Study on the shape of super-high face rockfill dam with sloping compound structure

Xi Lu¹,², Heng Zhou¹,², Jing Liu¹,² *, Shengjie Di¹,² and Ying Zhang¹,²

¹Northwest Engineering Corporation Limited, Power China, Xi’an, Shaanxi, 710065, China
²High Slope and Geological Hazard Research & Management branch, National Energy and Hydropower Engineering Technology R&D Center, Xi’an, Shaanxi, 710065, China

*Corresponding author’s e-mail: liujing@nwh.cn

Abstract. In order to solve the problems of large deformation and difficult maintenance of the super-high face rockfill dam, this paper innovatively proposes the slope-attached compound structure face rockfill dam. The difference between this dam and conventional rockfill dam is that a certain height of slope-attached concrete is set upstream of face rockfill dam. The finite element method is used to demonstrate the shape of the new structure, and the stress and deformation safety of the slope structure, face plate and rockfill dam are analyzed and evaluated. The research results show that it is feasible to set a certain height of sloping concrete in the upstream of the face rockfill dam, and it is also feasible to set a maintenance and drainage corridor in the slope, and this structure can effectively improve the safety and emergency maintenance conditions of the face rockfill dam.

1. Introduction

At present, a number of 200 m-level face rockfill dams have been built at home and abroad. The completed Shuibuya face rockfill dam in China has the highest dam height of 233 m, which is also the highest dam in the world [1]. The Bagong dam designed and constructed in China has the dam height of 203.5 m, which is the second highest dam in the world. With the construction and research of a batch of 200 m high concrete face slab dams at home and abroad, the dam construction technology and theoretical research level of 200 m high concrete face slab dams have gradually become mature. However, with the continuous increase of social demand, the construction demand of 300 m high face dam is increasing day by day, so it is urgent to study the corresponding dam construction technology. Several 200 m-level high face rockfill dams put into operation in recent years have been operating normally, but there are still some problems, such as large deformation of dam body, face panel extrusion damage or large leakage. Engineering technicians have expressed concerns and doubts about whether 300 m-level or higher dams can be built safely [3 ~ 4]. The development of concrete face rockfill dams in China is faced with the challenge from 200 m dam height to 300 m dam height. Compared with 200 m high face rockfill dam, the stress field in 300 m high face rockfill dam is higher, and the safety problems and risks will be more prominent.

In order to solve the problems such as large deformation, face plate damage, large leakage, poor maintainability, difficult to meet the safety and emergency management requirements of 300 m super high face rockfill dam. This article proposed the slope type compound structure face rockfill dam, and
uses the finite element to carry out the preliminary body demonstration. The stress and deformation safety of slope structure, face slab and rockfill dam are studied and analyzed, which provides an important reference for the design of 300 m super-high face rockfill dam.

2. Research scheme and analysis model

2.1. Research scheme
The dam of the project is a concrete face rockfill dam, the maximum height of which is 247 m. In this paper, based on the project, to stick a face rockfill dam slope type double entry structure are studied, revealing the concrete of the dam heel is set to stick poe's influence on the deformation and stress of the dam body, especially the anti-seepage body, key analysis completion period and storage period, and compared with the calculation results of conventional rockfill dam analysis, comprehensive comparing the pros and cons of the new compound structure. Typical sections of face rockfill dams are shown in Figure 1.

![Figure 1. Computed profile diagram.](image)

The height of the concrete slope in the research scheme is all 50 m, as follows:
- Scheme 1: Slope thickness is 10 m.
- Scheme 2: Uneven slope thickness, 8 m at the top and 10 m at the bottom.
- Scheme 3: Slope thickness is 5m, and there are two drainage corridors for maintenance in the middle of the slope body.
- Scheme 4: Slope thickness is 4m, and there are two drainage corridors for maintenance in the middle of the slope.
- Scheme 5: Slope thickness is 3m, and the middle part of the slope body is equipped with two drainage corridors for maintenance.

2.2. Analyze the model and parameters
The static and dynamic model parameters of dam material needed for calculation are determined by large indoor triaxial test, and the model parameters of contact surface are determined by combining with engineering experience. Duncan E-B nonlinear model is adopted for rockfill, and the model parameters are shown in Table 1. The concrete slabs, toe slabs and bedrock were considered according to the linear elastic model. Goodman element was used to simulate the connection between the concrete slabs and the cushion, and the joints around the slabs and vertical joints were used to simulate the connection element. The grid size of rockfill and foundation is 5 ~ 8 m, and the grid size of concrete structure is 2 ~ 3 m. The computed finite element model and the body shape of the slope are shown in Figure 1 and Figure 2.

![Figure 2. Finite element analysis model.](image)

![Figure 3. Slope concrete sign.](image)
Table 1. Material parameters.

| Material          | ρ (g/cm³) | φ₀ (º) | Δφ (º) | k    | n    | Rₑ    | kₑ    | Kₑ    | m    |
|-------------------|-----------|--------|--------|------|------|-------|-------|-------|------|
| Cushion           | 2.31      | 49.3   | 5.9    | 920.2| 0.29 | 0.67  | 1840  | 497.6 | 0.31 |
| Transition        | 2.31      | 49.3   | 5.9    | 920.2| 0.29 | 0.67  | 1840  | 497.6 | 0.31 |
| Main pile stone   | 2.27      | 50.1   | 6.3    | 1294.1| 0.32 | 0.74  | 2588  | 545.9 | 0.26 |
| Secondary rockfill| 2.22      | 53.2   | 9.0    | 1104.4| 0.22 | 0.65  | 2209  | 547.3 | 0.08 |
| Concrete          | 2.40      |        |        |      |      |       |       |       |      |

3. The toe slab stable

The peripheral joint directly determines the stability of the slope type concrete, so the stability of the toe plate under the condition of different peripheral joint inclination is analyzed, and the reasonable peripheral joint inclination is selected. Calculate 0°, 5°, 7.5° and 10° respectively. Figure 4 shows the location of the peripheral joints. The calculation results are shown in Table 2 and Table 3. Table 2 shows the anti-slip stability coefficient and the displacement value of the peripheral joints of the toe plates with different inclination angles. Table 3 shows the anti-sliding stability results of toe plates with different L lengths and different inclination angles.

When the dip angle of toe slabs is larger and steeper, the completion period of anti-skid stability coefficient of toe slabs is smaller than that in the impoundment period; while when the dip angle of toe slabs is smaller and the crack is flatter, the completion period of anti-skid stability coefficient of toe slabs is larger than that in the impoundment period. This indicates that the steeper the toe seam is, the more dangerous the anti-sliding stability is in the completion period than in the impoundment period. During the completion period, the sliding-resistance stability coefficient generally decreases with the increase of the dip angle of the toe slabs. The sliding-resistance stability coefficient of the toe slabs does not show obvious regular changes with the change of the dip angle during the water storage period. In general, the sliding-resistance stability coefficient of the toe slabs during the water storage period is large and the safety reserve is high.

During the completion period, with the increase of seam inclination angle, the shear deformation and compression deformation decrease, during the impounding period, with the increase of joint inclination angle, the shear deformation increases while the compression deformation decreases. When the front length of toe plate L = 5.0 is reduced to 4.0 m, the sliding - resistance stability coefficient of toe plate decreases to some extent, but the law is the same as that of 5.0 m. The sliding - resistance stability coefficient of toe plate during the completion period and the water storage period can meet the requirements of the code.

From the perspective of maximizing the anti-skid stability of toe slab, the safety margin of anti-skid stability of toe slab is the largest when the inclination angle is 0 degree. However, during the completion period of this scheme, the seam shear deformation is large, and the angle of the slope sticking body is too small on the upstream surface of the contact part of toe slab, which is easy to produce stress concentration, which is detrimental to the safety of toe sticking body is large, the safety margin of anti-sliding stability of toe plate is small in the completion period, and the shear deformation of seam is large in the impoundment period. Based on the above calculation results, the inclination angle of 5 or 7.5 are recommended.
Table 2. Sliding resistance and stability of toe plate with peripheral seam displacement (L = 4.0 m).

| Dip angle of toe slabs seam | Slip resistance stability factor | Peripheral seam displacement (mm) |
|-----------------------------|---------------------------------|----------------------------------|
|                             | Completion period | Storage period | Completion period | Storage period |
|                             | Shear | Compression | Shear | Compression |
| 2.853                       | 6.091 | 12.3        | 8.7   | 46.7        | 21.1 |
| 2.610                       | 8.157 | 18.7        | 13.5  | 31.7        | 26.0 |
| 5.154                       | 4.178 | 21.8        | 16.3  | 26.5        | 31.3 |
| 12.870                      | 11.074 | 42.1 | 21.3 | 25.1 | 39.2 |

Table 3. Anti-slip stability results of toe plates with different lengths and different inclination angles.

| Dip angle of toe slabs seam | Slip resistance stability factor |
|-----------------------------|---------------------------------|
|                             | L = 5.0 m Completion period | Storage period | L = 4.0 m Completion period | Storage period |
|                             | Shear | Compression | Shear | Compression |
| 2.937                       | 6.307 | 2.853        | 6.091 |
| 2.699                       | 8.609 | 2.610        | 8.157 |
| 5.323                       | 4.522 | 5.154        | 4.178 |
| 13.287                      | 11.855 | 12.870 | 11.074 |

4. Comparative study of dam body shape

4.1. Stress and deformation of the dam

Table 4 lists the extreme stress and deformation values of the dam body in each scheme. The calculation results show that the shape of the sloping body has little influence on the stress and deformation extremum of the rockfill dam body and face.

Table 4. Maximum stress and deformation value of rockfill with different slope body shape.

| Scheme                  | Completion period | Storage period |
|-------------------------|-------------------|----------------|
|                         | 1     | 2     | 3     | 4     | 5     | 1     | 2     | 3     | 4     | 5     |
| Displacement along the river (cm) | Upstream | 42.3 | 42.3 | 44.9 | 44.9 | 19.4 | 19.3 | 20.8 | 20.8 | 20.8 |
|                         | Downstream | 56.2 | 56.3 | 55.9 | 55.9 | 67.9 | 67.9 | 67.9 | 67.9 | 67.9 |
| Settlement (cm)         | 189.2 | 189.2 | 189.2 | 189.2 | 195.6 | 195.6 | 195.6 | 195.6 | 195.6 |
| Maximum principal stress (MPa) | 4.91  | 4.91 | 4.91 | 4.91 | 5.36 | 5.35 | 5.36 | 5.36 | 5.36 |
| Small primary stress (MPa) | 1.70  | 1.70 | 1.70 | 1.70 | 1.70 | 1.70 | 1.70 | 1.70 | 1.70 |

4.2. Stress and deformation of concrete slope sticking body

During the completion period of schemes 1 ~ 5, the maximum principal stress of the slope sticking body is 3.68 MPa, 4.65 MPa, 6.94 MPa, 6.97 MPa and 6.88 MPa respectively, which all appear at the top of the slope sticking near the downstream side. The minimum values of small principal stresses in the slope sticking body of the schemes 1 ~ 5 during the completion period are -0.48 MPa, -0.54 MPa, -1.08 MPa, -0.91 MPa and -0.33 MPa respectively. The minimum values of small principal stresses in the schemes 1 and 2 appear at the top of the slope sticking body, and the minimum values of small principal stresses in the schemes 3 ~ 5 appear around the corridor at the bottom.

During the impoundment period of schemes 1 ~ 5, the maximum maximum principal stress of the sloping body is 5.11 MPa, 4.73 MPa, 10.16 MPa, 8.30 MPa and 7.81 MPa, respectively. The maximum principal stress of schemes 1 and 2 all appear on the upstream slope of the sloping body, and the maximum principal stress of schemes 3 ~ 5 all appear around the bottom corridor. In schemes 1 ~ 5, the minimum values of the small principal stress of the slope sticking body in the water storage
period are -1.12 MPa, -1.18 MPa, -1.25 MPa, -1.52 MPa and -1.42 MPa respectively, and the minimum values of the small principal stress all appear on the downstream slope of the slope sticking.

Due to the small size of scheme 2, the deflection is slightly greater than scheme 1, and the tensile and compressive stresses of scheme 2 during the completion period are also slightly greater than those of scheme 1. In the impoundment period, the compressive stress of scheme 2 is smaller than that of scheme 1, but the tensile stress is larger than that of scheme 2. In all schemes 3 ~ 5, corridors are set in the slope sticking body. With the decrease of the thickness of the slope sticking body, the deflection increases during the storage period. From the perspective of controlling the tensile stress of the slope sticking body, scheme 3 is better than scheme 4 and 5. Figure 5 shows the main stress distribution of the slope sticking body in the project 3 completion period, Figure 6 shows the main stress distribution of the slope sticking body in the project 3 storage period, and Figure 7 shows the deflection distribution of the slope sticking body in the project 3 storage period.

4.3. Peripheral seam displacement
Schemes 1 ~ 5 during the completion period, the joint subsidence deformation between the panel and the slope is 0.32 mm, 1.76 mm, 1.7 mm, 1.7 mm, 1.6 mm respectively, schemes 1 ~ 5 water storage periods are 0.97 mm, 0.28 mm, 3.5 mm, 3.4 mm, 3.1 mm.
Scheme 1 ~ 5 during the completion period, the opening deformation of joints between the panel and the slope is 38.7 mm, 38.6 mm, 34.2 mm, 33.1 mm, and 32.5 mm respectively, schemes 1 ~ 5 water storage periods are 40.7 mm, 40.1 mm, 36.1 mm, 35.6 mm and 34.6 mm respectively.

During the completion period of schemes 1 ~ 5, the shear deformation of the joint between the slope sticking body and toe plate is 18.6 mm, 17.2 mm, 6.2 mm, 6.1 mm, 6.0 mm respectively, schemes 1 ~ 5 water storage periods are 31.9 mm, 34.1 mm, 44.5 mm, 46.5 mm, 49.2 mm respectively.

Schemes 1 ~ 5 during the completion period, the compression deformation of the joint between the slope sticking body and the toe plate is 13.5 mm, 11.5 mm, 9.6 mm, 9.5 mm and 6.9 mm respectively. schemes 1 ~ 5 are 26.0 mm, 24.7mm, 6.1mm, 6.9mm, 6.6mm respectively.

Schemes 1 and 2, the deformation and stress of the face and water stop basic same, joint scheme 3 ~ 5 with vertical with panel shear displacement of peripheral joint type of the package 1, 2 has a larger increase, scheme 3 ~ 5, the shear displacement of storage period is 44.5 mm, 46.5 mm and 49.2 mm, scheme as posted in 3 ~ 5 increase with the decrease of the shear displacement of peripheral joint thickness of slope.

5. Conclusion
This paper proposes a post high concrete face rockfill dam slope type double entry structure, and on the analysis of the dam is studied, results show that the storage period, stick a tensile stress in the downstream slope position control is the key to determine the dam type size, so from the perspective of tensile stress in the slope control, selection scheme 1 (50 m stick slope height, thickness of 10 m, peripheral joint angle is 7.5 °) is appropriate. Although the tension and compressive stress and joint displacement of schemes 3 ~ 5 are greater than those of schemes 1 and 2, scheme 3 (the height and thickness of the slope are 50 m and 5 m, two drainage corridors are set in the middle of the slope) is appropriate if a corridor is taken into consideration. In addition, the larger stress in this scheme belongs to the stress concentration with limited scope, and the difference between the stress level of the other parts and the absence of corridors is not significant.

To sum up, it is feasible to set a certain height of slope concrete in the upstream of the face rockfill dam, and it is also feasible to set a drainage corridor for maintenance in the slope. This structure can effectively improve the safety and emergency maintenance conditions of face dam.

Acknowledgments
This work was financially supported by the National Key Research and Development Program of China (Grant No.2017YFC0404805) of Northwest Engineering Corporation Limited, Power China. In the meantime, we express thanks to our colleagues for their help and technical support.

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