Research on Emergency Drainage Technology for Extraordinary Water Inrush Disaster in Da Bashan Highway Tunnel

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Abstract. The occurrence of sudden water inrush disasters during tunnel construction is characterized by urgency and unpredictability, making the scientific and efficient emergency response capability crucial. Based on the context of the Daba Mountain Tunnel project under construction on the Anlan Expressway, this study utilizes field investigations and theoretical analysis to analyze the causes of tunnel water inrush disasters. It proposes schemes for dewatering through inclined shafts and adits, determines key parameters of the schemes, discusses pump operational performance, and applies these schemes in engineering practice. The results indicate that water is stored in limestone formations, geological structures control water, water inrush occurs in fault zones, and the combined effects of dynamic and static loads lead to water inrush disasters, resulting in a tunnel water volume of approximately 170,000 m3. The refuge chamber can serve as a location for water storage, with 5 DN200 pipes selected for inlet pipes and 3 DN300 pipes for outlet pipes. The static and mobile pump stations have respective lift capacities exceeding 160m and 80m. The fixed pump station comprises two 630kW + one 160kW horizontal centrifugal pumps, while the mobile pump station consists of five 55kW submersible pumps, meeting the performance and economic requirements of parallel operation for the fixed pump station. Consequently, a phased emergency dewatering technology, comprising "mobile pump station + fixed pump station + water storage chamber," is developed and applied in practical engineering, effectively addressing the emergency response to severe water inrush disasters in tunnels.

Keywords: water inrush disaster; adits and inclined shafts; pump station; dewatering

1 Introduction

In recent years, with the expansion of the scale of expressway construction in western mountainous areas, a large number of tunnel projects have emerged. The complexity of mountainous geological environments determines the frequent occurrence of disasters during tunnel construction, with sudden water inrush disasters being the most common

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[1]. For extra-long tunnels, in order to meet the requirements of construction access and operational ventilation and rescue, it is often necessary to construct long-distance and steeply inclined construction adits, which affect the timely and efficient emergency response to sudden water inrush disasters. Therefore, conducting research on emergency dewatering techniques for sudden water inrush disasters in extra-long tunnels is of great practical significance for enhancing the emergency response capability during the construction period.

Currently, research on tunnel water inrush mainly focuses on the prevention and control of water disasters during construction in aquiferous strata, while relatively less attention has been paid to emergency response and disposal of water inrush disasters during tunnel construction. Liu Hairong [2], through the use of double-arch fixed pump stations, pump selection and layout, pipeline series-parallel connection, and power supply networking, solved the problem of reverse slope dewatering in the Guanjiao Tunnel. Jia Feng [3], based on the analysis of the construction characteristics of steep slope reverse slope drainage in the Yingpanshan Tunnel, discussed the layout of the drainage system and verified its effectiveness. Zuo Yujie [4] studied the layout of the reverse slope drainage system in the Qiyue Mountain Tunnel of the Yiwu-Wanning Railway. However, there are two main issues in the aforementioned studies. Firstly, the utilization rate of pump stations installed in adits is relatively low. Secondly, there is a lack of clarity in understanding the operating performance of pumps. This paper, based on the background of the water disposal project in the Da Bashan Extra-long Highway Tunnel of the Anlan Expressway, analyzes the causes of water inrush disasters, proposes schemes for drainage in adits and main tunnels, determines key parameters of the schemes, discusses the operating performance of pumps, and develops a phased dewatering technique of "mobile pump stations + fixed pump stations + water storage tanks". This technique has been applied in practical engineering and effectively solved the water inrush disasters in the Da Bashan Tunnel, providing a reference for similar projects.

2 Project Profile

The Da Bashan Extra-long Tunnel of the Anlan Expressway is a key project of the National Highway Yinbai Line (G69) in Shaanxi, which spans from Ankang to Langu (the border between Shaanxi and Chongqing). It is located in Taohetown, Langao County, Ankang City, Shaanxi Province, crossing the boundary between Shaanxi and Chongqing and connecting with the Chengkai Expressway in Chongqing. The tunnel has a total length of 13.59 km and is jointly constructed by Shaanxi and Chongqing provinces. It is divided into three construction sections. Among them, Section 20 of the Anlan Expressway is the middle section of the three sections. Considering the convenience of construction and ventilation requirements, the inclined shaft method is adopted, with an inclined shaft length of 1582 meters and a comprehensive longitudinal slope of -9.6%.

3 Analysis of the Causes of Extra-Large Water Inrush Disasters

3.1 Basic Situation of Water Inrush Disasters

On June 13, 2022, when the left tunnel line advanced to the milestone ZK87+039, the surrounding rock was revealed to be slightly weathered limestone, with developed joint fissures and a layered structure, as well as fold development, and seepage occurring at the arch crown. From the late middle to early August, the ZK87+039 of the left line served as the source of water inflow, with the water flow ranging from 300m3/h to 2042.58 m3/h, flooding four faces of the inclined shaft and the main tunnel. The water depth in the main tunnel was approximately 6 to 7.0 meters, and the maximum inundation length of the inclined shaft was 895 meters, with a water depth of 6.5 to 8.0 meters. Water pumping operations were carried out during this phase, which ended on August 21. After the pumping operation ceased, only localized water accumulation was observed at the intersection of the Shaanxi right line and the inclined shaft, while single-joint and curtain-like small-scale seepage persisted in the direction of the Chongqing left line. The key time points of tunnel water inflow are shown in Figure 1, and the inundated areas of the tunnel are depicted in Figure 2.

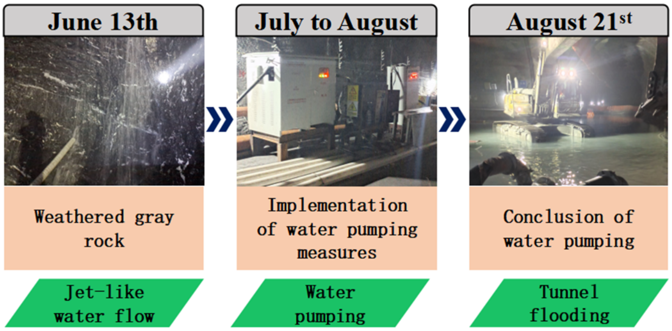

Fig. 1. Timeline of Major Events in Tunnel Water Inrush.

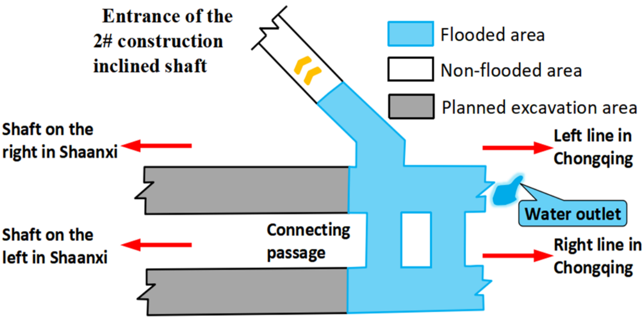

Fig. 2. Schematic diagram of flooded areas in the tunnel.

3.2 Analysis of the Causes of Sudden Water Inrush Disasters

A comprehensive analysis of the causes of water inflow disasters in the Daba Mountain Tunnel was conducted using a combination of on-site investigation, advanced geological forecasting, and expert consultation[5-7]. The analysis is as follows:

(1) The surrounding rock at the water outlet of the left tunnel line mainly consists of medium-thick layers of water-rich limestone, with developed folding. Additionally, it is close to the f94 fault zone between ZK86+937 and ZK86+961, providing objective conditions for groundwater storage;

(2) The water inflow occurs during the local high-water season from May to October, and it exhibits characteristics of single-joint and rainwater seepage. The water is clear, without turbidity or sediment, and there are no obvious surface cracks, exposed water sources, or replenishment channels. It is speculated to be a structurally controlled water inflow;

(3) Three-dimensional advanced geological radar detection results show the presence of a stratigraphic boundary intersecting the tunnel axis around K87+060 to K87+070, with two karst fissure flow channels. This type of water inflow is considered to belong to the category of fault-controlled water inflow in water-rich fault zones;

(4) During the tunnel excavation using the drilling and blasting method, the disruption and destruction of the water-resistant layer due to the action of dynamic and static loads across the fractured fault zone provided subjective conditions for water inflow in the tunnel. The combined effects of these four aspects constitute the reasons for the occurrence of water inflow disasters in the Daba Mountain Tunnel.

3.3 Water Inflow Prediction

Based on previous hydrogeological survey data and combined with measured data of water levels in the inclined shaft and tunnel drainage volume [8-11], it is calculated that the water inflow at the face will gradually stabilize at 1300 to 1500 m^3/h. For this plan, the tunnel water inflow is calculated at 1500 m^3/h, and the maximum accumulated water volume in the tunnel is approximately 1.7 × 10^5 m^3.

4 Reverse Slope Drainage Scheme and Equipment Configuration

The core issue affecting the normal construction of water-rich karst tunnels is the sudden influx of water during excavation and support operations, which leads to rock degradation and instability or tunnel flooding. The former can result in underground engineering disasters, while the latter can cause equipment damage and project delays, as is the case with the Daba Mountain Tunnel. Since the inclined shaft is the only passage connecting the tunnel to the outside, the implementation of an emergency drainage scheme follows the principle of "priority to drainage."

4.1 Tunnel Drainage Scheme Determination

The total length of the tunnel inclined shaft is 1574.3m, with a comprehensive slope of 10%, resulting in a height difference of 157.4m. Considering the water accumulation inside the tunnel, the effective drainage capacity of the tunnel is designed to be no less than 2000m³/h. Given the high lift and large water volume characteristics of pumping in the inclined shaft, a "mobile pump station + fixed pump station + water storage tank" approach is adopted for reverse slope drainage in the inclined shaft. In the main tunnel, a "horizontal centrifugal pump + water storage tank" drainage scheme is employed, as shown in Figure 3 below.

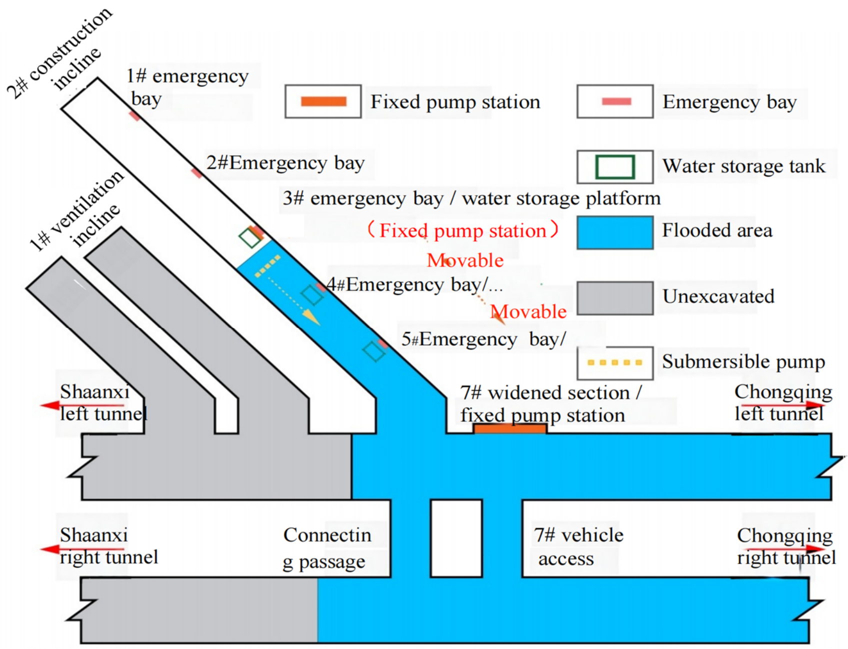

Fig. 3. Schematic diagram of the overall plan.

Inclined Shaft Drainage Scheme.

Firstly, to address the inundation of water inside the inclined shaft, a group of 5 submersible pumps are deployed as a mobile pump station to pump water into the 3# water storage tank at the refuge chamber. These pumps are paired with horizontal centrifugal pumps installed as fixed pump stations to pump water out of the tunnel. Secondly, as the water level inside the inclined shaft recedes, the mobile pump station will pump water into the 4# water storage tank, and the fixed pump station from the 3# water storage tank will be moved to the 4# water storage tank to pump water out of the tunnel. Finally, the mobile pump station will pump water into the 5# water storage tank, and the fixed pump station from the 4# water storage tank will be moved to the 5# water storage tank to pump water out of the tunnel. It is important to note that when transferring the fixed pump station, one spare horizontal centrifugal pump will be installed in the designated water storage tank to ensure uninterrupted operation. This transfer process ensures that the actual drainage capacity remains at no less than 2000m³/h.

Main Tunnel Drainage Scheme.

As the water level recedes to the main tunnel, water from the left tunnel face of the Chongqing-bound direction will be pumped into the 4# water storage tank. Subsequently, utilizing two 160kW pumps, the water will be relayed and pumped out of the tunnel. To overcome the water resistance and elevation differences along the main tunnel at the intersection with the inclined shaft, a fixed pump station will be added in the widened section at 7#, ensuring that water from the tunnel face can be efficiently drained downhill into the 4# water storage tank.

Layout of Water Storage Tanks.

Considering the actual conditions of the Da Ba Shan tunnel, the originally designed 3#, 4#, and 5# refuge chambers, which also serve as water collection chambers, will be converted into water storage tanks. The drainage of tunnel water will be conducted in four phases. Backup pumps will be provided to ensure the drainage capacity. A typical layout of the water storage tanks is illustrated in Figure 4.

Fig. 4. Refuge Bay Water Storage Tank.

4.2 Determination of Drainage Equipment Performance Parameters

Diameter of Inlet and Outlet Pipes for Water Storage Tanks.

Steel pipes are chosen for the pipeline material, with a flow velocity ranging from 2.0 to 3.0 m/s. A velocity of 2.5 m/s is selected. The diameter of the pipes is calculated using the following equation:

$$nD = (4Q_{h\max} / \pi V_p)^{1/2} \tag{1}$$

Where, Qhmax is the maximum drainage volume of the pipeline, measured in m³/h; D is the diameter of the pipeline, measured in mm. n is the number of pipelines; Vp is the velocity of water flow in the pipeline, measured in 2.5 m/s.

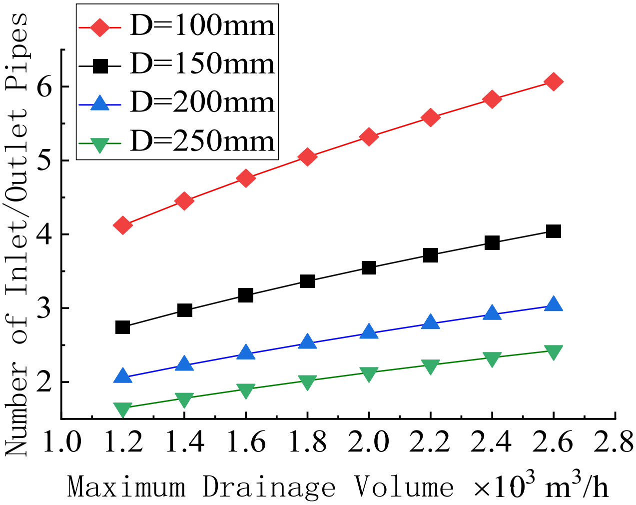

Fig. 5. Relationship between the number of pipelines and drainage volume.

The relationship between the number of pipelines and the maximum drainage volume shown in Figure 5 is positively correlated. When the maximum drainage volume is set to 2000 m³/h, selecting steel pipes with a diameter D=100 mm requires 6 pipes; for diameters ranging from 150 to 200 mm, 3 pipes are required; and for D=250 mm, 2 pipes are needed. Considering pipeline smoothness and failure risk, the diameter should not be too large to accommodate pump operation, ensuring an adequate number of pipes to avoid the risk of blockage. Therefore, the number of inlet pipes for the reservoir is set to 5 with D=200 mm, with the option to increase the number if necessary, while 3 outlet pipes with D=250 mm are chosen, and D=300 mm is selected.

Analysis of Pump Head.

Head is a critical parameter for pumps. When analyzing the head of a pump, it is necessary to consider both the along-pipeline and local hydraulic losses between water and the pipeline. The equation is as follows [12]:

$$H = Z + h_j + h_f \tag{2}$$

$$h_f = \lambda l v^2 / 2dg \tag{3}$$

$$h_j = \zeta v^2 / 2g \tag{4}$$

$$\zeta = \sin^2(\theta/2)\left[0.946 + 2.05\sin^2(\theta/2)\right] \tag{5}$$

In the above equation, hf represents the hydraulic losses along the pipeline, hj denotes the local hydraulic losses, d stands for the diameter of the pipeline, θ represents the angle of the pipeline bend, l denotes the length of the pipeline, λ signifies the coefficient of along-pipeline losses, which is set to 0.018, H is the pump head, and Z indicates the height difference between the water level at the inlet and the outlet.

Head of Fixed Pump Station.

Analyzing equation (2), the value of Z is determined based on the drainage layout of the incline well and the main tunnel. When the fixed pump station is located at the 5# fixed reservoir of the left tunnel, the maximum required head for the horizontal centrifugal pump is attained. At this point, the elevation of the fixed pump station bottom is 1181 meters, while the top elevation of the settling tank outside the tunnel is 1300 meters. Therefore, the actual height difference for water pumping at the fixed pump station is 120 meters, as depicted in Figure 6.

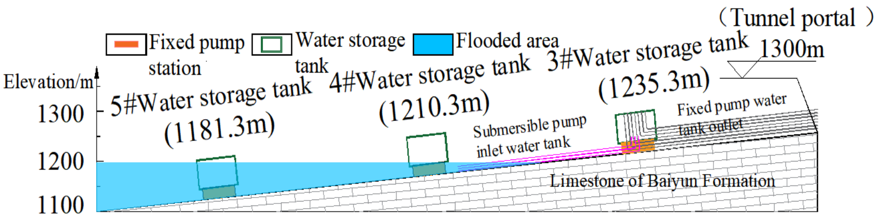

Fig. 6. Cross-sectional schematic diagram of the inclined well pumping equipment.

Next, according to equations (3) to (5), the hydraulic losses along the pipeline are calculated, with hf equaling 35.1 meters. Since the process of pumping water from the reservoir to the outside of the tunnel involves a much greater tunnel length than the flow through gentle bends, the angle of the pipeline bend is approximately horizontal, resulting in minimal local head losses, which can be disregarded. Therefore, the head of the fixed pump should be no less than 160 meters.

(2) Head of the Mobile Submersible Pump Group The maximum required head for the mobile submersible pump group occurs at the intersection of the 5# reservoir and the main tunnel, with an elevation of 1141.2 meters and a height difference Z of 40 meters. Continuing with the calculation of hydraulic losses along the pipeline using equations (3) to (5), hf is determined to be 40 meters. Similarly, the local head losses are negligible and can be disregarded. Therefore, the head of the mobile submersible pump group should be no less than 80 meters.

Preliminary Pump Selection.

Based on the variations in the reservoir positions (from 3# to 4# to 5# to the widened section at 7#), the drainage from the incline well and the main tunnel is divided into four stages, and the preliminary pump selection is shown in Tables 1 to 5 below.

Table 1. Pump Configuration for 3# Reservoir (Phase 1).

Pump Station Type	Power/kW	Theoretical Head/m	Equipment Head/m	Pumping Capacity /m3·h-1	Quantity	Note
Fixed Pump Station	630	102.3	280	1000	2	satisfy
	160	102.3	120-140	400	1	satisfy
Mobile Pump Station	55	42.9	35	500	5	35>(42.9/2)

Table 2. Pump Configuration for 4# Reservoir (Phase 2).

Pump Station Type	Power/kW	Theoretical Head/m	Equipment Head/m	Pumping Capacity /m3·h-1	Quantity	Note
Fixed Pump Station	630	102.3	280	1000	2	satisfy
	160	102.3	120-140	400	1	satisfy
Mobile Pump Station	55	48	35	500	5	35>(48/2)

Table 3. Pump Configuration for 5# Reservoir (Phase 3).

Pump Station Type	Power/kW	Theoretical Head/m	Equipment Head/m	Pumping Capacity /m3·h-1	Quantity	Note
Fixed Pump Station	630	102.3	280	1000	2	satisfy
	160	102.3	120-140	400	1	satisfy
Mobile Pump Station	55	48	35	500	5	35>(48/2)

Table 4. Pump Configuration for 7# Widening Section Reservoir (Phase 4).

Pump Station Type	Power/kW	Theoretical Head/m	Equipment Head/m	Pumping Capacity /m3·h-1	Quantity	Note
Fixed Pump Station	630	102.3	280	1000	2	satisfy
	160	102.3	120-140	400	1	satisfy
Mobile Pump Station	55	48	35	500	4	35>(48/2)

Table 5. Configuration of Backup Pumps.

Phase	Power/kW	Elevation/m	Pumping capacity/m3/h	Quantity
Second	160	120-140	400	1
Third	630	280	1000	1
Fourth	55	35	500	2

4.3 Pump Station Performance and Economic Analysis

Analysis of Centrifugal Pump Parallel Fixed Pump Station Performance. Regarding emergency drainage in the tunnel, the priority is to remove accumulated water from the tunnel as much as possible and ensure the safe operation of the pump station 24 hours a day. Therefore, it is necessary to analyze the rationality of parallel centrifugal pump performance. In parallel operation, pump selection should achieve the effect of jointly exceeding the flow rate of a single pump. The key is to construct a parallel performance curve and analyze the relationship between operating points and the efficient working range to obtain criteria for rational pump selection. Based on the theoretical head values calculated earlier, a parallel performance curve of the pumps is established using the method of superimposing flow rates with equal heads [13]:

The H~Q curve of a single pump conforms to a third-degree polynomial:

$$H = a_0 + a_1 Q + a_2 Q_2 + a_3 Q_3 \tag{6}$$

For n parallel pumps of the same model, the flow-head curve is obtained based on the superposition principle.

$$H = a_0 + a_1 Q/n + a_2 (Q/n)^2 + a_3 (Q/n)^3 \tag{7}$$

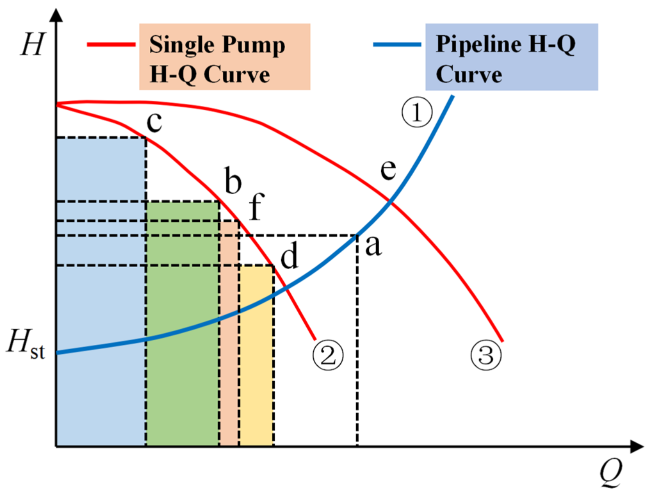

Fig. 7. Performance Curve of Two Centrifugal Pumps in Parallel Configuration.

As shown in Figure 7: Curve ① represents the pipeline H-Q curve; Curve ② represents the single pump H-Q curve; Curve ③ represents the H-Q curve after parallel operation; Point a is the design theoretical point. Zone cd indicates the efficient operating range; Point b is the rated operating point of the pump; Point c is the least favorable point after parallel operation; Point f is the position on the single pump head curve when Qa/n.

Based on the parameters provided by the pump manufacturer for the 630kW centrifugal pump, the performance model of the pump is established according to Equation (6), as shown in Figure 8.

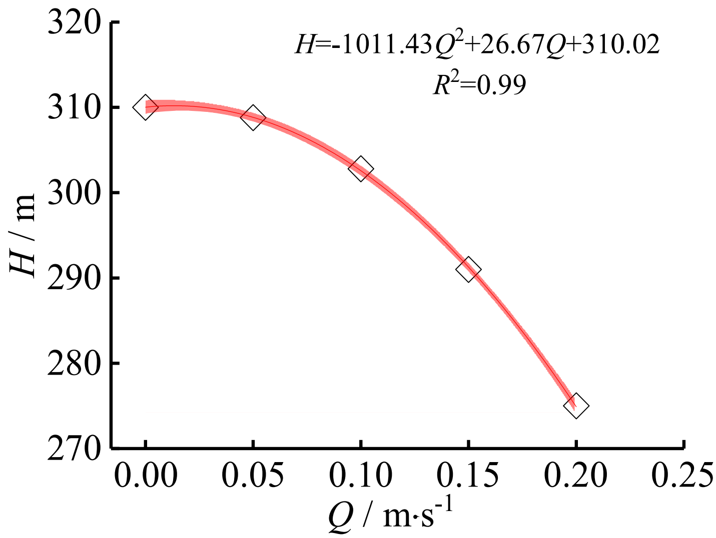

Fig. 8. Fitted Performance Curve of Water Pump.

For 2 fixed pumps, the performance model can be obtained using the flow superposition method[14]. A simpler approach is to substitute the polynomial coefficients fitted from Equation (6) into Equation (7), as follows:

$$H = -252.75Q^2 + 13.34Q + 323.36 \tag{8}$$

In addition, it is necessary to establish the pipeline's H-Q curve. According to the equation for the pipeline curve:

$$H = H_0 + \mu(L + \sum L_J)Q^2 \tag{9}$$

where H0 is the static head of the pump, 280m; μ is the pipe's friction coefficient, taken as 0.018 according to the Water Supply and Drainage Design Manual; L is the length of the pipeline, taken as the distance to the farthest 5# reservoir from the slant well mouth, 1400m; there are no fittings such as reducers or elbows, so the length of fittings can be disregarded.

Therefore, the expressions for the single pump and parallel pipeline curves can be determined as follows:

$$H = 280 + 25.2Q^2 \tag{10}$$

$$H = 280 + 6.3Q^2 \tag{11}$$

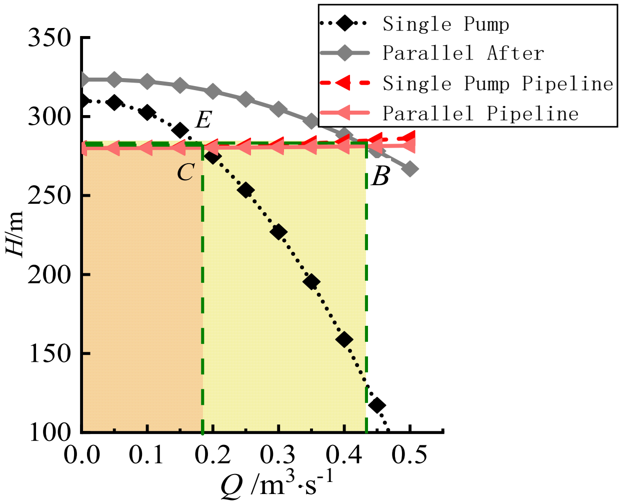

Fig. 9. Parallel Pump Operating Conditions Curve.

Based on the analysis provided and Figure 9, it can be concluded that the single pump operates at point C independently, with a flow rate of 0.19 m3/s and a head of 282 m. This point lies within the efficient operating range "cd" on the graph shown in Figure 5. Point A represents the parallel operation condition, with a flow rate of 0.43 m^3/s and a head of 288 m. Correspondingly, the operating condition of a single pump at point E also falls within the efficient operating range of the pump. Therefore, the design of two centrifugal pumps in parallel can be used to meet the pumping requirements of the four stages of the reservoir.

Economic Analysis of the Mobile Pump Station Scheme.

Furthermore, an economic analysis of the mobile pump station scheme is conducted. As previously calculated, the required head for the mobile submersible pumps is 80 m. The pumping capacity also needs to exceed the effective pumping capacity of the two fixed pumps in Tables 1-4, considering an efficiency of 85% [15].

$$Q_{移}=(1000\times 2+400)\times 0.85=2040\text{m}^3/\text{h} \tag{12}$$

For this purpose, the project team proposes two solutions: Solution 1 suggests creating a temporary storage tank between the two fixed reservoirs when the pump head cannot meet the drainage requirements, and relocating the fixed pumping station to the temporary station; Solution 2 proposes using high-power submersible pumps with a head greater than 62.3m to pump to the fixed reservoir. Based on market research, Solution 2 has strict requirements for armored cables, large pump size, and poor mobility, so Solution 1 is chosen. Additionally, an economic analysis is conducted on four types of mobile pump stations available in the market.

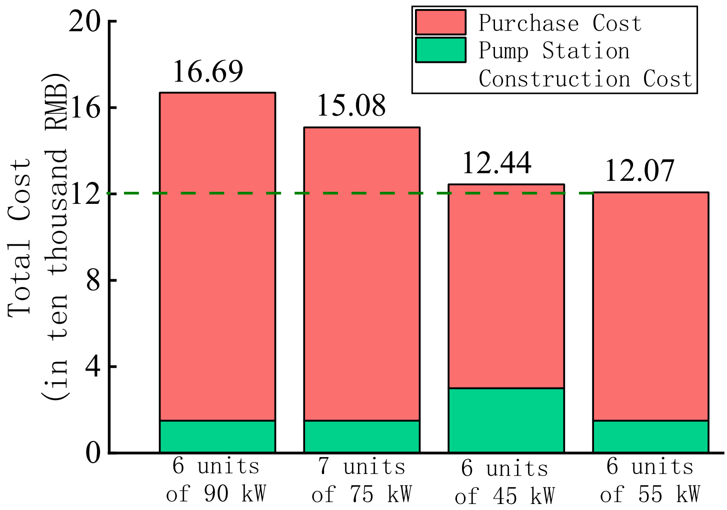

Fig. 10. Economic Analysis of Mobile Pump Stations.

As shown in Figure 10, comparing the total construction costs of mobile pump stations with four different power ratings, using six units of 55kW pumps is relatively economical compared to the other three options, indicating that the initial selection of the mobile pump station model is reasonably justified.

Discussion on Backup Pumps.

To ensure the normal drainage of the tunnel according to the drainage plan, backup pumps are set up with a drainage capacity of 70% of the working pump's capacity. During pump maintenance, the backup pumps are activated to ensure uninterrupted drainage operations. Moreover, when the water discharge in the tunnel exceeds 2000m³/h, two backup 55kW pumps are promptly placed at the water level, while one 630kW pump is stationed at the nearest fixed pumping station for coordinated operation in water drainage.

5 On-Site Implementation Effectiveness

The on-site implementation has been effective since the occurrence of the sudden water inflow during construction on June 13, 2022. The project construction team promptly took measures and initiated the emergency plan of "drainage to ensure safety". They adopted the "slanting well mobile pump + fixed pump in the main tunnel + water storage between pumps" technology for uphill drainage [16-17]. By August 21, 2022, all accumulated water in the slanting well and main tunnel had been successfully drained, ensuring the passage of pedestrians and construction vehicles, as shown in Figure 11 below.

a) Completion of Slope Draining b) Chongqing Left Line (Secondary Lining Completed)

Fig. 11. Implementation Effect of Engineering Applications.

6 Conclusion

Based on the study conducted at the Da bashan Tunnel of the Anlan Expressway, the following conclusions have been drawn regarding the emergency response to the significant water inflow disaster:

(1) The significant water inflow disaster in the Da bashan Tunnel is caused by the combined effects of water storage in the limestone formation, geological structure controlling water flow, water inflow from fault zones, and disturbances from dynamic and static loads.

(2) He positioning of refuge bays as locations for water storage reservoirs is effective. The inlet pipes for these reservoirs should be 5 pipes of DN200, while the outlet pipes should be 3 pipes of DN300. The head of the fixed and mobile pumping stations should be greater than 160m and 80m, respectively. The fixed pumping station should consist of 2 units of 630kW and 1 unit of 160kW, while the mobile pumping station should consist of 5 units of 55kW. Both fixed and mobile pumps should meet the requirements for parallel fixed pump station performance and economy.

(3) Proved by engineering application, the phased drainage technology of "mobile pumping station + fixed pumping station + water storage reservoir" is feasible. Its implementation effectively solves the emergency drainage problem of significant water inflow in the tunnel.

The engineering practice has proved that by adopting the resource allocation and construction method in this paper, the drainage of water in Daba Mountain tunnel has been successfully completed, which solves the construction problem of long distance reverse slope water pumping and drainage, and provides a reference scheme for the drainage of similar tunnels.

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