Pak. J. Engg. Appl. Sci. Vol. 33 July, 2025 (p. 17–25)

Sub Weir Influence on Barrage Tailwater using FLOW-3D Hydraulic Models

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Abstract

Barrages are used to divert the required water into canals and their design is said to be compromised if the required discharging capacity is reduced. In 2008, to restore tailwater levels, a sub weir was constructed downstream of Taunsa barrage, Pakistan. However, during 2010, the barrage didn't pass its design flow and consequently its left marginal bund was breached. This study aims to investigate the effects of sub weir on the tailwater level, water heads, discharge coefficient and drowning ratio using FLOW-3D hydraulic models. The results indicated a maximum 15% rise in head water which further revealed an overestimation of water heads over the prototype sub weir. The analysis further indicated that from 12.09 to 24.19 m3/s/m discharge, the rate of change of waterheads over the sub weir was found to be declined. Upon comparison, the simulated coefficients of discharge were found to deviate from the observed discharge coefficient and the maximum error reached 36 % at 12.09 m3/s/m. The findings of this study highlight the necessity of thoroughly assessing the design, operational feasibility, and hydraulic performance of such weirs across various discharge conditions to ensure their sustainability, particularly during extreme flood events.

Keywords: Subsidiary weir; Tailwater; Water head; Discharge coefficient; Drowning ratio

1. Introduction

In Pakistan, barrages play vital role in economy and are built based on the physical models' studies. They are constructed across the rivers to raise the upstream water levels for supplying the required amount of water to fulfill agricultural demands. Taunsa barrage, Punjab on mighty river Indus was built about 66 years ago for the design discharge capacity of 28,313 m3/s. The stilling basin of the barrage was a modified form of USBR Type-III basin which included two staggered rows of baffle and friction blocks. After the first operation in 1958, multiple issues, i.e., uprooting of baffle blocks, retrogression of riverbed and lowering of tailwater levels appeared on the barrage downstream. Additionally, [1], [2][3], [4] reported that due to the lowering of tailwater levels, hydraulic jumps (HJs) were found to be sweeping on the downstream floor which reasoned to be the uprooting of energy dissipators. To resolve these issues, based on the reduced scale model of 1:50, the stilling basin of Taunsa barrage was remodeled from the year 2005 to 2008. Under remodeling, the old basin's appurtenances were replaced with chute blocks and end sill. Additionally, a sub weir was also constructed about 274.30 m away from the crest of the main weir. On the contrary, to stabilize the HJ and its location, previous studied have utilized various baffle blocks, i.e., rectangular [5], [6], curved [7], T-shaped [8], triangular [9], and wedge-shaped baffle block [10]–[12]. Such devices control the HJs in case of less tailwater depths [13], [14], and reduce local scouring on the downstream [15].

Conventionally, for any new intervention and remodeling of the old hydraulic structures, i.e., sluice gate weirs and energy dissipators, reduced scale and laboratory models are developed, and their performance is assessed with USBR Monograph No. 25. The performance of these structures is said to be compromised; if the discharging capacity is reduced; HJs are not contained within the basins, and energy dissipation is reduced [13], [14], [16]. However, physical and laboratory models are considered to be expensive and time consuming. Additionally, due to the use of measuring devices and scaling effects, the results of desired output parameters are also affected in the reduced scale models. In contrast, with the advancement in computer technology and more efficient turbulence schemes, computational fluid dynamic (CFD) is widely used in hydraulic investigations of various graded control structures, i.e., dams, spillways, barrages, weir, canal, and sluice gates [17]–[19]. Therefore, for the last three decades, the use of CFD models in the design, testing and operation of hydraulic structures is becoming prevalent.

Many numerical codes such as Open Foam [20]; [21], ANSYS Fluent [22], REEF3D [23] and FLOW-3D [24] are employed to investigate the hydraulic jump and flow characteristics but herein the most relevant are highlighted. Free surface profiles over the rectangular broad-crested weir was investigated by [25] using Fluent software. The results indicated that flow depths on the upstream and in the rapidly varied regions over the weir crest were found in good agreement with the experiments. FLOW-3D and HEC-RAS models were employed by [26] to investigate the flow characteristics on upstream and downstream of single-step broad-crested weir. Out of the used models, the results of upstream flow depths, flow in gradually varied regions and the nappe profiles were well predicted by FLOW-3D models.

From bibliographical data, it is found that even after the Tuansa barrage's remodeling, during the year 2010, the barrage didn't pass super flood of 23.22 m3/s/m. In addition, after the remodeling the river also changed its course toward the left side. In view of the abovementioned, it is very crucial to reassess the hydraulics of sub weir downstream of Taunsa barrage. Therefore, the present study developed FLOW-3D hydraulic models to investigate the free surface profiles on upstream and downstream sides of the sub weir for a wide range of discharges. Additionally, the study also investigated the values of discharge coefficients for main and sub weirs. For the present numerical models, free surface tracking and turbulence is captured by the Volume of Fluid (VOF) and Renormalization Group (RNG K-ε) models, respectively.

2. Material and Methods

2.1 Brief of Main and Sub-Weir

Taunsa barrage was commissioned during the year 1958 for the design discharge capacity of 28,313 m^3/s. The barrage had modified United States Bureau of Reclamation (USBR) Type-III basin which consisted of baffle and friction blocks. Due to the retrogression and lessening of tailwater levels, the barrage was remodeled in 2008 in which the baffle and friction blocks were replaced with chute blocks and end sill as shown in Figure 1 (a). In addition, a sub weir was also constructed about 274.30 m downstream of the main weir as illustrated in Figure 1 (b). The crest level of the main weir was fixed to 130.44 m while the downstream floor level was changed from 126.79 m to 127.10 m. On the contrary, the crest of sub weir and downstream floor was fixed to 129.23 m and 124.34 m, respectively.

2.2 Numerical Model Implementation

For the present FLOW-3D models, Navier Stokes Equations (NSEs), i.e., continuity and momentum, are employed to solve the computational domain. As compared to Direct Numerical Simulation (DNS) and Large Eddy Simulation (LES), Reynold Averaged Navier Stokes Equation (RANS) models [27], [28] are preferred. For modelling of free surface, Volume of Flud method (VOF) was employed as presented in Equation (1) below.

$$\frac{\partial F}{\partial t} + \nabla * (\bar{u}F) = 0 \tag{1}$$

where F, ∇, and v are the fluid fraction, divergence operator and fluid velocity in present the computation domain in which F=0 indicates that cell is empty while F=1 shows that cell is fully occupied by fluid. However, if the F value ranged between 0< F<1, the cell represents a surface between the two different fluids. In the present study, a single fluid (Water) with free surface is considered for which 0.5 value is assigned in each computation cell. A single structured rectangular hexahedral mesh block was used to resolve the solid and liquid domains. For the present models, the overall length, width, and height of mesh block reached 347 m, 18.29 m, and 13.5 m, respectively. To resolve the finer element of the solid geometry such as chute blocks, the maximum cell size within the mesh block in x, y and z direction was fixed to 0.17 m. The quality of mesh cell size was monitored through adjacent cell size and aspects ratios, and it was ensured that these values must remain within the suggested ranges of FLOW-3D guidelines. Table 1 shows the mesh quality indicators for present models while the model's operational conditions are provided in Table 2.

Table 1: Details of cell number and mesh quality indicators

Number of real cells	Maximum adjacent ratio			Maximum aspect ratio		
X=2135 Y=110 Z=79	X	Y	Z	X-Y	Y-Z	Z-X
	1.00	1.00	1.00	1.02	1.02	1.00

In FLOW-3D hydraulic models, the solution convergence and steady state of the models can be achieved by comparing volume flow rate and mass averaged fluid kinetic energies at the inlet and outlet boundaries. At present, the volume flow rates are monitored at the outlet boundaries and these values are compared with the initial condition.

Table 2: Operational conditions for the numerical models

discharge through Single Bay ($m^3/s/m$)	Tailwater Levels (2003) (m)	Operation
2.42	129.69	Free Flow
4.84	130.45	
7.26	130.91	
9.67	131.15	
12.09	131.39	
14.51	131.58	
16.93	131.73	
19.35	131.88	
21.77	132.00	
24.19	132.06	

(a)

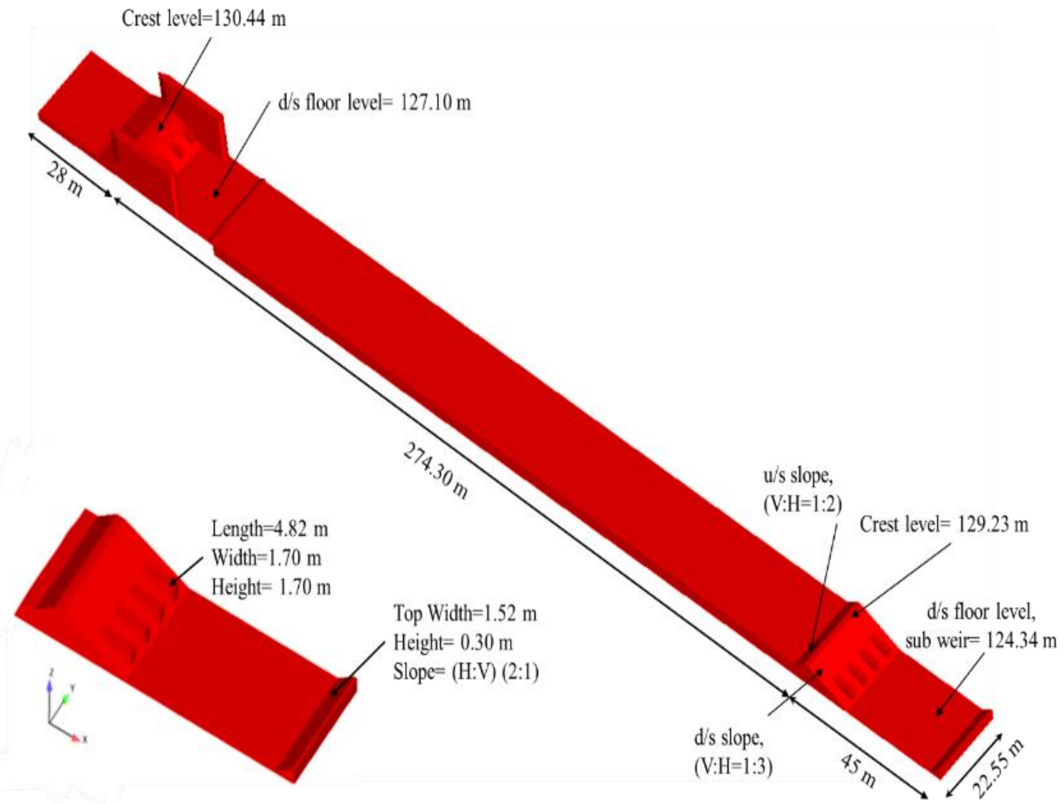


Fig. 1: (a) Remodeled Taunsa barrage stilling basin, **(b)** energy dissipation setup for sub weir

To reproduce actual conditions of the prototype barrage, volume flow rate (Q) was set for the upstream boundary (X_{min}) while downstream boundary (X_{max}) was set to specific pressure (P) with fluid elevation. It is essential to mention here that the fluid elevations at each discharge were given from the observed data for the year 2003 before remodeling of the barrage. On the lateral sides (Y_{min}, Y_{max}), no slip condition with zero tangential and normal velocity (u=v=w=0)) were imposed by setting the wall boundaries (W). The upper (Z_{max}) and lower (Z_{min}) boundaries were set to atmospheric pressure (P) and wall (W), respectively.

2.3 2Flow Analysis

To analysis the flow at the upstream of main and sub weirs, the following formula (2) is utilized to compute head above the weirs' crest (H (m)). measured in m.

$$H = (\frac{q}{c})^{2/3} \quad (2)$$

3. Result and Discussion

Using equation (2), free flow models were run at the available tailwater levels of the year 2003 and to assess the performance of sub weir, the existing barrage geometry was employed. The numerical model results were compared with field discharge measurements at different tailwater levels as shown in Figure 2. The analysis revealed that the model underestimated discharge at lower flow rates and overestimated it at higher flow rates. The errors ranged from -15.2% to -0.10%, highlighting variations in prediction accuracy based on discharge magnitude. To further assess the model's performance, statistical index such Mean Absolute Error (MAE), Root Mean Square Error (RMSE), Mean Bias Error (MBE), and Coefficient of Determination (R^2) were conducted between the modelled and the actual discharge values. On average, MAE showed that the numerical model deviates from the actual discharge by 5.44% while RMSE value reached to 6.99 % indicated slightly larger deviations in comparison to the MAE. On the other hand, MBE of -1.43% suggested that the model slightly underestimates discharge values. However, (R^2) of 0.990 indicated perfect correlation between prototype and simulated discharges, confirming the model's strong predictive accuracy.

3.1 Free Surface Profiles

Figure 3 shows the change of tailwater levels upstream of sub weir up to the design flow of 24.19 m3/s/m. At 2.42 m3/s/m, the model results indicated that sub weir raised water head up to 0.78 m while at 4.84 m3/s/m, the head above weir crest was found

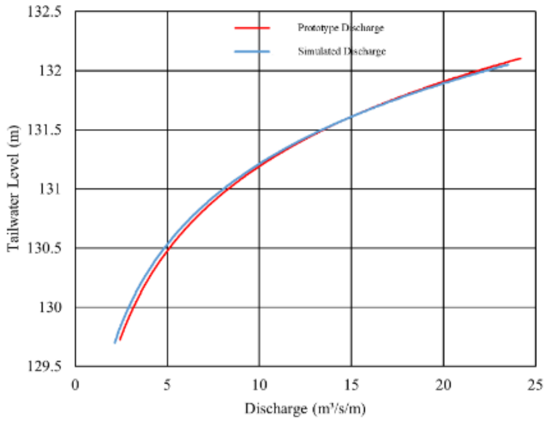
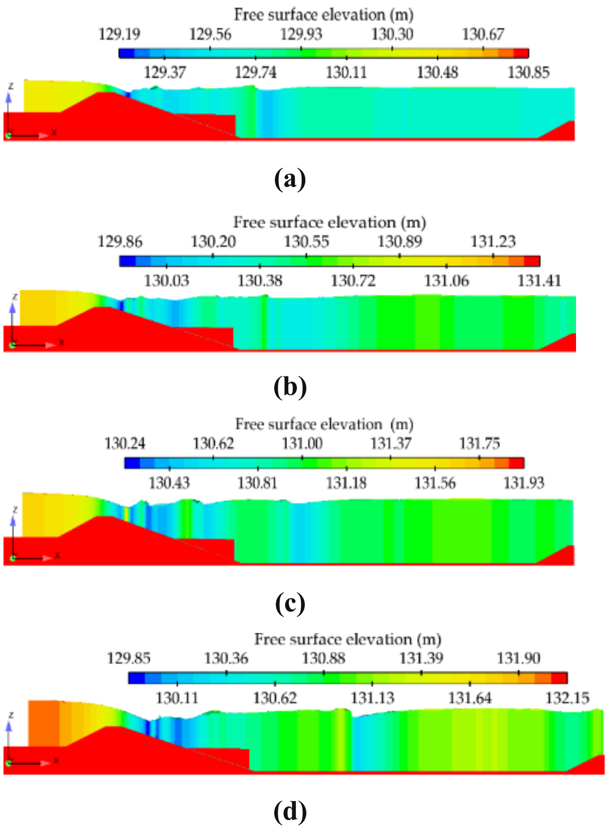


Fig. 2: Comparison of actual and simulated discharges

to be less. However, after the increase of discharge value up to the design flow, the water depth above the sub weir were noticed to be increased which reached 1.48 m at 24.19 m3/s/m. In addition, at all the tested discharge, the water surface profiles upstream of sub weir were stable while on the downstream side, due to the hydraulic jumps a turbulent free surface profiles were noticed as shown in Figure 3. However, after 12.09 m discharge, the free surface profiles of the downstream of sub weir were observed unstable and turbulent because at this discharge, the flow downstream of any diversion barrages found to be in the flood ranges as can be seen in Figure 3 (f-h).



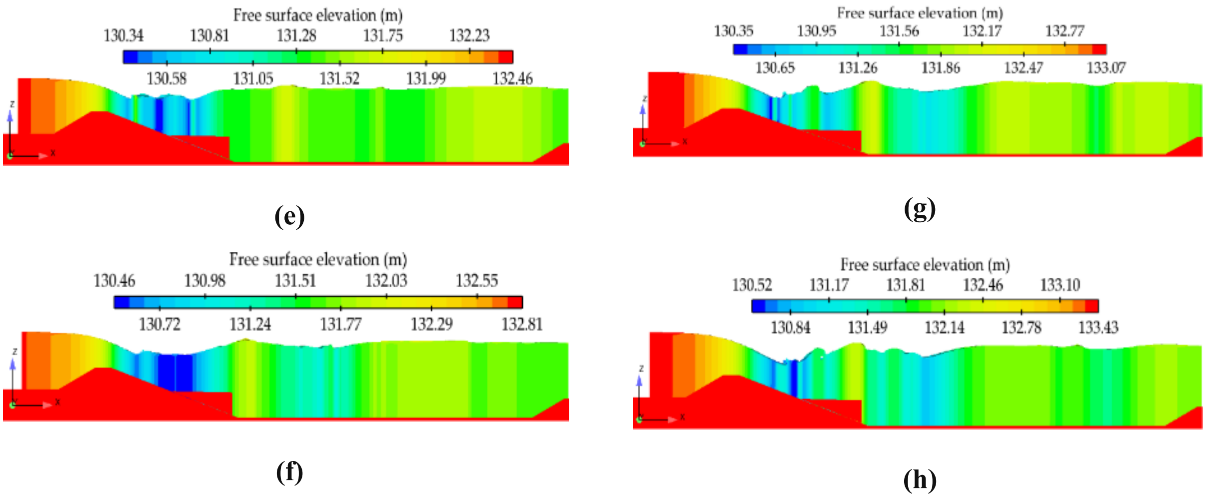


Fig 3: 2-D illustration of free surface profiles upstream and downstream of sub weir at various discharge

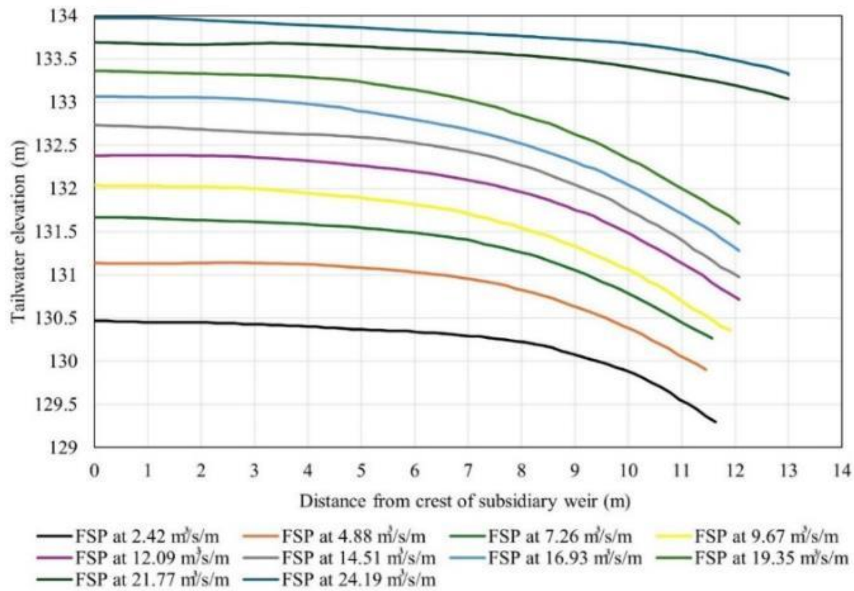


Fig. 4: Simulated free surface profiles upstream of sub weir

Figure 4 shows simulated free surface profiles upstream of sub weir using the tailwater levels (TWLs) of the year 2003 at various discharges. From Figure 4 it can be seen that as the discharge was increased the free surface profiles upstream of sub were also found to be raised, however, the relative change after 12.09 m3/s/m was found to be decreased. In addition, at 21.77 and 24.19 m3/s/m discharges, the relative change from the lower discharges were found to be negligible, which reached 12%. After comparing flood data of year 2010 [3], [4] the relative change of heads above the crest among the various discharges of simulated data was found to be deviated from the prototype data. A detailed comparison of the simulated and prototype data of free surface profiles upstream of sub weir is provided in Table 3. It is worth mentioning here that to draw the comparison between the simulated and prototype data, the relative change of heads above the crest of sub weir is computed from the year 2003 when there was no sub weir on the barrage downstream. From Table 4, the results indicated that as the discharge was increased, the relative increase of tailwater downstream of the main weir was limited to 15 % which further indicated that the sub weir didn't increase water head after 9.67 m3/s/m. On the contrary, at 9.67 m3/s/m, [3], [4] reported that the sub weir increased 25 % water head downstream of the main weir. From the analysis of literature and simulated data, it can be said there were some deficits in the operations of physical data for the design of sub weir which were also reported in [1], [2].

3.2 Water Heads and Coefficient of Discharge (C)

This section compares water heads above the crest of sub weir between the simulated and computed data of year 2010 downstream of Taunsa barrage. Figure 5 shows the simulated and observed water heads above the sub weir of Taunsa barrage. From Figure 4, the results revealed that at the initial discharge, the numerical model overestimated the water head while as the discharge was increased the values of waterhead over the sub weir were found to be less than that were observed at prototype during year 2010. The maximum difference of water head between the simulated and protype barrage was found at 12.09 m3/s/m discharge which reached 15%. However, the overall trends of simulated and observed waterheads were found to be similar. From the analysis of water heads, it can be said in comparison to the results of numerical models the overestimated values of water heads at the prototype and laboratory models are due to the operational conditions of the sub weir which have employed the erroneous data of tailwater levels [1], [2]. From Figure 6, it can be seen that as the discharge was increased that the simulated values (C_s) were also noted to be increased, which indicated a linear trend. In contrast, as compared to the C_s values, the observed coefficients of discharge (C_{ob}) over the sub weir were found to be less from 2.42 to 19.35 m3/s/m. However, after 19.35 m3/s/m, the C_{ob} were noticed to be increased which reached 2.329. From the simulated and observed values of discharge coefficients, the analysis indicated dissimilar trends of discharge coefficients, in particular as the discharge was increased the C_{ob} up to 19.35 m3/s/m was found to be decreased at the prototype subsidiary weir.

3.3 Drowning Ratio

Based on the model study report on scaled model of 1:50, Taunsa barrage was remodeled during 2008 which included subsidiary weir about 274.30 m downstream of the main weir. However, it was reported that during the super flood of year 2010, the barrage was unable to pass flood of 23.22 m3/s/m discharge. In addition, it indicated that subsidiary increased the water levels on the upstream of barrage which increased the drowning ratio (D_r) and decreased the discharge coefficient. Therefore, it is essential to analysis the results of drowning ratio for the simulated and observed data.

Table 3: Comparison of relative change in tailwater levels (TWLs) at the prototype and in numerical models

Q (m^3/s/m)	TWLs (2003)	TWLs at Prototype (2010)	Simulated TWLs	TWLs at Prototype after sub weir	Change of TWLs at prototype	Simulated TWLs after sub weir	Change of TWLs in numerical models
2.42	129.69	130.40	130.47	0.71	----	0.78	----
4.84	130.45	131.23	131.14	0.78	8%	0.69	-14%
7.26	130.91	131.91	131.66	1.00	23%	0.75	9%
9.67	131.15	132.47	132.04	1.32	24%	0.89	15%
12.09	131.39	132.95	132.38	1.56	15%	0.99	10%
14.51	131.58	133.31	132.73	1.73	10%	1.15	14%
16.93	131.73	133.56	133.07	1.83	5%	1.34	14%
19.35	131.88	133.74	133.36	1.86	2%	1.48	10%
21.77	132.00	133.86	133.69	1.86	0%	1.69	12%
24.19	132.06	133.99	133.97	1.93	4%	1.91	12%

By employing formula (3), the results of drowning ratios are also compared between the present numerical model and the observed data of the year 2010.

$$D_r = \frac{Downstream\ Water\ Level - Crest\ Level}{Upstream\ Water\ Level - Crest\ Level} \quad (3)$$

From Figure 7, the results of numerical models indicated a linear relationship of drowning ratio and discharge coefficient while a decreasing trend of drowning ratio was observed at the prototype. In numerical models and at prototype barrage, the maximum drowning ratios reached 68 % and 59.8 %, respectively. After comparing with [1], [2], the results of present numerical models showed agreement as at 23.22 m^3/s/m discharge, the barrage was not able to pass the flow. In addition, the analysis further revealed that at such discharges the drowning ratio at the prototype was increased which reduced the discharge capacity of the barrage as noticed during the 2010 flood.

Based on the results of hydraulic parameters such as free surface profile, head above the crest,

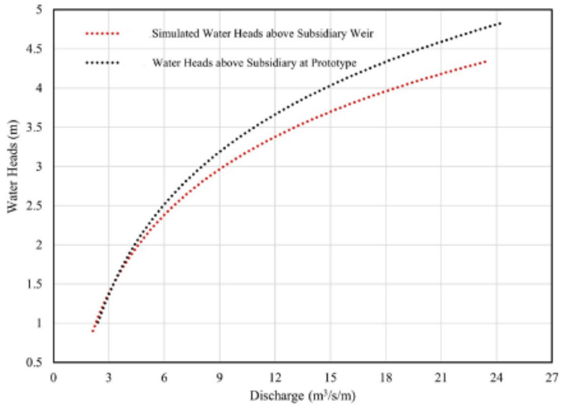


Fig. 5: Observed and simulated values of water heads above the crest of Subsidiary weir

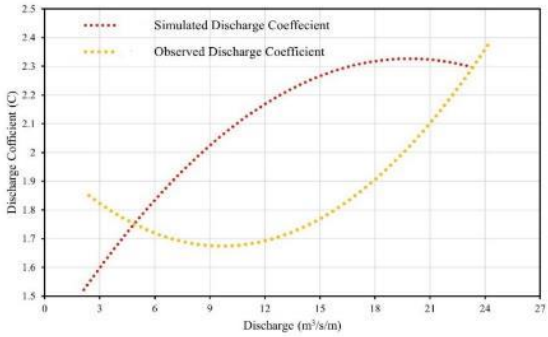


Fig. 6: Observed and Simulated discharge coefficient for different discharge

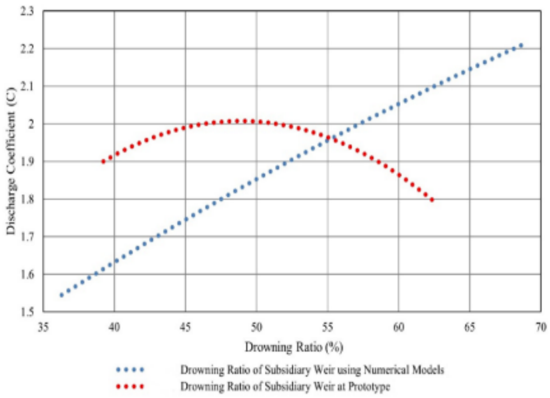


Fig. 7: Comparison of drowning ratios of numerical models and prototype data

coefficient of discharge, and drowning ratio, the deviation between the numerical and prototype values can be due to; (1) the use of erroneous tailwater levels employment on the reduced scale model of 1:50, as the relative change of head above the crest of sub weir at the higher discharge were found to be very less as compared to the present numerical models; (2) usually at the Prototype the measurements have localized fluctuations due to unsteady flow conditions, which are smoothed out in numerical modeling; (3) as the full-scale numerical model provided a more detailed representation of pressure distributions and turbulent structures that contributed to the deviated water levels; (4) based on employed models' condition, i.e., initial conditions, the simulated discharge coefficient showed a continuous increase while observed discharge coefficient initially decreased before increasing which showed higher discharge coefficient value at designed discharge. However, during the super flood of the year 2010, the barrage was unable to pass 23.22 $m^3/s/m$ that indicated due to submergence of sub weir at this discharge, flow resistance increased which led to decrease in the discharge coefficient. This analysis highlights the importance of conducting careful hydraulic investigations on reduced-scale models before implementing large-scale interventions. Ensuring accurate hydraulic data, i.e., tailwater levels, initial conditions, and operational parameters are crucial. Scale effects in physical models can introduce discrepancies when compared to prototype conditions which further emphasize precise calibration and validation.

4. Conclusion

The present numerical study employed FLOW-3D hydraulic models to investigate the effects of tailwater levels downstream of the Tuansa barrage. For free surface tracking, the VOF method is employed while turbulence is captured using RNG K-ε model. To operate the free flow models of subsidiary weir up to the design discharge, the study used similar operating conditions as were employed in the scaled model of 1:50. To check the effects of subsidiary, the study mainly focused on free surface profiles, water heads, coefficient of discharge and drowning ratios. The results of these hydraulic parameters are compared with prototype barrage data and with model study reports. Based on the models' results, the following conclusions are outlined.

The analysis of free surface profiles revealed that as the discharge was increased the relative change in the tailwater level downstream of main barrage was noticed to be decreased. In addition, in physical model and downstream of the prototype barrage, the water heads above the crest of subsidiary were found to be overestimated which further indicated that the head raising ability of subsidiary weir was reduced after 9.67 m3/s/m discharge.

The numerical models indicated higher discharge coefficients and drowning ratio which indicated a direct relationship with the discharge. In contrast, the analysis of observed data of these hydraulic parameters was found to be deviated which indicated nonlinear trends.

In conclusion, based on the results of water heads, discharge coefficient and drowning ratio, it can be said that the existing subsidiary weir downstream of the Taunsa barrage, Pakistan, is decreasing the discharging capacity of the barrage. However, as the present numerical models are operated under free flow conditions, therefore, the study suggests investigating the effects of subsidiary weir by employing orifice flow condition and other turbulence models.

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