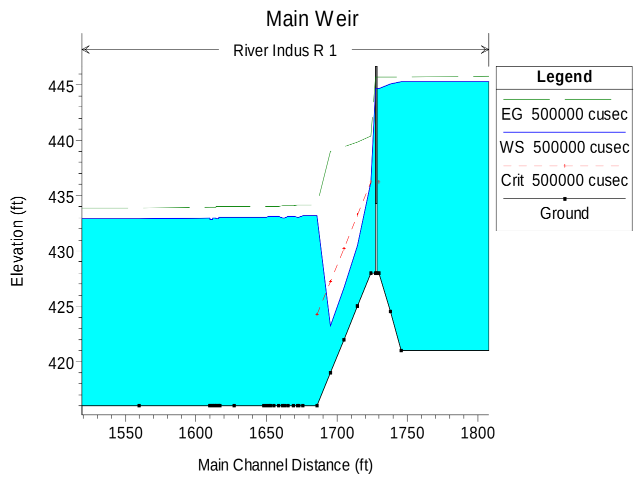

Figure 3 Profiles at weir section, for the discharge of 500000 cusec.

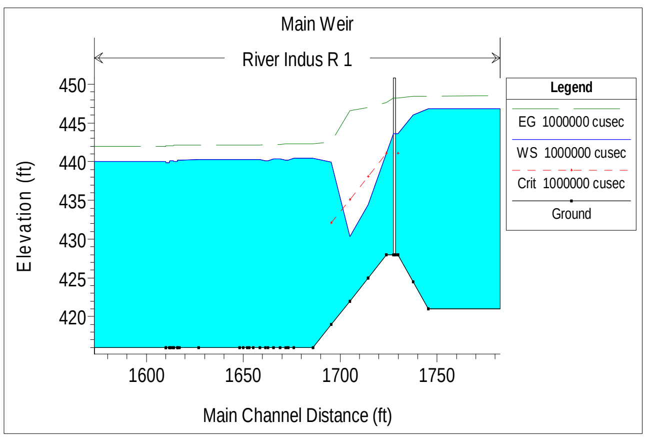

Figure 4 Profiles at weir section, for the discharge of 1000000 cusec.

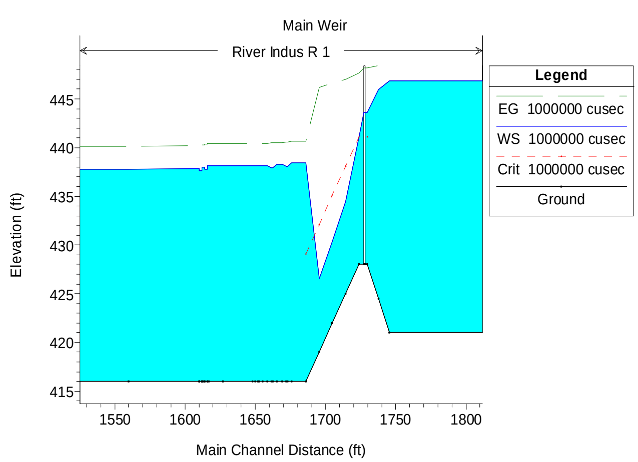

Figure 5 Profiles at weir section, for the discharge 1000000 cusec, (TWL EL437.70).

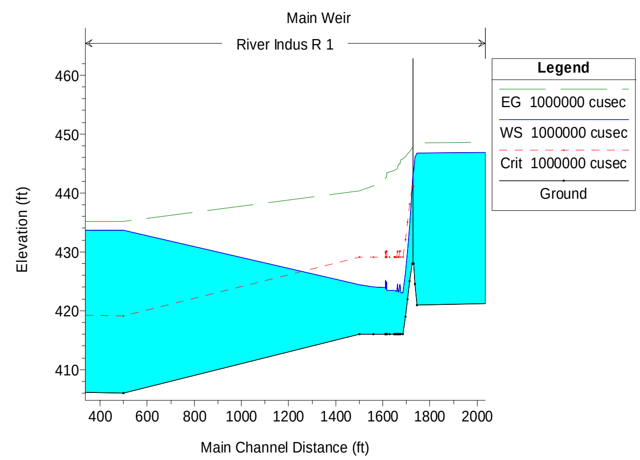

Figure 6 Profiles at weir section, for the discharge 1000000 cusec, (TWL EL433.4).

Table 1 RAS model results at weir section of the barrage.

2 Discharge cusec	At jump initiating point			TWL maintained	At jump termination	
	Water level	Velocity ft/sec	Froude number		Velocity ft/sec	Froude number
100000	420.00	26.28	4.58	423.43	3.62	0.23
300000	425.00	26.44	2.67	430.31	5.59	0.26
500000	426.70	28.64	2.33	433.80	7.55	0.32
1000000	430.29	32.41	1.98	440.14	11.12	0.40
1000000	426.55	35.58	2.28	437.70	12.95	0.50
1000000	423.01	38.32	2.55	433.50	36.42	2.36

Surface Flow Hydraulics after the Rehabilitation: RAS model developed for the weir section of the barrage was extended to incorporate subsidiary weir at a distance 900 ft, with crest EL424. Gate opening for various discharges, roughness coefficients, weir and orifice coefficients, river bed slope and retrogression were kept the same in both the models.

Model results showed that downstream water depth became higher, consequently the hydraulic jump moved up over the glacis (Figures 7-9). It was noted that upstream Froude number became less than 2.5 for most of the discharges indicating weak and unstable hydraulic jump to be developed. Froude number at jump initiating point further reduced at higher discharges and became 1.63 and 1.85 at the discharge of 500000 and 1000000 cusec, respectively. A series of small rollers developed in these scenarios, but the downstream water surface remain smooth.

Comparison of velocity with and without subsidiary weir revealed that decrease in velocity at the discharge of 500000 and 1000000 cusec was just 0.5 ft/sec and 0.36 ft/sec, respectively (Table 2). Similarly, the corresponding increase in water depth was 1.34 ft and 1.05 ft, indicating that the existing tail water levels were adequate. Figures 8 and 9 also showed that the subsidiary weir will be submerged and may develop hurdle at the higher discharges.

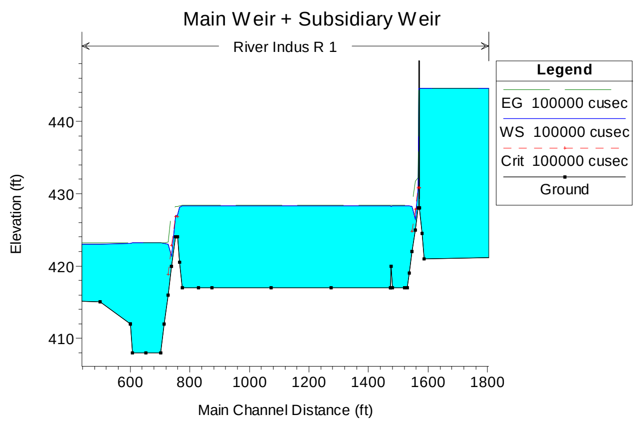

Figure 7 Profiles at the discharge 100000 cusec, after rehabilitation.

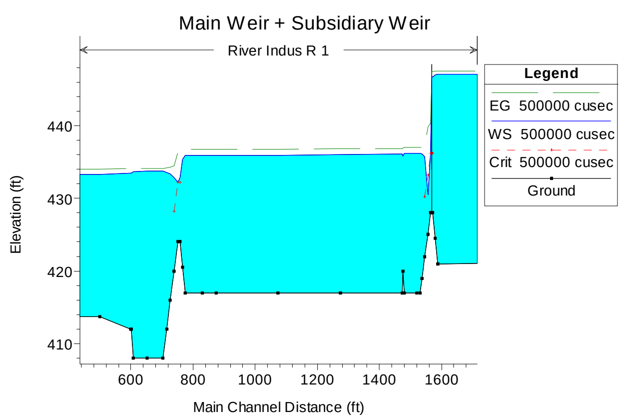

Figure 8 Profiles at the discharge of 500000 cusec, after rehabilitation.

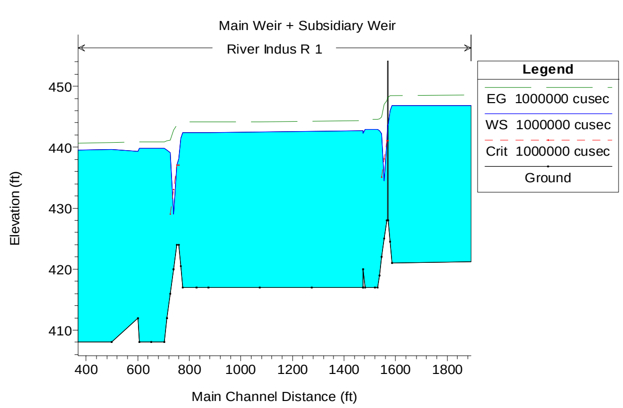

Figure 9 Profiles at the discharge of 1000000 cusec, after rehabilitation.

Table 2 RAS model results with subsidiary weir crest EL 424.

Discharge cusec	At jump initiating point			At jump termination			Increase in Water Depth Upstream of Subsidiary Weir
	Water level	Velocity ft/sec	Froude number	TWL observed	Velocity ft/sec	Froude number	
100000	426.45	18.57	2.72	428.31	2.38	0.12	2.01
300000	428.62	22.23	2.06	432.75	5.11	0.23	1.65
500000	430.47	24.54	1.85	436.04	7.05	0.28	1.34
1000000	434.44	28.45	1.63	442.67	10.46	0.36	1.05

Conclusions HEC-RAS model results for the Taunsa barrage were in good agreement with the physical model observations. RAS model results revealed that under gated and ungated flow the jump remained well on glacis. Downstream velocity remained within acceptable limits conforming that the loose stone apron remained safe at the Taunsa barrage.

RAS model results with subsidiary weir showed that the downstream water depth became higher, consequently the hydraulic jump moved up over the glacis. It was noted that the Froude number remained less than 2.5 for most of the discharges indicating weak and unstable hydraulic jump to be developed. Jump became undular at the discharge of 500000 and 1000000 cusec as upstream Froude number became 1.63 and 1.85, respectively.

Comparison of velocity with and without subsidiary weir revealed that the decrease in velocity at the discharge of 500000 and 1000000 cusec was 0.5 ft/sec and 0.36 ft/sec, respectively. Similarly, the corresponding increase in water depth was just 1.34 ft and 1.05 ft, indicating that at higher discharges the subsidiary weir will be submerged. Surface flow analysis confirmed that the hydraulic design of Taunsa barrage was adequate and the provision of an isolated structure (subsidiary weir) at 900 ft from the barrage crest is having hardly any justification.

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