Experimental and numerical study on hydraulic characteristics of multi-level intake for diversion project

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Abstract: Multi-level intake as an inlet of diversions has the advantage of selective withdrawal. To further understand the hydraulic characteristics of this inlet hydraulic structure, experimental model test and numerical simulation were carried out. The results of this study show that the water flow in the diversion channel is stable, and the flow pattern in the lock chamber section is disordered. In addition, the pressure near the entrance of the tunnel is linear, and the water head loss of the upper layer is the largest. Further, the formation of potential vortex flow can be prevented by controlling and scheduling. The results presented in the current work provide new insights into the hydraulics of flow in stratified intake structure and can be used in the design and operation of similar hydraulic projects.

Keywords: Multi-Level Intake; Experimental Model Test; Numerical Simulation; Flow Velocity; Pressure; Water Head Loss

1. Introduction

As people pay more attention to the protection of ecological environment and the improvement of life quality, stratified water intake is becoming more and more common[1-3]. The purpose is to selectively use water bodies at different elevations, realize the rational allocation of water resources, and reduce the impact on the ecological environment.

For multi-level intake, the flow pattern is more complex than that of deep-water intake. At present, research of multi-level water intake layout is mainly stop log gate, which is used in hydropower stations. The main hydraulic properties addressed in this research include flow pattern, vortex, head loss, water temperature. In this study both the model test and numerical simulation are adopted. Yang et al. summarized model design, velocity distribution and head loss law of pumped storage power station by analyzing research results of domestic stations[4]. Du et al. in their numerical research considered the intake of a hydropower station, and used the turbulence model to simulate the flow field of the intake, and classify the hydraulic characteristics of the intake of the hydropower station from the flow pattern, velocity and pressure distribution, head loss and other parameters[5]. Moreover, Dong and Yang both take a hydropower station as an example, and use both test and numerical method to simulate the hydraulic characteristics of stratified intake[6-7].
Therefore, the study of multi-level water intake can refer to the hydropower station. This investigation is conducted to understand how the flow behaves within this inlet type in such as the water surface flow at the intake, the vortex and the head loss as key elements of the intake design. To do so, the stratified intake of a diversion project is taken as research object, and the hydraulic characteristics of stratified intake are discussed by means of hydraulic model test and numerical simulation technology, so as to provide the basis for similar projects.

2. Research methods

2.1. Hydraulic model
To satisfy the similarity requirements for the model test, considering the conditions of the test site and test equipment, the model is scaled down by 1:20 taking into consideration the dynamic similarity between the prototype and model.

The water intake model is mainly composed of water supply system, upstream reservoir, diversion channel, sluice chamber, water tunnel, water measurement system and return water system. The upstream reservoir is placed 150m above the gate chamber inlet. The downstream part is the pressure tunnel with a diameter of 6.7m and a length of 150m. Both sides of the inlet are set to an elevation of 125m, with a total length of about 330m. The total length of the model is about 16.5m. Figure 1 shows the longitudinal and traverse section sketches of intake structure. The intake gate chamber and pressure tunnel shall be set out according to the design drawings and made of plexiglass. The upstream diversion channel and hillside shall be positioned with the panel and plastered with cement mortar.

![Figure 1. Sketch of intake structure (The elevations are in m, and the rest is in cm).](image)

The hydraulic physical model adopts electromagnetic flowmeter to control the flow. The free surface is measured by needle water level gauge, which is set in the reservoir. The flow velocity is measured by photoelectric propeller flow meter. The pressure is measured by piezometer.

2.2. Numerical model
In this study, CFD software and RNG $k - \varepsilon$ model and VOF method were used. The model includes reservoir and intake channel, sluice chamber section and tunnel section (Figure 2). The inlet size is the same as the physical model.

The shape of the computational domain adopts 3D model, and the grid is divided into Cartesian orthogonal structure grid. The whole computational domain is 430m long along the flow direction, 240m wide at the widest part and 70m deep, and is composed of 1 million hexahedral grids. It is found that when the number of grids is greater than $(0.9-1.0) \times 10^6$, there is no significant difference between the results, so the number of grids used is controlled to be $1.0 \times 10^6$. 


The inlet boundary of the computational area is set as the pressure inlet, and the corresponding water level is set. The outlet boundary is set as the average velocity, and a certain velocity and flow direction are set. The von Neumann boundary condition is adopted for the pressure on the outflow boundary. Wall boundary conditions can be no-slip velocity condition. The top surface in contact with the air is set as a symmetrical surface. The initial water body range is set for the reservoir, and the initial water level is set, and the pressure is hydrostatic pressure. The computational conditions include lower water intake (reservoir water level is 96 m) and upper, middle and lower water intake (reservoir water level is 118 m) respectively.

Numerical simulation of 3-D incompressible turbulent flow is adopted. The model consists of mass conservation equation, momentum conservation equation and energy conservation equation. The water temperature is 20℃.

3. Results and Discussion

3.1. Flow pattern and velocity

Table 1 shows the computational conditions. Figure 3 presents the velocity vector for different flow conditions. As it can be seen the velocities computed is closer to the measured data. The velocity distribution law of the computational value and the experimental value along the water depth direction is the same. The vertical velocity distribution in front of the intake is not uniform.

![Figure 2. Computational domain.](image)

Table 1. Computational condition.

| Condition | Water level (m) | Discharge (m³/s) | Remarks         |
|-----------|-----------------|------------------|-----------------|
| 1         | 96.0            | 40.0             | Lower gate intake |
| 2         | 96.0            | 50.0             | Lower gate intake |
| 3         | 118.0           | 50.0             | Upper gate intake |
| 4         | 118.0           | 50.0             | Middle gate intake |
| 5         | 118.0           | 50.0             | Lower gate intake |

Figure 4 is layout of velocity measuring point. Figures 5-8 are velocity vertical distribution for measuring points at various levels and for different flow discharges. When the upper, middle and lower gate takes water respectively, the velocity in a certain elevation range of the intake is larger. The maximum vertical velocity of upper, middle and lower gate is 0.23m/s, 0.21m/s and 0.33m/s respectively. The minimum vertical velocity is 0.05m/s, 0.07m/s and 0.06m/s respectively.
Therefore, in addition to individual points, it can be seen that the computational velocity distribution of typical cross-section is basically consistent with experimental one, and the magnitude of velocity is also close.

Figure 3. Velocity vector.

Figure 4. Layout of velocity point.
Figure 5. Vertical distribution of inlet velocity (water level is 96m, discharge is 40m$^3$/s).

Figure 6. Vertical distribution of inlet velocity (water level is 96m, discharge is 50m$^3$/s).

Figure 7. Vertical distribution of inlet velocity (1#, water level is 118m, discharge is 50m$^3$/s).
3.2. Pressure distribution

Figure 9 is the layout of pressure measuring points. Table 2 shows the pressure value. The pressure value increases linearly with the decrease of the elevation of the measuring point, and the distribution of the pressure is hydrostatic pressure. The pressure of the same measuring point increases with the rise of upstream water level, and the increase value of pressure is close to that of water depth. By increasing the flow rate, the law of pressure change is basically unchanged. It can be seen that in the range of test conditions, the pressure acting on each measuring point mainly depends on the water depth above the measuring point.

A comparison between the experimental and computational pressure head is shown in Figure 10. A good agreement can be seen between the computational and experimental results. It can be seen from the comparison that the pressure head difference is less than 1%.

| Number | Pressure hole          | Water level: 96m | Water level: 110m | Water level: 118m |
|-------|------------------------|-------------------|-------------------|-------------------|
|       | Elevation (m)          | Discharge: 50m³/s | Discharge: 40m³/s | Discharge: 2×50m³/s |
| Lower gate intake | Upper gate intake       |                   |                   |                   |
| 1     | 93.0                   | 12.9              | 26.9              | 33.8              |
| 2     | 89.8                   | 16.1              | 30.2              | 37.4              |
3.3. Head loss coefficient

Since the diversion flow of the project is small and the change of model test head loss is not obvious, the head loss coefficient is measured by increasing the flow (about 3.7 times of the design flow of 40m³/s). The head loss of the intake is computed by measuring the water level of the reservoir, the water surface elevation of the piezometer on the section of the headrace tunnel (about 2.5 times the pipe diameter from the transition section) and the discharge.

Table 3 shows the test results of the head loss of the intake. In the case of upper intake, the head loss coefficient is 1.256 (computational value is 1.132); in the case of middle intake, the head loss coefficient is 0.606 (computational value is 0.662); in the case of lower intake, the head loss coefficient is 0.428 (computational value is 0.487).

Table 3. Comparison of experimental and computational value of the head loss coefficient.

| Number | Discharge (m³/s) | Water level (m) | Experimental value | Head loss coefficient Average value | Computational value Average value | Remarks |
|--------|------------------|----------------|--------------------|-------------------------------------|----------------------------------|---------|
| 1      | 40               | 110            | 1.151              | 1.256                               | 1.133                            | 1.132   |
| 2      | 50               | 118            | 1.361              | 1.606                               | 1.647                            | 0.662   |
| 3      | 60               | 110            | 0.592              | 0.620                               | 0.620                            | 0.508   |
| 4      | 40               | 118            | 0.426              | 0.428                               | 0.466                            | 0.487   |
| 5      | 50               | 110            | 0.431              |                                     |                                  |         |
| 6      | 50               | 118            |                    |                                     |                                  |         |

The head loss of the upper and middle gate intake is mainly caused by the trumpet inlet, trash rack, breast wall, gate slot, curve section, right angle bifurcation and transition section. The empirical value of local head loss coefficient shows that the local head loss coefficient of intake shape is generally 0.2-0.25, and that of rectangular to circular transition section is about 0.05. The upper sluice is similar to right angle bifurcation, and the head loss coefficient is about 1.5. The water intake of middle sluice is similar to oblique bifurcation, and the head loss coefficient is about 0.5. The above empirical value is the local head loss coefficient of the independent components. The combined head loss coefficient of several components in a short distance is usually less than the sum of the head loss coefficients of the independent components. Therefore, it is reasonable that the head loss coefficients of the upper and middle water intakes are 1.256 and 0.606 respectively.

The head loss of the lower intake is mainly caused by the trumpet inlet, trash rack, breast wall, gate slot, right angle bifurcated pipe and transition section. The empirical value of the local head loss coefficient shows that the local head loss coefficient of the intake shape is generally 0.2-0.25, and that of the rectangular to circular transition section is about 0.05. The head loss coefficient of the right angle bifurcation is about 0.1. The above empirical value is that the sum of the local head loss
coefficient of the independent components is close to the test value, so the head loss coefficient of the lower intake obtained from the test is 0.428, which is reasonable.

Figure 11 shows that the computational head loss coefficient is very close to the experimental value, although there is a deviation. For the six cases studied here, the maximum error is 17.8%, and the minimum error is only 1.6%.

![Figure 11. Comparison between the experimental and computational head loss coefficient.](image)

3.4. Vortex

A three-level gate is set at the water inlet. When the water is taken from the lower layer, the water flows straight into the headrace tunnel. By observing the lowest operating water level, the water flow in front of the water inlet is smooth without vortex. When taking water from the middle layer and upper layer, the water flows into the diversion tunnel through the vertical channel after the entry, then through the trumpet shaped inlet and the working gate section. The water flow characteristics of the middle layer and upper layer are similar, so the test focuses on the inlet vortex of the upper layer.

The motion pattern of vortices is complex, including large-scale quasi stable vortices and small-scale random vortices. According to their motion patterns, they can be divided into three categories: surface depression vortices, intermittent inspiratory vortex and air-core vortex [8]. At present, there are many studies on the scale effect of vortices, but there is a large difference. There is no unified point of view. It is mainly based on the similarity criterion of gravity, the Reynolds number or the equal velocity criterion [9]. At present, the commonly used similarity design method of vortex model is to simulate according to Froude's criterion, so that the influence of viscosity and surface tension is in the secondary position, that is, to ensure the Reynolds number(Re) \(> 3 \times 10^4\) and Weber number(We) \(\geq 120\). When Re and We are lower than the above two values, the flow rate or velocity can be increased appropriately to reduce the scale effect [10]. The Re and We of the model flow under the operating condition of the lowest reservoir level (96m) are shown in Table 4. It can be seen from the table that the model Re of each working condition is greater than \(3 \times 10^4\), which meets the requirements of the critical value. The We is only 99 under working condition 1, which is less than the requirements of the above critical value, and there is a scale effect. At working conditions 2 and 3, the We is greater than the critical value. Therefore, the scale effect can be compensated by increasing the flow rate.
The submerged depth at the top of the tunnel is small, i.e. the submerged depth at the inlet is insufficient, harmful vortices and inspirations are likely to occur at the inlet. It is found that the vortices of this kind of intake arrangement are mainly in the vertical channel behind the gate. The relationship between the development state of the inlet vortex and the upstream water level is shown in Table 5. It can be judged from the table that when the water is diverted from the upper hole, the water level of the reservoir is more than 1.8m from the top of the upper hole, that is, the water level is more than 110.8m, and the intake will not have air-core vortex.

**Table 4. Re and We.**

| Condition | Water level (m) | Discharge (m³/s) | Average velocity at inlet (m/s) | Orifice height (m) | Submerged depth of orifice Center (m) | $Re_m \times 10^4$ | $We_m$ |
|-----------|----------------|-----------------|-------------------------------|------------------|------------------------------------|----------------|-------|
| 1         | 96             | 40              | 0.73                          | 5.50             | 6.75                               | 6.56           | 99    |
| 2         | 96             | 50              | 0.91                          | 5.50             | 6.75                               | 8.20           | 154   |
| 3         | 96             | 100             | 1.82                          | 5.50             | 6.75                               | 16.40          | 618   |

When the water depth at the top of the tunnel is small, i.e. the submerged depth at the inlet is insufficient, harmful vortices and inspirations are likely to occur at the inlet. It is found that the vortices of this kind of intake arrangement are mainly in the vertical channel behind the gate. The relationship between the development state of the inlet vortex and the upstream water level is shown in Table 5. It can be judged from the table that when the water is diverted from the upper hole, the water level of the reservoir is more than 1.8m from the top of the upper hole, that is, the water level is more than 110.8m, and the intake will not have air-core vortex.

**Table 5. Relationship between inlet vortex and water level (m) under typical flow rate (Upper intake).**

| Discharge (m³/s) | Vortex state | Vortex free | Surface depression vortex | Intermittent inspiratory vortex | Air-core vortex |
|-----------------|--------------|-------------|--------------------------|--------------------------------|-----------------|
| 40              | h>110.4      | 107.8<h<110 | 106.7<h<107.8            | h<106.7                        |
| 50              | h>110.8      | 109.2<h<110.8| 108.9<h<109.2 | h<108.9                        |
| 100             | h>110.4      | 108.6<h<110.4| 107<h<108.6            | h<107                          |

4. Conclusions

The multi-level and qualitative water intake of diversion project has the characteristics of selective withdrawal. In this paper, experimental model test and numerical simulation are used to analyze the hydraulic characteristics of multi-level intake of a diversion project, which can provide reference for similar projects in the future.

(1) By comparing different vertical velocity distribution of measuring point, we concluded that the computational velocity is close to experimental one.

(2) The pressure increases linearly with the decrease of the elevation of the measuring point and presents a hydrostatic pressure distribution.

(3) When the upper, middle and lower gate takes water respectively, the head loss decreases step by step, and the water head loss of the upper gate is the largest. The head loss coefficient of each level of the physical model is close to the computational value.

(4) Considering the observation of model test and the previous research results of vortex, when the designed lowest water level is 96m, no air-core vortex will occur. When taking water from the upper and middle gates and when the water level of the reservoir is 1.8m lower than the top elevation of gate, it is recommended to use the next level of intake to avoid the generation of air-core vortex.

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