Overloading performance of typical block in the left bank abutment of Baihetan arch dam in China

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ABSTRACT:
The stability of the rock mass of the high arch dam abutment is an important factor affecting the structural behavior and safety of the dam. The left bank of the Baihetan hydropower station dam site is a slant and dip slope. The gently inclined in-layer shear zones and the steep-dip faults chamfered slope constitute two typical blocks that affect the stability and deformation of the dam abutment. Because of the importance of the block stability for 289m high arch dam, a three-dimensional rigid body limit equilibrium method based on multi-arch beam overloading and a three-dimensional nonlinear finite element numerical overloading analysis method were proposed to study the performance of the typical blocks in Baihetan arch dam abutment. The analyzed results by applying of the two methods show that although the stability safety factors of the typical blocks are slightly less than 3.5 under normal operating conditions, the safety factors of the typical blocks are above 1.5 when the water load is overloaded 5.5 times, indicating overload capacity of the typical block is strong. It is recommended to take the basic treatment measures to ensure the stability safety factor of the typical block under normal operating conditions to meet the requirements of the specification, and to ensure the stability and safety of the abutment. The research results have been applied to the design of Baihetan hydropower station project, and the proposed overloading methods can be applied in the stability analysis of other arch dam abutment blocks.

1 INSTRUCTION

It has been recognized that the safety of concrete dams depends mainly on complex and varied foundations affected by man-made and natural environment. In the dams around the world, there are many cases of crash or impact on operation and efficiency, which are due to the development of geological defects such as faults and fissures. The literature (Kalustyan 1995) points out that by 1985 0.24% of the world's listed concrete high dam 6261 crashed, and the causes of the crash are mostly related to the dam base. According to incomplete statistics, 60% of the world's dams crashed are caused by the joint destruction of dam body and dam base or shoulder (geological body). The Malpasst arch dam (Habib 1987, Piosel et al. 1991) in France was completed in 1954 and the measured data in July 1959 showed that the displacement value of the dam body and dam base was too large. Then, the dam burst suddenly on the evening of December 2 of the same year, causing huge loss of life and property. Although there is a great deal of controversy about the real cause of the dam crash (Zhang 1998, Zhang 2003), it is agreed that an important factor causing dam failure is poor geological conditions of the dam site, especially the very poor rock mass in the left bank, which develops all kinds of fractures including schistosity, crack, joint and fault, with the characteristics of low shear strength and low deformation modulus for sandwiching with mud partially. The abutment stability of Longyangxia gravity arch dam in China (Chen et al. 1999, Liu et al. 2001) is one of the key technical problems for design and operation of the project. The abutments of Longyangxia gravity arch dam are developed with faults whose intersection forming a fracture zone wide up to 15m in width. The fracture makes the abutment mountains on both banks, especially on the left bank, relatively thin. The displacements of 4th dam section of Longyangxia dam on the left bank are toward the left bank obviously, and the displacements of 13th dam section are toward the right bank firsts and gradually turns to the left bank, and both of them increase gradually in operation of the dam. Therefore, in the design of high arch dam, it is necessary to pay full attention to the stability of abutment rock mass, which has also aroused the extensive attention of engineering construction research in recent years (Jiang et al. 2005, Liu et al. 2008, Wang et al. 2001, Yu et al. 2007). Based on the three-dimensional nonlinear finite element method, the arch beam subload method and the dam abutment three-dimensional rigid body limit equilibrium method, this paper has studied the overload performance of the typical blocks on the left abutment of Baihetan hydropower station, and has provided support for the dam foundation treatment and the structural design of the arch dam.

2 DESIGN OF BAIHETAN ARCH DAM AND LOCAL BLOCK CHARACTERISTICS ON THE LEFT BANK

2.1 *Basic scheme of Baihetan arch dam*

Baihetan Hydropower Station is located in Ningnan County, Sichuan Province and Qiaojia county, Yunnan Province, downstream of the Jinsha River in China. It is another hydropower station with an installed capacity of more than 10 million kilowatts in China after the Three Gorges and Xiluodu hydropower stations. Both banks of the dam site are monoclinic mountains, and the basalt like stratification structure constitutes the layered rock slope on both banks. The left bank is inclined consequent slope, and the right bank is inclined reverse slope. The basic scheme of the dam is concrete elliptical double curvature arch dam layout. The middle and lower parts of dam base mainly use the slightly weathered class II and class III_1 (columnar jointed basalt) rock mass, and the upper part of dam base near the dam crest uses the weakly weathered class III_1 and class III_2 rock mass.

The barrage of the hydropower station is a concrete double curvature arch dam, with an elevation of 834.0m, a maximum dam height of 289.0m, a bottom elevation of 545m, a crest centerline arc length of 714.9m, and a total thrust of 16.17 million tons under the normal pool level.

2.2 *Typical block characteristics on the left bank*

The steep dip F_{17} fault with an extension length of about 1400m, which is exposed at an elevation of about 680m on the left bank of the dam site, constitutes the determined surface of the possible sliding blocks controlling stability of the low and medium left abutment. The occurrence of F_{17} fault is N30 ° ~ 40 ° E, NW ∠ 70 ~ 80 °, and the fault zone with width of 1.2 ~ 3.0m is mainly structural breccia. No other long steep dip structural plane of NE strike was found in the upper plate (inner side) of F_{17} fault. There is no interlayer shear zone exposed in the dam abutment outside of the F_{17} fault, and the bottom slip surface of the blocks may be controlled by the inlayer shear zones only. There are inlayer shear zones named as LS_{3320}, LS_{3319}, LS_{3318}, LS_{3317} and LS_{331} exposed in this area. After the excavation of the dam foundation on the left bank, the blocks combined by F_{17} fault and multiple shallow inlayer shear zones in the dam foundation were excavated, which no longer form the possible sliding blocks of the dam abutment. The inlayer shear zones LS_{331} and LS_{3318} exposed in the low elevation area will still combine with F_{17} fault forming the possible sliding blocks. The relative position of the structural planes is shown in Figure 1. The schematic diagram of the combined typical blocks is shown in Figure 2, and the parameters of the main structural planes are shown in Table 1.

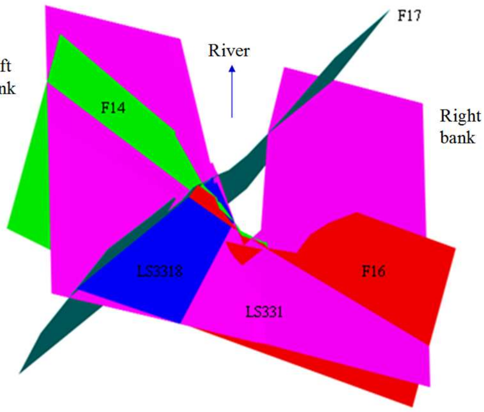

Figure 1. Diagram of the relative position of the structural planes

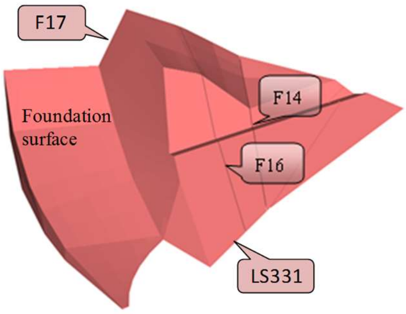

Figure 2. Three-dimensional diagram of the typical block on the left abutment

Table 1. Physical and mechanical parameters of the structural planes

Names	Location	E (GPa)	μ	Unit weight (kN/m^3)	Thickness (cm)	Shear-friction strength	
						f'	c' (MPa)
LS_{3318}	North of F_{16}	1.0	0.35	22.0	20~30	0.38	0.07
	South of F_{16}	1.5	0.35	22.0	5～25	0.50	0.10
LS_{331}	West of F_{17}	1.5	0.35	22.0	3~5	0.65	0.15
	South of F_{16}	1.5	0.35	22.0	2~15	0.55	0.15
	North of F_{16}	1.0	0.35	22.0	15	0.45	0.10
F_{14}		1.0	0.35	22.0	40	0.35	0.05
F_{16}		1.0	0.35	22.0	40	0.35	0.05
F_{17}	Upstream of the F_{14}	1.5	0.35	22.0	60	0.45	0.10
	Within 60m of the F_{14} downstream	0.5	0.35	22.0	80	0.35	0.05
	60m away from F_{14} downstream	0.5	0.35	22.0	80	0.25	0.02

3 ANALYSIS ON OVERLOAD PERFORMANCE OF TYPICAL BLOCKS BASED ON THREE-DIMENSIONAL RIGID BODY LIMIT EQUILIBRIUM METHOD

Because inlayer shear zones that form the possible sliding abutment blocks on the left bank incline to the upstream and the right bank, self-weight of the blocks is beneficial to their stability and the sliding mechanism of the blocks is different from that of other traditional blocks dip toward downstream. In addition to reflect stability state of the typical blocks under special conditions, the abutment stability analyses were carried out by using arch beam water overload method to compute the arch dam thrust force and by applying three-dimensional rigid body limit equilibrium method.

For comparison, the design values of self-weight, uplift pressure, shear capacity parameters of the blocks are still used for overload analyses. The results, i.e. stability factors of the typical blocks, are shown in Figure 3. The stability safety factors of the blocks decrease gradually with the increase of water overloads, but finally tend to be stable. When the water overload multiple is 5.5 times, the shear-friction stability safety factors of the two typical blocks are still greater than 1.7.

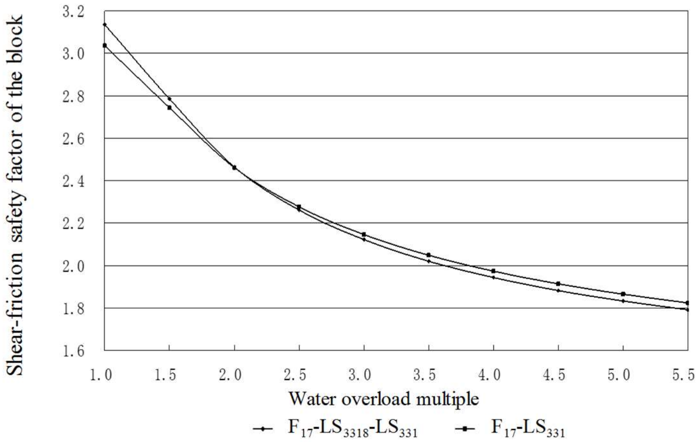

Figure 3. Shear-friction safety factors of the blocks vs. water overload multiples

Although the difference between the overload analyses and the actual operation of the arch dam is large, the results can basically reflect the characteristics and the degree of safety reserve of the two typical blocks on the left bank abutment. That is, with the increase of the arch dam loads, safety factors of the blocks will decrease, but their reduction rates will gradually decrease, and the stability safety factors will eventually tend to a fixed value.

4 ANALYSIS ON OVERLOAD PERFORMANCE OF THE BLOCKS BASED ON 3D NONLINEAR FINITE ELEMENT METHOD

4.1 *Analysis method and model*

4.1.1 *Analysis method*

The whole model of arch dam and foundation was established, and the stress state of the structural surfaces and stability of the blocks in the left abutment were analyzed by using the three-dimensional nonlinear finite element program. The normal operation conditions and water overloading process of the arch dam were simulated, to study the overload capacity of the two typical blocks. The elastic-plastic model based on the Ottosen four-parameter criterion was adopted for the dam body concrete. The physical and mechanical parameters of the dam body concrete are shown in Table 2. The elastic-plastic model based on Drucker-Prager criterion was adopted for the bedrock, and the physical and mechanical parameters were determined according to the rock classification. The elastic-plastic model based on Mohr Coulomb criterion was adopted for faults and inlayer shear zones in parallel direction, and the non-tensile model is adopted for the structural surfaces in perpendicular direction.

Table 2. Physical and mechanical parameters of the dam concrete

E（GPa）	μ	Unit weight (kN/m^3)	Tensile strength (MPa)	Compressive strength (MPa)	Thermal expansion coefficient
2.1×10^4	0.167	24.0	4.0	40	1.0×10^{-5} / °C

When the block is sliding on its both sides, the sliding direction of the block must be along the intersection direction of the two sliding surfaces. The calculation formula of the anti-sliding safety factor of the block is

$$K_z = \frac{\sum \sigma_{1i} f_{1i} A_{1i} + \sum c_{1i} A_{1i} + \sum \sigma_{2i} f_{2i} A_{2i} + \sum c_{2i} A_{2i}}{\sum \tau_{1i} A_{1i} + \sum \tau_{2i} A_{2i}} \tag{1}$$

where K_Z= the overall anti-sliding safety factor of the block; f_{1i} and c_{1i} = the shear strength parameters at the calculation point of the first sliding surface of the block; σ_{1i} = the normal compressive stress at the calculation point of the first sliding surface of the block; τ_{1i} = the shear stress in the sliding direction at the calculation point of the first sliding surface of the block; A_{1i} = the sub area of the first sliding surface of the block; f_{2i} and c_{2i} = shear strength parameters at the calculation point of the second sliding surface of the block; σ_{2i} = normal compressive stress at the calculation point of the second sliding surface of the block; τ_{2i} = the shear stress in the sliding direction at the calculation point of the second sliding surface of the block; A_{2i} = the sub area of the second sliding surface of the block.

4.1.2 *Analysis model and calculation conditions*

In the nonlinear three-dimensional finite element stability analysis model, the range of foundation is about 1.5 times of dam height on the left and right banks, upstream side and down the base, and is about 2.5 times of dam height on the downstream side. Faults F_{17}, F_{16}, F_{14} and inlayer shear zones LS_{3318}, LS_{331} were simulated, and the relative positions of these geological structures are shown in Figure 1. The 8-node hexahedron isoparametric element and 6-node pentahedron isoparametric element were used to simulate the dam body and bedrock, and the 8-node hexahedron thin-layer element was used to simulate the faults and the inlayer shear zones. The dam body was divided into 6 layers along the thickness direction and 24 layers along the height direction. The total number of model elements is 30049, the total number of nodes is 32537. The grid diagram of finite element model is shown in Figure 4.

The loads involved in the calculation mainly include water load, sand load, temperature load, dam self-weight, uplift pressure. The uplift pressure load is determined according to Chinese design standard. The method of water bulk density overload was adopted to analyze overloading performance of the dam and abutment blocks.

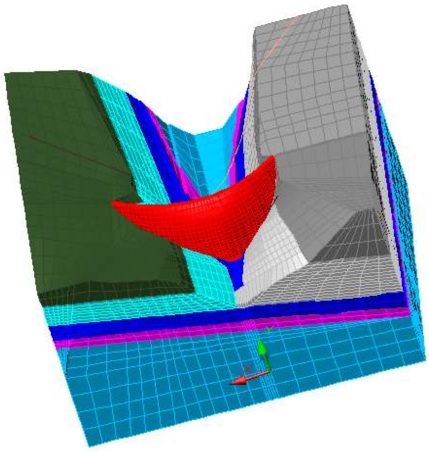

Figure 4. Finite element model grid of the dam and foundation

4.2 ***Analyses On overload performance***

(1) Progress of yield and cracking zones of the foundation surface

The distribution of yield and cracking zones of foundation surface under different water overload times *k* is shown in Figure 5. Progress of the yield zones shows that under the basic combination load condition ($k = 1.0$), a layer element of yield unit appears on the upstream side of the foundation surface of the arch dam below the elevation 620.0m on the left bank and 571.0m of the right bank, and the depth of the yield zone is about 1 / 6 of the dam thickness. When $k = 1.5$, the yield area on the upstream side of the left bank foundation contact surface extends upward to the elevation of 692.0m, and the yield zones of the right bank extends upward to the elevation of 660.0m. At the same time, a layer of unit on the upstream side under the elevation of 586.0m on the left bank and 547.0m on the right bank appears cracking. When $k = 4.5$, the downstream face of the dam toe appears the buckling unit, and the foundation surface of the riverbed appears the through cracking area. When $k = 5.0$, the cracking and yield zones at the lower part of the crown beam are through from upstream side to downstream side. The yield overload coefficient of arch dam is between 4.5 and 5.0.

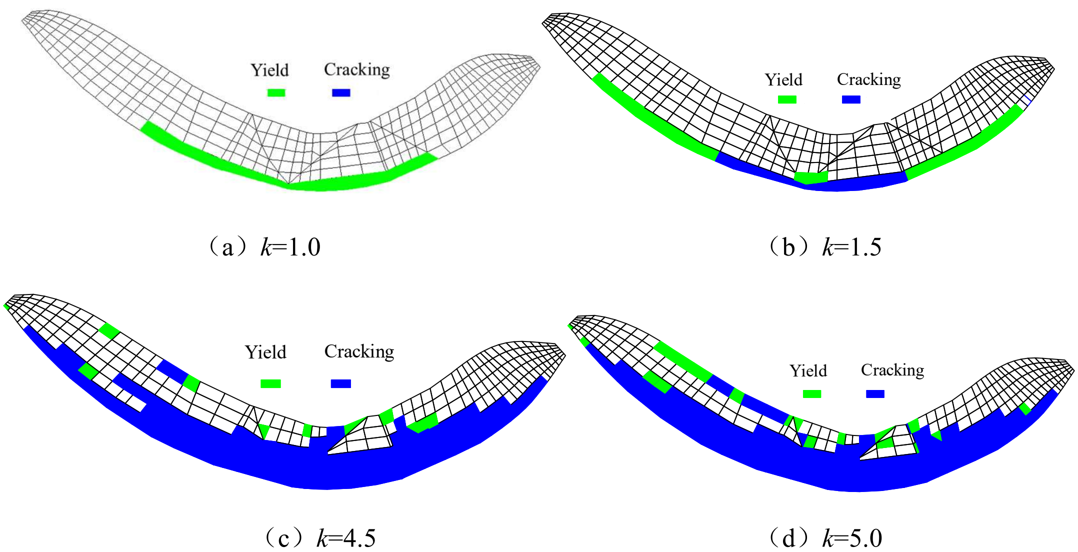

Figure 5. The yield and cracking zones of the foundation surface under different overload multiples

(2) Stability of F_{17}-LS_{331} composite block

The variation process of the stability safety factor of F_{17}-LS_{331} composite block gotten from numerical analyses is shown in Figure 7. The following results can be seen from the figure. When overload multiple k = 1.0, the stability safety factor of the block is 3.17. With the increase of overload multiple, the stability safety factor of the block gradually decreases and tends to converge: when k = 1.5, it is 2.85; when k = 2.0, it is 2.67; when k increases to 6.0, the stability safety factor of the block converges to 1.86.

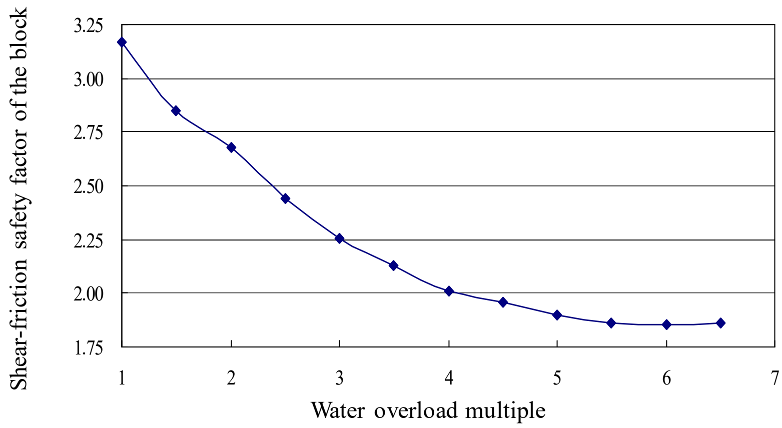

Figure 7. Shear-friction safety factors of F_{17}-LS_{331} composite block vs. water overload multiples

(3) Stability of F_{17}-LS_{3318}-LS_{331} composite block

The variation process of the stability safety factor of F_{17}-LS_{3318}-LS_{331} composite block gotten from numerical analyses is shown in Figure 8. When overload multiple k = 1.0, the stability safety factor of the block is 3.27. With the increase of overload multiple, the stability safety factor of the block gradually decreases and tends to converge: when k = 1.5, it is 2.75; when k = 2.0, it is 2.48; when k increases to 5.5, the stability safety factor of the block converges to 1.82.

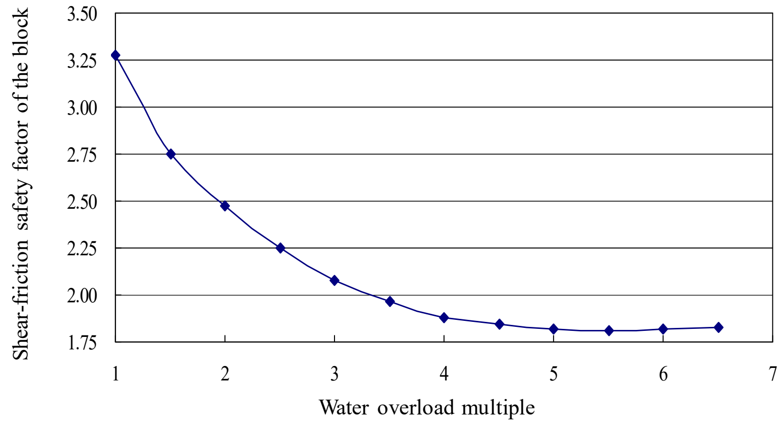

Figure 8. Shear-friction safety factors of F_{17}-LS_{3318}-LS_{331} composite block vs. water overload multiples

5 CONCLUSION

There are two typical blocks (F_{17}-LS_{3318}-LS_{331} and F_{17}-LS_{331}) composed by fault F_{17}, inlayer shear

zones LS_{3318} and LS_{331} exposed in the scope of left abutment of Baihetan arch dam. The shear-friction stability safety factors of the blocks in the basic scheme of arch dam under normal combination conditions are 3.14 and 3.04 respectively obtained by using rigid body limit equilibrium method, which cannot reach 3.5, i.e. the Chinese standard requirement. Because the inlayer shear zones that form the abutment blocks on the left bank incline to the upstream and to the right bank, the effect of block self-weight on stability and the sliding mechanism of the blocks are different from those of traditional blocks dip toward downstream in other dams. Therefore, the water overload means based on the arch beam sub-load method and the three-dimensional nonlinear finite element method is proposed to analyze overloading capacity of the typical blocks. The following conclusions are arrived from the analyses.
(1) The stability safety factors of two local blocks obtained by using rigid body limit equilibrium method decrease synchronously with the increase of water overload multiple, but their decrease rates gradually debase, and the stability safety factors gradually tend to certain values. When the water overload multiple is 5.5 times, the safety factors can still reach 1.7, reflecting the particularity of the stability conditions of the typical blocks in Baihetan left bank abutment. It can be known that the overload capacity of the typical blocks is very strong.
(2) The results of numerical overload analyses are consistent with the results of overload analyses based on the arch beam sub-load method. Under normal operation conditions, the safety factors of the F_{17}-LS_{3318}-LS_{331} and F_{17}-LS_{331} blocks are 3.17 and 3.28 respectively. When the water overload multiple is over 6.0 times, the shear-friction stability safety factors of the two typical blocks converge to 1.86 and 1.82 respectively, which are greater than 1.5, indicating that the two blocks have a high safety margin. In the process of water overloading, no adverse effect of the typical blocks on the yield and cracking of the foundation surface was found. The comprehensive analyses show that the overload coefficient when Baihetan arch dam yielding is between 4.5 and 5.0, and that the overall safety factor is equivalent to that of similar projects in China and is moderate in similar dams.
In conclusion, although the stability safety coefficients of two local blocks on the left bank have not met the requirements of $K \geqslant 3.5$ when applying the shear resistance formula of rigid body limit equilibrium method, the results of overload method analyses show that both typical blocks have sufficient safety margin. Therefore, the comprehensively evaluated anti-sliding stability of the left abutment blocks can meet the requirements of Chinese standards. In view of the importance of the stability of the abutment blocks, it is suggested that the stability of the blocks should be improved properly in combination with the design of anti-seepage and drainage measures.

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