Numerical Evaluation of a 70-m Deep Hydro-Power Station Foundation Pit Dewatering

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Abstract

The average depth of Yamansu hydropower station foundation pit is 70 m. The foundation pit was excavated with five-level slopes. The lowering of groundwater level is significant for excavation. Drainage and pumping were adopted 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 utilized as a drainage channel to lower the groundwater level. The 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 lower 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 referred by similar engineering, was performed and verified successfully.

1 Introduction

Groundwater level is often lowered to meet the excavation requirement. The foundation pit of most of hydro-power stations is located in rock mass, and groundwater can be pumped without worrying about soil erosion and hydraulic failure together with slope stability. Improper dewatering in soil layers results in disaster, including quicksand, piping, or pit collapse. Thus, most of the urban deep foundation pit is excavated using vertical-retaining structure and water-proof curtain, and pumping wells are often used to extract the groundwater and lower groundwater level. Most foundation pits that are deeper than 40 m are shaft with limited area where the groundwater level can be lowered easily. The foundation pit of Yamansu Hydropower Station (YHS) belongs to a hydro-power station foundation pit. Its maximum excavation depth reaches 70 m. For a large foundation pit, existing experiences cannot solve the dewatering problem.

Nowadays, most studies focus on urban underground space engineering, such as subway, industrial, and civil foundation pits. For the limitation of laboratory experiments, in-site pumping and dewatering tests are usually performed to determine the hydraulic parameters of a dewatering engineering. In-situ pumping–recharge tests, single-well pumping and multi-well recharge test, and multi-well pumping–recharge tests are carried out to verify the non-Darcy flow and feasibility of artificial recharge in deep confined aquifer (Wang et al. 2013, 2014, 2017; Zhang et al. 2017a, 2017b; Wang et al. 2019; Li et al 2019). The data from field measurement of excavation and support were analyzed to study the deformation law of adjacent building structures, surface settlement, displacement of supporting piles, axial force, and water level variation during different excavation stages of deep excavation construction (Wang and Wang 2019). Model tests are also used to verify the working mechanism of land subsidence because of the difficulty in reappearing the deformation law in in-site tests (Wang et al. 2018). 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. 2021; Xie et al. 2021). The impacts of the interaction between

the waterproof curtain and pumping wells on the surroundings are analyzed by numerical simulation (Wang et al. 2019). Cutoff wall must be combined with pumping well for the dewatering. In-site test and numerical simulation are widely used in dewatering system. In dewatering systems that combine the cut and cover method with deep wells, performing water tightness assessment test (WTAT) before the excavation stage is suggested and applied to the excavation (Pujades et al. 2012A, 2012b, 2014; Wang et al 2009, 2016; Wu et al 2016; Wang et al 2018; Xu et al 2019; Cao et al 2020; Wang and Xu 2021). Dewatering test and numerical models are used to simulate the dewatering process and pit deformation behavior during dewatering (Zeng et al., 2019). Land subsidence and settlement are also the focus of studies in dewatering (Xu et al 2017; Wu et al 2019).

However, most existing works on dewatering focus on the foundation pit of industry or civil engineering where foundation pits have limited scale in terms of area and depth. The vertical structure and cut off wall were used, and the dewatering conditions are relatively perfect than those of a hydro power foundation pit. Lowering the water level of a 70-m deep pit for the engineering is urgent.

2 Material And Methods

2.1 Prototype description

YHS is located in Akesu Wushi County, Xinjiang Uygur Autonomous Region. AHS is located in the Tashigan River, the left bank of the Gobi Desert. The length of YHS Water Diversion Project reaches 55 km. Power station is superimposed with hydropower station tail water power. The diversion line is found along the left bank of the Gobi Desert from west to east along the contour layout1. The channel gap of power generation contains tail water. Yamansu Hydropower Plant is composed of pressure front pool, pressure steel pipe, main workshop, auxiliary workshop, drain trough, and tailrace canal, which is located in the lower part of the front of the mountain. The ground elevation is 1550–1540 m while the base elevation is 1479.67 m. The maximum depth of the excavation reaches 70 m. For a large project, lowering the groundwater level is crucial for excavation.

The outcrop strata of the plant are mainly sand gravel layer that is based on the Pleistocene alluvium in the Quaternary System (Q_3^{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; the limestone composition; sandstone-dominated; has a general particle size of 2–15 cm; and mainly fine-grained fillings, thereby meeting the requirements of the foundation bearing capacity of the workshop. The local sand–gravel distribution of Quaternary Holocene alluvium (Q_4^{al+pl}) on the local gully site development has the following properties: has an average thickness of 0.5 m; gray to green gray; egg; 60% to 70% gravel content; round and second round; better local sorting; the limestone composition; sandstone-dominated; has a general particle size of 2–15 cm; and fine-grained fillings are mainly composed of silt and loose structure and need to be removed. In addition, according to the 1971–2010 data statistics of Wushi County Meteorological station: the years of average annual precipitation is

112 mm. Precipitation is mainly concentrated in May to September, accounting for approximately 73% of the annual precipitation.

According to the borehole data, the slope is composed of the alluvium (Q_3^{al+pl}) sand gravel in the Quaternary series, which is green gray, has a dense structure, poor sorting, and the secondary angular and round. Gravel with a general particle size of 6–15 cm and 3–5 cm has approximately 5%–10% and 65%–70% of limestone composition, mainly sandstone, respectively. The remaining composition corresponds to fine silt. No sand or sand lens body are observed in the 80-m depth of drilling, and it has the forming condition of the slope of the workshop.

From top to bottom, the powerhouse strata of YHS are the alluvium (Q_3^{al+pl}) sand gravel layer and the Quaternary lower-Middle Pleistocene (Q_{1-2}^{gl}) semi-cementation conglomerate in the Quaternary series. Meanwhile, the bearing layer is the fourth Upper Pleistocene (Q_{1-2}^{gl}) half cementation conglomerate. The hydraulic conductivity of sand gravel is 2×10^{-2} cm/s– 9×10^{-3} cm/s, thereby belonging to medium permeability, and the hydraulic conductivity of half cementation conglomerate is 1×10^{-2} – 8×10^{-3} cm/s. The basic conditions of the foundation pit of Yamansu hydropower plant are shown in **Table 1**.

YHS (**Fig. 1**) is located in Akesu, Xinjiang and belongs to the Ⅱ hydrogeological area. Its groundwater is phreatic water. The excavation of the foundation pit is approximately 70 m. Based on the hydraulic parameters calibrated in situ pumping tests, the use of finite difference method was suggested to validate the proposed dewatering scheme and to optimize the layout of the tube well. Moreover, the influence of the depth of the cutoff wall, the permeability coefficient, and the thickness on the drainage must be discussed urgently. The stability of the slope and the failure sensitivity of the tube are analyzed. Finally, the optimization suggestion of the dewatering scheme is given.

2.2 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 extends downstream was used as the drainage channel. Groundwater was drained to the canal and finally flown into the Tashigan river. In this stage, the groundwater level was lowered to 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. In 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 due was about 12 months.

2.3 Mathematical model

Based on the concept model, the mathematical model was presented as follows:

$$\begin{cases} \frac{\partial}{\partial x} \left(K_{xx} \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_{yy} \frac{\partial H}{\partial y} \right) + \frac{\partial}{\partial z} \left(K_{zz} \frac{\partial H}{\partial z} \right) + W = S_s \frac{\partial H}{\partial t}, \\ H(x, y, z, t) \Big|_{t=0} = \varphi_1(x, y, z, t), \quad (x, y, z) \in s_1, t \geq 0, \\ K \frac{\partial H}{\partial n} \Big|_{s_2} = q_1(x, y, z, t), \quad (x, y, z) \in s_2, t \geq 0, \\ H(x, y, z, t) \Big|_{t=0} = H_0(x, y, z), \quad (x, y, z) \in \Omega, \end{cases} \quad (1)$$

where k is the hydraulic conductivity, (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), (m^3/d); S_s is the water storage rate; ω is the seepage region; h is water level, (m); t is time (d); and h_0 is the initial time water level (m).

Several iterative solutions were used in the numerical simulation. To enable the program structure to satisfy any iterative solution format, the difference equation form of the unknown and known head are merged, the model that contained the calculated units written the difference equation, and then the linear equation group was obtained:

$$\{A\}\{h\} = \{q\} \quad (2)$$

where $\{A\}$ was the water head coefficient matrix, $\{h\}$ is the desired head matrix, and $\{Q\}$ is all the constant and known items contained in each equation. Based on the two matrices, the iteration method is used to solve $\{h\}$.

2.4 Numerical model

According to the ground topographic map of YHS workshop and excavation plan, the influence radius of dewatering was 1000 m, and the model size was 2300 m $\times$ 2300 m.

Non-uniform rectangular mesh was used in horizontal and vertical directions by using hexahedral mesh subdivision. First, the 23 m $\times$ 23 m grid was used to divide the three-dimensional model. To improve the calculation accuracy, the mesh in the vicinity of the foundation pit was encrypted as 1 m $\times$ 1 m. The planar meshes were divided into 526 rows and 526 columns. A total of 315,844 grids were generated (Fig. 2).

According to the investigation report, the initial groundwater level of the foundation pit is 1512.86 m, and the ground water table is not fluctuating. The initial head of the first stage drainage was set as 1512.86 m. According to the original drainage design scheme, the first stage of slope drainage groundwater level was reduced to 1503.00 m, and the initial head of the second stage was 1503.00 m.

The area of the excavation of the workshop was set as the constant head boundary. The head value was 1512.86 m. The bottom was the zero-flux boundary, and the top of the phreatic aquifer was the free surface boundary (0 flow boundary). Drain PLATFORM was set in 1503.10 m, and the drain water level was 1502.00 m. The hydraulic conductivity of the drain ditch was 0.02 cm/s. The hydraulic conductivity of the gravel layer was 2×10^{-2} cm/s.

2.5 Working condition

According to the dewatering and drainage schemes, the numerical simulations were carried out in three kinds of working conditions:

Working condition 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 foundation pit. By drainage method, the ground water level of the foundation pit was reduced from 1512.86 m to 1501.00 m, that is, to the plant platform (1502.80 m), to meet the requirements of drawdown, hydraulic gradient, and slope stability.

Working Condition 2: In the late stage, pumping wells were used to drain groundwater without cut-off wall to check whether the 40 pumping wells on 1502.86 m platform reduce water level to 1478.67m (1.0 m below the factory building).

Working Condition 3: If working condition 2 cannot meet the drawdown requirements, then simulation was performed to check whether the 40 pumping wells on the 1502.80 m platform with cut off wall can decrease groundwater level to 1478.67 m.

2.6 Parameter reversion and model calibration using field pumping test

2.6.1 Pumping and observation well

The arrangement of pumping and observation wells is shown in **Fig. 3**. Pumping wells S1 to 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 ground settlement observation points were arranged on the opening line of 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, S3 was 52 m; and 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 a XRB30 series multi-channel 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. After the experiment was completed, screening and analysis was conducted. The whole experiment was set up at intervals of 1 minute, collecting and storing the water level gauge readings and pumping well flow rates of each well. After completing a set of tests, the data were exported and analyzed in a timely manner.

According to the design requirements, observation points were installed in 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 observation of settlement observation point.

2.6.2 Pumping test process

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

First, single well pumping test with constant pumping rate was carried out. The water level was fully recovered between two pumping tests. In the course of the test, the water level and the observation well were synchronized with the water level observation, and the ground subsidence and synchronous monitoring were carried out.

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 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.

During the pumping test, the pumping rate was also being observed. The time interval was 30 min. The readings of the flow meter were used, and the accuracy was less than 0.1m^3 . If the pumping rate was too small 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. In the process of the whole pumping test, the pumping rate in the pumping well was kept constant.

On November 12, 2016, test 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 finished.

2.6.3 Parameter reversion

The information of single well pumping test (S5) is listed in **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, and the corresponding methods were selected for calculation and comparison (**Fig. 5**).

Theis unsteady flow fitting calculation method was adopted using the preliminary data. According to the fitting curve, the relevant hydrogeological parameters were calculated as shown in **Table 4**.

3 Results

3.1 Working Condition 1

According to the dewatering requirements, slope drainage was adopted in working condition 1 to lower 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 steady flow method is shown in **Fig. 6(a)**. 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 meet the requirements of dewatering design.

The drained water head calculated using unsteady flow method for 10, 20, 30, 60, and 90 days is shown in **Figs. 6(b)–(f)**.

The water level observation sections on the 1503.10 m platform after drainage for 90 days are shown in **Fig. 7(a)**.

Observation wells OW-1 to OW-6 were set in the foundation pit platform (**Fig. 7(a)**). The water level of the observation wells are shown in **Fig. 7(a)**. When the pit was excavated to the platform of 1503.00 m after drainage for 90 days, 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 8-8 section meet the requirement to lower the water level below 1501 m. The slope drainage cannot meet 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. The water level of 2-2 sections and 6-6 section is shown in **Fig. 7(b)**. All six sections meet the requirements of gradient calculation.

3.2 Working Condition 2

According to the requirement of drainage design, 40 pumping wells were adopted on the 1503.10 m platform in working condition 2. The reduced water level was 1478.67 m, which is 1.0 m lower than the foundation pit ground platform (1479.67 m). The water level calculated using stable flow for working conditions 2 is shown in **Fig. 8(a)**.

The water head can be decreased to 1477.23 m to meet the design dewatering requirements. The water level calculated using unsteady flow for 20, 40, 60, 80, and 100 days is shown in **Figs. 8(b)–(f)**.

Four observation wells were arranged at the bottom of the excavation at the second dewatering stage (**Fig. 9(a)**). The 100-day dewatering can reduce the water level of foundation pit bottom in the second

stage to 1477.60 m (**Fig 8(f)**), which can meet the requirements of water drainage design to reduce to 1478.67 m (1.0 m below the bottom of the foundation pit). According to **Figs. 8 (b)–(f)**, the water head can be lowered to 1478.5m after the second dewatering stage started 80 days.

According to the requirement of drainage design, the allowable value of the hydraulic gradient was 0.24, the hydraulic gradient was verified according to the section shown in **Fig. 9(b)**, and the results are shown in **Table 6**.

The hydraulic gradient in section 1-1 and 2-2 cannot meet the requirement of allowable hydraulic gradient (**Table 6**). Piping damage may occur in this position, and taking measures or optimizing the drainage design plan in advance is recommended.

For 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 water drainage system, the calculation of the working condition 2 was simulated, and the regional head contour is shown in **Fig. 10(a)**.

To decrease the water level to 1478.67 m, which was lower than the foundation pit ground platform (1479.67m) 1 m, stable flow was used to calculate the dewatering water level (**Fig. 10(b)**).

The water level of the bottom of the foundation pit was 1480.02 m, which was higher than 1478.67 m (**Fig. 10(b)**), cannot meet the dewatering requirements. The water level near the edge of the foundation pit was less than 1480.02 m, thereby meeting the design requirements. The calculation of the water level of 30, 40, 60 and 90 days using unsteady flow is shown in **Fig. 10(c)**. The water level of the central position of the bottom of the foundation pit at 90 days of dewatering was 1481.56 m, the water level of the deepest position at the bottom of the foundation pit was about 1481.9 m, and hence was unable to meet the design requirements. The water level at 2-2, 3-3, and 6-6 was above the bottom of the base and cannot meet the dewatering requirements. According to the drainage design requirement, the allowable value of hydraulic gradient was 0.24, and the calculation of 6 sections satisfies the requirement, and the results are shown in **Table 6**.

3.3 Working Condition 3

Cutoff wall was adopted in working condition 3 in calculation, water level contour chart is shown in **Fig. 11(a)**. The calculation of the water head isoline of 30, 60, and 90 days in the foundation pit by unsteady flow is shown in **Figs. 11(b)–(d)**.

The allowable value of the hydraulic gradient was 0.24, and the results are shown in **Table 7**.

According to **Fig. 11(c)**, after adding the cutoff wall, the water level of the foundation pit did not meet the dewatering requirement of 90 days. According to **Fig. 11(d)**, in sections 2-2, 3-3, and 6-6, the water level was above the bottom of the foundation pit and cannot meet the requirement of deep drawdown. From **Table 7**, the hydraulic gradient at 6 sections satisfied the requirement of allowable hydraulic gradient.

3.4 Working Condition 4

Assuming no initial drainage existed, the water level was directly lowered using pumping, and the contour diagram of the regional water head is shown in **Fig. 12(a)**. The contour of the foundation pit head calculated using steady flow is shown in **Fig. 12(b)**. The water level can meet 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 water level of the foundation pit calculated using unsteady flow for 30, 60, and 90 days is shown in **Fig. 12(c)**. The drawdown is shown in **Fig. 12(d)**.

The hydraulic gradient was verified according to the section is shown in **Table 11**.

According to **Fig. 12(a)**, the dewatering cannot satisfy the second excavation requirement by only using well dewatering. According to **Figs. 12(c)–(d)**, the water level was above the bottom of the foundation pit at the section 2-2, 3-3, and 6-6, which cannot meet the requirement of drawdown. From **Table 12**, the hydraulic gradients at all 6 sections can meet the requirements of allowable hydraulic gradients.

4 Discussion

4.1 Pumping well layout and cut off wall optimization, dewatering verification

Given that the distance between the pumping well bottom and foundation pit bottom was large in the original design, and the water level was designed to be lowered to 1478.67 m, the positions of pumping wells were optimized (**Fig 13**). A total of 40 pumping wells were adopted. A total of 33 were installed on the platform of 1503.00 m, and the well distance was optimized as 14.0 m. Seven were installed on the platform of 1485.40 m, and the well distance was optimized as 7.0 m. The water level for the revision dewatering scheme is shown in **Fig. 14(a)**.

In the optimization scheme, 40 m deep cut off 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 (1479.67m). The water levels calculated for 30, 60, and 90 days are shown in **Figs. 14 (b)–(d)**. The hydraulic gradients are shown in **Table 6**. The lowered groundwater level met the deepest groundwater level requirement (1478.67 m) (**Fig. 14(c)**). The groundwater water levels at the 8 sections were all below the foundation pit bottom (**Fig. 14(d)**), which can meet the requirement of dewatering drawdown. The hydraulic gradient of the six sections satisfied the allowable hydraulic gradient (**Table 6**).

4.2 Influence of cutoff wall on dewatering

4.2.1 Influence of cutoff wall depth on dewatering

To verify whether the cut off wall depth can be shortened, the depth of cut off wall was set to 40 m according to the adjusted well position layout. The water level calculated for 30, 60, and 90 days using unsteady flow is shown in **Fig. 14**.

When the cut off wall was set as 35 m deep under the adjusted well position, most of the pit bottom can meet the requirement of deep drawdown. However, the deepest position of the pit bottom cannot meet the requirements (1480.90 m).

The pumping capacity of all wells for 30, 60, and 90 days when cut off wall depth were 40 m and 35 m was statistically analyzed and shown in **Table 7**, which shows that the deeper the depth of the wall is, the smaller the total pumping capacity of the tube well will be (**Fig. 15**).

4.2.2 Influence of hydraulic conductivity of cutoff wall on dewatering

Based on the adjusted well position, 0.5 m wide and 50 m deep cutoff wall was adopted. The hydraulic conductivity of the cutoff wall was set as 2×10^{-3} cm/s, 2×10^{-4} cm/s, and 5×10^{-7} cm/s. The water level for 30 days is shown in **Fig. 16**.

When the hydraulic conductivity of cutoff wall was 2×10^{-3} cm/s, a strong hydraulic connection inside and outside the cutoff wall was still observed. When the dewatering due was 30 days, the lowest water level outside the cutoff wall was about 1485.37 m. The lowest water level at the bottom of the pit was about 1480.94 m and cannot meet the design of dewatering requirements. When the hydraulic conductivity of the cutoff wall was 2×10^{-4} cm/s, the hydraulic connection inside and outside the cutoff wall was obviously weakened. When the dewatering due was 30 days, the lowest water level outside the cutoff wall was about 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 meet the requirement of the dewatering design. When the hydraulic conductivity of the cutoff wall was 5×10^{-7} cm/s, the hydraulic connection between the cutoff wall and the inside and outside was obviously weakened. The lowest water level in the foundation pit was about 1478.30 m, which can meet the design of dewatering requirements.

4.2.3 Influence of cutoff wall thickness on dewatering

Based on the adjusted well position, when the hydraulic conductivity of the cutoff wall was 5×10^{-7} cm/s, the thickness of the cutoff wall was set to 0.5 m and 0.6 m. The hydraulic conductivity of the cutoff wall was set to 1×10^{-15} cm/s, and the thickness of cutoff wall was set to 0.4, 0.5, and 0.6 m, dewatering for 6 days. The water level of 18-day and 30-day is shown in **Fig. 17**.

When the hydraulic conductivity of cutoff wall was 5×10^{-7} cm/s and 1×10^{-15} cm/s, the thickness was 0.5 m and 0.6 m, respectively. The water level was basically consistent with the internal and external head of the foundation pit for 6, 18, and 30 days, indicating that when the hydraulic conductivity of the cutoff wall was certain, the wall thickness had minimal effect on the dewatering of the foundation pit.

4.3 Water discharge capacity of single well and inflow of foundation pit

4.3.1 Water discharge capacity of 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 chosen to determine the maximum effect of the level interference in the pumping of wells:

$$q = \frac{l' d}{\alpha'} \times 24, \quad (3)$$

where Q is the water pipe water capacity (m^3/d), D is the outside diameter of filter (mm), α' is the empirical coefficient related to the hydraulic conductivity of aquifer, and l' is the submerged segment length of filter (m).

The aquifer thickness ≥ 20 m, $\alpha' = 50$ because of the hydraulic conductivity of the study area $K' = 17$ m/d. The calculation result is shown in **Table 8**. The discharge capacity of a single tube diameter of 700 mm was larger than the diameter of 340 mm. With the length of the filter tube increasing, the water capacity of the single tube was increasing.

4.3.2 Water inflow in foundation pit

The pumping wells in the pit were summarized as constant water level boundary (1479.86 m). The water inflow calculated using steady flow numerical simulation method for 25, 30, 35, and 40 m deep cutoff wall was calculated (**Table 9**). The water inflow decreased with the increasing cut off wall depth but without obvious difference.

4.4 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 time without considering the lag effect. One-dimensional consolidation formula was used to calculate the land subsidence:

$$\Delta S = \frac{\gamma_w}{E_s} \Delta h H, \quad (4)$$

where Δs is the deformation of gravel layer (mm); Δh 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).

The adjusted pumping well position was the basis to implement the dewatering. The land subsidence after dewatering for 15, 30, 90, and 365 days was calculated. Because of the lack of the physical parameter data of the external stratum of the foundation pit, the calculation of the soil layer was in accordance with the sand gravel layer, and the compression modulus was 25 MPa. The calculation result is shown in **Fig. 18**. The maximum subsidence of the foundation pit after dewatering for 15 days was about 1 mm, 30 days was about 1.1 mm, 90 days was about 1.2 mm, and 365 days was about 1.2 mm.

With the dewatering of the foundation pit, the subsidence of the ground surface of the foundation pit was gradually increasing slowly, and the subsidence rate of the late stage decreased until it becomes stable. In addition, the subsidence of the foundation pit was reduced by the center of the foundation pit, and the subsidence is the largest near the foundation pit.

4.5 Optimization of drainage scheme for foundation pit dewatering

According to the working condition, drainage design scheme, and the numerical calculation of the three-dimensional 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 meet the requirements of 1501.10 m. Gutter was suggested to be excavated when digging to 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 be able to meet 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 about 1481.00 m, which cannot meet the requirements of the drainage design. Therefore, to ensure the total number of tube wells in the case of the original tube position adjustment, after three-dimensional numerical calculation, the adjusted tube position can meet the requirements of precipitation.

4.6 Slope stability of optimization scheme

Using the proposed optimization scheme, first, the water level of the slope was calculated, and then the safety factor of slope stability and the slip surface of minimum safety factor were calculated by using the numerical calculation software. The calculated A section and B sections are shown in **Fig. 19**. Because of the symmetry, the calculation was taken in half. According to the investigation report, the parameters used were the weight heavy γ_0 of sand and gravel layer is 20.9 KN/m³, friction angle φ is 32°, and cohesion force C is 0. Because the slope groundwater level in the small area was approximate to the straight line, the straight line was used in the calculation.

The numerical software was used to search for the potential sliding surface of A profile and B profile automatically, and different methods were used to calculate the stability coefficients of the most dangerous landslide, respectively (**Fig. 19**).

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

4.7 Pumping well failure sensitivity analysis in optimization scheme

The pumping well position of the optimization scheme was located at the elevation 1502.80 m and 1485.40 m. First, 33 pipe wells (well1-well33) were used to reduce the water level to 1484.40 m, and 7 pumping wells (Well 34-40) were installed in 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 well 34-well 40, uniform random function (the function can generate random numbers within a certain range) was used to produce the potential 5%, 10%, 15%, and 20% failure pumping wells in well1-well11, well12-well22, well23-well33. 2, 4, 6, 8 tube wells were selected and supposed as failure from near the pit the deepest and upstream by random methods. The water level for 90-day dewatering is shown in **Fig. 20**. When the number of failure wells in pit was less than 15% (6 wells), the water level satisfied the dewatering requirement. However, when the number of failure wells was increased to 20% (8 wells), the water level cannot meet the dewatering requirement.

Well1-well33 tube wells were assumed to be working normally, and one well was randomly selected from the well 34-well 40 for calculation. The water level for the 90-day dewatering is shown in **Fig. 20**. The water level cannot meet the dewatering requirements at the bottom of the failure tube location. The internal well 34-well 40 cannot exhibit failure condition; otherwise, the dewatering requirements cannot be satisfied. The optimized dewatering scheme was performed and verified successfully (**Fig. 21**).

5 Conclusions

According to the YHS drainage design scheme, the design optimizations were performed using numerical simulation method, and the following conclusions were reached:

(1) Working condition 1 cannot reduce the water level to the lower 2 m position (1501.1m) by means of slope drainage in the tailrace canal. The foundation pit platform can only meet the requirement in the downstream near the tailrace canal position head.

(2) Working condition 2 used tube well dewatering to reduce the groundwater head to the 1478.5 m in 80 days during the second dewatering phase. However, the hydraulic gradient at 1–1 and 2–2 sections cannot meet the requirement of allowable hydraulic gradient.

Piping damage may occur during the second excavation stage. Preventive measures should be taken in time or the drainage design plan must be further optimized.

(3) On the basis of working condition 2, considering the rainfall recharge, the central position of the bottom of the foundation pit was 1481.56 m, and the deepest position of the bottom of the foundation pit was approximately 1481.90 m, which cannot meet the design requirements. Therefore, the position of tube well was adjusted, and the cutoff wall was adopted in the dewatering simulation. The water head basically met the design requirements, and the deeper the wall depth of the cutoff wall, the smaller the total pumping amount of the well tube.

(4) Working Condition 4 did not consider the initial drainage, and only well dewatering was used. The water level for 30, 60, and 90 days of foundation pit head cannot meet the dewatering requirements, but the hydraulic gradient can.

(5) When the hydraulic conductivity of the cutoff wall was large, obvious hydraulic connections inside and outside the cutoff wall were observed, the dewatering effect in the foundation pit was poor, and the hydraulic only occurred through the bottom of the wall. The dewatering effect of the foundation pit was the best. When the hydraulic conductivity was constant, the thickness of the cutoff wall has minimal effect on dewatering.

(6) The water inflow decreased gradually with the depth of the cutoff wall; however, the difference was not obvious.

(7) Under dewatering conditions, the subsidence nearby the foundation pit gradually increased slowly, and the subsidence rate decreased until reaching its stable state. The total subsidence of the 365 days was 1.2 mm and did not affect the perimeter building safety.

(8) According to the slope stability analysis under the normal dewatering conditions, the minimum safety factor of slopes A and B were 1.268 and 1.272, which exceeded 1.10 and satisfied the requirement of correlative specification, respectively.

(9) According to the failure sensitivity analysis under uniform distribution, the safety margin of the pumping wells was 15% for the peripheral wells (Well1 to well 33), 15% failure of the pumping wells, that is, 6 wells, can be accepted. Meanwhile, no failure can be accepted for the internal wells (Wells 34–40).

Declarations

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Tables

Table 1 Basic conditions of the foundation pit of Yamansu hydropower

Item		Content	
Stratum system	Geographical Position	Outside the flood dike of Bulakelexi Village, Yamansu Township	
	Geomorphic location	Slope toe of Northwest piedmont alluvial proluvial inclined plain	
	Foundation pit	Retaining form Sloping excavation	
	Geologic Age	Q_3^{al+pl}	Q_{1-2}^{gl}
	lithology	Alluvial proluvial sand gravel layer	Semi cemented conglomerate
	Thickness (m)	>100	>8
	Permeability coefficient (cm / s)	$2 \times 10^{-2} - 9 \times 10^{-3}$	$1 \times 10^{-2} - 8 \times 10^{-3}$
	Average critical gradient	0.48	-
	Allowable hydraulic gradient	0.24	-
	Ground	1545.00	
Altitude system (m)	Powerhouse foundation pit bottom (foundation surface)	1479.67	
	Rear slope opening line	1545.00	
	Large area of the plant	1503.20	
	Maximum excavation depth (slope height)	65.00	
	Elevation of generator floor in plant area	1503.20	
	Maximum permanent slope height	42.00	
	Groundwater level in flood season	1512.86	
	Super high groundwater level in flood season	33.19	
	Design water level elevation	1478.67	Pit middle
	Design drawdown	34.19	Pit middle

Table 2 Pumping well system

No.	Elevation (m)	Depth (m)	Radius (cm)	Initial water level (m)	Filter tube	Pump	Water level gauge	Flowmeter
S1	1515.0	56	34	1512.9	Concrete filter pipe, 16 m-long (G1 pipe 4 m), wrapped with 50- 100 mesh filter screen, filled with 19 cm-thick pea stone with particle size of 3- 20 mm	Deep well submersible pump 250QJ140 -90/6 Lift 90 m, flow 140 m ³ /h, power 55 kw, Outlet pipe diameter 125 mm	Input level gauge TY-1002 N -50 m Maximum range 50 m	Electromagnetic flowmeter LDB- 125 SM2112 Maximum range 220 m ³ /h
S2	1515.0	52	34	1512.9				
S3	1515.0	52	34	1512.9				
S4	1515.0	56	34	1512.9				
S5	1515.0	56	34	1512.9				
S6	1515.0	56	34	1512.9				
G1	1515.0	40	34	1512.9		-		-
G2	1515.0	56	34	1512.9		-		-
G3	1515.0	56	34	1512.9		-		-

Table 3 Pumping test scheme

Experiment stage	Pumping well number	Pumping well	Observation well	Experiment due (h)
I	1	S1	S2, S4, G2 S3, S5, G3 S6	4-8
		S2	S1, S4, G1, G2	4-8
II	2	S1, S4	S2, S5, S6, G1, G2	24
		S3, S5	S1, S6, G1, G3	24
III	3	S1, S3, S5	S2, S4, S6, G1, G3	48
IV	4	S1, S4, S5, S6	S2, S3, G1, G2, G3	48

Table 4 Hydraulic parameters of pumping test

Pumping well	Observation well	T (m ² /d)	S_y	K (cm/s)
S1	G1	5.22E+02	8.68E-02	4.03E-03
	G2	4.59E+02	5.88E-02	3.54E-03
	G3	2.57E+02	4.63E-02	1.98E-03
S2	G1	1.31E+03	6.36E-04	1.01E-02
	G2	3.60E+02	3.26E-02	2.78E-03
	G3	2.35E+02	5.40E-02	1.81E-03
S3	G1	7.75E+02	9.90E-01	5.98E-03
	G2	2.97E+02	4.84E-01	2.29E-03
	G3	3.37E+02	9.90E-01	2.60E-03
S4	G1	1.49E+03	7.01E-04	1.15E-02
	G2	1.02E+03	2.41E-01	7.87E-03
	G3	2.66E+02	1.25E-01	2.05E-03
S6	G1	1.54E+00	8.24E-04	1.19E-05
	G2	5.56E+02	1.21E-01	4.29E-03
	G3	4.64E+02	9.00E-02	3.58E-03
Average		4.62E+02	2.77E-1	3.57E-3

Table 6 Verification of hydraulic gradient

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	N
		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

Table 7 Water inflow (m^3) under different cut-off wall depths and pumping dues

Pumping due (d)	Cut-off wall depth (m)	
	40	35
30	2241817	2527067
60	4232751	4776307
90	6126210	6936336

Table 8 Single pumping rate (m^3/d)

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	10080

Table 9 Water inflow of different cut-off walls

Cut-off wall depth (m)	25	30	35	40
Water inflow (m ³ /d)	53068.53	52913.62	52397.05	51958.88

Figures

(a) YHS foundation pit and tailrace canal

(b) YHS foundation pit (In plane)

(c) YHS foundation pit (In section)

Figure 1

(a) Numerial model

Figure 2

YHS FDM numerical model (unit: m)

Figure 3

Arrangement of pumping and observation wells

(a) Pumping well

(b) Observation well

Figure 4

Pumping test results with well S5 as pumping well

(a) S1 (Pumping well)

(b) S2 (Pumping well)

(c) S3 (Pumping well)

(d) S4 (Pumping well)

(e) S5 (Pumping well)

Figure 5

Verification of YHS hydraulic parameters

(a) Observation well and line arrangement

(b) Calculation groundwater level

Figure 7

Water level in observation wells

(a) Steady flow

(b) 20 d

(c) 40 d

(d) 60 d

(e) 80 d

(f) 100 d

Figure 8

Water level after dewatering of WC 2

(a) Observation well arrangement

(b) Calculation groundwater level

Figure 9

Water level in section in WC2

(a) Regional water level

(b) 30 d

(c) 60 d

(d) 90 d

Figure 11

Water level of WC3

(a) Regional water level

(b) Steady flow

(c) 30 d

(d) 60 d

(e) 90 d

Figure 12

Water level of WC4

Figure 13

Location of optimized pumping wells

Figure 14

Water level when cut off wall was set as 40 m deep

Figure 15

Water level when the cut off wall is 35 m deep

(a) Hydraulic conductivity 2×10^{-3} cm/s

(b) Hydraulic conductivity 2×10^{-4} cm/s

(c) Hydraulic conductivity 5×10^{-7} cm/s

Figure 16

Water level under different hydraulic conductivity cut off walls in 30 days

Figure 17

Drawdown under different thickness after dewatering

(a) Dewatering 15d

(b) Dewatering 30d

(c) Dewatering 90d

(d) Dewatering 365d

Figure 18

Subsidence after dewatering for 15,30, 90 and 365 (unit: mm)

Figure 19

Potential sliding surface search for B-B section

Figure 20

Water level after 90 dewatering when 5%, 10%, 15%, and 20% pumping wells fail

Figure 21

Successfully excavated YHS foundation pit