Numerical analysis of groundwater flow behaviour at a dam site in Karst area during its reservoir impoundment

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Abstract: The Jiangpinghe concrete-faced rockfill dam, with a maximum height of 219 m, is located in the karst area in western Hubei Province, China. The bedrocks at the dam site are highly dissolved, in which karst conduits, faults, fractures and weak interlayers are developed. Given the complex geological conditions, a large-scale grouting curtain covering an area of 376,000 m² was built to control the seepage in the dam foundation. However, the piezometers in the middle section of the lowest grouting tunnel at the right bank manifest an abnormal rise of hydraulic head after reservoir impounding. The hydraulic head varies with a good correlation with the reservoir water level. This abnormal rise of hydraulic head can be attributed to higher permeability of strata where small-scale karst pipes and conduits are developed. In this study, an inverse modelling procedure combined orthogonal design, finite element forward modelling of transient groundwater flow, artificial neural network, and genetic algorithm was adopted to evaluate the permeability of the strata. The inverse results show that the permeability of shallow-buried rocks is one order of magnitude larger than that of deep-buried rocks in the right bank. The numerical simulations show good agreement with field measurements. Based on the inverted results, the groundwater flow behaviour at the dam site was analysed and the performance of the seepage control system was evaluated. It is demonstrated that the abnormal rise of hydraulic head is induced by the permeable structures developed along karst strata, and the seepage control system performs well in lowering the hydraulic head and limiting the amount of leakage through the dam foundation.

1. Introduction

Quite a number of high dams have been constructed in karst areas in China. As a result of karstification, the foundation rocks are typically characteristic of strong heterogeneity and complex flow geometry. The permeability of the bedrocks is hence of strong spatial variability and anisotropy, and uncertainties arise when the permeability is evaluated with a certain amount of hydraulic tests. This is exactly why leakage events frequently occur in high-dam foundations even though impervious barriers are designed and constructed (Chen et al., 2016a; Wang et al., 2016). It is, therefore, of great
importance to evaluate the groundwater flow behavior and assess the performance of seepage control system in dam foundations during reservoir impoundment.

The objective of this study is to characterize the groundwater flow behavior in the foundation rocks at the Jiangpinghe (JPH) dam site. Inverse modelling is performed to evaluate the permeability of the karst strata using the groundwater observations during reservoir impounding. The abnormal rise of hydraulic head at the piezometers in the right bank is discussed and the effectiveness of the seepage control system is then assessed with the inverted hydraulic parameters.

2. Site characterization

2.1. Project description and geological settings

Located in the upper Loushui River in western Hubei, the JPH Hydropower Station mainly consists of a concrete-faced rockfill dam of 219 m high, two hydraulic tunnels and a ground powerhouse in the left bank, and a diversion tunnel and two spillway tunnels in the right bank (Figure 1). The impoundment of the reservoir started on November 9, 2019, and the reservoir water level is rising to the normal pool level (470 m) in September 2021 by about 180 m.

![Figure 1. Layout of the JPH Hydropower Station.](image)

The bedrock outcropping at the dam site is highly dissolved, which mainly consists of limestone and clastic sedimentary rocks. As shown in Figure 2, the bedrocks contain the Longwangmiao formation (C1l) of the lower Cambrian system, and the Gaotai formation (C2g) and the Kongwangxi formation (C2k) of the middle Cambrian system, with an orientation being N30°–40°W/SW < 10°–18°. The Longwangmiao (C1l) and Kongwangxi (C2k) formations are further subdivided into multiple layers according to the structural characteristics of the strata. Formation C1l is weakly karstified, and is regarded as an aquifuge. The overlying strata have experienced different degrees of karstification, with weak interlayers developing along these strata. In formation C2g, that is highly dissolved in the left bank, a large-scale karst conduit develops along the subvertical fault F71, and there are karst fissures along the fault F11. Small-scale karst pipes and cavities are present along the strata C2g and C2k, which provide potential flow paths through the dam foundation. Given that the degree of karstification decreases with increasing depth, the strata C2g and C2k are further subdivided into a near-bank zone (marked by Zone A) and a deep-buried zone (Zone B), as shown in Figure 2. The statistics of hydraulic conductivity of these two zones obtained from borehole packer tests are given in Table 1, showing that Zone A is of slightly higher permeability than Zone B. Noticing that part of the tests
failed due to high permeability in Zone A, we suppose that the permeability of Zone A is underestimated and needs further evaluation.

![Diagram](image)

**Figure 2.** Geological cross-section along the dam axis.

| Zone   | Arithmetic mean | Median   | Geometric mean | 95% confidence intervals          |
|--------|-----------------|----------|----------------|----------------------------------|
| Zone A | 4.71×10⁻⁵       | 2.85×10⁻⁵ | 3.15×10⁻⁵     | [5.43×10⁻⁶, 1.38×10⁻⁴]           |
| Zone B | 2.85×10⁻⁵       | 2.27×10⁻⁵ | 2.17×10⁻⁵     | [5.40×10⁻⁶, 8.78×10⁻⁵]           |

2.2. Seepage control and monitoring systems

Given the poor geological conditions at the dam site, a grout curtain covering an area of 376,000 m² was designed and constructed to control the groundwater flow behaviour in the dam foundation (with its trace shown in Figure 1). Figure 3 shows the layout of the grout curtain in the right bank. The grout curtain was constructed from four layers of grouting tunnels and was extended downwards to elevation 170 m where the permeability of rocks is sufficiently low. A groundwater monitoring system consisting of 65 piezometers and 9 weirs was installed at the dam site. The number of piezometers installed at the dam base (numbered from P1-1 to P1-5) and in the left and right banks are 5, 24 and 36, respectively. All piezometers except P1-1 are located on the downstream side of the grout curtain. Each grouting tunnel was installed with a weir to collect the discharge into the tunnel and one weir was installed at the downstream side of the dam to monitoring the discharge through the dam foundation.

As shown in Figure 4, the measurements at piezometers OHR4-1 ~ OHR4-13, installed in the lowest grouting tunnel (Figure 3), show a good correlation with the reservoir water level. An abnormal rise of hydraulic head, however, has been recorded by the piezometers (OHR4-5 ~ OHR4-8) in the middle section of the lowest grouting tunnel. For example, when the reservoir water level was raised by about 162 m by January 2021, the hydraulic head at piezometer OHR4-7 increased by about 110 m. This increase of hydraulic head is much higher than the values of 44.52 m and 43.72 m recorded at the neighbouring piezometers OHR4-2 and OHR4-11, respectively. This abnormal rise of hydraulic head and its good correlation with pool water level evidence that there exist flow paths from reservoir to the middle section, and the small-scale karst pipes and cavities in Zone A are highly likely to play this role, which will be justified in the following section by inverse modelling.
3. Numerical modelling

3.1. Finite element model and boundary conditions
Given the dimension of the dam and the relatively intensive discontinuities at the site, we adopted the continuum approach for groundwater flow modelling. To further clarify the mechanism of the abnormal head rise in the right bank, a large-scale finite element mesh that well represents the site conditions and hydraulic structures was created (Figure 5). The dimension of the mesh is 2000 m along the river flow direction and 2400 m along the dam axis. The mesh contains 7,273,816 brick and tetrahedral elements and 1,746,933 nodes.

The boundary conditions were specified as follows. The time-variant reservoir water level was prescribed on the surface of the dam and ground surface submerged in the reservoir. The ground surface and downstream surface of the dam submerged in the downstream river channel were prescribed with the corresponding water level, which fluctuated around 292 m under the influence of tail water. The other surface of the dam and ground above the water levels was prescribed as potential seepage boundary. The boundary of the right-bank mountainside (ABCD in Figure 5) and the base of the model was regarded as impermeable according to the site conditions. The groundwater level of the
left-bank mountainside of the model (EFGH) is relatively high, and its distribution could be represented by a quadratic curve. The parabolic variational inequality (PVI) formulation (Chen et al., 2011) was employed for modelling the transient flow at the dam site. The hydraulic parameters in the transient flow model were listed in Table 2, including the hydraulic conductivity \( K_\parallel \) (the component parallel to bedding or structural plane) and \( K_\perp \) (the component perpendicular to the bedding or structural plane), the specific storage \( S_s \), and the specific yield \( \mu \). Most of these parameters were determined based on the packer tests and the site conditions.

![Figure 5. Finite element mesh of the JPH dam site.](image)

**Table 2.** Hydraulic properties of rocks at the dam site.

| Materials          | \( K_\parallel \) (cm/s) | \( K_\perp \) (cm/s) | \( \mu \) | \( S_s \) (m\(^{-1}\)) |
|--------------------|--------------------------|----------------------|---------|-------------------------|
| \( C_{1L}^{1} \) (Left bank) | 3.1\times10^{-5} | 1.0\times10^{-5} | 0.008  | 9.62\times10^{-7} |
| \( C_{2L}^{1} \) (Right bank) | 3.9\times10^{-5} | 1.0\times10^{-5} | 0.008  | 9.62\times10^{-7} |
| \( C_{1L}^{2-2}, C_{2L}^{1-1}, C_{2L}^{1-3}, C_{2L}^{1-5} \) (L) | 5.2\times10^{-5} | 1.7\times10^{-5} | 0.03   | 7.94\times10^{-7} |
| \( C_{1R}^{2-1}, C_{1R}^{1-3}, C_{2R}^{1-5} \) (R) | 4.6\times10^{-5} | 1.5\times10^{-5} | 0.01   | 7.94\times10^{-7} |
| \( C_{2R}^{1-1}, C_{2R}^{1-3}, C_{2R}^{1-5} \) (L) | 8.6\times10^{-5} | 1.0\times10^{-5} | 0.03   | 1.33\times10^{-6} |
| \( C_{1R}^{2-2}, C_{1R}^{1-3}, C_{2R}^{1-5} \) (R) | 7.8\times10^{-5} | 1.0\times10^{-5} | 0.03   | 1.33\times10^{-6} |
| Fault F\(_{371}\) | 1.5\times10^{-4}–1.5\times10^{-3} | 5.0\times10^{-6}–5.0\times10^{-5} | 0.06   | 3.55\times10^{-6} |
| Fault F\(_{11}\) | 1.5\times10^{-4}–1.5\times10^{-3} | 5.0\times10^{-6}–5.0\times10^{-5} | 0.06   | 3.55\times10^{-6} |
| Zone IIIA | 2.0\times10^{-6}–5.2\times10^{-4} | 2.0\times10^{-6}–5.2\times10^{-5} | 0.03   | 1.61\times10^{-6} |
| Zone IIIB | 5.4\times10^{-6}–8.8\times10^{-5} | 1.0\times10^{-6}–5.0\times10^{-5} | 0.03   | 1.61\times10^{-6} |

**Table 3.** Back-calculated values of hydraulic conductivities of rocks and faults (unit: cm/s).

| Hydraulic conductivities | Zone A | Zone B | Fault F\(_{371}\) | Fault F\(_{11}\) |
|--------------------------|--------|--------|-------------------|-------------------|
| \( K_\parallel \) | 3.7\times10^{-4} | 3.6\times10^{-5} | 4.5\times10^{-4} | 6.4\times10^{-4} |
| \( K_\perp \) | 4.2\times10^{-5} | 8.0\times10^{-6} | 5.0\times10^{-5} | 1.5\times10^{-5} |

3.2. Inverse modelling of the groundwater flow

Inverse modelling was widely used to determine the parameters of aquifers in engineering (Yang et al., 2004; Ayvaz et al., 2007; Virbulis et al., 2013). In this study, an inverse modelling procedure combined
orthogonal design, finite element forward modelling of transient groundwater flow, artificial neural network, and genetic algorithm was adopted to evaluate the hydraulic conductivities of the strata and faults based on the time-series measurements at the site (Zhou et al., 2015; Chen et al., 2016b; Hong et al., 2017). The parameters to be estimated include the hydraulic conductivities of Zone A, Zone B, F_371 and F_11, and their bounds are given in Table 2. An objective function is constructed using the time-series of pore pressure data to improve the reliability of the inversed results:

$$
\text{min } f_i(K) = \left( \sum_{i=1}^{M} \frac{\left\| \phi(K) - \bar{\phi}_i \right\|^2}{\left\| \bar{\phi}_i \right\|^2} \right)^{1/2}
$$

(1)

where $K$ is a vector of the hydraulic conductivities to be estimated; $\phi(K)$ and $\bar{\phi}_i$ are the time-series of calculated and measured water heads at piezometer $i$, respectively; $M$ is the number of the piezometers involved in inverse modelling; $\left\| \cdot \right\|_2$ denotes the Euclidean norm of a vector.

Different parameter combinations were designed by orthogonal design within the bounds of the parameters to be estimated. Finite element forward modelling of transient groundwater flow was then invoked to obtain the time-series data of water heads for each parameter combination. The parameter combinations (as input) and calculations (as output) were used to train and construct a BP neural network. Finally, a genetic algorithm was used to optimize the objective function, and obtain the optimal estimate of the parameters. Details of the algorithm can be found in Chen et al. (2020).

4. Numerical results

4.1. Result of the inverse modelling

The hydraulic conductivities of the strata and faults obtained from inverse modelling are listed in Table 3. The inverse results show that the near-bank rocks in Zone A are of much higher permeability than the deep-buried rocks in Zone B, and the values of hydraulic conductivity differ by one order of magnitude. Consequently, groundwater flow paths are formed along the small-scale karst pipes and cavities in Zone A, and the reservoir water pressure is transmitted to the middle section of the lowest grouting tunnel. Faults F_371 and F_11 also have high in-plane permeability, hence resulting in local rise of hydraulic head along the fault zones. As a fact that the surrounding rocks of the lowest grouting tunnel were grouted during construction, the discharge into this tunnel is negligible, and the abnormal rise of hydraulic head in the middle section does not cause leakage along the flow paths.

With the reversed hydraulic properties of rocks and faults, transient flow simulation was performed. Figure 6 shows a comparison of the measured and calculated hydraulic heads at some typical piezometers, indicating that the numerical results well reproduce the groundwater flow behaviours at the dam site. As shown in Figure 7, the discharges of weirs in the lowest grouting tunnel shows a difference between the measurements and simulations, due mainly to the impact of construction at the site. But the simulations do reflect that the amount of discharge into the tunnels has been well reduced to a sufficient low magnitude.

4.2. Groundwater flow behaviour in the dam foundation

Figure 8 shows the predicted phreatic surface and hydraulic head contours at the normal pool level (470 m) along cross-section A-A. Also plotted in Figure 8 are the transient locations of phreatic surface as the reservoir water level was raised from 297.36 m on November 9, 2019 to 459.85 m on December 31, 2020. It can be observed from the plot that the seepage-proof system containing concrete-faced slab and grouting curtain is effective in depressing the phreatic surface and reducing the pore pressure in the dam and its foundation during the reservoir filling.

Figure 9 plots the hydraulic head contours at the horizontal section of elevation 278 m, at which the piezometers in the lowest grouting tunnel are located. This plot clearly shows the infiltration of reservoir water along the high-permeability Zone A, which results in an abnormal rise of hydraulic
head in the middle section of the grouting tunnel. The discharge into the grouting tunnel (shown in Figure 7) has a sufficient small amount, indicating that the grout curtain is effective in cutting off the leakage paths from reservoir to the downstream side.

Figure 6. Comparison between the measured and calculated time-series of hydraulic head in the dam foundation (a), the left bank (b), and the right bank (c and d).

Figure 7. Comparison between the measured and calculated discharge of weirs in the lowest grouting tunnels (a) in the right bank, and (b) in the left bank.
Figure 8. Hydraulic head contours and variations of phreatic surface along cross-section A-A (with its trace shown in Figure 1).

Figure 9. Hydraulic head contours at the horizontal section of elevation 278 m.

5. Conclusion
This study focuses on the site characterization and inverse modeling of the abnormal rise of hydraulic head during reservoir impoundment in the middle section in the right-bank foundation of the JPH rockfill dam. It has been clarified that this event mainly occurs along the strata where small-scale karst pipes and cavities are developed. The flow paths only transmit the water pressure, and the amount of leakage is effectively controlled by the seepage-proof system.

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