Safety Assessment Technology of Concrete Arch Dam

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Abstract. Most of China’s reservoir dams were built before the 1980s, long-term safety issues of which have become increasingly prominent. In recent years, with the "high dam, large reservoir, long-distance water diversion" water conservancy projects are underway, the safety assessment of some major water conservancy projects will be a major long-term problem. Taking Gomal Zam Dam as an example, this paper introduced and summarized the safety assessment technology of concrete arch dams, which will provide a reference for the safety assessment and technical development of arch dams in the future.

1. Introduction

China tops the world in terms of the number of reservoirs, dams and water-diversion projects it has initiated. Up to 2013, there were over 98,000 dams of all description in China, 85% of which have been in operation for more than 30 years and about a third above 50 years. Long-term safety concerns of these dams have surfaced gradually, marked by degradation problems such as structural cracking, freeze-thaw damages, corrosion and carbonization, together with increasing seepage, substandard flood control as well as decrease of stress, stability and safety margin. With the “high dam, large reservoir and long-distance water diversion” initiative under way in these years, safety assessment of some major water conservancy projects are becoming a long-standing issue to be addressed.

Safety assessment is imperative when problems emerge in projects, such as substandard design, inferior quality below par, damage-caused performance degradation, changes of operation conditions and difference between the new and old norms. Such assessment usually includes analysis of engineering quality, flood control capacity, structural security, seepage control and aseismic performance of the dam, and relies on experience, elaborate check and accurate calculation to judge the dam’s safety condition and take countermeasures. The following are detailed introduction of safety assessment techniques for concrete arch dams with a specific project as the study case.

2. Research Background

Gomal Zam Dam impounds the Gomal River, a tributary of Indus River, on the northwestern border of Pakistan, with its highest flood stage at 761.5 meters and its largest storage capacity at 2 billion m³. The major dam is a curved roller-compacted concrete gravity dam with a height of 133 meters, with its the average height of foundation along the riverbed at 630 meters, its dam-crest standing 763 meters high and stretching 231 meters long. Flow discharge facilities in the dam include a four-arch chute spillway, a two-arch flow discharge and a one-arch scouring hole. The dam was built through two phases of projects: in the first phase (about 15 years), the normal storage level is 743.2 meters and...
lowest water operating level is 711 meters; the normal storage level increased to 750.40 meters in the second phase.

The geological conditions of Gomal Zam Dam are unfavorable, teeming with fragmented rocks with poor deformation modulus. Severe seepage occurs before the water stored reaches the height of 711.00 meters, and meanwhile the degree of observed deformation shows changes. Safety of a water-storage dam matters a great deal, so comprehensive safety assessment is required to be done on the basis of additional surveys, on-site tests, calculation and analysis of existing observation data.

3. Technical Solution

Comprehensive safety assessment of Gomal Zam Dam was conducted according to the current design and engineering standards to uncover problems and hidden risks in light of specific research results. In accordance with the standards of safety assessment, a series of specific studies were done on its stress and deformation, stability of the dam foundation and abutment, stability and volume of seepage and its seismic performance. To be specific, the studies included on-site detection and testing of quality of consolidation grouting and curtain grouting of the dam’s foundation, analysis of rock mechanical parameters and permeability parameters of the dam foundation, analysis of monitoring data, analysis of the dam’s stability and stress, analysis of the dam’s 3D seepage field and analysis of the dam’s seismic performance. Through analysis of data of design, construction and observation of Gomal Zam Dam and on the basis of specific research results and engineering experience in China and abroad, comprehensive safety assessment was made.

4. Key Content and Techniques of Safety Assessment

4.1. Review of physical and mechanical parameters of the dam foundation rock mass

Through on-site drilling, hydrostatic tests, geophysical prospecting, borehole imaging, borehole modulus testing and on-site testing of the rock bearing plate, the physical and mechanical parameters of rocks in the dam foundation were identified. According to results of on-site tests and geophysical prospecting of the foundation rock mass after consolidation grouting as well as preliminary research, the following principles were applied to defining the physical and mechanical parameters of the foundation rock mass: (1) the rock is mainly layered cataclasite which belongs to Category III-2 and Category IV-1; (2) the passing rate of hydrostatic tests after consolidation grouting is merely 30.1%; (3) the stress condition of the whole curved gravity dam is taken as the stress condition for stress analysis; (4) the deformation modulus on the vertical direction in tests of the bearing plate is taken as the deformation modulus of the rock mass on the abutment, as this direction is perpendicular to the rock’s strike direction and close to the impact direction of the rock mass in the abutment; (5) because tests on the borehole deformation modulus didn’t take into account the strike direction of the rock strata, which would make the smoothness level of the borehole wall unable to meet the test requirements, and cement was grouted to seal the borehole and new boreholes were drilled, the test results would be influenced, so the testing result of the borehole deformation modulus was only taken as a reference for the final analysis; (6) the value of \( f' \) and \( c' \) was identified according to the range of rock classification and preliminary test results; (7) the parameters of rocks in the grouting area which were determined according to the preliminary test results were slightly adjusted based on engineering experience.

The physical and mechanical parameters of the rock mass in the dam foundation are shown in Table 1 and Table 2, and corresponding division of the rock mass in the dam foundation is shown in Fig.1. The structural plane of the rock mass in the abutment includes the rock strata plane, the fault and fracture zone, stress-relief cracks and others. According to the type and filling conditions of the structural plane, the recommended standard mechanical parameters, preliminary test results and experience drawn from similar projects, the parameters of shearing strength of each structural plane were determined: (1) for the F2 and F3 faults and the fault and fracture zone, \( f'=0.4\text{–}0.5 \) and \( c'=0.04\text{MPa}; \) (2) for the stratum on the strong stress-relief zone, \( f'=0.6 \) and \( c'=0.1\text{MPa}; \) and for the weak stress-relief zone, \( f'=0.7 \) and \( c'=0.2\text{MPa}; \) (4) for the stress-relief crack of the low-angle dip
toward the right bank downstream J3, the shearing parameter was defined by taking 20% as the connection rate, the parameter for the structural plane was determined as recommended in (2), and comparison of parameters was conducted by assuming the rock mass was thoroughly sheared (i.e. the discontinuity of the structural plane was neglected).

Table 1. The recommended parameters of rock mass in the foundation of consolidation grouting area

| Position (elevation) | Divition number | Recommended value | Original design value | Value of bearing plate test |
|----------------------|-----------------|-------------------|-----------------------|-----------------------------|
|                      | f' C'(MPa)      | E(GPa)            | f' C'(MPa)            | E(GPa)                      |
| Left bank            | 632-675         | 0.95              | 0.45                  | 4.0                         | 0.80                        | 0.45                       | 5.0                        | 2.98                        |
| 675-720              | 0.90            | 0.40              | 3.5                   | 0.75                        | 0.40                        | 4.0                         | 3.69                        |
| 720-763              | 0.65            | 0.30              | 2.0                   | 0.65                        | 0.25                        | 1.9                         | 2.55                        |
| River bed            | below 632       | 0.95              | 0.50                  | 5.0                         | 0.9                         | 0.5                         | 6.0~7.0                     | 3.0~4.82                    |
| Right bank           | 632-675         | 0.80              | 0.40                  | 3.0                         | 0.75                        | 0.40                        | 5.0                         | 2.22~3.15                   |
| 675-720              | 0.80            | 0.40              | 3.0                   | 0.75                        | 0.40                        | 5.0                         | 3.69                        |
| 720-763              | 0.70            | 0.35              | 2.0                   | 0.70                        | 0.35                        | 3.0                         | 6.35                        |

Table 2. The recommended parameters of rock mass in foundation outside consolidation grouting area

| Position          | Divition number | Recommended value | Original design value |
|-------------------|-----------------|-------------------|-----------------------|
| f' C'(MPa)        | E(GPa)          | f' C'(MPa)        | E(GPa)                |
| Left bank         | 13              | 1.1               | 0.45                  | 3.0                         | 0.80                        | 0.45                       | 2.6                         |
| 14                | 1.1             | 0.45              | 3.0                   | 0.85                        | 0.45                        | 3.0                         |
| River bed         | 15              | 1.1               | 0.50                  | 5.0                         | 1.05                        | 0.55                        | 3.9                         |
| 16                | 1.1             | 0.45              | 3.0                   | 0.85                        | 0.45                        | 2.6                         |

4.2. Stress and deformation analysis

Stress and deformation analysis was made through the multi-arch cantilever method and the finite element method respectively. When applying the multi-arch cantilever method, the special connection of the arch dam and the gravity abutment was taken into account and sensitivity analysis was made. When applying the finite element method, the actual physical and mechanical parameters and the processing range were considered to calculate the dam’s deformation and stress; the equivalent stress was also calculated to analyze whether there would be damages in the dam and where the damages would possibly occur so as to assess safety of the dam above the current basement rock mass.

Research through the multi-arch cantilever method showed that the maximum principal stress on the upstream and downstream plane of the dam in the first phase of water storage was below the allowable stress, thus meeting the standard; in the second phase, the maximum tensile stress exceeded the allowable stress, thus not meeting the standard, but the exceeding degree was within 10%. The dam’s maximum tensile stress was below the allowable stress which met the standard and had a large margin. Figure 2 showed the finite element model used in the calculation. The stress of the upstream plane was mainly around the foundation surface of the left and right arch, with prominent concentration of stress around the heel and toe of the dam. After conversion into equivalent stress, the maximum tensile stress of the basic combination of work conditions was 1.45MPa, the maximum tensile stress of casual combination of work conditions was 1.50MPa, with the maximum pressure reaching -3.30MPa. Both the tensile and compression stress of the dam met the standard, with a certain margin.
To sum up, the dam’s stress basically meets the design standard under different operating conditions, its distribution of displacement follows the general law of displacement of arch dams, and analysis of its stress and deformation meets the safety standard.

4.3. Analysis of abutment stability

There are extensive stress-relief cracks with a steep-dip structural plane parallel to the river on the rock mass in the dam’s left and right abutments. The right abutment shows F2 and F3 faults parallel to the river, and quite a lot of small low-dip structural planes that are parallel to the river inside the rock mass of the abutment. These two structural planes, when intersecting with the tension fracture plane, creates a typical wedge on the left and right abutments, and under extraneous forces including gravity, the arch thrust and water pressure, the abutments may slide downstream and lose stability. Analysis of stability of the abutment were made through three methods, i.e. the 3D rigid-body limit equilibrium method, the finite-element-rigid-body limit equilibrium method and the finite-difference-rigid-body limit equilibrium method.

4.3.1. 3D rigid-body limit equilibrium method. There may be three sliding modes on the left abutment, that is, with the river-parallel stress-relief crack as the sideslip plane, the abutment forms a typical wedge with the rock strata, the slow-dip structural plane J3 and the horizontal plane respectively to generate shear zones of different elevations. The anti-sliding stability and safety coefficients of all these three sliding modes meet the requirements.

Three sliding modes of the right abutment were analysed, that is, with the F2 and F3 faults as the lateral crack plane, the abutment formed a typical wedge with the strata, the slow-dip structural plane J3 and the horizontal plane respectively to generate shear zones of different elevations. The sliding mode of the wedge formed by the F2 and F3 faults and the lateral crack plane showed hyper-stability; as for the sliding mode of the wedge formed by the F2 fault and the slow-dip structural plane J3, the anti-sliding stability and safety coefficients of the controlled sliding mode were 2.08 and 1.80 for the normal water storage levels in the first and second phase respectively under temperature drop; as for the sliding mode of the wedge formed by the F3 fault and the slow-dip structural plane J3, the anti-slide stability and safety coefficients of the controlled sliding mode are 1.95 and 1.80 for the normal water storage levels in the first and second phase respectively under temperature drop. That is to say, the anti-slide stability of the right abutment was not up to par.

4.3.2. Finite-element rigid-body limit equilibrium method. The result worked out by this method basically conformed to that by the method above. The anti-slide stability and safety coefficients of the left abutment under the normal water storage level and the maximum flood level in the first and second phase meet the standard. The anti-slide stability and safety coefficients of the right abutment under most slide modes are below the standard, among which the minimum safety coefficients for the
normal water storage level and the maximum flood level in the first and second phase are 2.04, 1.95 and 1.80, which are all below the standard values.

Among the major loads contributing to the sliding force, the rock’s gravity plays the controlling role, while the hydrostatic pressure and the seepage pressure collectively account for 20% of the total load; the hydrostatic pressure and the seepage pressure can be neglected in the calculation of the anti-slide force. Thus, the rock’s gravity plays a major role in ensuring anti-slide stability of the right abutment.

4.3.3. Finite-difference rigid-body limit equilibrium method. For the water storage level in the first phase and second phase respectively, the anti-slide stability and safety coefficients of the left abutment under the most dangerous slide mode are 3.49 and 3.3 (under temperature drop condition) and 3.55 and 3.36 (under temperature rise condition). The corresponding figures for the right abutment are 2.17 and 2.03 (under temperature drop condition) and 2.22 and 2.09 (under temperature rise condition). The anti-slide stability and safety coefficients are not up to the standard, and the stability of the left abutment is higher than that of the right abutment.

In general, the left abutment is more stable than the right abutment. The anti-slide stability and safety coefficients of the left abutment under the three possible slide modes all meet the standard, but those of the right abutment do not.

4.4. Analysis of holistic safety of the dam
A finite-element model which could reflect the dam foundation’s geological conditions and the dam’s structure was established, and non-linear analysis was made based on the given parameters to work out the range of crack and yield zones under the current water level and the normal water level; meanwhile, the dam’s overload safety coefficient was obtained through non-linear overload analysis, and sensitivity analysis was made with consideration of different parameters. With the above research results and comparison results with similar projects (Kolnberin Dam, Xiaowan Dam and etc.), comprehensive safety assessment of the dam was conducted.

Non-linear finite-element analysis of the DP model showed that though regional yield occurred in the F2/F3 fault under the design load of normal water storage level, the dam remained in the elastic operating state without tendency to overall instability. Safety analysis made under three damage modes, i.e. head overload, volume weight overload and strength reduction, showed that the safety degree for head overload was between 1.4 and 1.5, and that for strength reduction was 2.6, the safety margin of which was smaller than similar projects in China. The safety degree for volume weight overload of the dam was 6.0, which was close to that of similar domestic projects. The volume weight safety degree of the F2/F3 fault was 2.0, the safety margin of which was lower than similar domestic projects.

Analysis of strength reduction and volume weight overload by the Mohr–Coulomb non-linear finite-element method showed that, with the whole project’s loss of bearing capacity as the standard, the overall safety degree of Xiaowan Dam was around 3.5, that of Lijiaxia Dam was around 4.5, and that of Tengzigou Dam and Shimen Dam was around 5.0 and 3.0 respectively. The overall safety degree of this project is obviously low, between 2.0 and 2.5.

4.5. Analysis of seepage control
The rock structure of the dam consists of fractured rock mass with strong permeability and poor seepage control stability, and the designed curtain depth and extension range are not enough, so the reservoir faces the problem of limestone fracture seepage. Under the substandard anti-seepage condition, when the reservoir’s water level is at 771 meters, the total seepage reaches 2~3m3/s, close to the long-time average 3.77m3/s. According to the completed hydraulic test of the curtain grouting check hole, the two-layer grout curtain achieves better effect than the single-layer curtain, the overall permeability rate of the curtain rock mass fails to reach the standard of curtain design.
Based on the total seepage, the Bailey bridge seepage and distribution of underground water when the water storage level reaches 711 meters, regressive analysis of the project’s permeability rate was made, and further analysis about the 3D seepage field distribution features, seepage changes and seepage control stability of the left and right abutments was carried out. Analysis of seepage field showed that the seepage slope was steep at the junction of the curtain and the mountain and at the riverbed beside the apron, which was liable to problems in seepage control stability. Around-dam seepage is one of the major seepage modes under different water storage levels, which leads to high-pressure water head on the left and right abutments, thus undermining the abutments’ stability. Supplementing of each curtain can effectively reduce the seepage of the reservoir and the Bailey bridge, thus contributing to the stability of the dam.

According to the results of on-site check and seepage analysis, systematic and pronged measures including consolidation, replacement, seepage control and water discharge shall be taken to ensure the integration, balance, anti-deformation capacity of the rock mass and improve seepage stability. As prescribed in the standard, “the main curtain shall be completed before the reservoir begins to store water”, and prior engineering experience has also proved that it will be difficult to ensure the quality of the grout curtain when the water storage level is high, so how to ensure the quality of the grout curtain according to the actualities of the project becomes the key problem.

4.6. Analysis of seismic safety
Aside from the static load combination, factors under the effect of earthquakes including static-dynamic combined stress, displacement and seismic stability of the abutment were fully considered to make thorough aseismic safety analysis of the dam-foundation system. Specific research included the following: (1) according to “Specifications for Seismic Design of Hydraulic Structures” and based on the special “heavy top, arch bottom” structure of the dam, the static and dynamic arch-cantilever trial loading method was adopted to make static and dynamic analysis and seismic safety assessment of the dam. (2) With the dam and the ground foundation as an integrated system, the following factors were taken into consideration: dynamic responses from of the dam and the foundation during the earthquake, uneven input of infinite foundation radiation damping and seismic oscillation, dynamic deformation coupling between the dam and the foundation, dynamic motion and displacement of the transverse cracks of the dam. On the basis of dam-foundation non-linear static response analysis, non-linear analysis of response to seismic oscillation was made to assess the strength of the dam; meanwhile, the rigid-body limit equilibrium method with consideration of the arch thrust time course was used to study the stability of the rock mass of the abutment and make comprehensive analysis of the dam’s seismic performance.

As the result of the dynamic and static arch-cantilever trial load method shows, the maximum principal compressive stress on the upstream and downstream of the dam under the designed seismic effect is smaller than the dynamic allowable compressive stress of the concrete of the dam; the static maximum principal tensile stress for some parts of the upstream of the dam slightly exceeds the dynamic allowable tensile stress, but the exceeding volume is small; the static maximum principal tensile stress for the downstream of the dam far exceeds the dynamic allowable tensile stress of the concrete of the dam, and the exceeding volume is large.

Holistic non-linear finite-element analysis on the seismic response of the dam-foundation system shows that the regions where the static-dynamic maximum tensile stress exceeds the dynamic allowable tensile stress of the concrete are mainly stress-concentrating regions such as the left and right arches on the dam crest, the bevel of two gravity abutments and the medium-elevation arch abutments, the area of which is small. The dynamic-static combined maximum principal compressive stress is not large and below the dynamic allowable compressive stress of the concrete of the dam. Analysis of seismic stability of the abutment shows that the minimum anti-slide stability safety coefficient of the slide blocks on both sides is close to the standard value, 1.2, within the seismic time course, and only falls under 1.2 for short time periods, so it can be considered that the seismic safety of the abutment meets the standard.
To sum up, the non-linear finite-element dynamic analysis properly simulated the influence of joints of the dam and the infinite foundation radiation damping, which reflected the actual operating conditions of the dam under seismic effects in a more authentic way. Under the effect of designed earthquake, the stress and seismic stability of the dam both meet the requirement of seismic safety, but the seismic safety margin of the abutment is small.

4.7. Summary
Analysis and assessment of the design, concrete, fundamental processing techniques and safety monitoring of the dam have also been conducted. In safety assessment of Gomal Zam Dam, specific research on its engineering geological conditions, dam design, fundamental processing, stress and deformation, stability of the abutment, the overall safety, seepage control ability and seismic performance, concrete construction and safety monitoring was carried out targeted at its poor geological conditions and seepage in initial stages of water storage. In particular, various methods and advanced analysis techniques were adopted to make comprehensive assessment of safety of the dam.

5. Conclusion
This paper introduced the features of dam safety assessment techniques and expounded on the techniques for safety assessment of concrete dams. With Gomal Zam Dam as the study case, it detailed the technical solutions and major techniques for safety assessment of concrete dams in hopes of providing reference for safety assessment of old dams and damaged dams.

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