Stress-strain analysis of Aikou rockfill dam with asphalt-concrete core

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Received 13 December 2010; received in revised form 4 March 2011; accepted 24 March 2011

Abstract: Aikou rockfill dam with asphalt-concrete core is situated in a karst area in Chongqing City, China. In order to study the operative conditions of the rockfill dam, especially those of the asphalt-concrete core, the Duncan model is adopted to compute the stress and strain of both the rockfill dam and the asphalt-concrete core after karst grouting and other treatments. The results indicate that the complicated stress and deformation of both the dam body and the core are within reasonable ranges. It is shown that structure design and foundation treatment of the dam are feasible and can be used as a reference for other similar projects.

Key words: asphalt-concrete core; rockfill dam; Aikou reservoir; stress and deformation

1 Introduction

Aikou rockfill dam with asphalt-concrete core is located on the Pingjiang River, about 1.7 km away from Aikou Town in Xiushan County, Chongqing City, and about 0.6 km downstream of the estuary of the Liangqiao River merging with the Cenlong River.

Some data related to the reservoir, the dam and the core are listed in Table 1.

<table>
<thead>
<tr>
<th>Normal water level</th>
<th>Safety check flood level</th>
<th>Elevation of dam top</th>
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</tr>
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<th>Core width at elevation of 481.0 m</th>
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The core width expands to 3.0 m from the elevation of 481.0 m to the top of the grouting gallery, following the ratio of 1:0.3. Copper seals are set to ensure the connection between the concrete and the asphalt-concrete core.

From upstream to downstream of the river, the dam body is arranged as follows: upstream main rockfill, transition layer, roller-compacted asphalt-concrete core, cushion layer, transition layer, and downstream main rockfill. The maximal cross-sectional profile of the dam is shown in Fig.1.

The main stratum of the dam foundation is composed of dolomite ε3 of Houba group of Cambrian system, limestone with dolomite of Moda group; Ordovician system O1t in dam left abutment, limestone O1h, and a small amount of dolomite limestone. Here, εln, O1t, O1h and O1h are karst layers, εln is normally a weak karst layer, and O1t and O1d are water-resistant layers. Karst is also strongly developed at the fault. The descriptions of the rocks corresponding to different strata are shown in Table 2.

Table 2 Data related to the reservoir, the dam and the core.

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Karst is developed in the whole dam site. On the left bank, a karst system of Kw3 and Kw2 is developed, which is duct-like and relatively isolated. The dam foundation is located on the layer ε1m, a strongly developed karst layer, where the minimal straight rate of limestone cavern is 5.08% and the maximum is 50%. On the right bank, the abutment is mainly located on an ancient streambed, where karst layers ε3 and ε3h are fully developed, and Kw12, Kw51 and W103 systems exist. The elevation of karst layer in the dam right abutment and dam right bed can reach 330–370 m.

The hydraulic conductivity of rock mass is moderate on the left bank, but it is very strong on the riverbed. Among the total tested sections, 38.9% have a value of
q > 10 Lu. The hydraulic conductivity of rock mass on the right bank is the strongest, with 67.2% having a value of $q > 10$ Lu, and 29.4% having a value of $q < 5$ Lu.

The karst at the dam base is strongly, asymmetrically and unevenly developed. The karst straight rate of the foundation at the depth of 0–40 m is 0%–78%, averaging 21.15%. The karst cave is filled with mud, sand and gravel with an average filling rate of more than 77%. The largest cave is up to 12.92 m in diameter. Therefore, local collapsing of dam foundation probably exists.

In order to prevent collapsing, and to ensure the stability of the dam foundation, two reinforcement measures are adopted. First, karst caves were dug up in the depth of 3–5 m underground, and then they were backfilled with concrete C15. Second, consolidation grouting was employed. The grouting holes were gradually deepened and thickened by using the so-called three-sequence-aperture method. The distance between the aperture pitches of sequence I is 12 m, and the depth of the aperture pitches varies from 36 m in the dam’s foundation to 13 m at the dam’s toe, heel and abutment. Similarly, the distance between the aperture pitches of sequence II is 6 m, and the depth varies from 21 to 13 m; the distance between the aperture pitches of sequence III is 3 m, and the depth is 13 m.

During construction, if there appeared any karst cave with the diameter more than 12 m at the bottom of the apertures of sequence I, the nearby apertures of sequences II and III would be deepened to the depth of sequence I. If the diameter of a karst cave at the bottom of the apertures of sequence II was more than 8 m, the around apertures of sequence I would be deepened to the depth of sequence II [1, 2].

Based on these measures, it is indicated that the stability of the dam base can be ensured [3]. In this paper, the deformation and safety of the dam body and the asphalt-concrete core are studied [4].
2 Computational methods and parameters

Since asphalt-concrete is a hybrid material with multi-phase and multi-component characteristics, its compacted structural and physico-mechanical properties are not only correlated with the material quality and the physical characteristics of the adoptive minerals, but also related to the physico-chemical characteristics of the minerals. Triaxial tests of asphalt-concrete have been conducted at Tongji University. Experimental results show that, when the lateral pressure reaches 0.6–1.2 MPa, the stress-strain relationship of asphalt-concrete can be described approximately with hyperbola. Thus, it is reasonable to use the Duncan model to describe the stress-strain relationship of high rockfill dam with asphalt-concrete core [5, 6].

Many studies have been carried out about rockfill dams with the asphalt-concrete core. The results show that with the Duncan model, the stress and deformation analyses of dams are close to the actual ones, which demonstrates the application of the Duncan model [7–10]. The physico-mechanical parameters of the materials used by Zhang et al. [7] are shown in Table 3, where \( K, n, G, F \) and \( D \) are the experimental parameters.

Table 3 Physico-mechanical parameters used in the analysis of rockfill dam with asphalt-concrete core [7].

<table>
<thead>
<tr>
<th>Material</th>
<th>Internal friction angle, ( \phi ) (°)</th>
<th>Cohesion, ( c ) (kPa)</th>
<th>Failure ratio, ( R_t )</th>
<th>( K )</th>
<th>( n )</th>
<th>( G )</th>
<th>( F )</th>
<th>( D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rockfill</td>
<td>45.4</td>
<td>0.046</td>
<td>0.81</td>
<td>644</td>
<td>0.37</td>
<td>0.41</td>
<td>0.08</td>
<td>1.44</td>
</tr>
<tr>
<td>Asphalt-concrete</td>
<td>26.9</td>
<td>0.24</td>
<td>0.72</td>
<td>273</td>
<td>0.28</td>
<td>0.48</td>
<td>0.08</td>
<td>0</td>
</tr>
<tr>
<td>Backfill</td>
<td>47.4</td>
<td>0.045</td>
<td>0.90</td>
<td>950</td>
<td>0.39</td>
<td>0.47</td>
<td>0.09</td>
<td>1.48</td>
</tr>
</tbody>
</table>

The tangent modulus \( E_i \) of the Duncan model is given by

\[
E_i = (1 - R_t S) E_i^0
\]

where \( R_t \) is the failure ratio, \( S \) is the stress level, and \( E_i^0 \) is the initial tangent modulus.

The Mohr-Coulomb criterion can be written as

\[
(\sigma_i - \sigma_s) = \frac{2c \cos \phi + 2\sigma_s \sin \phi}{1 - \sin \phi}
\]

The tangent Poisson’s ratio \( \mu_i \) is given by

\[
\mu_i = \frac{g - F \log \left( \frac{\sigma_i}{P_s} \right)}{(1 - A)^2}
\]

\[
A = K \rho_i \left( \frac{\sigma_i}{P_s} \right)^n \left[ 1 - R_t (1 - \sin \phi) (\sigma_i - \sigma_s) \right] \frac{2c \cos \phi + 2\sigma_s \sin \phi}{1 - \sin \phi}
\]

According to geological data, the karst filling materials are mostly clayey soil with low-strength crushed stone.

The physico-mechanical parameters of materials employed in Aikou rockfill dam with asphalt-concrete core are shown in Table 4.

Table 4 Physico-mechanical parameters of materials of Aikou rockfill dam with asphalt-concrete core.

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit weight, ( \gamma ) (kN/m³)</th>
<th>Internal friction angle, ( \phi ) (°)</th>
<th>Cohesion, ( c ) (kPa)</th>
<th>Failure ratio, ( R_t )</th>
<th>( K )</th>
<th>( n )</th>
<th>( G )</th>
<th>( F )</th>
<th>( D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rockfill</td>
<td>21</td>
<td>35</td>
<td>0</td>
<td>0.8</td>
<td>1</td>
<td>0.03</td>
<td>0.37</td>
<td>0.1</td>
<td>11</td>
</tr>
<tr>
<td>Transition material</td>
<td>21.5</td>
<td>33</td>
<td>0</td>
<td>0.78</td>
<td>0.3</td>
<td>0.42</td>
<td>0.08</td>
<td>0.08</td>
<td>11</td>
</tr>
<tr>
<td>Asphalt-concrete</td>
<td>23.5</td>
<td>24</td>
<td>120</td>
<td>0.82</td>
<td>0.4</td>
<td>0.42</td>
<td>0.08</td>
<td>0.08</td>
<td>7</td>
</tr>
<tr>
<td>Cushion material</td>
<td>22</td>
<td>30</td>
<td>0</td>
<td>0.8</td>
<td>0.3</td>
<td>0.40</td>
<td>0.08</td>
<td>0.08</td>
<td>7</td>
</tr>
<tr>
<td>Backfill</td>
<td>20</td>
<td>35</td>
<td>0</td>
<td>0.75</td>
<td>0.4</td>
<td>0.38</td>
<td>0.15</td>
<td>2.7</td>
<td></td>
</tr>
</tbody>
</table>

3 Stress-strain analysis

3.1 Classification of calculated loads

The deadweight of the dam body is the dominant load during construction period. But in impoundment period, the load also includes the water pressure acting on the core and the buoyancy acting on the dam body.

In this paper, based on the construction schedule, the deadweight in construction period is divided into 10 levels, and the water pressure in water storage period is divided into 8 levels. In the whole process, the water level is considered to rise up to normal pool level and then drop to deadwater level and finally rise up again [9, 10].

3.2 Analysis of computational results

For simplicity, the computational results of deformation and stress patterns of the dam after construction and under the condition of normal pool level are listed Figs.2–6.

The horizontal and vertical displacements of the asphalt-concrete core and the stresses under different calculation conditions are shown in Fig.7.
(a) Vertical displacement.
(b) Horizontal displacement.

**Fig.2** Displacement isolines of dam body after construction (unit: mm).

**Fig.3** Stress level isolines of dam body after construction.

(a) Vertical displacement.
(b) Horizontal displacement.

**Fig.4** Displacement contour lines of dam body under the condition of normal pool level (unit: mm).
Fig. 5 The maximum principal stress isolines of dam body under the condition of normal pool level (unit: MPa).

Fig. 6 Stress level isolines of dam body under the condition of normal pool level.

Fig. 7 Comparisons of horizontal and vertical displacements and stress level of core wall under different operative conditions.

The displacement during the special impounding process, in which the water level rises to the normal pool level and then drops to 520 m and finally rises again, is shown in Fig. 8.

The computational results indicate that:

1) At all loads, the shear stress ratio (stress level) between dam body and core is less than 1, which shows that the stress rupture does not happen. The maximum stress level is about 0.8 after dam construction and it appears near the upstream face of the dam. Because the deadwater level is comparatively low, it has a little influence on the stress level of the dam body. At water level of 520 m, the stress level of the upstream face of the dam increases, whereas the
Fig. 8 Comparisons of horizontal and vertical displacements of the core under conditions that the water level drops from the normal water level to 520 m and then rises again.

(1) The asphalt-concrete core has a relatively larger horizontal displacement. After dam construction, at the bottom of the core and 2/3 of the dam height, an upward displacement with a maximum value of 27.78 mm occurs. As the water level rises, the horizontal displacement of the core gradually changes its direction to downstream and the maximum value occurs at the top of the dam. At the normal pool level, the maximum downward displacement at the dam top is 302.8 mm and the relative displacement at the bottom of the core is 269.0 mm. At the safety check flood level, the maximum downward displacement at the dam top is 379.4 mm and the relative displacement at the bottom of the core is 340 mm. The maximum settlement after completion of dam construction is 281 mm. At the normal pool level and the safety check flood level, those values are 206 and 198 mm, respectively. All of these happen at the height of 1/2–1/3 of the core. After impoundment, the top of the core wall is lifted in different states, reaching 100 mm in height at the safety check flood level. This indicates that the change of displacement in the core is very complex. The stress level is less than 0.65 under different conditions, so the core is safe. The change pattern of the shear strength is basically consistent with that of the core displacement. In order to cope with the complicated changes of stress and displacement, which may induce pore increasing in the asphalt-concrete and the material deformation limitation to be exceeded, high construction quality of the asphalt-concrete core must be ensured. At the same time, the tests on asphalt-concrete, cushion material and transition material are needed to control all materials' moduli within a reasonable range, and to make sure that the deformation of the asphalt-concrete is acceptable.

(2) The stress level and displacement near the bottom of the core and the interface of the grouting gallery are both small, and their changes are comparatively mild and slow. This shows that the structural shape and geometrical size adopted in the design can guarantee the safety of the core.

(3) It can be observed from the isolines of the maximum principal stress (σ₁) that, at the same elevation, the maximum principal stress of the asphalt-concrete core is smaller than that of the transition-cushion material on both sides, and the settlement of the core is smaller than that in the fillings on both sides of the core, which indicates the occurrence of “arch effect”. On the other hand, the differences of maximum principal stress and settlement between the core and the transition-cushion material on both sides are small, which indicates that “arch effect” is not very obvious, and the transition and cushion layers play an important role. It can also be seen from
the physico-mechanical parameters of the rockfill that meet the engineering requirements. Especially after the deformation moduli of the cushion and transition materials should be compatible with that of the asphalt-concrete.

(5) At the normal pool level, the normal stress of the core in the vertical direction and the hydraulic pressure at corresponding elevations are shown in Fig.9. It can be seen that the hydraulic pressure at any point of the core is less than the normal stress. Therefore, no hydraulic fracturing occurs.

![Fig.9 Normal stress and hydraulic pressure of the core at various elevations.](image)

(6) The isolines of the stress level at different heights and under different conditions show that after impounding, the stress level in the middle and upper core decreases. In other words, the impounding is advantage for the upper part of the core. But in the lower part of the core, the stress level is always relatively higher.

(7) Unloading and reloading cycles will reduce the horizontal displacement and increase the settlement slightly. It implies that the water level fluctuation has a little impact on the displacement of dam body, which may be partly due to the little wetting deformation of the rockfill materials.

(8) Unloading and reloading cycles have an impact on the stress level, especially on the stress level of the upstream dam face at the elevation of 535 m (about 2/3 of the dam height), where the stress level of a few individual elements approaches 1. However, considering the actual adoption of precast concrete block revetment, as long as the precast blocks are mutually closely dogged, the cushion layer and the project quality are ensured, no failure of these units happens.

4 Conclusions

By applying the schemes proposed in this paper to the karst base in Aikou dam project, the stress, displacement and stability of the dam foundation can