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Hydraulics of Jinnah Barrage; Existing Structure and Rehabilitation Alternatives

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Abstract

Jinnah barrage has passed more than seventy years of its useful life and is facing some structural and hydraulic problems. Irrigation & Power Department, Government of Punjab, Pakistan engaged Evaluation Consultants (EC) in the Year 1998 to review physical health of the Jinnah barrage. Evaluation consultants noted that the energy dissipation problems persist downstream of the barrage.

Feasibility study on "Rehabilitation and Modernization of Jinnah Barrage" completed in Year 2005, proposed subsidiary weir with crest at EL676, to be constructed across the river, at a distance of about 800 ft, downstream of the barrage to address energy dissipation problems. The crest level of proposed subsidiary weir (EL676) is lower by 2 ft and higher by 1 ft as compared with crest level of weir and undersluices sections of the barrage, respectively. The proposed structure if constructed, adversely affect the hydraulic performance of the barrage. A detailed surface flow analysis was carried out using computer software HEC-RAS for the existing structure, the proposed subsidiary weir and its alternatives, to optimize rehabilitation works. This paper discusses the calibration of HEC-RAS model, the surface flow hydraulics of subsidiary weir and its alternatives.

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

1. Introduction

For successful rehabilitation and modernization of an existing hydraulic structure, precise identification of hydraulic problems is of paramount important [1, 2]. Otherwise the huge investment may go partially or completely in waste. Recently billions of rupees have been spent on rehabilitation and modernization of Taunsa barrage by constructing a subsidiary weir at about 800ft downstream of the barrage. The existing concrete floor was overlaid by RCC slab and replenishment of loose stone apron was carried out. The performance of barrage, silt excluders, and rehabilitation structures are yet not tested at higher discharges. Furthermore, the placing of hydropower complex between the barrage and subsidiary weir is almost not feasible; consequently the option of hydropower development at the barrage is not viable.

Mahboob [3, 4] concluded that the design of Jinnah barrage is acceptable. The energy dissipation mechanism at the barrage under prevailing water level conditions was studied in detail [5, 6, 7 & 8]. Although the Feasibility Report [7] recommended subsidiary weir; whereas the numerical studies do not reflect the hydraulics of the existing structure; rather depict barrage designing as per USBR design guidelines under various design scenarios.

2. Barrage Details

Jinnah barrage consisted of 42 weir bays; two undersluices each consisting of 7 bays with clear span of 60 ft (18.3 m). Barrage width between the abutments is 3781 ft (1152.4 m), whereas the clear waterways for the weir and undersluices sections are 2520 ft (768.1 m) and 420 ft (128 m), respectively. Weir and undersluices crest and floor levels are at EL678, EL670, EL675 and EL667, respectively.

Due to the advancement in computer software, the 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 in channels. The HEC-RAS is one dimensional model whereas the flow in stilling basin in such cases is three dimensional. The HEC-RAS model results give good comparison but the modular should have thorough understanding of the hydraulics of real flow problems.

Two divide walls 350 ft (106.7 m) long), bifurcate weir and undersluices sections of the barrage. In left and right undersluices, two fish ladders are provided along the divide walls.

Jinnah barrage has a 20 ft (6.1 m) wide navigation bay and silt exclusion system in its right and left undersluices, respectively. The barrage is designed for a flood of 950,000 cusec (26725.6 cumec); however, a flood of 1,100,000 cusec (30945.8 cumec) can be passed as the barrage guide banks have enough freeboard. Normal pond level is at EL692, which will get raised at EL694 to meet 10,000 cusec (283.2 cumec) of remodeled capacity of Thal canal.

3. Energy Dissipation Mechanism at Jinnah Barrage

Jinnah barrage energy dissipation system consists of 70 ft long stilling basin, two rows of baffle/impact blocks and two rows of friction blocks. The baffle/impact blocks are placed at a distance of 10 ft from the toe of glacis, whereas the friction blocks are provided instead of the end sill wall. If the downstream water depth becomes less that the conjugate depth the baffle/impact blocks direct water upward, consequently increasing water depth and reduces water velocity. This will help to stabilize and terminate the hydraulic jump over the paved floor. Friction blocks control water depth at low discharges, dissipate some of the energy and allow passing of gravel and pebbles. This arrangement is quite efficient at low flows and is not very sensitive to downstream water depth [6]. The Froude number before the jump remains within 2.0 to 4.5 for most of the discharges as shown in Table 1. The Froude number after the jump becomes less than 0.5 even at higher discharges indicating subcritical flow conditions. Chaudhry [2] observed that the jump swept but baffle/impact blocks help to terminate the jump over paved floor.

4. Modeling of Flow using Computer Software HEC-RAS

Real challenge in this modeling was to develop flow conditions replica of the corresponding flow on the prototype structure. The emphasis was to develop water surface profile, with special reference to jump location and energy dissipation. For this purpose one complete bay and two half bays (134 ft) along with piers, gates and other structural arrangement were modeled using HEC-RAS software. The longitudinal sections for the barrage and various alternative rehabilitation scenarios are shown in Figure 1.

Barrage upstream water level at gated control flow was maintained at EL694 by controlling gate opening. For higher discharges (ungated flow) the upstream water level were maintained at observed/projected level by adjusting river bed slope and Manning's roughness coefficient. The downstream water level was adjusted at prevailing level by changing river bed slope, roughness coefficient and retrogression. The basic computer model (model for the existing structure) so developed was upgraded, to model subsidiary weir and its alternatives for various design scenarios.

5. Surface flow Hydraulics of the Existing Structure

The surface flow analysis was carried out both at weir and undersluices sections of the barrage. Stilling basin alongwith baffle/impact and friction blocks were modeled and the analysis was carried out, for various discharges under the prevailing water level conditions **(Fig 1A & 1B)**.

The results indicate that under gated control flow the jump swept at weir section of the barrage and downstream water depth became less than the conjugate depth. The baffle/impact and friction blocks help to stabilize and terminate the jump on the paved floor **(Figure 2 & 3).** At higher discharges the jump seems to be developed on the glacis and downstream Froude number remains less than 0.5. The downstream velocity increases with the increase in discharge **(Table 1)** and became greater than the limiting velocity (9 ft/sec) which initiate the displacement of loose stone apron.

Barrage undersluices perform better as compared with the weir section. The jump developed on glacis of the undersluices and downstream Froude number remained less than 0.4 **(Table 2).** The hydraulic jump as per USBR guidelines is characterized as low Froude number jump.

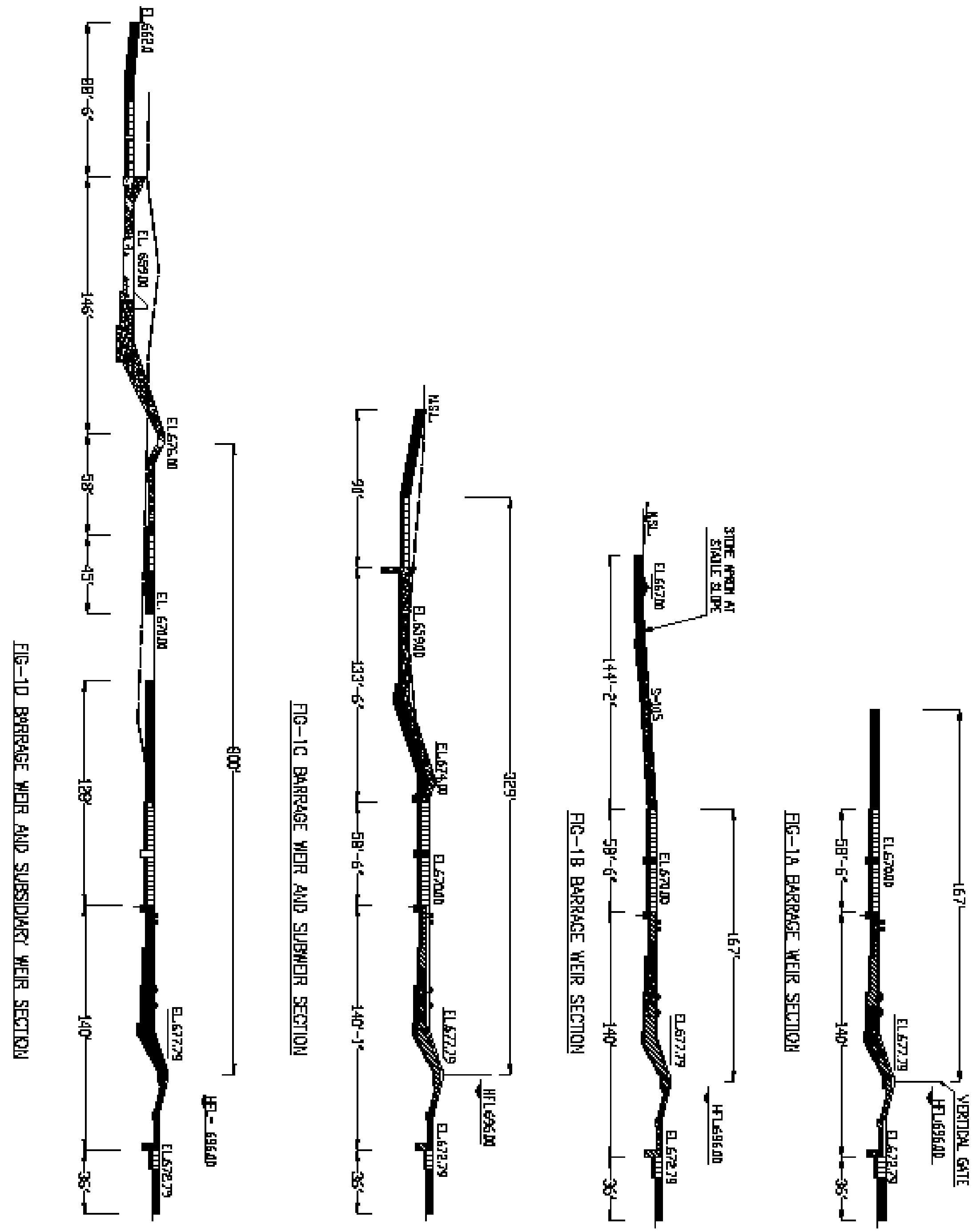

Figure 1: Cross Sections through barrage and various rehabilitation scenarios

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

Discharge cusec	Water level U/S of barrage ft	At jump initiating point		After the jump		
		Water level	Froude number	Water level	Velocity ft/sec	Froude number
100000	694.0	671.1	4.26	675.2	5.16	0.40
200000	694.0	672.1	3.10	677.6	6.98	0.45
300000	694.0	673.0	2.78	679.9	8.08	0.45
500000	694.0	674.3	2.61	683.4	9.98	0.48
842000	694.6	679.2	2.06	688.6	12.16	0.50
950000	695.9	679.9	2.01	690.7	12.29	0.48

Table 2 RAS model results of stilling basin at undersluices sections of the barrage.

Discharge cusec	Water level U/S of barrage ft	At jump initiating point		After the jump		
		Water level	Froude number	Water level	Velocity ft/sec	Froude number
100000	694.17	672.3	3.07	674.66	3.98	0.27
200000	693.96	673.4	2.52	677.37	5.14	0.28
300000	693.93	674.4	2.27	679.9	6.19	0.30
500000	694.07	676.13	2.02	683.26	8.20	0.36
700000	690.79	677.70	1.88	685.35	10.17	0.42
842000	692.78	681.75	1.53	688.78	10.30	0.39
950000	694.19	689.19	0.83	691.30	10.42	*0.37*

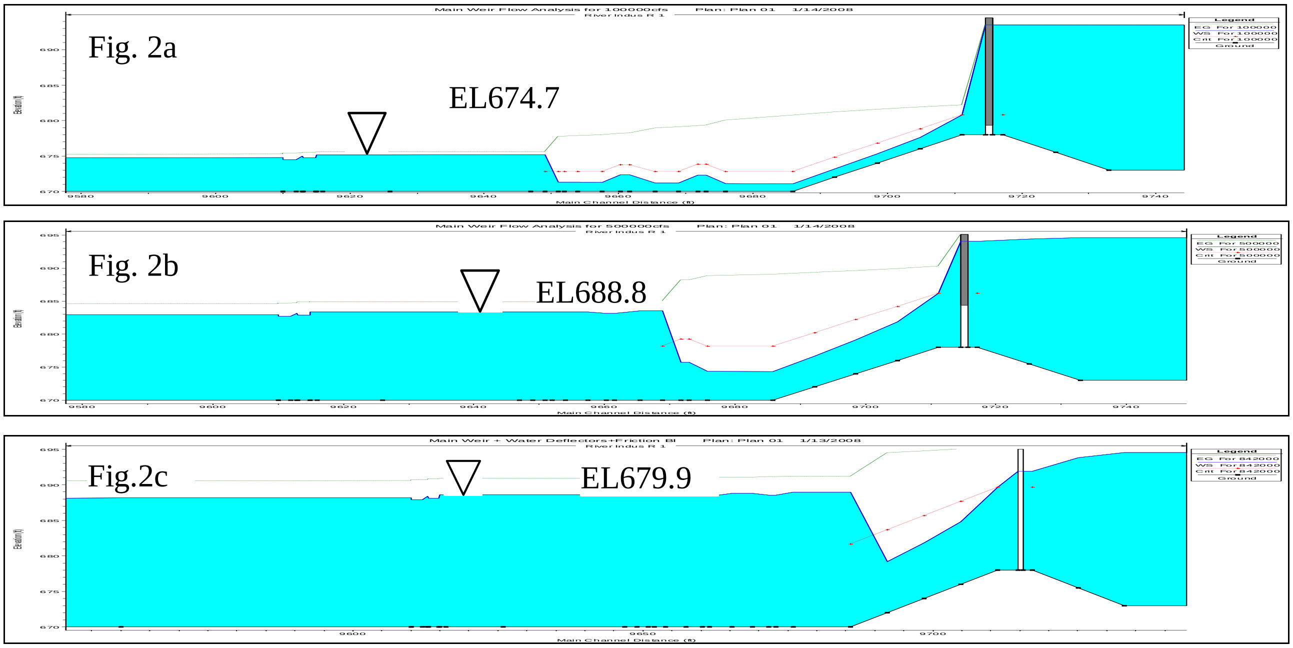

Figure 2 Water surface profiles at weir section, for the discharge of 100000, 500000 and 8420000cusec, respectively

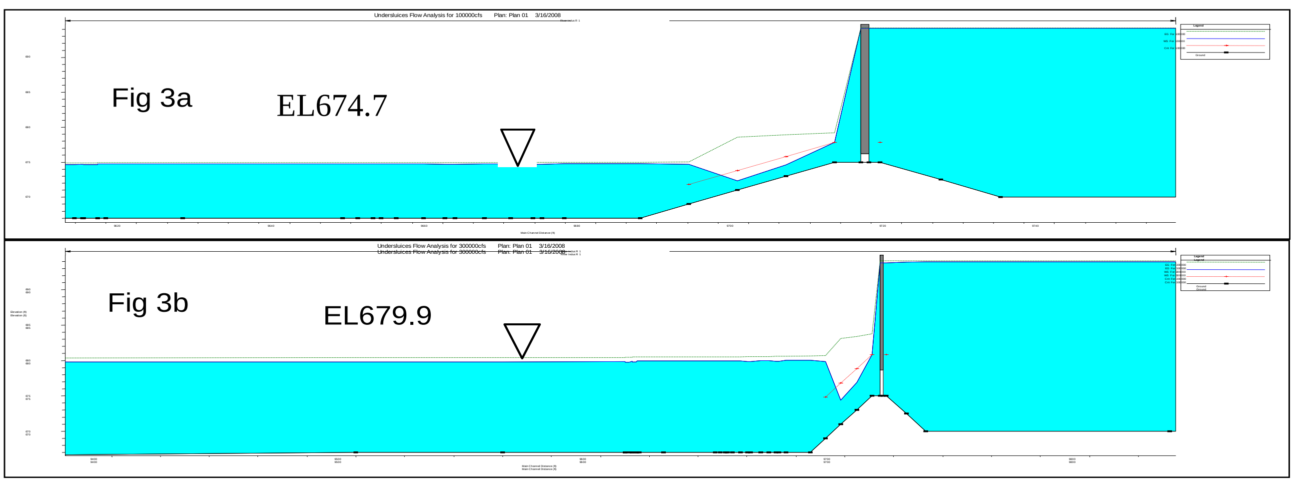

Figure 3 Water surface profiles at undersluices section, for the discharge of 100000 and 300000cusec, respectively

6. Surface Flow Hydraulics of the Subsidiary Weir

The HEC-RAS model developed for the existing barrage structure was modified to incorporate subsidiary weir at a distance of 800 ft with crest EL676 and EL675, (Fig 1D). Gate openings 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 the downstream water depth became higher than the conjugate depth, consequently the hydraulic jump moved up over the glacis (Table 3 & 4, Figure 4). It was further noted that the upstream Froude became less than 2.5 for most of the discharges indicating weak and unstable hydraulic jump to be developed. The jump became undular at the discharge of 842,000 cusec. Velocity downstream of the jump remained greater than 9 ft/sec at higher discharges, which indicate that the proposed subsidiary cannot control the displacement of loose stone apron downstream of the barrage.

Table 3 RAS model results of the barrage with subsidiary weir crest EL 676.

Discharge cusec	Water Level u/s Barrage	At jump initiating point			At jump termination		
		Water level	Velocity ft/sec	Froude number	Water Level	Velocity ft/sec	Froude number
100000	694.0	677.6	16.63	2.31	680.41	2.61	0.14
200000	694.0	678.8	18.84	1.97	682.72	4.22	0.21
300000	694.0	679.9	20.42	1.81	684.70	5.48	0.25
500000	694.0	681.3	22.78	1.65	688.01	7.45	0.31
842000	695.7	690.3	18.42	1.00	692.61	10.00	0.37
950000	697.17	690.9	19.69	0.96	693.90	10.67	0.38

Table 4 RAS model results of the barrage with subsidiary weir crest EL 675.

Discharge cusec	Water Level u/s Barrage	At jump initiating point			At jump termination		
		Water Level	Velocity ft/sec	Froude number	Water Level	Velocity ft/sec	Froude number
100000	694.0	675.3	20.1	3.01	678.3	3.24	0.20
200000	694.0	676.4	22.2	2.52	680.3	5.04	0.27
300000	694.0	677.4	23.7	2.27	683.7	5.90	0.28
500000	694.0	679.2	26.0	2.02	686.9	7.93	0.34
842000	695.7	684.8	25.7	1.53	691.5	10.53	0.40
950000	697.17	685.6	26.43	1.50	692.7	11.21	0.41

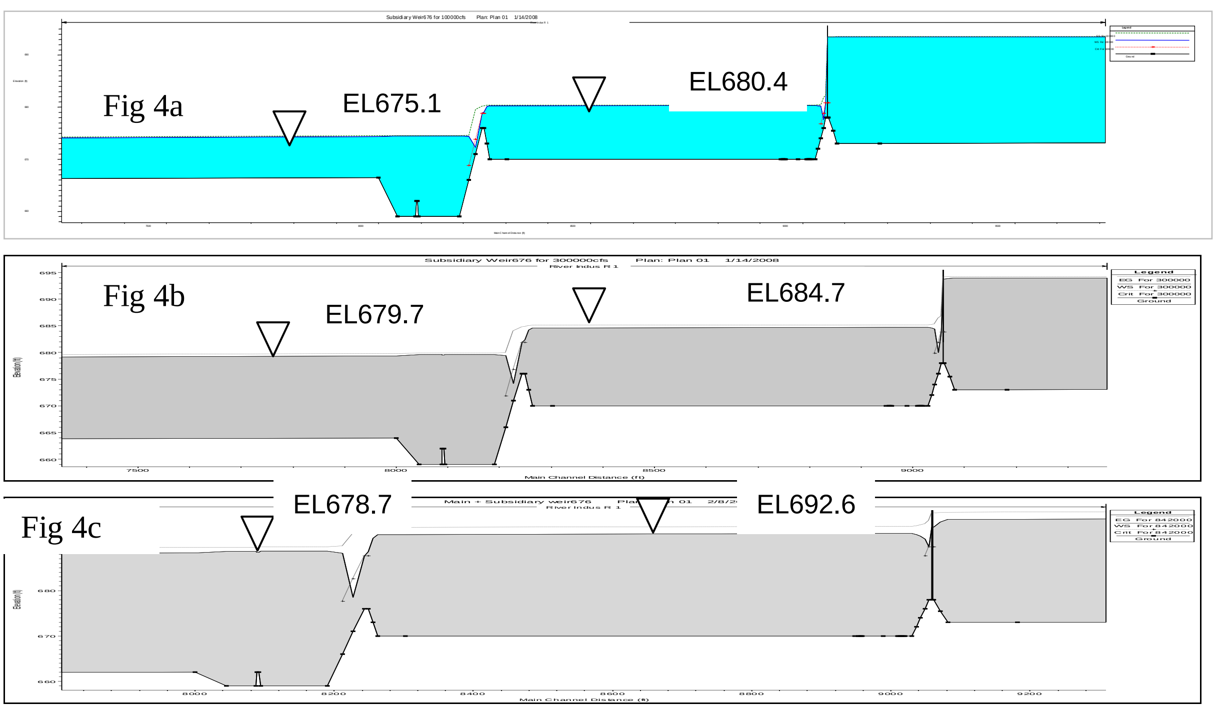

Figure 4 Water surface profiles with subsidiary weir crest EL676, for the discharge of 100000, 300000 and 842000 cusec, respectively.

The analysis further revealed that the hydraulic control was shifted at the subsidiary weir as its crest level was higher than the corresponding crest level of the undersluices. The discharging capacity of the undersluices and efficiency of silt exclusion system would be affected badly and existing guide banks have to be raised to pass the design flood of 950,000 cusec.

7. Surface Flow Hydraulics of Two-step Weir

An alternative to subsidiary weir, the two-step weir is proposed with crest EL674 in weir section of the level of the second stilling basin was kept at EL659, which was the same as of the subsidiary weir already discussed. The existing stone apron downstream of barrage was replaced by concrete block floor having inverted filter underneath. This barrage just downstream of the existing stilling basin (Figure 1C). The proposed and existing structures act hydraulically and structurally integral components of each other. The downstream floor will make the two-step weir an integral part of the existing structure. This arrangement allows the release of seeping water and control uplift pressure under the barrage at the prevailing level.

The HEC-RAS analysis showed **(Figure 5)** that the hydraulic jump was developed over the barrage glacis for all the discharges. A second jump developed over the glacis of the two-step weir and dissipates part of the remaining energy. The downstream velocity became 7 ft/sec as compared with the prevailing velocity of 12.2 ft/sec for the discharge 842,000 cusec **(Table 1 & 5).** The possibility of displacement of loose stone apron at higher discharges became minimal with two-step stilling basin arrangement.

Table 5 HEC-RAS model results for Two-step basin crest EL674

Discharge cusec	U/S OF JUMP			D/S OF JUMP		
	Water Level	Velocity ft/sec	Froude Number	Water Level	Velocity ft/sec	Froude Number
100000	675.34	20.08	3.06	678.20	3.27	0.20
	D/S of two-step basin			674.50	1.73	0.08
200000	676.42	22.21	2.52	680.51	5.11	0.28
				677.19	2.95	0.12
300000	677.39	23.74	2.27	682.39	6.50	0.33
	D/S of two-step basin			679.67	3.90	0.15
500000	676.69	28.60	2.33	685.53	8.64	0.39
	D/S of two-step basin			683.33	5.52	0.20
842000	681.84	28.83	1.81	690.15	11.22	0.44
	D/S of two-step basin			689.90	7.32	0.23

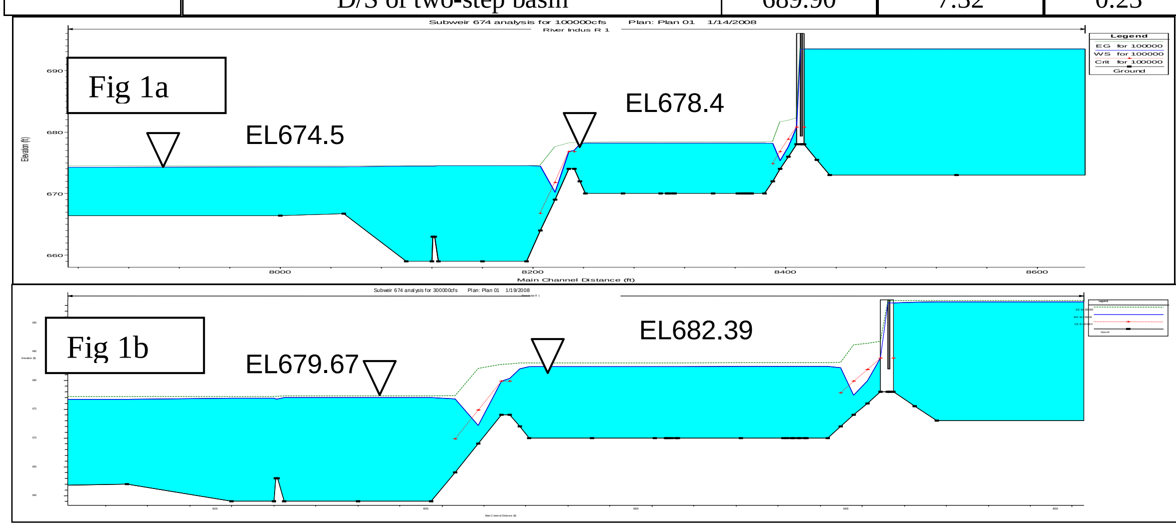

Figure 5 Profiles developed with Two-step basin, crest EL 674, for the discharge of 100000 and 300000 cusec, respectively.

8. Conclusions

HEC-RAS model results for the Jinnah barrage were in good agreement with the prototype observations. Analysis of the existing structure with HEC-RAS model revealed that under gated control flow the jump swept and developed on the horizontal floor whereas at higher discharges it shifted on the glacis. The baffle/impact and frictions blocks help to stabilize and terminate the hydraulic jump over paved floor.

Model showed that the subsidiary weir arrangement with crest EL676 submerged the barrage at higher discharges and hydraulic control shifted at the proposed structure. The displacement of loose stone apron downstream of the barrage cannot be completely eliminated with the provision of subsidiary weir as the velocity remained greater than 9 ft/sec at higher discharges. The lowering of crest elevation below EL676 further increases the velocity upstream of subsidiary weir and consequently the loose stone apron displacement will be more.

The two-step weir arrangement with crest EL 674 shifted the jump on the glacis at low discharges whereas at higher discharges the water level risen downstream of the barrage remained within acceptable limits. Furthermore the loose stone apron downstream of two-step weir was proposed at EL659, as compared with the existing level EL670; therefore the repeated replenishment of loose stone shall be minimized. Two-step weir arrangement if constructed shall not change the flow conditions at undersluices, the working of fish ladders and silt exclusion system.

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9. References

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