Numerical evaluation of a 70-m deep hydropower station foundation pit dewatering

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Abstract
The foundation pit of Yamansu hydropower station has an average depth of 70 m and has been excavated with five-level slopes. Lowering of groundwater level is important in excavation. In this study, drainage and pumping were conducted in dewatering. Pumping tests were performed to reverse hydraulic parameters and calibrate numerical models. Numerical simulations were performed to evaluate the dewatering scheme. In the first stage, powerhouse tailrace canal was excavated and used as a drainage channel to lower the groundwater level. Drawdown and hydraulic gradient were evaluated to prevent potential water inrush, erosion, piping, and slope failure. In the second stage, cutoff wall and pumping wells were evaluated to reduce the groundwater level. The influences of depth, permeability, and thickness of the cutoff wall were evaluated. Dewatering scheme was revised accordingly based on the evaluation. The optimized dewatering scheme, which can be used as a reference in similar engineering projects, was performed and verified successfully.

Keywords Deep foundation pit · Dewatering · Numerical simulation · Field experiment · Dewatering scheme optimization

Introduction
Groundwater level has to be lowered to excavate in dry conditions, provide stability to the excavation bottom, prevent water inrush, avoid hydraulic erosion, and prevent hydraulic failure. Most foundation pits of hydropower stations are excavated in rock mass (Deng and Chen, 2021). Groundwater can be pumped without worrying about soil erosion, hydraulic failure, and slope stability. However, inappropriate foundation pit dewatering in soil layers may result in disaster including quicksand, piping, or pit collapse (Chow and Ou 1999; Shaqour and Hasan 2008; Shen et al. 2014; Tan and Lu 2017). Most foundation pits that are deeper than 40 m are shafts with limited areas, where the groundwater level can be lowered easily (Yan et al. 2019; Guo et al. 2020; Zhou et al. 2020). Yamansu Hydropower Station (YHS) has a hydropower station type of foundation pit with a maximum excavation depth of 70 m. For such a deep and large foundation pit, existing cases do not provide sufficient insights to help solve the dewatering problem.

So far, most studies have focused on urban underground space engineering, such as subway station, industrial, and civil foundation pits. Onsite pumping and dewatering tests are usually performed to determine the hydraulic parameters of the soil of a dewatering engineering project. The deformation law of adjacent building structures, surface settlement, displacement of supporting piles, axial force, and water level variation during deep foundation pit dewatering are studied (Wang and Wang 2018). Model tests are also used to verify the working mechanism of land subsidence because of the difficulty in reapplying the deformation law in on-site tests (Wang et al. 2018a, b). Numerical simulations are widely used in the analysis of foundation dewatering considering seepage (Sumer et al. 1989; Zaadnoordijk 2010; Ahmad et al. 2019; Zhang et al. 2020) and seepage–deformation coupling (Luo et al. 2008; Li et al. 2020; Zeng et al. 2021a, b; Xie et al. 2021). The effects of the interaction between the waterproof curtain and pumping wells on the surroundings are analyzed by numerical simulation (Wang et al. 2019a, b). Vertical curtain is often used to be combined with a
pumping well for the dewatering. On-site test and numerical simulations are widely used in foundation pit dewatering. In dewatering systems, water tightness assessment test (WTAT) is often suggested prior to excavation (Pujades et al. 2012a, b, 2014; Wang et al 2009, 2016; Wu et al 2016; Wang et al 2018a, b; Xu et al 2019; Cao et al 2020; Wang and Xu 2021). Dewatering test and numerical models are used to verify the dewatering process and pit deformation behavior during dewatering (Zeng et al. 2019; 2021a, b). The studies on dewatering also focus on land subsidence and settlement (Xu et al 2017; Wu et al 2019). However, most existing works on dewatering focus on the foundation pit of industry or civil engineering, together with tunnels, where foundation pits have limited scale in area and depth. The working conditions of the above dewatering engineering were better than that of a hydropower station foundation pit.

The existing dewatering method cannot deal with the YHS foundation pit dewatering. In this study, new initial stage drainage and second-stage pumping method were developed to lower groundwater and prevent seepage failure. On-site pumping test, numerical simulation, and analytical analysis were conducted to verify the drawdown and seepage failure. The original scheme was verified and optimized successfully. The combined dewatering and optimizing methods can be referred to by other similar engineering.

Material and methods

Prototype description

YHS is located in Akesu Wushi County, Xinjiang Uygur Autonomous Region in China (Fig. 1). The power station is located in the Tuoshigan River, the left bank of the Gobi Desert. The length of the YHS Water Diversion Project is 55 km. The power station is superimposed with a hydropower station that generates tail water power. The diversion line is found along the left bank of the Gobi Desert from west to east along with the contour layout 1. The channel gap of power generation contains tail water. YHS is composed of a pressure front pool, pressure steel pipe, main workshop, auxiliary workshop, drain trough, and tailrace canal; it is located in the lower part of the front of the mountain. The ground elevation is 1550–1540 m, whereas the base elevation is 1479.67 m. The maximum depth of the excavation reaches 70 m.

Multimethods were integrated to characterize soil, including geological mapping (1:1000), drilling, exploratory well, exploratory trench, borehole shear wave test, comprehensive geophysical profile test, borehole gravity II dynamic penetration, particle size, water content, natural density, relative density, soil chemistry, water injection test, quick shear test at the joint of sand gravel and concrete, and shear test of remolded coarse-grained soil.

The outcrop strata (CNFC-NSDRI 2014) of the plant are mainly sand gravel layer based on the Pleistocene alluvium in the Quaternary system (Q₃ al+pl). The sand gravel that is widely distributed in the engineering area has the following properties: gray to green gray; egg; 60% to 70% gravel content; round and second round; better local sorting; dense; with limestone composition; sandstone-dominated; has a general particle size of 2–15 cm; mainly with fine-grained fillings, thereby satisfying the requirements of the foundation bearing capacity of the workshop. The local sand–gravel distribution of Quaternary Holocene alluvium (Q₄ al+pl) on the local gully site development has the following properties: average thickness of 0.5 m; gray to green-gray; egg; 60% to 70% gravel content; round and second round; better local sorting; with limestone composition; sandstone-dominated; has a general particle size of 2–15 cm; fine-grained fillings are mainly composed of silt with a loose structure and should be removed. In addition, according to the 1971–2010 statistics from the Wushi County Meteorological station, the average annual precipitation is 112 mm. Precipitation is mainly concentrated in May to September, accounting for approximately 73% of the annual level.

According to the borehole data, the slope is composed of alluvium (Q₅ al+pl) sand gravel in the Quaternary series, which is green-gray, has a dense structure, poor sorting, and hypo-edge angle and round. Gravel with a general particle size of 6–15 and 3–5 cm has approximately 5%–10% and 65%–70% composition of limestone and sandstone. The remaining composition corresponds to fine silt. No sand or sand lens body is observed in the 80-m depth of drilling, and this area forms the condition of the slope of the workshop.

From top to bottom, the powerhouse strata of YHS are the alluvium (Q₅ al+pl) sand gravel layer and Quaternary lower-middle Pleistocene (Q₁₂ gl) semi-cementation conglomerate in the Quaternary series. Meanwhile, the bearing layer is the fourth Upper Pleistocene (Q₃ al+pl) half cementation conglomerate. The hydraulic conductivity of sand gravel is 2 × 10⁻² cm/s–9 × 10⁻³ cm/s (CNFC-NSDRI 2014), which is characterized by medium permeability. The hydraulic conductivity of a half cementation conglomerate is 1 × 10⁻²–8 × 10⁻³ cm/s. The basic conditions of the foundation pit of YHS are shown in Table 1.

The groundwater of the power plant is phreatic water. The excavation of the foundation pit is approximately 70 m. Based on the hydraulic parameters calibrated in in-situ pumping tests (CNFC-NSDRI 2014), the Modflow software using the finite difference method (Harbaugh et al. 2017) was used to validate the proposed dewatering scheme and optimize the layout of the tube well. Moreover, the influence
of the depth of the cutoff wall, permeability coefficient, and thickness of the drainage has to be discussed. The stability of the slope and failure sensitivity of the tube are analyzed. Finally, the proposed optimization process of the dewatering scheme is provided.

The deep foundation pit of YHS was large and deep, and the dewatering conditions were more complicated than the normal foundation pit:

1. The groundwater level in the plant is high. During the construction of the deep foundation pit project, the groundwater level needs to be reduced, and the draw-down is large, reaching 34.19 m.
2. The soil around the foundation pit has good permeability. The hydraulic conductivity of the alluvial proluvial sand gravel layer reaches the order of $10^{-2}$ cm/s, the
1. The water yield is large, and the task of pumping and discharging groundwater is heavy.

3. The excavation depth of the foundation pit is large. The maximum excavation depth of the foundation pit reaches 65 m, with high energy consumption and large lift for pumping and discharging groundwater.

4. Insufficient safety reserve for foundation pit support. The deep foundation pit adopts a simple graded slope excavation form, the lower slope ratio is large, and the safety reserve is insufficient. The impact of dewatering during construction on slope stability must be strictly evaluated.

5. Excavation construction and dewatering construction interfere with each other. The slope section is unfavorable to the construction of the dewatering well and cut-off wall.

6. Seepage damage has a possibility of occurring. During the natural drainage period of construction, the possibility of seepage failure caused by a large hydraulic gradient should be considered to prevent seepage stability of the slope.

7. The basis of the existing design scheme is insufficient. The conventional plane seepage analysis cannot accurately simulate the effect of the dewatering pipe well, and the calculated seepage flow has a large error.

**Concept model**

The YHS foundation pit dewatering was divided into initial drainage and later dewatering stages. In the initial drainage stage, the tail canal that extended downstream was used as the drainage channel. Groundwater was drained to the canal to flow into the Tashigan River. At this stage, the groundwater level was lowered below the 1503.10 m workshop platform. The groundwater level was reduced from 1512.91 m to 1501.00 m. The drawdown was approximately 12.00 m. At the dewatering stage, the cutoff wall and pumping wells were installed on the 1502.80 m platform. The groundwater level was decreased to 1478.67 m (1.0 m below the building base surface). The excavation under the 1502.80 m platform was performed with the gradual descent of the foundation pit water level.
The pumping and drainage period was approximately 12 months.

Mathematical model

Based on the concept model, an unsteady groundwater flow mathematical model was presented as follows:

\[
\begin{align*}
\frac{\partial}{\partial x} (K_{xx} \frac{\partial H}{\partial x}) + \frac{\partial}{\partial y} (K_{yy} \frac{\partial H}{\partial y}) + \frac{\partial}{\partial z} (K_{zz} \frac{\partial H}{\partial z}) + W &= S_s \frac{\partial H}{\partial s}, \\
H(x, y, z, t)\big|_{s_1} &= \varphi_1(x, y, z, t), \quad (x, y, z) \in s_1, t \geq 0, \\
K_{ij} \frac{\partial H}{\partial n}\big|_{s_2} &= q_1(x, y, z, t), \quad (x, y, z) \in s_2, t \geq 0, \\
H(x, y, z, t)\big|_{t=0} &= H_0(x, y, z), \quad (x, y, z) \in \Omega, 
\end{align*}
\]

where \(K_{xx}, K_{yy}, K_{zz}\) represent hydraulic conductivity, \((\text{m/d})\); \(w\) is the amount of water flowing in or out of the aquifer from the vertical direction under unit time (inflow is positive, outflow is negative), \((\text{m}^3/\text{d})\); \(S_s\) is the water storage coefficient; \(\omega\) is the seepage region; \(h\) is water level, \((\text{m})\); \(t\) is time \((\text{d})\); and \(h_0\) is the initial time water level \((\text{m})\).

Several iterative solutions were used in the numerical simulation. To enable the program structure to satisfy any iterative solution format, the different equation forms of the unknown items in each equation. Based on the two matrices, the iteration method is used to solve \(h\).

Numerical model

According to a similar local engineering experience (CDC 2015), the influence radius of dewatering considering cutoff measures was determined as approximately 1000 m, and the model size was 2300 m × 2300 m.

Non-uniform rectangular mesh was used in horizontal and vertical directions through hexahedral mesh subdivision. The 23 m × 23 m grid was used to divide the 3D model. The mesh in the vicinity of the foundation pit was encrypted as 1 m × 1 m to improve the calculation accuracy. The planar meshes were divided into 526 rows and 526 columns. A total of 315,844 elements were generated (Fig. 2).

According to observation during an investigation (CNF-CNSDRI 2014), the initial groundwater level of the foundation pit is 1512.86 m, and the groundwater table does not fluctuate. The initial head of the first stage drainage was set to 1512.86 m. According to the original drainage design scheme, the first stage of the slope drainage groundwater level was reduced to 1503.00 m, and the initial head of the second stage was 1503.00 m.

The boundary far from the excavation area of the workshop was set as the constant head boundary. The head value was 1512.86 m. The bottom was set as zero-flux boundary, and the top of the phreatic aquifer was set as a free surface boundary (0 flow boundary). A platform used for drainage was constructed in 1503.10 m, and the drain water level was 1502.00 m. The hydraulic conductivity of the gravel layer was \(2 \times 10^{-2}\) cm/s.

Working condition

According to the dewatering and drainage schemes, numerical simulations were conducted in four working conditions, as follows:

1. Due to the initial use of slope drainage in the foundation pit of the workshop, the tailrace canal, which has been extended downstream, was used as the channel of groundwater drainage in the initial stage of the foundation pit. Through the drainage method, whether the ground water level of the foundation pit can be reduced from 1512.86 to 1501.00 m, that is, to the plant platform (1502.80 m), to satisfy the requirements of drawdown, hydraulic gradient, and slope stability was checked.

2. In the late stage, pumping wells were used to drain groundwater without a vertical curtain to check whether the 40 pumping wells on the 1502.86 m platform reduce the groundwater level to 1478.67 m (1.0 m below the factory building).
3. If working condition 2 cannot satisfy the drawdown requirements, then simulation was performed to check whether the 40 pumping wells (CDC and TJU 2016) on the 1502.80 m platform with the cutoff wall can decrease the groundwater level to 1478.67 m.

4. A total of 40 pumping wells were used to drain groundwater without initial drainage and vertical curtain to check whether the groundwater level can be reduced to 1478.67 m.

**Parameter reversion and model calibration using field pumping test**

**Pumping and observation well**

The arrangement of pumping and observation wells is shown in Fig. 1. Pumping wells S1–S6 were used as pumping/observation wells. Observation well G1 was installed in the center of the rectangle layout. Pumping/observation well G2 was installed in the S1\S2\S4 extension line layout. Pumping/observation well G3 was installed in the direction of the extension line of S1\S3\S5. The structures of pumping and observation wells are plotted in Fig. 3. The ground settlement observation points were arranged on the opening line of the excavation slope within the range of the pumping test.

According to the design requirements, the main tests in the first stage were performed on the north side. The depth of wells S1, S4, S5, G2, G3, and S6 was 56 m; the depth of wells S2 and S3 was 52 m; the depth of observation well G1 was 40 m. The main technical parameters of each well are shown in Table 2.

The experimental observation was equipped with an XRB30 series multichannel data recorder, which automatically collected and stored the detection data transmitted by the water-level meter and the flowmeter according to the set time and frequency. Screening and analysis were conducted after the experiment was completed. The entire experiment was set up at intervals of 1 min; it involved collecting and storing the water-level gauge readings and pumping well flow rates of each well. After a set of tests was completed, the data were exported and analyzed in a timely manner.

According to the design requirements, observation points were installed on the EL.1522 road and the opening line plant upstream to monitor pumping test and ground settlement. The settlement observation plan in the foundation pit right layout fixed the control points, and the total station set up regular settlement observations of the settlement observation point.

**Pumping test process**

The water pumping test scheme and process are shown in Table 3.

First, a single-well pumping test with a constant pumping rate was conducted. The water level was fully recovered between two pumping tests. During the test, the water level and observation well were synchronized with the water level observation, and the ground subsidence and synchronous monitoring were performed.

When the pumping wells were open, the water level was observed at the specified time interval, including 1', 2', 3', 4', 6', 8', 10', 15', 20', 25', 30', 40', 50', 60', 90', and 120'. The observation interval after 120' was 30 min. The
The observation from 480’ to 1200’ interval was 60 min. The observation interval until the pump was stopped was 2 h. The water level was recovered after the pump was stopped, and the time interval was recorded with the pumping test.

The pumping rate was also observed during the pumping test. The time interval was 30 min. The readings of the flow meter were used, and the accuracy was less than 0.1 m$^3$. If the pumping rate was extremely low and the water level decreasing rate was slow, then a larger flow pump was used. The observation times of the pumping rate were synchronized with the observation of the groundwater level. During the entire pumping test, the pumping rate in the pumping well was maintained constant.

On November 12, 2016, pumping and observation wells were installed. On November 15, pumping equipment and instrument installation were completed. On November 18, the pumping test began. On December 2, the experiments were completed.

**Parameter reversion**

The information on the single-well pumping test (S5) is performed. The pumping rate and water levels in observation wells were obtained (Fig. 4). The hydrogeological parameters were calculated on the basis of steady and unsteady flow test data. The hydraulic conductivity of the aquifer was determined according to the specific hydrogeological conditions of the test field. A trial and test method was used to inverse hydraulic parameters.

Theis unsteady-flow fitting calculation method was adopted to calculate the initial hydraulic parameters using the preliminary data (CNFC-NSDRI 2014). According to the fitting curves (Fig. 5), the relevant hydrogeological parameters were calculated, as shown in Table 4.

## Results

### Working Condition 1

According to the dewatering requirements, slope drainage was adopted in working condition 1 to reduce the water level to 1501.00 m. The bottom elevation of the drainage ditch on the platform was 1502.00 m. The bottom elevation of the tailrace canal was 1494.20 m. The drained water level calculated using the steady flow method is shown in Fig. 6a. The water level cannot be decreased to 1501 m, except for the position near the side of the tailrace canal. Most of the foundation pits cannot satisfy the requirements of the dewatering design.

The water level observation sections on the 1503.10 m platform after drainage for 90 d are shown in Fig. 6b.
Observation wells OW-1 to OW-6 were set in the foundation pit platform. The water levels of the observation wells are shown in Fig. 7. When the pit was excavated to the platform of 1503.00 m after drainage for 90 d, the water level of the upstream position of the platform was 1505.08 m (drawdown: 7.63 m); the water level of the downstream of the platform near the tail canal was 1500.16 m. Only the water level of the 8–8 section...
satisfied the requirement to reduce the water level below 1501 m. The slope drainage cannot satisfy the requirement during the first excavation stage of the foundation pit.

The water inflow when the foundation pit platform was lowered to the elevation of 1502.80 m is 42752 m³. All six sections satisfy the requirements of gradient calculation.

Fig. 5 Verification of YHS hydraulic parameters
Working Condition 2

According to the requirements of drainage design, 40 pumping wells were adopted on the 1503.10 m platform according to engineering experience (CDC and TJU 2016) in working condition 2. The reduced water level was 1478.67 m, which was 1.0 m lower than the foundation pit ground platform (1479.67 m). The water head can be decreased to 1477.23 m to satisfy the design dewatering requirements.

Four observation wells were arranged at the bottom of the excavation at the second dewatering stage. The 100-day dewatering can reduce the water level of the foundation pit bottom in the second stage to 1477.60 m, satisfying the requirements of water drainage design to reduce the water level to 1478.67 m (1.0 m below the bottom of the foundation pit). According to Fig. 8, the water head can be lowered to 1478.5 m after the second dewatering stage that started after 80 d.

According to the drainage design requirements, the allowable value of the hydraulic gradient was 0.24, the hydraulic gradient was verified according to the section shown in Table 5.

The hydraulic gradient in sections 1–1 and 2–2 cannot satisfy the requirements of the allowable hydraulic gradient (Table 5). Piping damage may occur in this position, and taking measures or optimizing the drainage design plan in advance is recommended.

For a more accurate simulation, the recharge conditions of rainfall infiltration under the average annual rainfall of Wushi County is 112 mm. According to the design of the water drainage system, the calculation of working condition 2 was simulated.

Stable flow was used to calculate the dewatering water level and decrease the water level to 1478.67 m, which was lower than the foundation pit ground platform (1479.67 m) at 1 m.

The water level of the bottom of the foundation pit was 1480.02 m, which was higher than 1478.67 m, and does not satisfy the dewatering requirements. The water level near the edge of the foundation pit was less than 1480.02 m, thereby satisfying the design requirements. The water level of the

Table 4  Hydraulic parameters of pumping test

| Pumping well | Observation well | T (m²/day) | S | K (cm/s) |
|--------------|-----------------|-----------|---|----------|
| S1 G1        | 5.22 × 10²      | 8.68 × 10⁻²| 4.66 × 10⁻⁶|
| S1 G2        | 4.59 × 10²      | 5.88 × 10⁻²| 4.10 × 10⁻⁶|
| S1 G3        | 2.57 × 10²      | 4.63 × 10⁻²| 2.29 × 10⁻⁶|
| S2 G1        | 1.31 × 10³      | 6.36 × 10⁻⁴| 1.17 × 10⁻³|
| S2 G2        | 3.60 × 10²      | 3.26 × 10⁻²| 3.22 × 10⁻⁶|
| S2 G3        | 2.35 × 10²      | 5.40 × 10⁻²| 2.09 × 10⁻⁶|
| S3 G1        | 7.75 × 10²      | 9.90 × 10⁻¹| 6.92 × 10⁻⁶|
| S3 G2        | 2.97 × 10²      | 4.84 × 10⁻¹| 2.65 × 10⁻⁶|
| S3 G3        | 3.37 × 10²      | 9.90 × 10⁻¹| 3.01 × 10⁻⁶|
| S4 G1        | 1.49 × 10³      | 7.01 × 10⁻⁴| 1.33 × 10⁻³|
| S4 G2        | 1.02 × 10³      | 2.41 × 10⁻¹| 9.11 × 10⁻⁶|
| S4 G3        | 2.66 × 10²      | 1.25 × 10⁻¹| 2.37 × 10⁻⁶|
| S6 G1        | 1.54            | 8.24 × 10⁻⁴| 1.38 × 10⁻⁴|
| S6 G2        | 5.56 × 10²      | 1.21 × 10⁻¹| 4.97 × 10⁻⁶|
| S6 G3        | 4.64 × 10²      | 9.00 × 10⁻²| 4.14 × 10⁻⁶|
| Average      | 4.62 × 10²      | 2.77 × 10⁻¹| 3.57 × 10⁻³|

Fig. 6 Water level under self-drainage conditions
central position of the bottom of the foundation pit at 90 d of dewatering was 1481.56 m, the water level of the deepest position at the bottom of the foundation pit was approximately 1481.9 m, and therefore was unable to satisfy the design requirements. The water levels at 2–2, 3–3, and 6–6 were above the bottom of the base and cannot satisfy the dewatering requirements. According to the drainage design requirement, the allowable value of the hydraulic gradient was 0.24, and the calculation of six sections satisfies the requirement. The results are shown in Table 5.

**Working Condition 3**

A vertical curtain was adopted in working condition 3 in the calculation. The allowable value of the hydraulic gradient was 0.24, and the results are shown in Table 5. After the cutoff wall was added, the water level of the foundation pit did not meet the dewatering requirement of 90 d. In sections 2–2, 3–3, and 6–6, the water level was above the bottom of the foundation pit and could not satisfy the requirement of deep drawdown. As shown in Table 5, the hydraulic gradient at six sections satisfied the requirement of the allowable hydraulic gradient.

**Working Condition 4**

Based on the assumption that no initial drainage existed, the water level was directly lowered by pumping. The water level can satisfy the requirements of the first stage excavation, that is, the water head of the 1503.00 m platform is 1489.43 m, less than 1501.00 m. The hydraulic gradient was verified according to the section shown in Table 5. The dewatering cannot satisfy the second excavation requirement only through well dewatering. The water level was above the bottom of the foundation pit at sections 2–2, 3–3, and 6–6, which cannot satisfy the requirements of drawdown. Table 5 shows that the hydraulic gradients at all six sections can satisfy the requirements of allowable hydraulic gradients.

**Discussion**

**Influence of cutoff wall**

**Depth of cutoff wall**

The depth of the cutoff wall was set to 40 m according to the adjusted well position layout to verify whether the cutoff wall depth can be shortened. When the cutoff wall was set to 35 m under the adjusted well position, most of the pit bottom can satisfy the requirements of drawdown. However, the deepest position of the pit bottom cannot satisfy the requirements (1480.90 m).

The pumping capacity of all wells when the cutoff wall depths were 40 and 35 m was statistically analyzed for 30, 60, and 90 days. The results are reported in Table 5, which shows that the deeper wall indicates a smaller total pumping capacity of the tube well (Fig. 9).

**Hydraulic conductivity the of cutoff wall**

Based on the adjusted well position, a cutoff wall that is 0.5 m wide and 50 m deep was adopted. The hydraulic conductivity of the cutoff wall was set to $2 \times 10^{-3}$ cm/s, $2 \times 10^{-4}$ cm/s, and $5 \times 10^{-7}$ cm/s, and the variation of water level with hydraulic conductivity is shown in Fig. 10. When the hydraulic conductivity of the cutoff wall was $2 \times 10^{-3}$ cm/s, a strong hydraulic connection inside and outside the cutoff wall was still observed. When the dewatering
period was 30 days, the lowest water level outside the cutoff wall was approximately 1485.37 m. The lowest water level at the bottom of the pit was about 1480.94 m and cannot satisfy the dewatering requirements. When the hydraulic conductivity of the cutoff wall was $2 \times 10^{-4}$ cm/s, the hydraulic connection inside and outside the cutoff wall was evidently weakened. When the dewatering period was 30 days, the lowest water level outside the cutoff wall was approximately 1487.27 m, and the lowest water level at the bottom of the pit was about 1478.45 m. The water level in the pit can satisfy the requirement of the dewatering design. When the hydraulic conductivity of the cutoff wall was $5 \times 10^{-7}$ cm/s, the hydraulic connection between the cutoff wall and the inside and outside was evidently weakened. The lowest water level in the foundation pit was approximately 1478.30 m, which can satisfy the dewatering requirements.

### Thickness of cutoff wall

Two hydraulic conductivities of the cutoff wall were assumed considering different cutoff qualities. When the hydraulic conductivity of the cutoff wall was set to $5 \times 10^{-7}$ cm/s, the calculated cutoff wall thickness were selected as 0.5 and 0.6 m. When the hydraulic conductivity

| WC | Stage | Section | Head difference (m) | Seepage length (m) | Hydraulic gradient | Safe hydraulic gradient | Verify |
|----|-------|---------|---------------------|--------------------|--------------------|-------------------------|--------|
| 1  | I     | 1–1     | 0.2                 | 4.04               | 0.05               | 0.24                    | Y      |
|    |       | 2–2     | 0.2                 | 4.88               | 0.04               | 0.24                    | Y      |
|    |       | 3–3     | 0.2                 | 8.25               | 0.02               | 0.24                    | Y      |
|    |       | 4–4     | 0.2                 | 8.96               | 0.02               | 0.24                    | Y      |
|    |       | 5–5     | 0.2                 | 9.02               | 0.02               | 0.24                    | Y      |
|    |       | 6–6     | 0.2                 | 8.05               | 0.02               | 0.24                    | Y      |
| II |       | 1–1     | 1.03                | 4.17               | 0.25               | 0.24                    | Y      |
|    |       | 2–2     | 1.03                | 4.17               | 0.25               | 0.24                    | N      |
|    |       | 3–3     | 1.03                | 6.72               | 0.15               | 0.24                    | Y      |
|    |       | 4–4     | 1.03                | 6.91               | 0.15               | 0.24                    | Y      |
|    |       | 5–5     | 1.03                | 6.90               | 0.15               | 0.24                    | Y      |
|    |       | 6–6     | 1.03                | 6.71               | 0.15               | 0.24                    | Y      |
| 2  | I     | 1–1     | 1.54                | 11.62              | 0.13               | 0.24                    | Y      |
|    |       | 2–2     | 1.54                | 11.12              | 0.14               | 0.24                    | Y      |
|    |       | 3–3     | 1.54                | 10.98              | 0.14               | 0.24                    | Y      |
|    |       | 4–4     | 1.54                | 11.53              | 0.13               | 0.24                    | Y      |
|    |       | 5–5     | 1.54                | 11.34              | 0.14               | 0.24                    | Y      |
|    |       | 6–6     | 1.54                | 10.51              | 0.15               | 0.24                    | Y      |
| II |       | 1–1     | 1.89                | 10.39              | 0.18               | 0.24                    | Y      |
|    |       | 2–2     | 1.89                | 11.03              | 0.17               | 0.24                    | Y      |
|    |       | 3–3     | 1.89                | 11.92              | 0.16               | 0.24                    | Y      |
|    |       | 4–4     | 1.89                | 8.10               | 0.23               | 0.24                    | Y      |
|    |       | 5–5     | 1.89                | 8.05               | 0.23               | 0.24                    | Y      |
|    |       | 6–6     | 1.89                | 10.52              | 0.18               | 0.24                    | Y      |
| 3  | I     | 1–1     | 2.54                | 24.62              | 0.10               | 0.24                    | Y      |
|    |       | 2–2     | 2.63                | 24.33              | 0.11               | 0.24                    | Y      |
|    |       | 3–3     | 3.66                | 35.81              | 0.10               | 0.24                    | Y      |
|    |       | 4–4     | 3.18                | 30.25              | 0.11               | 0.24                    | Y      |
|    |       | 5–5     | 3.25                | 30.35              | 0.11               | 0.24                    | Y      |
|    |       | 6–6     | 3.46                | 35.91              | 0.10               | 0.24                    | Y      |
| II |       | 1–1     | 2.14                | 11.75              | 0.18               | 0.24                    | Y      |
|    |       | 2–2     | 2.14                | 10.81              | 0.20               | 0.24                    | Y      |
|    |       | 3–3     | 2.14                | 9.04               | 0.237              | 0.24                    | Y      |
|    |       | 4–4     | 2.14                | 9.4                | 0.23               | 0.24                    | Y      |
|    |       | 5–5     | 2.14                | 9.4                | 0.23               | 0.24                    | Y      |
|    |       | 6–6     | 2.14                | 8.9                | 0.24               | 0.24                    | Y      |
of the cutoff wall was set to $1 \times 10^{-15}$ cm/s, the calculated cutoff thicknesses was selected as 0.4, 0.5, and 0.6 m. When the hydraulic conductivity of the cutoff wall was $5 \times 10^{-7}$ and $1 \times 10^{-15}$ cm/s, the thickness was 0.5 and 0.6 m, respectively. The water level was basically consistent for 6, 18, and 30 days, indicating that the wall thickness had a minimal effect on the dewatering of the foundation pit.

**Pumping rate of single well and inflow of foundation pit**

**Water discharge capacity of a single well**

The drainage discharge capacity and the outer diameter of the corresponding drain in the second dewatering stage were calculated. The water capacity of the dewatering pipe well should be selected to determine the maximum effect of the level interference in the pumping of wells, as follows:

$$q = \frac{\rho \cdot d}{\alpha'} \times 24,$$

where $q$ is the discharge capacity (m$^3$/d), $d$ is the outside diameter of filter (mm), $\alpha'$ is the empirical coefficient related to the hydraulic conductivity of the aquifer, and $l'$ is the submerged segment length of filter (m).

The aquifer thickness $\geq 20$ m, $\alpha' = 50$ because of the hydraulic conductivity of the study area $K' = 17$ m/d. The calculation results are shown in Table 6. The discharge capacity of a single pumping with a diameter of 700 mm was larger than the diameter of 340 mm. The water discharge capacity of the single well increased with the increasing length of the filter tube.

**Water inflow in the foundation pit**

The pumping wells in the pit were summarized as constant water level boundary (1479.86 m). The water inflow rate calculated using the steady flow numerical simulation method for 25 m, 30 m, 35 m, and 40 m deep cutoff wall is presented in Table 7. The inflow yield under various cutoff wall depth and pumping period is presented in Table 8. The water inflow rate and inflow yield decreased with the increasing cutoff wall depth.

### Table 6: Single pumping rate (m$^3$/day)

| Filter tube length (m) | 15  | 18  | 21  | 24  | 27  | 30  |
|------------------------|-----|-----|-----|-----|-----|-----|
| Filter tube diameter (mm) | 340 | 2448| 2937.6| 3427.2| 3916.8| 4406.4| 4896|
| 700                    | 5040| 6048| 7056| 8064| 9072| 10,080|

### Table 7: Water inflow rate (m$^3$/day) of foundation pit under various depth cutoff walls

| Cut-off wall depth (m) | 25  | 30  | 35  | 40  |
|------------------------|-----|-----|-----|-----|
| Water inflow rate (m$^3$/day) | 53,068.53 | 52,913.62 | 52,397.05 | 51,958.88 |
Optimization of pumping well layout and cutoff wall depth

According to the working condition, drainage design scheme, and results of the 3D numerical model, the drainage plan of the foundation pit (including the tailrace ditch foundation pit) of the plant was optimized.

During the first stage of foundation pit excavation, according to the original drainage design scheme, the water level can only be lowered to 1503.00 m, which cannot satisfy the requirements of 1501.10 m. Gutter was suggested to be excavated when digging to the 1501.10 m platform in the vicinity of the platform excavation, and the water was drained to the tailrace canal. The depth of the gutter should satisfy the requirements for water drainage.

During the second dewatering stage, according to the original drainage design scheme, the lowest water level in the foundation pit can only be reduced to approximately 1481.00 m, which cannot satisfy the requirements of the drainage design.

In the original dewatering scheme, the distance between pumping wells was 12 m, and the water level was designed to be reduced to 1478.67 m. The positions of pumping wells were optimized, and 40 pumping wells were adopted. A total of 33 pumping wells were installed on the platform of 1503.00 m, and the well distance was optimized to 14.0 m. Then, seven wells were installed on the platform of 1485.40 m, and the well distance was optimized to 7.0 m. The water level for the optimized dewatering scheme is shown in Fig. 11.

In the optimization scheme, 40 m deep cutoff wall was adopted according to the optimized well arrangement. The water level was decreased to 1478.67 m, 1 m below the foundation pit bottom platform (1479.67 m) required by technical note (MHURD 2017). The water levels calculated for 30, 60, and 90 d are shown in Fig. 12. The hydraulic gradients are presented in Table 5. The lowered groundwater level satisfied the deepest groundwater level requirement (1478.67 m) (Fig. 12c). The groundwater levels at the eight sections were below the foundation pit bottom, which can satisfy

| Pumping period (d) | Cutoff wall depth (m) |
|--------------------|----------------------|
|                    | 40                   |
| 30                 | 2,241,817            |
| 60                 | 4,232,751            |
| 90                 | 6,126,210            |
|                    | 35                   |
| 30                 | 2,527,067            |
| 60                 | 4,776,307            |
| 90                 | 6,936,336            |
the requirements of dewatering drawdown. The hydraulic gradient of the six sections satisfied the allowable hydraulic gradient (Table 5).

Therefore, to ensure the total number of pumping wells in the case of the original pumping wells position adjustment, the adjusted pumping wells position can satisfy the dewatering requirements.

**Slope stability of optimization scheme**

Using the proposed optimization scheme, the water level of the slope was initially calculated, and then the safety factor of slope stability and slip surface of minimum safety factor were studied using a numerical method. The calculated A and B sections are shown in Fig. 10. The calculation was performed in half because of the symmetry. According to the investigation report, the parameters used were the following: weight $\gamma_0$ of sand and gravel layer is 20.9 kN/m$^3$, friction angle $\varphi$ is 32°, and cohesion force $C$ is 0 kPa. The
Fig. 13 Potential sliding surface search for B-B section

(a) Swedish strip method

(b) Bishop method

(c) Janbu method
straight line was used in the calculation because the slope of groundwater level in the small area was approximate to the straight line.

The numerical method was used to search for the potential sliding surface of A and B profiles automatically, and different methods were used to calculate the stability coefficients of the most dangerous landslide (Fig. 13).

Under the normal dewatering conditions, the minimum safety factor of A profile was 1.268 as calculated by the Swedish strip method. The B section of the slope adopted the ordinary and Janbu methods to calculate the minimum safety factor, all of which were 1.272. Under the optimal scheme of dewatering, the anti-sliding stability of the slopes satisfies the standard requirements.

Pumping well failure sensitivity analysis

The pumping well position of the optimization scheme was located at the elevation of 1502.80 and 1485.40 m. Initially, 33 pipe wells (wells 1–33) were used to reduce the water level to 1484.40 m, and seven pumping wells (wells 34–40) were installed in the pit. A total of 40 wells can lower the groundwater level of the pit bottom to 1478.86 m.

For the 40 wells, regardless of wells 34–40, a uniform random function was used to produce the potential 5%, 10%, 15%, and 20% failure pumping wells in 1–11, 12–22, and 23–33 (the function can generate random numbers within a certain range). Then, two, four, six, and eight tube wells were selected and supposed to fail by random methods. The water level for 90-d dewatering is shown in Fig. 14. When the number of failure wells in the pit was less than 15% (six wells), the water level satisfied the dewatering requirement. However, when the number of failure wells was increased to 20% (eight wells), the water level cannot satisfy the dewatering requirement.

Pumping wells 1–33 were assumed to be working normally, and one well was randomly assumed to fail from wells 34–40. The water level for the 90-d dewatering is shown in Fig. 14. The water level cannot satisfy the dewatering requirements at the bottom of the failure well location. If one of the internal wells 34–40 failed, the dewatering requirements cannot be satisfied.

Land subsidence caused by foundation pit dewatering

The stratum of YHS was sand gravel layer, which has good permeability, and the deformation caused by dewatering can be completed in a short period without considering the lag effect. A two-step method was used to calculate the land subsidence. The groundwater level drawdown was calculated initially. Then, the land subsidence was calculated using 1D consolidation formula to calculate the land subsidence, as follows:

\[ \Delta S = \frac{\gamma_w}{E_s} \Delta h \]  

where \( \Delta S \) is the deformation of gravel layer (mm), \( \Delta h \) is the change value of groundwater level, (m), \( H \) is the original thickness of sand gravel layer (m), and \( E_s \) is the volume compression modulus (kPa).

Based on the optimization scheme, the land subsidence after dewatering for 15, 30, 90, and 365 days was calculated. The calculation of the soil layer was in accordance with that of the sand gravel layer, and the compression modulus was 25 MPa according to the field test (CNFC-NSDRI 2014), owing to the lack of physical parameter data of the external stratum of the foundation pit. The calculation results are shown in Fig. 15. The maximum subsidence outside the foundation pit after dewatering for 15 days was approximately 1 mm, around 1.1 mm for 30 days, approximately 1.2 mm for 90 days, and about 1.2 mm for 365 days. With the dewatering of the foundation pit, the subsidence of the ground surface outside the foundation pit gradually increased, and the subsidence rate of the late stage decreased until it stabilized. In addition, the subsidence outside the foundation pit was the largest near the foundation pit and decreased with the increasing distance outward.

The optimized dewatering scheme was applied and verified, as shown in Fig. 16.

Conclusions

1. For a hydraulic power station slope and similar engineering slope, canal, such as tailrace canal, can be used to reduce the initial groundwater level when dewatering by using only the pumping well that cannot satisfy the designed value.
2. When the drawdown of cannel drainage cannot satisfy the requirement of dewatering, tube wells can be introduced to reduce the groundwater head to the designed value.
3. When piping may occur during the excavation stage, preventive measures should be considered in time or the drainage design plan must be further optimized.
4. The position of the tube well should be adjusted to optimize the dewatering scheme. Cutoff wall can be adopted during dewatering when the drawdown and piping cannot be satisfied. The deeper cutoff wall indicates a smaller total pumping amount of the well tube.
5. When the hydraulic conductivity of the cutoff wall was large, evident hydraulic connections inside and outside the cutoff wall were observed, the dewatering effect...
Fig. 14 Water level after 90 dewatering when 5%, 10%, 15%, and 20% pumping wells fail

(a) 5% pumping wells failure

(b) 10% pumping wells failure

(c) 15% pumping wells failure

(d) 20% pumping wells failure

(e) One well occurred in well34-well40s
on the foundation pit was poor, and the hydraulic only occurred through the bottom of the wall.

6. Under dewatering conditions, the subsidence near the foundation pit gradually increases, and the subsidence rate decreases until it reaches a stable state.

7. According to the slope stability analysis under the normal dewatering conditions, the minimum safety factor of slopes can be determined.

8. According to the failure sensitivity analysis under the uniform distribution, the safety margin of the pumping wells can be determined.
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Declarations

Conflict of interest The authors declared that there is no conflict of interest.

Availability of data and material All data are included in the manuscript.

References

Ahmad I, Tayyab M, Zaman M, Anjum MN, Dong XH (2019) Finite-difference numerical simulation of dewatering system in a large deep foundation pit at Taunsa Barrage, Pakistan. Sustainability 11:1–17

Cao CY, Shi CH, Liu LH, Liu JW (2020) Evaluation of the effectiveness of an alternative to control groundwater inflow during a deep excavation into confined aquifers. Environ Earth Sci 79:502. https://doi.org/10.1007/s12665-020-09253-3

Chow HL, Ou CY (1999) Boiling failure and resumption of deep excavation. J Perform Constr Facil 13:114–120

CNFC North Survey, Design and Research Institute Co., Ltd (NSDRI) (2014) Geotechnical investigation report of Xinjiang Tuoshigang River Yamansu hydropower project, January 2014

Deng K, Chen M (2021) Blasting excavation and stability control technology for ultra-high steep rock slope of hydropower engineering in China: a review. Eur J Remote Sens 54(supp1):92–106

Guo HQ, Tao SZ, Zhang Q (2020) Research on stress and deformation of super deep foundation pits based on the centrifugal model test. Chin J Undergr Space Eng 16(1):177–186

Harbaugh AW, Langevin CD, Hughes JD, Niswonger RN, and Konikow LF (2017) MODFLOW-2005 version 1.12.00, the U.S. Geological Survey modular groundwater model: U.S. Geological Survey Software Release, 03 February 2017. https://doi.org/10.5066/F7RF5S7G.

Li MG, Chen JJ, Xia XH, Zhang YQ, Wang DF (2020) Statistical and hydro-mechanical coupling analyses on groundwater drawdown and soil deformation caused by dewatering in a multi-aquifer-aquitard system. J Hydrol 589:125365. https://doi.org/10.1016/j.jhydrol.2020.125365

Luo ZJ, Zhang YY, Wu YX (2008) Finite element numerical simulation of three-dimensional seepage control for deep foundation pit dewatering. J Hydrodyn Ser B 20:596–602

Ministry of Housing and Urban-Rural Development of the People’s Republic of China. (MHURD) (2017) Technical code for groundwater control in building and municipal engineering. JGJ/T111–2016.

Fig. 16 Successfully excavated YHS foundation pit
Power China Chengdu Engineering Co. Ltd. (CDC) (2015) Plant
dewatering and drainage scheme design (bidding stage),
2015.10.
Power China Chengdu Engineering Co. Ltd. (CDC), Tongji University
(TJU) (2016) Optimization of dewatering and drainage scheme,
three-dimensional seepage and slope stability analysis of deep
foundation pit in the plant area of Yamansu hydropower station
in Xinjiang, 2016.08.
Pujades E, Carrera J, Vázquez-Suñé E, Jurado A, Mascaruñano-Salvador
E (2012a) Hydraulic characterization of diaphragm walls for cut
and cover tunneling. Eng Geol 125:1–10. https://doi.org/10.
1016/j.enggeo.2011.10.012

Pujades E, López A, Enric C, Carrera J, Vázquez-Suñé E, Jurado A
(2012b) Barrier effect of underground structures on aquifer. Eng
Geol 145–146:41–49. https://doi.org/10.1016/j.enggeo.2012.07.004

Pujades E, Vazquez-Sune E, Carrera J, Jurado A (2014) Dewater-
ing of a deep excavation undertaken in a layered soil. Eng Geol
178:15–27

Shaqour FM, Hasan SE (2008) Groundwater control for construc-
tion purposes: a case study from Kuwait. Environ Geol 53(8):1603–
1612. https://doi.org/10.1007/s00254-007-0768-9

Shen SL, Wu HN, Cui YJ, Yin ZY (2014) Long-term settlement
behaviour of metro tunnels in the soft deposits of Shanghai. Tunn
Undergr Space Technol 40(1):309–323. https://doi.org/10.1016/j.
tust.2013.10.013

Sumer SM, Elton JJ, Tapics JA (1989) Dewatering optimization using
a groundwater flow model at the Whitewood open-pit coal mine,
Alberta. Int J Rock Mech Min Sci Geomech Abstr 26:A213.
https://doi.org/10.1016/0148-9062(89)92831-3

Tan Y, Lu Y (2017) Forensic diagnosis of a leaking accident during
excavation. J Perform Constr Facil 31(5):04017061

Wang Z, Wang C (2018) Analysis of deep foundation pit construc-
tion monitoring in a metro station in Jinan City. Geotech Geol Eng
37:813–822

Wang XX, Xu YS (2021) Impact of the depth of diaphragm wall
on the groundwater drawdown during foundation dewatering consid-
ering anisotropic permeability of aquifer. Water 13:418

Wang J, Wang PC, Wang WD, Zhou SQ (2019a) Optimization analysis
of deformation of underlying tunnel in dewatering and excavation
of phreatic aquifer. Adv Mat Sci Eng 11:1–15. https://doi.org/10.
1155/2019/2461817

Xue SL, Wu HN, Wang BZF, Yang TL (2017) Dewatering induced
subsidence during excavation in a Shanghai soft deposit. Environ
Earth Sci 76:351

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