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Experimental and numerical investigation on hydraulic characteristics of the water-retaining weir in the long-distance water conveyances system

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ABSTRACT

Water-retaining weirs are key local flow-control structures in long-distance water conveyance systems, and their hydraulic performance is of great significance for maintaining flow depth, improving conveyance conditions, and ensuring operational safety. This study aimed to optimize the structural design of water-retaining weirs in a long-distance water conveyance system to improve their hydraulic performance. Physical model tests were first conducted using a 1:20-scale model to investigate four distinct water-retaining weir structures. Based on the experimental results, the variation of key hydraulic parameters, namely the local head loss coefficient (ζ), submergence coefficient (σ) and the discharge coefficient (m), were systematically investigated. To compensate for the limitations of the physical model tests and to further clarify the complex flow-field characteristics, three-dimensional numerical simulations were subsequently performed. Experimental data were combined with simulated flow patterns to conduct a comparative analysis of the two weir types with and without a dissipation weir. This study demonstrated that the structure without an energy dissipation pool, combined with a higher weir crest, yielded a smaller local head-loss coefficient and a larger submergence coefficient. Additionally, removing the energy dissipation pool resulted in a more stable flow regime with fewer fragmented vortices under well-flow. The installation of a downstream energy dissipation pool had a minor effect on the discharge coefficient. On this basis, the weir structure was further optimized, resulting in a hydraulically efficient design that was better suited to practical engineering applications. These findings from this research enhance the understanding of the hydraulic behaviour of water-retaining weirs and provide a useful basis for the design and optimization of long-distance water conveyance systems.

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Highlights

- The variation laws of the key hydraulic characteristics of water-retaining weirs in long-distance water conveyance systems were fully revealed.
- Based on physical model tests and computational fluid dynamics simulations, four water-retaining weir configurations were systematically compared in terms of local head loss coefficient, submergence coefficient, discharge coefficient, and flow-field characteristics.
- Removing the downstream energy dissipation pool, combined with a higher weir crest yielded a smaller local head-loss coefficient, a larger submergence coefficient, and a more stable flow regime under well flow, while a lower weir crest provided a greater flow coefficient under free overflow.

1. Introduction

Water resources are essential to human survival, economic development, and ecological security. However, due to variations in climate, topography, and socio-economic conditions, many countries exhibit a pronounced spatial mismatch among water availability, population distribution, and water demand. As a result, inter-basin

and long-distance water transfer projects have become important-engineering measures for alleviating regional water scarcity (Rollason et al., 2022). Major rivers and lakes around the world, including the River Thames in the United Kingdom, the Yangtze River in China, the Amazon River in South America, the Ganges in India, the Nile in Egypt, the Great Lakes and the Mississippi

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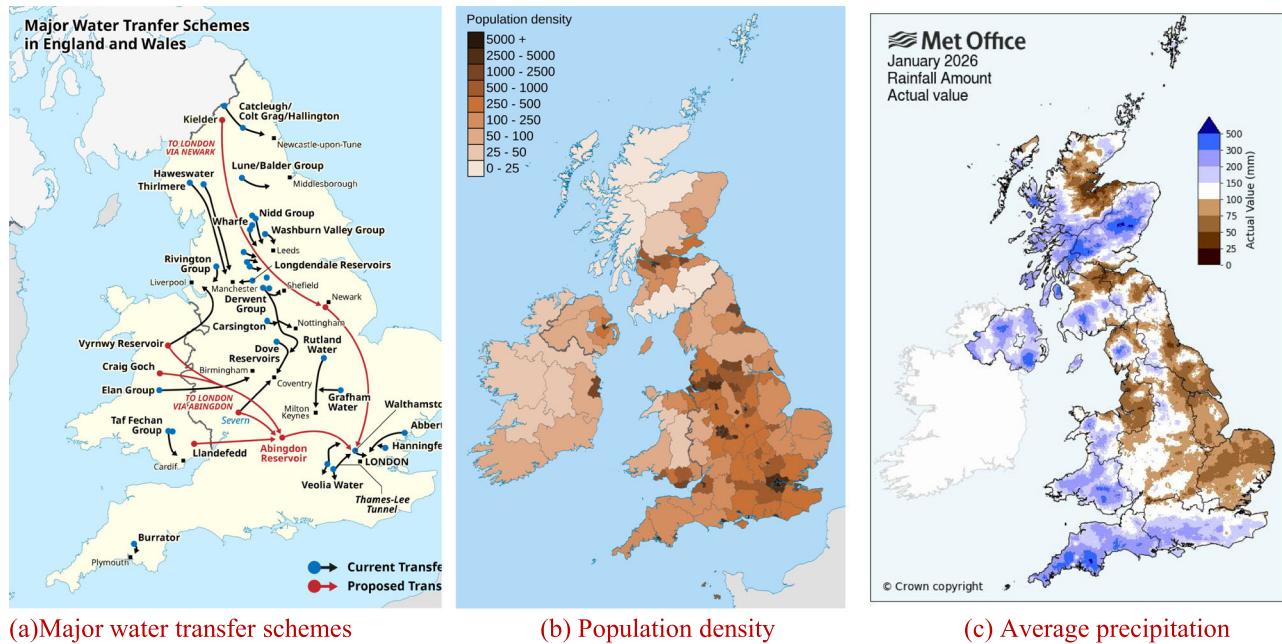


Figure 1. Regional context in England and Wales. (a) Major water transfer schemes. (b) Population density. (c) Average precipitation.

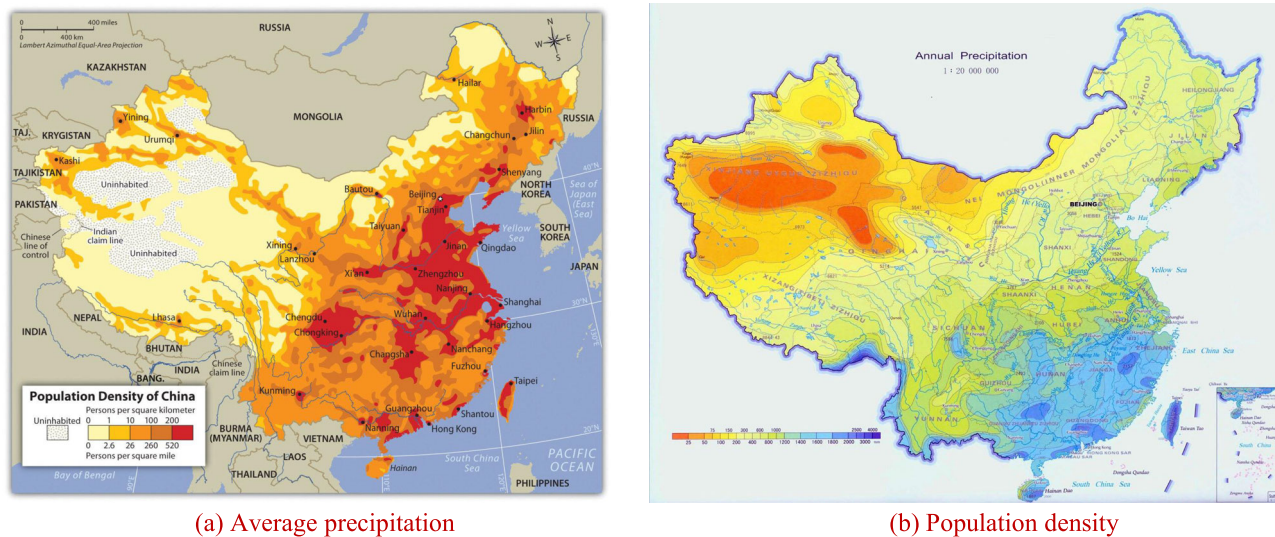


Figure 2. Regional context in China. (a) Average precipitation. (b) Population density.

River in North America have also been associated with water transfer projects. More specifically, in the United Kingdom, significant regional differences were observed in population distribution, rainfall patterns, and the configuration of water transfer systems. As a result, water transfer and redistribution are closely linked to major river basins such as the Thames, Severn, Trent, and Great Ouse, as well as reservoir-based supply systems including Lake Vyrnwy and Haweswater Reservoir, which support long-distance water delivery to major urban areas such as London, Liverpool, and Manchester (Figure 1(a-c)) (Water, 2022). In China, precipitation generally exhibits a spatial gradient characterized by greater rainfall in the

south than in the north, and in the east than in the west, whereas some population and economic activities remained concentrated in several water-deficient regions. Therefore, numerous water transfer projects have been developed to convey water from the south to the north in order to alleviate the mismatch between water availability and the distribution of population and economic activities (Figure 2(a,b)) (China, 2023; Ma et al., 2020; Sun et al., 2021). This imbalance between water supply and water demand indicates that local water resources alone are often insufficient to satisfy the requirements of regional development.

Long-distance water conveyance systems are widely used to address the uneven distribution of water resources and to enable their more efficient use. Because restarting the water supply after a service interruption requires a relatively long refilling time, several feasible structures and devices for water retention after the initial filling process have been designed and improved for practical application (Shi et al., 2023). Shi et al. (2025) developed a cascaded neural network surrogate model for real-time decision-making in long-distance water supply distribution, demonstrating that accurate flow monitoring and coordinated regulation of pumps and valves are essential for the efficient and reliable operation of long-distance water conveyance systems. Although such system-level studies have improved the understanding of operational regulation in water conveyance projects, limited attention has been paid to water-retaining weirs with complex internal configurations. In water conveyance systems, water retention refers to maintaining the water diversion tunnel, box culvert and penstock in a pressurized state during low-discharge operation or short-term non-operation. It is essential for ensuring timely resumption of water conveyance and preventing water disengagement, thereby reducing the risk of prolonged refilling, air entrainment, hydraulic fluctuations, and operational instability during system restart. For example, a downstream valve or sluice gate can be used to retain water in a pressurized water transfer system, although supplementary facilities such as air valves are usually required to control possible higher water-hammer pressure. For gently sloping conveyance pipelines, a downstream overflow weir, which is also called a water-retaining weir, is the preferred option. The hydraulic performance of the water-retaining weir directly affects the operational efficiency and stability of the entire system. Therefore, selecting an optimal water-retaining weir structure is essential to balance the need for water retention with the requirement of minimizing energy dissipation during high-flow periods. This approach can be used in long-distance water conveyance tunnel and low-pressure box culvert, promising potential for hydraulic control in gravity water transfer system. The Tianjin main line of the Middle Route Project of South-to-North Water Diversion adopted the water-retaining weir and played an important role in ensuring reliable system operation (Hua et al., 2017).

The key challenge for reliable water transfer is to achieve smooth hydraulic connections among adjacent pressurized segments. After detailed comparative analysis and verification, water-retaining weirs were adopted as the hydraulic connecting structures in this stepped low-pressure water supply system. A typical water-retaining weir consists of a pressurized inlet, an upstream

weir well, an overflow weir, a downstream weir well and a pressurized outlet. As the downstream water depth changes, the water-retaining weir exhibits three typical overflow regimes, namely well flow, submerged overflow and free overflow (Shariq et al., 2022). Its advantage is that it keeps the water-filled state and prevents higher water hammer in a pressurized water conveyance system under various operation conditions. The main functionality includes ensuring that the system retains water during the water-filling state and prolonging the service life of pipe, by maintaining the submergence of neighbouring inlet (outlet) pressurized pipes. Using water-retaining weirs can ensure uninterrupted and immediate water supply at each instant, while the whole pressurized pipeline is always under a relatively low pressure.

Along the Tianjin Middle Route Project of South-to-North Water Diversion, eight water-retaining weirs with heights from 10 m to 6 m were constructed. Water-retaining weirs with different heights are used to satisfy different water-level regulation and conveyance requirements. Therefore, the highest (Model 1) and lowest (Model 2) weir geometries were selected as representative boundary cases to facilitate a clear comparison of their hydraulic performance. The high-weir geometry represents the upper-limit condition with stronger obstruction to the flow and the low-weir geometry represents the lower-limit condition with weaker obstruction and relatively higher conveyance capacity. Comparing these two geometries therefore helps reveal how the structural scale of the water-retaining weir influences the hydraulic coefficients and downstream flow structure. Water from the upstream reservoir flows into the bottom of the upstream weir well from the upstream box culvert, and overflows the weir into the downstream box culvert. When the water flows through water-retaining weir, the flow patterns become complex with rapidly bending flow lines under the constraint of the well sidewall and weir boundary, the circular anti-vortex beams. An exact understanding of hydraulic characteristic of water-retaining weirs will be beneficial for predicting the hydraulic response of the whole system. Moreover, it is necessary to prevent the hydraulic jump into the downstream box culvert.

There are different operating water levels in the weir well, with some complex weir flows and potentially harmful flow patterns. Therefore, it is necessary to accurately reveal the complete hydraulic characteristics of water-retaining weirs based on experimental and simulated research (Wang, Xu, et al., 2024). Sun et al. (2022) studied the variation law of the key hydraulic characteristics, like the critical submergence degree, head loss and velocity distribution of the flow field over the weir using both experimental and numerical approaches, and verified the reliability of the numerical simulation. In addition, the

mathematical models of overflows and pressurized flow were built to simulate the procedure of alternate flow in water-retaining weirs by Wan, but the flow coefficient and local head-loss coefficient were adopted based on some assumption, and not proved in detail by experimental research (Wan et al., 2006). Wan (2006) conducted experimental research to analyze the hydraulic characteristic of overflow weirs located at the outlet of long water conveyance tunnel in detail. This was to identify reasonable area of connection pool and submerged height of overflows weir, which were required to avoid air admission and water disengaging. Guven et al. (2013) investigated the hydraulic characteristics of a broad-crested weir and box culverts through experimental testing, concluding that the discharge coefficient was significantly influenced by parameters such as upstream head, weir side slope, and culvert geometry, and providing empirical equations to optimize the design of such combined water-retaining structures. Previous studies (Saadatnejadgharhassanlou et al., 2017) conducted experimental research on the discharge coefficient of the weir. Their study revealed that water head and weir height significantly affected the discharge coefficient. The experimental setup and testing, as described, ensured high precision by using scaled-down models and appropriate discharge measurement techniques. Bagheri and Kabiri-Samani (2020) employed the Joukowsky transform to design various streamlined weir models and measured key hydraulic parameters, including discharge coefficient, velocity profiles, and water surface distributions. This result demonstrated that streamlined weir performed better than traditional weirs, offering higher discharge coefficients and improved stability. Ma et al. (2016) used a tall physical dropshaft model to investigate flow breakup, air entrainment, and air-pressure variation in a plunging-flow dropshaft. Zhang et al. (2024) used hydraulic model testing, theoretical analysis, and numerical simulation to investigate the flow behaviour and discharge performance of complex spillway systems in tailings dam.

During condition switch of long-distance water conveyance system, the flow patterns of overflow weir undergo a transition from free overflows to submerged overflows, even to well overflows, and the submergence coefficient is varied according to different overflow patterns while the adoption of a fixed value leads to obvious error. In order to ensure the simulation accuracy, it is necessary to adopt a reasonable submergence coefficient based on the justification of real-time overflow pattern. A dynamic method to choose the submergence coefficient under different transient flow patterns of overflows weirs in transient flow computation was improved to realize the accurate simulation for change law of flow patterns (Li et al., 2007). Li et al. (Jiang et al., 2026)

investigated the effect of water-retaining weir height on the discharge capacity of a segmented low-pressure water conveyance project, demonstrating that weir height is an important geometric factor controlling discharge performance. After the flow patterns of the water-retaining weir were simulated by the coupling method of standard $k-\varepsilon$ turbulence model and VOF model and better simulation results were showed for free-water surface and flow turbulence, its optimized design was investigated by comparative analysis of difference flow fields (Yang et al., 2008). Aydin et al. (2020) used a two-phase CFD model to analyze spillway aerators in high-head dams and demonstrated the applicability of evaluating air entrainment and hydraulic performance to prevent the cavitation damage. Aydin and Ulu (2023) further applied a CFD approach to investigate the hydraulic performance of a labyrinth-shaft spillway, including the discharge capacity and discharge coefficient variation under different geometric and hydraulic conditions. Nourani et al. (2021b) reported that, unlike the traditional empirical formulas, numerical simulations provide a high-resolution view of the contour surrounding the weir. This allows researchers to analyze complex phenomena that are difficult to measure in physical models, such as the exact location of flow separation at the weir crest and the formation of the downstream recirculation zone. Muhammad et al. (Asif et al., 2024) used the CFD package fluent to observed the distribution of the flow in the central region and the complex pattern of secondary flow. Beside of these, the results revealed that submergence state markedly affects both the blockage of flow and the distribution of discharge among different regions.

Although previous studies had widely applied CFD methods, physical experiments and emerging data-driven approaches to analyze free surface flow, discharge capacity, and energy dissipation of conventional weirs and spillways, as well as the operational regulation of long-distance water conveyance systems. However, the relationships among flow patterns, weir geometry, and key hydraulic coefficients have not yet been studied for water-retaining weirs. Considering that, with the application and development of numerical simulation technology in hydraulic research, the visualization of the numerical simulation flow field greatly facilitates the study of hydraulic performance (Ansari & Patil, 2022; Kartal & Emiroglu, 2024; Samadi & Arvanaghi, 2014). On the premise of the actual measurement and control requirements of water-retaining weirs, this study used the method of combining model experiments and numerical simulation to study the water-retaining weir. Due to the complex structure of water-retaining weirs, their local head-loss coefficient (ζ), submergence coefficient (σ) and discharge coefficient (m) were difficult to obtain

without empirical data. Therefore, the primary objective was to derive three key hydraulic parameters (ζ , σ and m) for empirical governing equations (Zai et al., 2025). These parameters served as the quantitative basis for evaluating the discharge capacity and energy consumption of each weir type. While physical experiments provide reliable data, they struggle to capture the complex, micro-level turbulence and internal flow structures behind the weir and it is not always possible to set up experimental studies for all designs being considered (Li, Zhou, et al., 2024). To compensate for these experimental limitations, a three-dimensional (3D) numerical simulation was employed using computational fluid dynamics (CFD). This approach allowed for a high-resolution observation of the flow patterns downstream of the overflow weir, including the distribution of vortices and the transition process of the flow (Wang, Cheng, et al., 2024).

2. Experimental setup

2.1. Design of experimental model

Water-retaining weir have played a critical role in the combined operation of the long-distance water conveyance system. Under steady or unsteady flow conditions, the piezometric head in the outlet section of upstream box culvert and inlet section of downstream box culvert are physically linked by water-retaining weir well. The central overflow weir features a free water surface, while both sides experience pressurized flow. With different downstream water depths h , the water-retaining weir exhibits three typical overflow regimes including well flow (h_1), submerged overflow (h_2) and free overflow (h_3). Here, h denotes the downstream water depth used to distinguish the different overflow regimes, as shown in Figure 3. The experimental model was designed as geometric normal model with length proportion $\lambda_L = 20$ according to gravity similarity criterion,

therefore, other proportions were the velocity proportion $\lambda_V = \lambda_L^{0.5} = 4.472$, pressure proportion $\lambda_P = \lambda_L = 20$ and discharge proportion $\lambda_Q = \lambda_L^{2.5} = 1788.854$.

In order to ensure the similarity of flow patterns and water surface transient in the overflow weir for the model and prototype, the simulation range included the upstream and downstream three passages pressurized box culverts besides the main part of the water-retaining weir between the upstream and downstream transition segment. The main parts of the model including overflow weir and three passages pressurized box culverts, which were made of polymethy methacrylate for flow pattern observations. The whole water-retaining weir model was designed as a module type and was easy to disassemble for comparative analysis and optimization. In order to realize smooth inflow and discharge control, an intake pool was located upstream of this model. The intake pools for two water-retaining weirs were arranged separately for individual water supplying and to avoid no mutual disturbance. The discharge for each water-retaining weir model was adjusted by a gate valve and measured by a standard triangular weir. A tail-water pool with sufficient area and an adjustable end gate was set at the outlet of each water-retaining weir model for water level regulation and stabilization, as shown in Figure 3. Figure 4(a,b) showed the physical experimental model of Model 1 and Model 2.

2.2. Experimental content

By leveraging the terrain's natural features, the system incorporated weir sections to enable segmented gravity flow control. This effectively manages the maximum water pressure in each segment and significantly simplifies the complexity of the water conveyance control system. Therefore, conducting in-depth research into the hydraulic characteristics of the water-retaining weir will help to predict the hydraulic response of

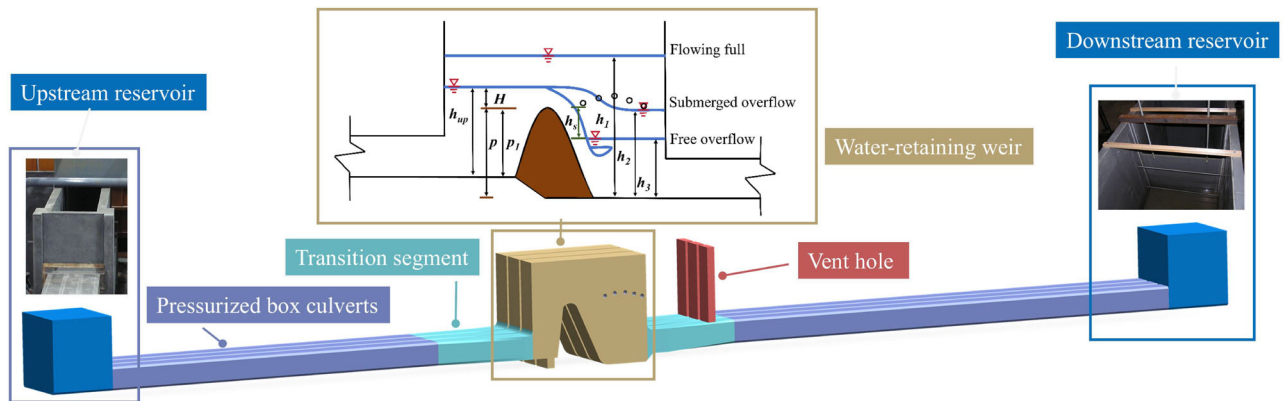
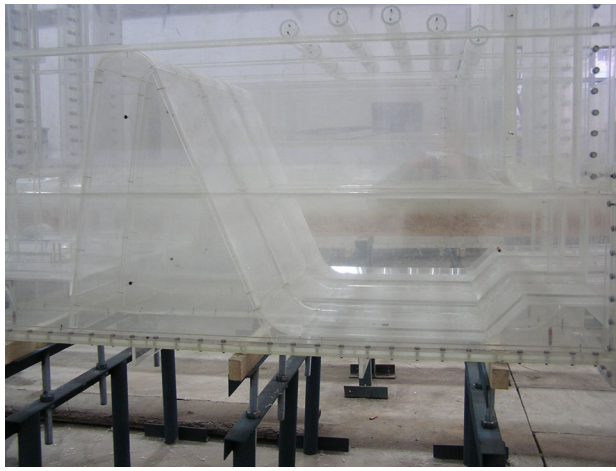


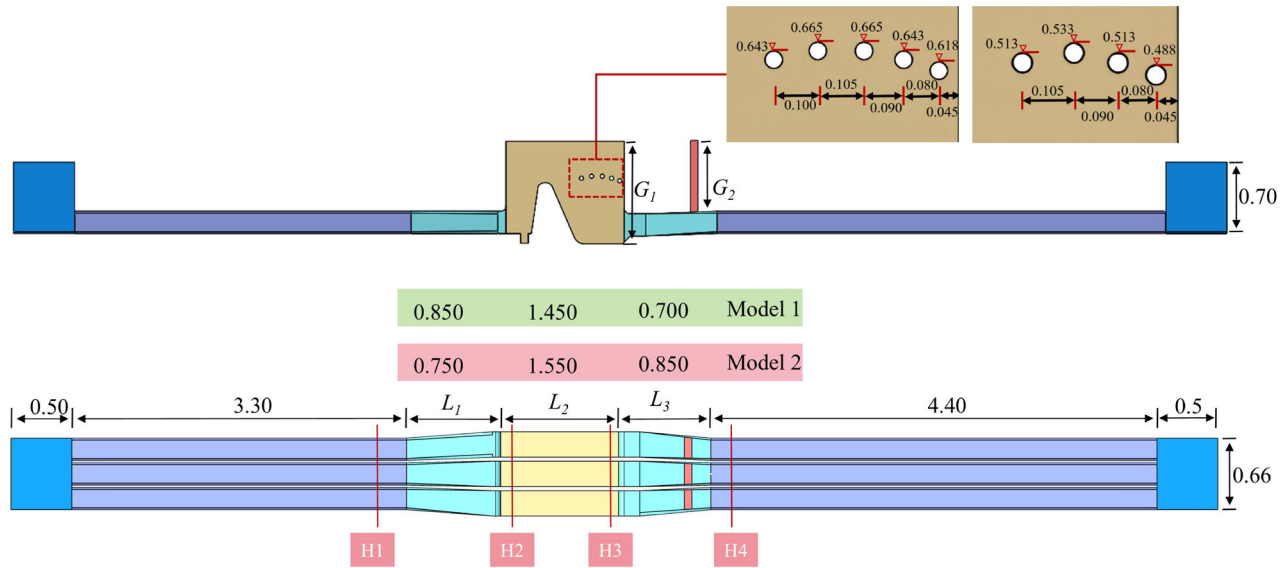
Figure 3. Schematic diagram of the long-distance conveyance system.



(a) Model 1



(b) Model 2

Figure 4. Side views of the experimental models. (a) Model 1. (b) Model 2.**Figure 5.** Measurement section layout diagram of water-retaining weir.

the entire system, thus ensuring the safe operation of the project. Therefore, the experimental content of the water-retaining weir mainly includes the measurement of piezometric heads at different measurement sections and the inspection of flow patterns. Then, according to the measured data, the local head-loss coefficients, discharge coefficients and submerged coefficients are analyzed based on their definitions and corresponding formulas.

2.3. Measurement technology

Along the water-retaining weir model including the upstream and downstream three passages pressurized box culverts, four measuring sections (H1 ~ H4) were

arranged and designed with several pressure holes connected to ring pipe for pressure-equalizing, which were shown in Figure 5. Along the water flow direction, the specified measuring sections are:

- Section H1 was located at the upstream-pressurized box culvert close to water-retaining weir well and was used to check whether the inlet discharge reached a balanced condition and to calculate the cross-sectional velocity.
- Section H2 was between the outlet of upstream-pressurized box culvert and water-retaining weir, and was used to measure the upstream water depth of the weir.
- Section H3 was arranged in downstream water-retaining weir well close to downstream pressurized

box culvert, and was used to measure the downstream water depth of the weir.

- Section H4 was located at the downstream pressurized box culvert close to air vent and was used to check whether the outlet discharge reached a balanced condition and to calculate the cross-sectional velocity.

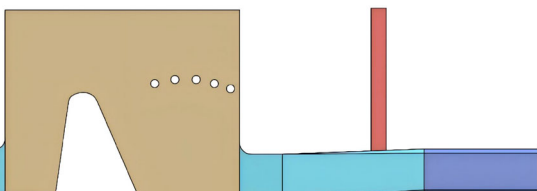
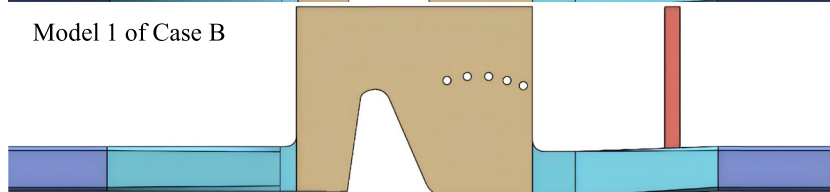
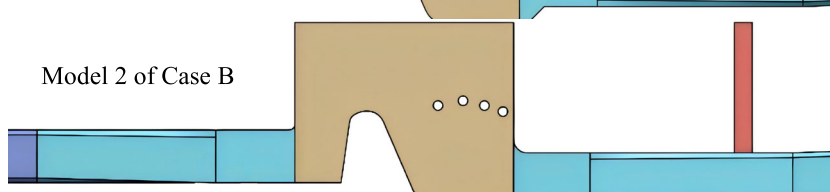
The piezometric head at the above section was connected to the measuring cylinder via transparent plastic tubes. A probe was used for measurement, with readings accuracy to 0.1 mm. The measuring cylinder was fixed to the wall to minimize water level fluctuations and ensure measurement accuracy. Both of Model 1 and Model 2 were equipped with a set of measuring cylinders. The measuring cylinders of the Model 1 and Model 2 were connected with transparent plastic tubes for water injection. After ensuring there was no air bubbles in the connecting tubes prior the experiment for water injection, the system was left to stand for 24 h to verify the baseline of each probe in the set. Meanwhile, the probe baseline for Model 2 was also verified using the same method.

Two water-retaining weir designed (Model 1 and Model 2) with different internal structures were selected for comparative analysis. The detailed dimensions were listed in Table 1. The conveyance box culvert was

kept identical for both models, with a cross-section of $0.22 \text{ m} \times 0.22 \text{ m}$. Model 1 (weir crest was 0.500 m) included five circular anti-vortex beams, whereas Model 2 (weir crest was 0.300 m) included four, and all beams had a diameter of 0.04 m. This component was used to suppress local vortex formation and reduce rotational flow before the water entered the downstream box culvert. Since the anti-vortex component was identical in all experimental schemes for the same model, it mainly served to stabilize the inflow condition and was not treated as an independent experimental variable. The detailed position and dimensions of the anti-vortex component were shown in Figure 5. In addition, vent holes were installed on the pressurized culvert at 0.650 m (Model 1) and 0.925 m (Model 2) downstream of the weir well to maintain smooth and stable flow conditions. Details of both Model 1 and are summarized in Table 1.

There were two cases considered for each model: Case A included an energy dissipation pool on the downstream side of the weir well. The bottom elevation of the dissipation pool was 0.075 m lower than the bottom at the entrance of the downstream box culvert, and it was connected to the culvert floor by a 1:1 sloped transition. In Case B, the water-retaining weir structure and all model dimensions were identical to those in Case A, but no

Table 1. Schematic diagrams of two different internal structures for each model (Model 1 and Model 2).

Model	Geometric details (m)		Schematic of the internal structure	
1	G_1	0.950 (Case A) 0.875 (Case B)	Model 1 of Case A	
	G_2	0.640		
	P	0.600		
	P_1	0.500		
	L_1	0.850		
	L_2	1.450	Model 1 of Case B	
	L_3	0.700		
2	G_1	0.780 (Case A) 0.705 (Case B)	Model 2 of Case A	
	G_2	0.555		
	P	0.470		
	P_1	0.300		
	L_1	0.750		
	L_2	1.550	Model 2 of Case B	
	L_3	0.850		

energy dissipation pool was included. All other structural details remained unchanged, as shown in Figure 5.

3. CFD simulation

3.1. Watershed model establishment

The 3D solid model was produced using SolidWorks based on the 2D water-retaining weir CAD drawings. To improve the agreement between the 3D numerical results and the experimental model tests, the 3D numerical model was likewise constructed at a 1:20 structural scale. The $x-t$ file format was used to import the 3D model into ANSYS Workbench, and the channel model was shown in Figure 10.

3.2. Numerical simulation methods

The 3D numerical flow simulation in the long-distance water conveyance system was carried out using ANSYS FLUENT software (Azimi et al., 2023). The working media was air and water. The boundary conditions of the model were set according to the actual experimental conditions, as shown in Figure 6. Combined with the actual working conditions of the water-retaining weir, the boundary conditions were pressure inlet, pressure outlet and velocity inlet.

The water-air two-phase flow around the water-retaining weir was treated as incompressible. The flow field was solved using the Reynolds-averaged Navier-Stokes equations. The continuity equation is expressed as:

$$\frac{\partial u_i}{\partial x_i} = 0 \quad (1)$$

$$\begin{aligned} \frac{\partial(\rho u_i)}{\partial t} + \frac{\partial(\rho u_i u_j)}{\partial x_j} \\ = -\frac{\partial p}{\partial x_i} + \frac{\partial}{\partial x_j} \left[(\mu + \mu_t) \left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) \right] \\ + \rho g_i \end{aligned} \quad (2)$$

where u_i and u_j are the velocity components, x_i and x_j are the coordinate directions, ρ is the mixture density, p

is the pressure, μ is the dynamic viscosity, μ_t is the turbulent viscosity, and g_i is the gravitational acceleration component.

The realizable $k-\varepsilon$ model was selected in this work. Compared with the standard $k-\varepsilon$ model, this model improves the dissipation rate equation and accounts for the influence of mean rotation on turbulent stresses, allowing the model to have high simulation accuracy and numerical stability. The modelled transport equations for k and ε can be given as follows:

$$\begin{aligned} \rho_m \frac{dk}{dt} = \frac{\partial}{\partial x_i} \left[\left(\mu + \frac{\mu_t}{\sigma_k} \right) \frac{\partial k}{\partial x_i} \right] \\ + G_k + G_b - \rho_m \varepsilon - Y_M \end{aligned} \quad (3)$$

$$\begin{aligned} \rho_m \frac{d\varepsilon}{dt} = \frac{\partial}{\partial x_i} \left[\left(\mu + \frac{\mu_t}{\sigma_\varepsilon} \right) \frac{\partial \varepsilon}{\partial x_i} \right] \\ + \rho_m C_1 S_\varepsilon - C_2 \rho_m \frac{\varepsilon^2}{k + \sqrt{\nu \varepsilon}} \\ + C_{1\varepsilon} \frac{\varepsilon}{k} C_{3\varepsilon} G_b \end{aligned} \quad (4)$$

where G_h and G_b are the generation of turbulence kinetic energy due to the mean velocity and buoyancy, σ_k and σ_ε are the turbulent Prandtl numbers for k and ε , and Y_M is the influence of compressible turbulence on the overall dissipation rate. Herein, $Y_M = 0$. Other constants include $C_1 = 1.44$, $C_2 = 1.9$, $\sigma_k = 1.0$, and $\sigma_\varepsilon = 1.2$.

No slip wall condition was implemented; the VOF model was adopted to track the phase volume fractions across the entire domain and to simulate the interaction of multiple immiscible fluids (Li, Deng, et al., 2024). In the numerical simulations, both air and water phases were modelled according to the free surface homogeneous model, where the two species share the same dynamic fields of pressure, velocity and turbulence. Therefore, the VOF multiphase model was applied to solve and capture the three-phase flow characteristics of an air-liquid interphase (Hirt & Nichols, 1981; Surhone et al., 2010). The water and air volume fractions satisfy:

$$\alpha_w + \alpha_a = 1 \quad (5)$$

where α_w and α_a are the volume fractions of water and air, respectively.

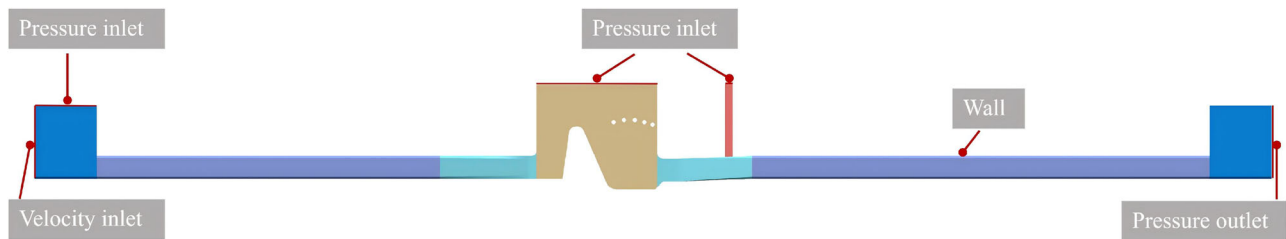


Figure 6. Boundary conditions of water-retaining weir.

A computational cell is filled with water when $\alpha_w = 1$, filled with air when $\alpha_w = 0$, and contains the free surface when $0 < \alpha_w < 1$.

Global initialization was performed according to the velocity-inlet condition. Besides, the mass flow rate was monitored during the calculation process. It was assumed that the calculation had converged when the mass flow rate value approached zero in a steady state.

3.3. Mesh generation and irrelevance verification

The flow field was meshed using ANSYS meshing, considering that the geography of the water-retaining weir, the grids around the weir was divided by finer meshes, which could better resolve the variations in the flow field. The entire area was divided into poly-hex core meshes as shown in Figure 7. As shown in Figure 7, local mesh refinement was applied to regions with complex flow variations, including the area near the water-retaining weir and the upstream and downstream transition sections. Four grid numbers were used to verify grid independence. The grid was regenerated by decreasing and increasing the grid size of the background domain. Four mesh sizes of 0.02, 0.015, 0.01, 0.008 and 0.005 m were set respectively for simulation calculation.

As shown in Table 2, in order to verify the reliability of numerical simulation, the upstream water depth

and cross-sectional mean velocity were compared simulation and experiment. With a grid size of 0.008 m, the relative errors were 1.05% for upstream water depth and 2.89% for the cross-sectional mean velocity (the cross-sectional mean velocity referred to average flow velocity at location H4 for the three downstream pressurized box-culvert sections). Further mesh refinement increased the total grid number by 3.61 million, yet the relative errors decreased only marginally by 0.29% and 0.26%, respectively. The comparison between simulated and experimental results showed that they were in good agreement, the maximum error of two indicators was 2.89%, supporting the accuracy of the numerical predictions (Xia et al., 2013). Therefore, after comparing with calculation time cost, simulation efficiency and accuracy, it was finally determined that the most suitable size was 0.008 m. The upstream water depth is H_{up} , which was in shown in Figure 3.

To further quantify the mesh sensitivity on numerical simulation, a Grid Convergence Index (GCI) analysis was performed using Case-1 (3 = coarse), Case-3 (2 = medium) and Case-5 (1 = fine) were selected as the coarse, medium and fine triplet for the GCI calculation. The corresponding refinement ratios were both greater than 1.30, satisfying the American Society of Mechanical Engineering (ASME) recommendation for grid refinement (Celik et al., 2008).

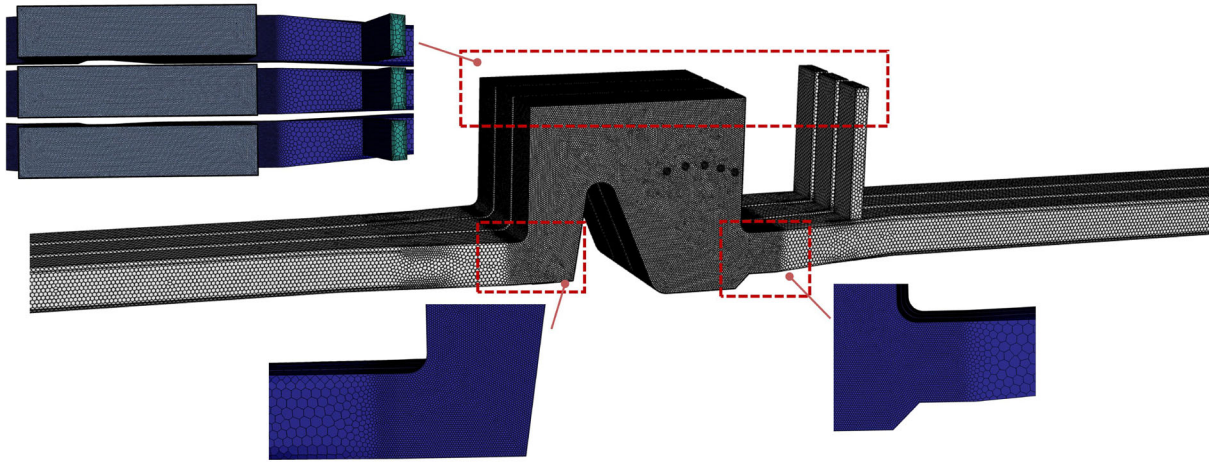


Figure 7. Mesh graph of computational domain.

Table 2. Grid independence verification table.

Case	Grid numbers	Upstream water depth (H_{up}) (m)			Cross-sectional mean velocity (m/s)		
		Numerical	Experimental	Relative error	Numerical	Experimental	Relative error
Case-1	480502	0.5243	0.542	3.33%	0.0684	0.0760	10.00%
Case-2	1030550	0.5288		2.44%	0.0702		7.63%
Case-3	2109861	0.5334		1.59%	0.0718		5.53%
Case-4	3483057	0.5363		1.05%	0.0738		2.89%
Case-5	7092517	0.5379		0.76%	0.0740		2.63%

The apparent order of accuracy p_χ is obtained by iteratively solving:

$$p_\chi = \frac{1}{\ln r_{21}} \left| \ln \left| \frac{\varepsilon_{32}}{\varepsilon_{21}} \right| + q(p_\chi) \right| \quad (6)$$

$$q(p_\chi) = \ln \left(\frac{r_{21}^{p_\chi} - s}{r_{32}^{p_\chi} - s} \right) \quad (7)$$

where $\varepsilon_{ij} = \varphi_i - \varphi_j$ and $s = \text{sgn} \left(\frac{\varepsilon_{32}}{\varepsilon_{21}} \right)$. φ_k represents the solution in the k th grid. The extrapolated values and the GCI are calculated form:

$$\varphi_{\text{ext}}^{21} = \frac{r_{21}^{p_\chi} \varphi_1 - \varphi_2}{r_{21}^{p_\chi} - 1} \quad (8)$$

$$\text{GCI}_{\text{fine}}^{21} = \frac{1.25 e_a^{21}}{r_{21}^{p_\chi} - 1}, e_a^{21} = \left| \frac{\varphi_1 - \varphi_2}{\varphi_1} \right| \quad (9)$$

$$\text{GCI}_{\text{coarse}}^{21} = \frac{1.25 e_a^{21} r_{21}^{p_\chi}}{r_{21}^{p_\chi} - 1} \quad (10)$$

Where p_χ is the local order of accuracy. Similarly, $\text{GCI}_{\text{coarse}}^{32}$ and $\text{GCI}_{\text{fine}}^{32}$ can be calculated.

The GCI analysis showed that the fine-grid GCI values for upstream water depth and cross-sectional mean velocity were 0.84% and 2.99%, respectively, indicating that the mesh-related uncertainty was small and that the numerical results exhibited good convergence. Although Case-5 was the finest mesh, the differences between Case-4 and Case-5 were only approximately 0.29% for upstream water depth and 0.26% for cross-sectional mean velocity. When the mesh was further refined from Case-4 to Case-5, the improvement in accuracy was limited, whereas the number of cells increased substantially from 3,483,057 to 7,092,517. Therefore, considering both computational accuracy and efficiency, Case-4 was adopted for the subsequent numerical simulations.

4. Results and application

4.1. Local head-loss coefficient

Local loss coefficient is an important indicator for evaluating the performance of the water-retaining weir. The smaller loss coefficient means the flow measurement facility has better flow capacity. As the water-retaining weir changes the flow section of the channel, the flow direction here is deflected and the flow turbulence is enhanced, which are the main reasons for the large energy loss. The local head loss is much greater than the frictional head loss, so the latter can be negligible.

The head loss between measuring section i and section j , denoted as ΔH_{ij} , can be obtained from the energy

equation as follows:

$$\Delta H_{ij} = H_i - H_j = \left(Z_i + \frac{p_i}{\gamma} + \alpha_i \frac{v_i^2}{2g} \right) - \left(Z_j + \frac{p_j}{\gamma} + \alpha_j \frac{v_j^2}{2g} \right) \quad (11)$$

Where H_i and H_j are the total heads at sections i and j ; Z_i and Z_j are the elevation heads; p_i and p_j are the pressures; v_i and v_j are the cross-sectional mean velocities; γ is the specific weight of water and α_i and α_j are the kinetic-energy correction coefficients.

In the present calculation, the kinetic-energy correction coefficients were approximately taken as unity, namely $\alpha_i = \alpha_j = 1$. Therefore, the head loss can be simplified as:

$$\Delta H_{ij} = \left(Z_i + \frac{p_i}{\gamma} \right) - \left(Z_j + \frac{p_j}{\gamma} \right) + \frac{v_i^2 - v_j^2}{2g} \quad (12)$$

Accordingly, the local head-loss coefficient between sections i and j is given by:

$$\zeta_{ij} = \frac{2g \Delta H_{ij}}{V^2} \quad (13)$$

Where V is the characteristic velocity, calculated as the ratio of the experimental discharge Q to the total flow area of the three $0.22 \text{ m} \times 0.22 \text{ m}$ pressurized box culverts.

Based on the overall trends observed in the experimental results of Model 1 and Model 2, the local head-loss coefficient exhibited a monotonic decay with increasing relative downstream water depth (h/p , h represents the downstream water depth and p represents the height of the weir). Specifically, the local head-loss coefficient decreased sharply at low relative depths and gradually approached stable values as the submergence increased. This characteristic pattern closely aligns with the mathematical behaviour of an exponential decay function. Accordingly, the relationship is fitted using an exponential decay function in the form of:

$$y = y_0 + A \times e^{-\frac{x-x_0}{\omega}} \quad (14)$$

where y_0 , A , x_0 , and ω are fitting constants.

The fitted relationships between the local head-loss coefficient (measured at Sections 1–4) and the relative downstream water depth h/p for Cases A and B of Model 1 and Model 2 under submerged flow conditions are showed in Figure 6. It can be seen that, for both Model 1 and Model 2 under Cases A and B, if the downstream relative depth, namely h/p , was less than 1.2, the total local head-loss coefficients flowing through overflow weirs increased rapidly with the reduction of h/p .

As the water flow passed the water-retaining weir, local head-loss coefficient arose from the abrupt change in channel structure at the inlet and outlet, the obstruction imposed by the weir wall and the energy dissipation by roller and vortex downstream of the weir. Most energy was dissipated through the blockage at the overflow weir and vortex-induced mixing downstream. With increasing relative submergence, the effective blockage became weaker and the downstream region provided more space for the recirculating flow to develop, resulting in a smaller local head-loss coefficient. Once the relative submergence exceeded a certain level and the water-retaining weir was sufficiently submerged, the local head-loss coefficient approached an approximately constant value. In contrast, when the relative submergence fell below a threshold, reduced submergence increased the blockage effect and restricted the downstream region, leading to congested and unstable recirculation and more disordered flow. Under these conditions, the local head-loss coefficient increased, corresponding to the steeper segment in the Figure 8(a–d). Ultimately, when the relative depth in downstream weir well was greater than 1.4, the total head-loss coefficient of Model 1-Case A tended to approach a constant value of 2.84, while the constant value for Model 2-Case A was 3.86. For Case B, the constant value for Model 1 was 2.43 while the constant value for Model 2 was 3.13.

When the water-retaining weir structure was modified, comparison across the two structures, namely Case A and B, showed that increasing the overflow weir height markedly decreased the local head-loss coefficient in the submergence cases. In addition, replacing a downstream concave bed with a flatbed transition further reduced the loss. These effects occurred because a higher overflow weir height raised the upstream backwater level and enlarged the upstream flow area, while a flat downstream bed promoted a smoother transition and suppressed local backflow downstream of the weir.

4.2. Submergence and discharge coefficient

The discharge coefficient was used to represent the overall capacity of the water-retaining weir under free-overflow conditions. It contained the combined influences of weir structure and flow behaviour on the effective flow area, velocity distribution and associated energy losses. Depending on the operating discharge, the water-retaining weir exhibited free-weir flow, in which the downstream water depth had a negligible effect on the outflow and a clear water-level difference existed between the upstream and downstream sides. When the downstream water level rose exceeding the crest elevation and began to constrain the outflow, the overflow

weir transitioned to submerged discharge, where the upstream-downstream water-level difference became relatively small. Under free discharge, the discharge coefficient was treated as approximately constant for a given weir configuration and flow condition. Therefore, the flow coefficients for different experimental discharges under free overflows were listed in Tables 3 and 4, including the average flow coefficients.

The water-retaining weir can be looked as a wide crest weir. According to Fritz and Hager (1998), the flow equation under free flow condition for this type of weirs can be calculated by

$$Q = C_d \sigma B N \sqrt{2g} H_0^{1.5} \quad (15)$$

Where C_d is flow coefficient, σ is submergence coefficient, B is weir width (m), N is the number of gate holes, H_0 is weir head excluding the upstream kinetic head (m).

$$C_d = \frac{Q}{\sigma B N \sqrt{2g} H_0^{1.5}} = f(Q, B, N, H_0, \sigma) \quad (16)$$

For the water-retaining weir shown in Figure 3, the weir width (B) and the number of gate holes (N) is set to be 0.22 m and 1, respectively. Then, the discharge coefficient at different flow discharge is calculated.

The study showed that the discharge coefficient increased as the weir height decreased, although the improvement was modest. The discharge coefficients obtained from the present experiments ranged from 0.409 to 0.479, which was consistent with the range documented in the literature (Azimi et al., 2014; Nourani et al., 2021a). Under free-flow conditions, the mean discharge coefficient for Model 1 in Case A was 0.429 and 0.442 for Model 2, representing a 3.0% increase. In Case B, the corresponding mean values were 0.433 (Model 1) and 0.443 (Model 2), representing a 2.3% increase. This trend was attributed to the changed flow velocity associated with a reduced weir height, which raised the discharge coefficient. However, the increase was constrained by the weir structure and its flow-regulation mechanism. Moreover, it can be seen from the Tables 3 and 4 that the installation of an energy dissipation pool had only a minor effect on the discharge coefficient, indicating that the absence of a downstream energy dissipation pool had no significant influence on the discharge capacity of the water-retaining weir. The upstream head and the height of the weir mainly controlled the overflow process over the weir crest.

When the weir configuration and inlet discharge remain unchanged, the discharge coefficient can be treated as approximately constant and selected from Tables 3 and 4 for the corresponding operating condition. If an increase in downstream water depth leads

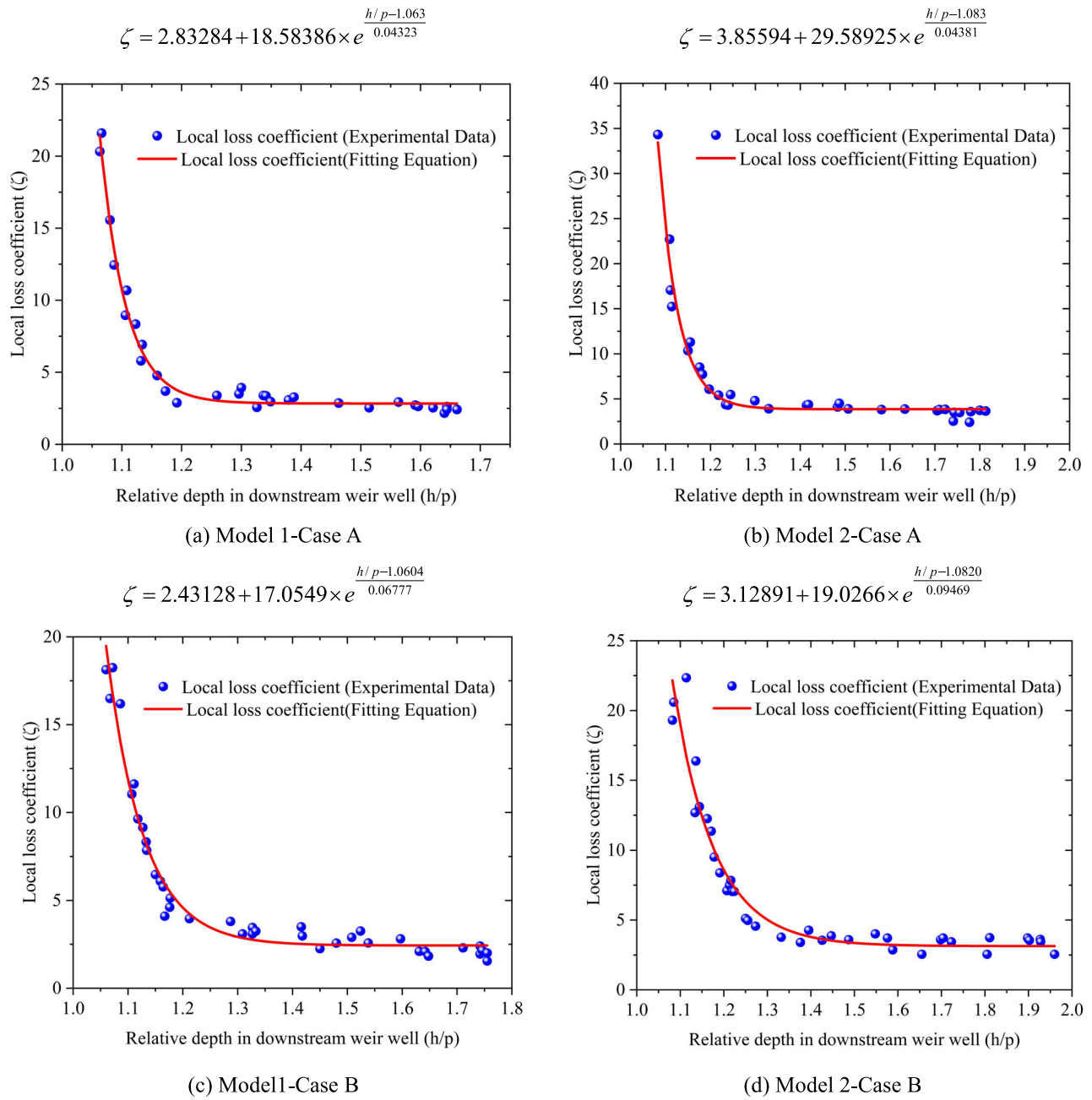


Figure 8. Relation curve of local head loss coefficient and relative depth in downstream weir well for various Models. (a) Model 1-Case A $\zeta = 2.83284 + 18.58386 \times e^{\frac{h/p - 1.063}{0.04323}}$. (b) Model 2-Case A $\zeta = 3.85594 + 29.58925 \times e^{\frac{h/p - 1.083}{0.04381}}$. (c) Model 1-Case B $\zeta = 2.43128 + 17.0549 \times e^{\frac{h/p - 1.0604}{0.06777}}$. (d) Model 2-Case B $\zeta = 3.12891 + 19.0266 \times e^{\frac{h/p - 1.0820}{0.09469}}$.

Table 3. Flow coefficients for water-retaining weirs in Case A.

Experimental discharge (m^3/s)	Model 1		Model 2	
	Flow coefficient (m)	Average value (m_{avr})	Flow coefficient (m)	Average value (m_{avr})
0.0112	0.405	0.429	0.409	0.442
0.0168	0.414		0.434	
0.0224	0.434		0.438	
0.0280	0.446		0.455	
0.0335	0.445		0.476	

Table 4. Flow coefficients for water-retaining weirs in Case B.

Experimental discharge (m ³ /s)	Model 1		Model 2	
	Flow coefficient (m)	Average value (m_{avr})	Flow coefficient (m)	Average value (m_{avr})
0.0112	0.407	0.433	0.410	0.443
0.0168	0.419		0.424	
0.0224	0.439		0.434	
0.0280	0.445		0.466	
0.0335	0.455		0.481	

Table 5. Different empirical function forms and fitted equations for the submergence correction factor.

Function number	Function	Fitted equation
1	Lorentzian function	$\sigma = 1.00506 - \frac{0.07014}{4 \times (1.07404 - x)^2 + 0.04888}$
2	Exponential function	$\sigma = 0.01721 + 0.97019[1 - \exp(-8.79666(1 - x))]$
3	Power-law function	$\sigma = 0.83208(1 - x)^{0.24914}$
4	Fourth-order polynomial function	$\sigma = 1.10114 - 0.30376x - 0.43689x^2 - 0.14321x^3 - 0.01329x^4$

Table 6. Performance evaluation of different empirical functions for fitting the submergence correction factor.

Function number	1	2	3	4
R^2	0.9835	0.9821	0.7874	0.8475
RMSE	0.0498	0.0518	0.1785	0.1512
MAE	0.0331	0.0372	0.1665	0.1290

to submergence, the submergence coefficient for each submergence level can be calculated using Equation (16).

$$\delta = \frac{Q}{C_d B N \sqrt{2g} H_0^{1.5}} = f(Q, B, N, H_0, C_d) \quad (17)$$

To represent the nonlinear relationship between the submergence correction factor σ and the no dimensional submergence index h_s/H , the experimental data were fitted with using a Lorentzian function. This function reproduces the observed trend—approximately constant at low submergence, a rapid transition near the critical condition, and a marked decrease thereafter while offering a more stable fit than high-order polynomials.

The Lorentzian function is written as

$$L = \frac{1}{\pi} \times \frac{\gamma}{(x_0 - x)^2 + \gamma^2} \quad (18)$$

Here, x_0 denotes the location of the transition, and γ controls the width of the curve. By defining $\omega = 2\gamma$, the expression can be written equivalently as

$$L(x) = \frac{2}{\pi} \times \frac{\omega}{4(x_0 - x)^2 + \omega^2} \quad (19)$$

And to fit the submergence correction factor, a scaling parameter A and a baseline level y_0 are introduced, yielding a ‘downward’ Lorentzian form for σ

$$\sigma(x) = y_0 - \frac{2A}{\pi} \times \frac{\omega}{4(x_0 - x)^2 + \omega^2}, x = \frac{h_s}{H} \quad (20)$$

To further justify the selection of the empirical equation for the submergence correction factor, the Lorentzian

function was compared with commonly used empirical function forms. The same experimental dataset was fitted using the Lorentzian, exponential, power-law, and fourth-order polynomial functions. Table 5 listed the candidate empirical function forms and their fitted equations, and Table 6 presented the corresponding fitting-performance indicators. The evaluation indicators included the coefficient of determination R^2 , root mean square error RMSE, and mean absolute error MAE. A value of R^2 closer to 1 indicates better overall goodness of fit, while smaller RMSE and MAE values indicate smaller deviations between predicted and experimental values.

As shown in Table 6, the R^2 , RMSE, and MAE of the Lorentzian function were 0.9835, 0.0498, and 0.0331, respectively, giving the best performance among the four tested functions. Therefore, the Lorentzian function was adopted as the empirical relationship between the submergence correction factor and the dimensionless controlling variable x .

As shown in Figures 9(a–d), for Cases A and B of Model 1 and Model 2, the submergence ratio x varied from -0.5 to 1 , and the corresponding submergence coefficient ranged from 0 to 1 . Although the four types differed in weir height and well structure, they exhibited a broadly consistent relationship between the submergence coefficient and the submergence ratio. Taking the weir crest as the datum, the water levels below the crest represent as $-h_s$, while those above the crest by $+h_s$.

When the downstream water level behind the weir was slightly below the weir crest (i.e. x was negative and approaching 0), the crest discharge was affected by the downstream water level, indicating a transitional regime between free overflow and submerged weir flow. The submergence coefficient remained close to 1 , indicating that the submergence coefficient was mainly affected by the upstream head and was only

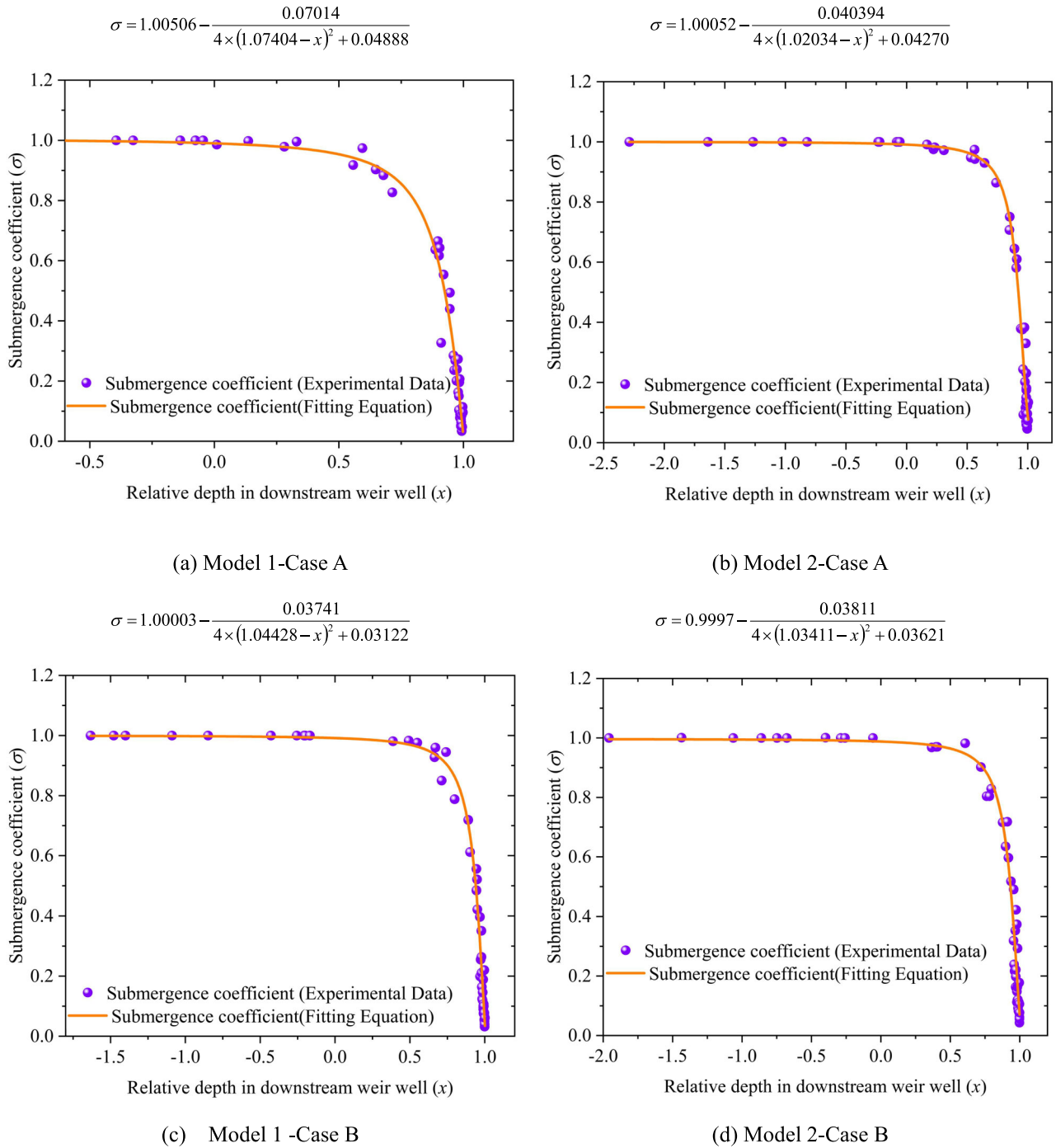


Figure 9. Relation curve of submergence coefficient σ and submergence degree x for various Models. (a) Model 1-Case A $\sigma = 1.00506 - \frac{0.07014}{4 \times (1.07404 - x)^2 + 0.04888}$. (b) Model 2-Case A $\sigma = 1.00052 - \frac{0.040394}{4 \times (1.02034 - x)^2 + 0.04270}$. (c) Model 1-Case B $\sigma = 1.00003 - \frac{0.03741}{4 \times (1.04428 - x)^2 + 0.03122}$. (d) Model 2-Case B $\sigma = 0.9997 - \frac{0.03811}{4 \times (1.03411 - x)^2 + 0.03621}$.

weakly constrained by the tailwater level. As the submergence ratio continued to increase, the rising tailwater reduced the conveyance capacity and the flow shifted from free overflow toward submerged conditions, causing the submergence coefficient to decrease monotonically with increasing submergence. It reflected that the submergence coefficient was controlled by

the downstream conditions, thereby markedly reducing conveyance capacity. Once the submergence ratio exceeded the critical range ($h_s/H > 0.72$ for Model 1 and $h_s/H > 0.65$ for Model 2), the submergence coefficient exhibited a sharp nonlinear drop. As the downstream water level increased, the accumulated water behind the weir exerted a distinct ‘jacking effect’ on the discharge,

Table 7. Comparison of submergence coefficient among water-retaining weir structures under representative submergence conditions.

Submergence ratio (x)			0.95	0.90	0.80	0.50
σ	Case A	Model 1	0.5208	0.6809	0.8034	0.9241
		Model 2	0.5854	0.7208	0.8212	0.9229
	Case B	Model 1	0.6211	0.7516	0.8470	0.9400
		Model 2	0.6068	0.7400	0.8375	0.9341

and the transmission of the counter-pressure caused the hydraulic slope of the flow section to flatten. This phenomenon affected dissipation of a portion of the potential energy when it was analyzed from the perspective of the energy equation, significantly reduced the effective head available for conversion into kinetic energy, which in turn led to a nonlinear plunge of the submergence coefficient beyond the critical threshold.

As shown in Table 7, four representative submergence ratios ($x = 0.95, 0.90, 0.80$ and 0.50) were selected to characterize the water-retaining weir response from incipient submergence to deep submergence. Specifically, $x = 0.95$ indicated the initial emergence of submergence effects, $x = 0.90$ – 0.80 corresponded to the clear submergence-controlled regime, and $x = 0.50$ represent a typical condition with a severe submergence region. Table 5 showed that, under identical operating conditions, the submergence coefficient was generally higher for the structures without an energy dissipation pool than for those with an energy dissipation pool (Case A and B). This indicated that removing the energy dissipation pool allowed the tailwater level downstream of the water-retaining weir to rise more readily, thereby strengthening the backwater effect and increasing the degree of submergence. Across these conditions, Model 1-Case B consistently exhibited the largest submergence coefficient, suggesting superior discharge performance under the same submergence when tailwater rose or backwater intensified. This behaviour reduced discharge fluctuations induced by downstream disturbances and improved the hydraulic stability of the release process.

In this study, equation fitting was conducted only for four typical water-retaining weir configurations, namely Model 1-Case A, Model 2-Case A, Model 1-Case B, and Model 2-Case B. The Eqs. in Figures 8 and 9 were empirical relationships fitted from the experimental data in this study, and their applicability was limited by the tested configurations and parameter ranges. Table 8 listed the valid parameter ranges corresponding to each fitted equation.

The fitted equations were used to transform discrete experimental data into continuous parameter relationships, thereby revealing the basic variation patterns of local head loss, submergence effect, and discharge capacity of water-retaining weirs under different flow regimes.

Table 8. Applicable parameter ranges of the fitted equations for the water-retaining weir configurations.

		ζ	h/p	σ	x
Case A	Model 1	(2.417, 21.593)	(1.063, 1.661)	(0.034, 1)	(−5.94, 1)
	Model 2	(2.011, 18.244)	(1.06, 1.755)	(0.032, 1)	(−1.63, 1)
Case B	Model 1	(3.652, 34.319)	(1.083, 1.813)	(0.046, 1)	(−2.29, 1)
	Model 2	(2.55, 22.347)	(1.082, 1.96)	(0.043, 1)	(−1.96, 1)

The specific coefficients and numerical relationships in the fitted equations varied with the geometry of the water-retaining weir. Therefore, these equations are basically applicable to the four tested configurations and the corresponding experimental data ranges in this study, but for other untested geometries, the identified trends provide reliable techniques and methods for analyzing similar problems with further experimental or numerical validation.

4.3. Flow pattern in weir well

As the flow passed over the water-retaining weir, vortices formed downstream. Post-processing of the contour fields identified their locations and distribution, as shown in Figures 10–13. Four operating conditions were selected for comparative analysis, and the corresponding parameters were summarized in Table 9. The inlet discharge was fixed ($40/\lambda_Q \text{ m}^3/\text{s}$) for all schemes, while adjusting the depth of downstream weir controlled the flow pattern. Owing to practical constraint and measurement accuracy, the target depth of downstream weir was maintained at approximately 0.540 m for Model 1 and 0.465 m for Model 2. A comparison of the weir head between the experiments and numerical simulations showed that they were in good agreement, the relative error of all schemes remained with 5%. The simulated contour flow pattern can therefore be further compared with the observed flow-pattern images to examine differences in flow development in more detail.

During the physical model tests, the downstream flow patterns under representative conditions were visually observed. The numerical simulation was mainly used to complement the physical model experiments by revealing downstream flow field structures that were difficult to observe directly. In the numerical simulations, a steady state was assumed once the difference between the inlet

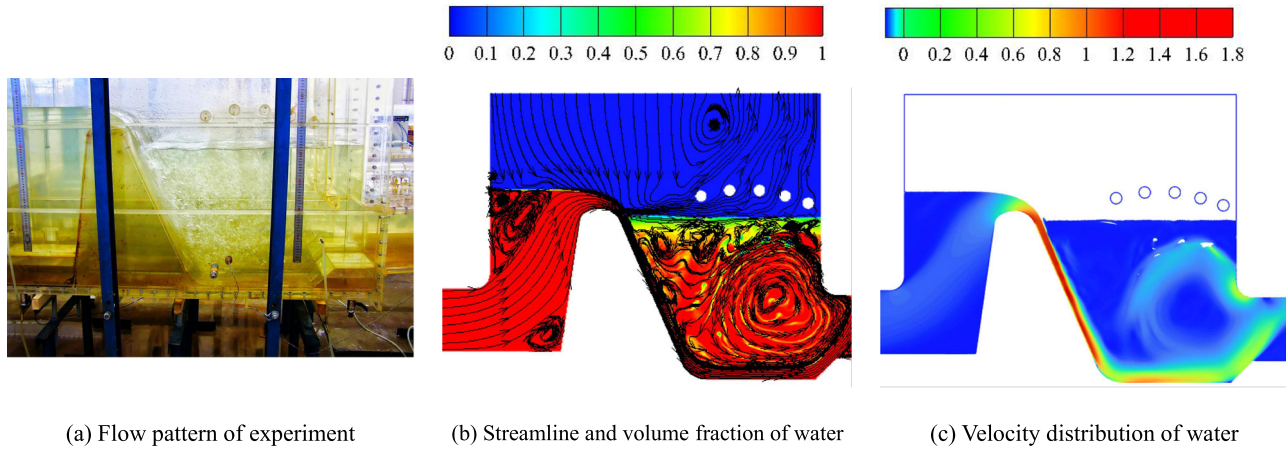


Figure 10. Comparison of the flow pattern between the experiment and simulation (Model 1-Case-A). (a) Flow pattern of experiment. (b) Streamline and volume fraction of water. (c) Velocity distribution of water.

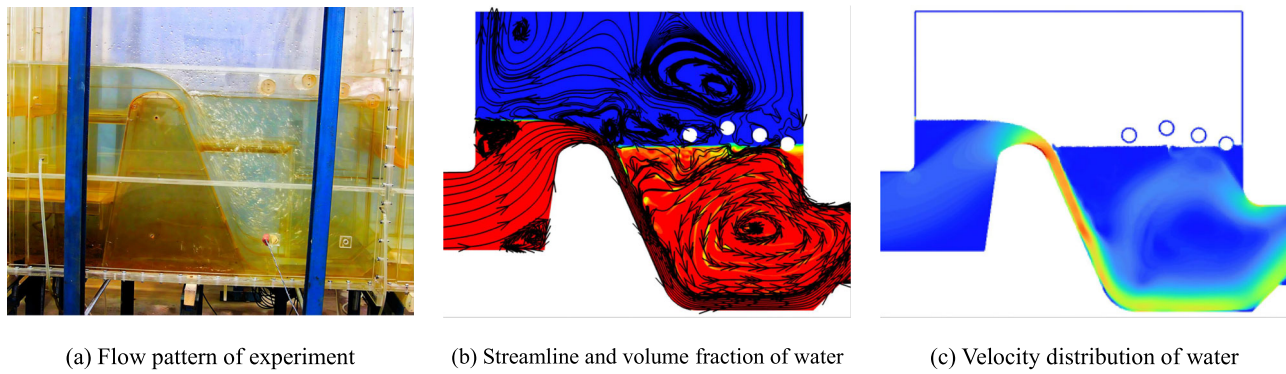


Figure 11. Comparison of the flow pattern between the experiment and simulation (Model 2-Case-A). (a) Flow pattern of experiment. (b) Streamline and volume fraction of water. (c) Velocity distribution of water.

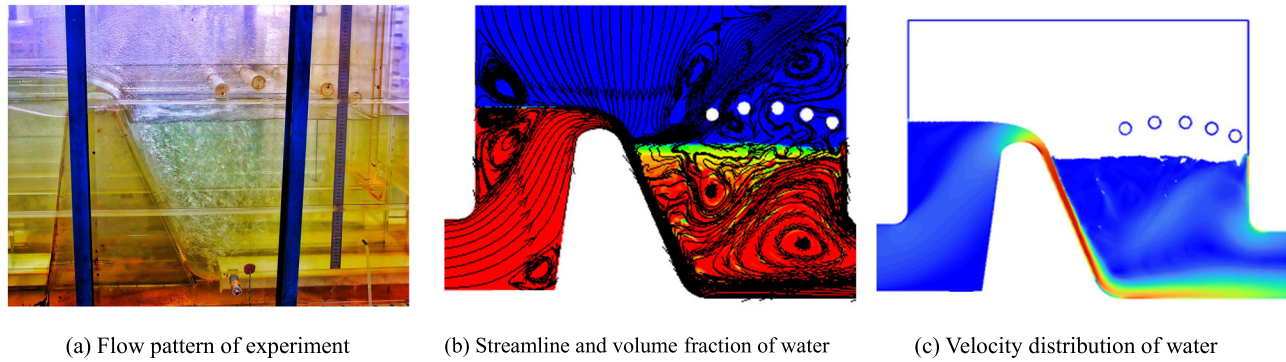


Figure 12. Comparison of the flow pattern between the experiment and simulation (Model 1-Case-B). (a) Flow pattern of experiment. (b) Streamline and volume fraction of water. (c) Velocity distribution of water.

and outlet discharges fell below 2%. Contour and streamline maps were then obtained at the corresponding stabilized time for each case. Figures 10–13 compared the downstream flow patterns for Model 1 and Model 2 under Cases A and B. By analyzing the experiments and numerical simulations, the key hydraulic performance of the Models was assessed in detail. In each figure, subplots (a) presented experimental photographs capturing the

free-surface profile and overall flow pattern. Subplots (b) showed simulated streamlines together with the VOF volume-fraction field to identify flow separation and backflow. And subplots (c) reported the velocity distribution of the liquid (water) phase obtained by masking the computational domain using a liquid volume-fraction threshold (≥ 0.5).

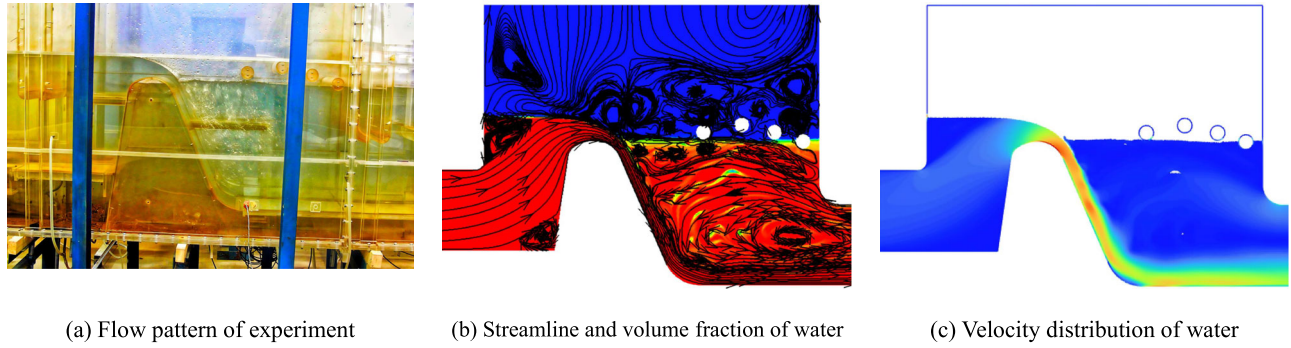


Figure 13. Comparison of the flow pattern between the experiment and simulation (Model 2-Case-B). (a) Flow pattern of experiment. (b) Streamline and volume fraction of water. (c) Velocity distribution of water.

Table 9. Detailed parameters for four schemes.

Scheme	Case	Model	Discharge (m ³ /s)	Depth of downstream weir(m)	Weir head (m)		Relation error (%)
					Experiment	Simulation	
1	A	1	0.0224	0.541	0.064	0.065	1.56
2		2		0.464	0.066	0.064	−3.03
3		1		0.540	0.061	0.064	4.92
4		2		0.466	0.063	0.064	1.59

When the upstream water level was substantially higher than the downstream water level, the flow accelerated along the overflow weir and entered the downstream weir as a high-velocity jet. A continuous high-velocity jet formed from the weir crest to the downstream slope. The strong velocity contrast between the velocity of jet and the relatively stationary downstream water produced intense shear at the interface, which broke the free surface and air entrainment. This process produced bubble and rolling flow within the weir well, forming a highly mixed air–water region. A relatively large velocity gradient existed between the high-velocity jet region and the low-velocity recirculation zone, which was an important reason for enhanced local shear mixing and energy dissipation. The CFD results provide additional evidence for explaining the differences in local head loss and energy dissipation among different water-retaining weir configurations. Both experimental photographs and numerical results (in Figures 10–13) showed that the upstream water surface of the water-retaining weir remained relatively stable, and the simulated upstream stable water-surface position was generally consistent with the experimental observation, as shown in Table 9.

Downstream of the water-retaining weir, the flow patterns were restricted by the available depth of downstream and ultimately reached a dynamic equilibrium. An obvious vortex structure developed behind the overflow weir in each operating conditions, characterized closed streamline that displayed the downstream main-recirculation domain. In addition, a certain degree of free-surface fluctuation and bubble activity appeared

near the downstream free surface in all cases. The spatial distribution of vortices within the weir chamber varied substantially among the different weir configurations. Using the weir surface and anti-vortex beams as visual references, the overflow nappe shape, falling position of the overtopping flow, and downstream water-surface development observed in the experiments show good correspondence with the simulated free-surface pattern and streamline distribution, as shown in Figures 10–13.

The experimental photographs showed that, in both Cases A and B, Model 2 exhibited a weaker surface roller of the downstream water-retaining weir and reduced bubble activity within the weir well compared with Model 1. The interface near the free surface (represented by the transition band around $\alpha \approx 0.5$) appeared more continuous and stable, consistent with the changes observed in the simulated velocity field and streamline. For the numerical results, Model 2 consistently exhibited a lower peak velocity in the near-wall jet and a more uniform velocity distribution within the recirculation domain, indicating weaker jet impact and shear-layer intensity and thus improved flow stability. The free-surface transition zone in Model 2 was also thinner, smoother, and more continuous. The main flow pathway shifted towards the downstream box culvert more smoothly in Case B. This indicated that flow momentum was transported more directly along the bottom toward the downstream box culvert rather than being dissipated through strong internal circulation within the downstream weir. In Case

A, the downstream flow was dominated by a clearer large-scale primary recirculation structure, while small-scale vortices were markedly reduced.

In contrast, Model 1 tended to generate multiple small-scale vortical structures outside the primary vortex in both cases. This behaviour was attributed to the higher weir height in Model 1, which produced a larger post-weir drop and stronger energy variation, thereby increasing flow unsteadiness. These consistencies confirm that the numerical results reliably reproduce the essential flow features and can therefore be used to explain the hydraulic differences between the two weir configurations.

5. Conclusions

This study combined experiments with 3D numerical simulations to investigate the hydraulic performance of the water-retaining weirs in a long-distance water supply system. A CFD model was developed using the realizable k -epsilon turbulence model and the VOF method to analysis free-surface flow. The effects of the water-retaining weir structure on the local head-loss coefficient, submergence coefficient, discharge coefficient, and downstream flow patterns were further analyzed, and the results were used to propose structural optimization measures for the water-retaining weir. The main conclusions are as follows:

- (1) The local head-loss coefficient was primarily controlled by the relative submerged depth. With increasing relative submerged depth, the coefficient gradually levelled off and eventually approached a nearly constant value. The calculations also indicated that the flat-bed layout downstream of the water-retaining weir weakened recirculation and reduced local head loss, thereby enhancing hydraulic conveyance capacity.
- (2) A reduction in weir height led to an increase in the discharge coefficient, and the extent of improvement was relatively small. Under free-flow conditions, the lower water-retaining weir consistently exhibited higher discharge coefficients than the higher one under the same downstream arrangement. The installation of a downstream energy dissipation pool had only a minor effect on the discharge coefficient.
- (3) The submergence coefficient remained almost unchanged when the downstream water depth was lower than the downstream weir height, suggesting that tailwater had little effect on discharge in this range. As the downstream water depth increased towards the weir height, the coefficient began to

decline. Once the downstream water depth exceeded the weir height, submerged overflow became evident and the coefficient decreased markedly. In the submerged-flow regime, the cases without an energy dissipation pool exhibited higher submergence coefficients and therefore a higher discharge capacity.

- (4) Flow visualization revealed that compared with the concave-bed configuration, the flat-bed configuration exhibited a more stable flow pattern and fewer small-scale vortices. This indicated that the concave bed was associated with stronger local recirculation and more pronounced energy dissipation, while the flat bed provided a smoother downstream flow regime with smaller local losses and better discharge conditions.

From an engineering perspective, the present results indicate that the application of a water-retaining weir is necessary for the long-distance conveyance system and sensitive to structural design details. Within the range examines in this study, the low-crest weir with a flat downstream bed performed most favourably in terms of head loss, discharge efficiency and flow stability.

Given the scope of this study, related research inevitably has certain limitations. Considering the rapid changes in the flow streamline in the upstream weir well and the flow complexity in the downstream weir well, the local head losses generated at the culvert-like inlet and outlet transitions could not be quantified separately. Therefore, the experiments mainly measured the head difference between the upstream and downstream observation sections of the water-retaining weir. In addition, the numerical simulation mainly focused on representative cases to explain downstream vortices, recirculation, and velocity distributions that were difficult to observe directly in the physical experiments, and a full parametric analysis covering all discharges, water depths, and weir configurations has not yet been conducted. Second, the air vents in this study were retained as part of the prototype structure and experimental model arrangement to represent the ventilation condition in the actual engineering configuration. However, the size, location, and venting capacity of the air vents were not investigated as independent variables.

Future work should add pressure or water-level measurement points at the inlet and outlet transition sections of the water-retaining weir to conduct their head losses analysis under typical flow patterns, such as well flow. The CFD simulation range should also be expanded, and quantitative indicators such as vorticity, turbulent kinetic energy, and energy dissipation should be introduced to more systematically evaluate the hydraulic performance and local-loss components of different water-retaining

weir configurations. The effects of air-vent size, location, number, and venting capacity on air–water exchange, pressure fluctuation, flow stability, and overall hydraulic performance should also be further investigated.

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Data availability statement

The data that support the finding of this study are available from the corresponding author upon reasonable request.

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