Properties and Engineering Influence of Corewall Filter I with Different Fine Particle Contents

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Abstract. The protection provided by filter I for the corewall constitutes an important safety guarantee for the impervious bodies of corewall dams. In particular, the fine particle content (the content of particles < 0.075 mm in size) of filter I provides a filter protection effect for the corewall. Furthermore, this also determines its own drainage characteristics as an important gradation requirement for filter design. In practical engineering, due to the nature of the aggregate source, the crushing and screening process, and rolling construction, the fine particle content may exceed relevant standards. Relying on the field measured gradation envelope of the filter of a gravelly earth core rockfill dam project, this study prepared two classes of filter I (i.e., filter I of class I with a fine particle content of 8%, and filter I of class II with a fine particle content of 12%). Permeability property test, large-scale compression test, large-scale static triaxial test, and large-scale dynamic triaxial test were conducted. Furthermore, dynamic finite-element calculation of the dam under seismic action was performed, and the anti-seismic liquefaction property of the filter was analyzed. According to the permeability property test, the permeability coefficients of the two classes of filter I were both $1 \sim 5 \times 10^{-3}$ cm/s. Lower content of particles $< 0.075$ mm in size increased the permeability coefficient. According to the mechanical test, the stress-strain test curves of both classes of filter I presented nonlinearity, compressive hardening, elastoplasticity, and other general laws. The strength index and deformation resistance both increased with increasing relative density. Under the same relative density, both strength index and deformation resistance declined with increasing fine particle content. According to the dynamic finite-element anti-seismic liquefaction analysis on filter I of class II (with a content of particles $< 0.075$ mm in size of 12%) under seismic action, the liquefaction degree had a maximum value of 0.58 (less than 0.8), and the anti-liquefaction safety coefficient had a minimum value of 1.72, which would not trigger liquefaction of filter I. This experimental study and computational analysis offers references for the exploration of the gradation design of filter I of earth core rockfill dams and investigates the related engineering influence.

1. Introduction
Earth-rockfill dams offer a series of advantages, such as locally available materials, low construction cost, simple structure, and good anti-seismic performance. With the continuous development of studies on the properties of rockfill materials and the continuous advance of construction techniques, this dam type has become one of the mainstream types in high dam construction worldwide[1].
Among already built-up and under-construction large-scale high dams of China, most have adopted the dam type of high earth-rockfill dams, e.g., the Nuozhadu Rockfill Dam, the Lianghekou Rockfill Dam, and the Liangjiangkou Rockfill Dam. In the corewall dam design, to protect the earth materials of the corewall, usually, filters are mounted at both the upper and lower sides of the corewall according to the principle of filter design. They are used to drain filtering soil, and prevent piping, soil flow, and other seepage deformation failures of earth structures at sites of seepage overflows. When the corewall cracks due to hydraulic fracturing or an earthquake disaster, fine particles in the filter can fill up and repair the corewall cracks[2]. In case of an earthquake, if the filter experiences a liquefaction failure under seismic action, the dam shell will be disassociated from the corewall, resulting in corewall failure due to the loss of support.

The influence of the fine particle content on the void pressure of sandy soil can be explained with the microstructure characteristics of sandy soil that contains fine particles [3-4]. Depending on specific particle sizes, 0.075 mm can be introduced as the dividing line to classify fine particles in sandy soil into two groups, i.e., a coarse particle group and a fine particle group. When the fine particle content is lower than a critical threshold, the dynamic characteristics of sandy soil containing fine particles depend on the coarse particle group. With the gradual increase of the fine particle content, the void ratio es of the framework formed by coarse particles gradually increases. In response to a further increase of void ratio es, the number of contact points among soil particles shows a continuous decrease, accompanied by a gradual decrease in the interaction forces among the internal chains in the soil mass. Thus, at the same strain level, deformation resistance also declines with it, resulting in a decline of the elastic modulus. When the fine particle content further increases and exceeds a critical threshold, the large number of fine particles not only fills the voids among coarse particles, but also wraps up these coarse particles, making it impossible for the latter to exert their framework role. In contrast, the void ratio ef among fine particles gradually declines with increasing fine particle content.

For filter I as a protection for the corewall, the fine particle content (content of particles < 0.075 mm in size) not only influences its strength, mechanical characteristics, and permeability, but also affects the filter protection effect on corewall and serves as an important gradation requirement for filter design. Therefore, the design code has specified that the content of particles < 0.075 mm in size in filter I may not exceed 5% [5]. However, in practical engineering, due to the nature of the aggregate source, the crushing and screening process, and rolling construction, the fine particle content may exceed relevant standards. Considering that the field measured fine particle content of filter I in the filling process of a gravelly earth core rockfill dam exceeded this specification, this study prepared two classes of filter I (i.e., filter I of class I with a fine particle content of 8%, and filter I of class II with a fine particle content of 12%), and permeability property test, large-scale compression test, large-scale static triaxial test, and large-scale dynamic triaxial test were conducted. Furthermore, the dynamic finite element method was adopted to perform dynamic characteristic and anti-Seismic liquefaction analysis on filter I under seismic action. Via both experimental study and computational analysis, the properties and engineering influence of corewall filter I were studied with different fine particle contents.

2. Test materials and test equipment

2.1. Test materials

For filter I of class I, the content of particles < 0.075 mm in size was 8%, and the relative density was set to 0.8 and 0.94 for sample preparation. For filter I of class II, the content of particles < 0.075 mm in size was 12%, and relative density was set to 0.8 and 0.85. According to the field measured gradation envelope of a gravelly earth core rockfill dam in the filling process, the equivalent replacement method [6] was used for indoor blending. For the upper, average, and lower envelopes of the two classes of filter I, the maximum particle sizes were uniformly < 60 mm; therefore, the measured gradation was the simulated gradation. Table 1 provides the indoor simulation test densities corresponding to different relative densities of the two classes of filter I.
Figure 1. Gradation curve of filter I

Table 1. Indoor simulation test densities of filter I

| Filter | Gradation properties | Dry density (g/cm³) | Test density (g/cm³) |
|--------|----------------------|---------------------|----------------------|
| I      | Upper envelope       | 1.60                | 1.97                 |
|        | Mean line            | 1.65                | 2.01                 |
|        | Lower envelope       | 1.69                | 2.04                 |
| I      | Upper envelope       | 1.59                | 1.97                 |
|        | Mean line            | 1.64                | 2.00                 |
|        | Lower envelope       | 1.68                | 2.02                 |

2.2. Test equipment
The main technical parameters of the large-scale dynamic/static triaxial compression tester adopted in the test were: maximum axial static load: 1,500 kN (three grades: 300 kN, 800 kN, and 1,500 kN); maximum axial dynamic load: 500 kN; maximum confining pressure: 4.0 MPa; maximum back pressure: 0.5 MPa; maximum axial stroke: 210 mm; dynamic load frequency: 0.01 ~ 5 Hz; dynamic load waveforms: sine wave, triangular wave, and rectangular wave; sample diameters: 200 mm and 300 mm. This instrument is currently the largest and most advanced dynamic/static triaxial compression tester in China.
Figure 2. Large-scale dynamic/static triaxial compression apparatus

3. Static characteristic test

3.1. Triaxial shear test

The samples used for the test were weighed within six particle size ranges of 10-5 mm, 5~2 mm, 2~1 mm, 1~0.25 mm, 0.25~0.075 mm, and 0.075~0 mm, and in each case, after natural drying, they were divided into five equal portions (uniformly blended). The head method was adopted to saturate samples. Confining pressure was applied as required for sample consolidation; after that, a consolidated drained shear test was conducted in drained status, or a consolidated undrained test was conducted in undrained status after closing the upper and lower drain valves. The confining pressures were set to 300 kPa, 800 kPa, 1,200 kPa, and 1,600 kPa, respectively. The large-scale triaxial shear test was conducted on a total of 12 groups of filter I, and the total stress strength index and effective stress strength index under saturated conditions were obtained from test results, as shown in Table 2.

With regard to the saturated samples of upper envelope gradation, Figure 3 and 4 show the test curves of filter I of class I and filter I of class II, respectively.

As indicated by the test results, under the same relative density, the strength index declined with increasing fine particle content. For filter I of the same class, the strength index uniformly increased with increasing relative density. Under the same relative density, the strength index declined with increasing fine particle content. The stress-strain test curves of both classes of filter I presented nonlinearity, compressive hardening, elastoplasticity, and other general laws.

Table 3 lists the parameters of Shen Zhujiang’s double-yield-surface model obtained from the results of the large-scale static triaxial consolidated drained shear test.

| Filter I | Relative density |
|----------|------------------|
| I        | 0.80             |
|          | 0.94             |
| II       | 0.80             |
|          | 0.85             |

Table 2. Strength indexes of the triaxial shear test on filter I

| Filter I | Relative density | Total stress strength indexes | Effective stress strength indexes | Strength indexes |
|----------|------------------|-------------------------------|-----------------------------------|------------------|
| I        | 0.80             | 107.3                         | 33.2                              | 64.8             |
| I        | 0.94             | 149.2                         | 36.1                              | 121.2            |
| II       | 0.80             | 103.6                         | 32.8                              | 63.7             |
| II       | 0.85             | 117.2                         | 33.4                              | 64.9             |

Note: Strength indexes are uniformly upper gradation envelopes.
Figure 3. Large-scale triaxial test curve of filter I of class I

Figure 4. Large-scale triaxial test curve of filter I of class II
Table 3. Parameters of Shen Zhujiang’s double-yield-surface model for the triaxial shear test on filter I

| Filter properties | Gradation | \( \phi_0 \) (°) | \( \Delta \phi \) (°) | \( K \) | \( n \) | \( R_f \) | \( c_d \) (%) | \( n_d \) | \( R_d \) |
|------------------|-----------|-----------------|-----------------|------|-----|-------|------------|-----|-------|
| Upper envelope   | 197       | 46.9            | 6.3             | 680.2| 0.38| 0.70  | /          | /   | /     |
|                  | Saturated | 44.4            | 4.9             | 532.9| 0.41| 0.71  | 0.71       | 0.45| 0.73  |
|                  | Drying    | 50.5            | 7.9             | 928.2| 0.32| 0.69  | /          | /   | /     |
|                  | Saturated | 49.0            | 7.4             | 783.6| 0.33| 0.70  | 0.70       | 0.25| 0.78  |
|                  | Drying    | 45.7            | 5.6             | 635.3| 0.40| 0.74  | /          | /   | /     |
|                  | Saturated | 43.4            | 4.5             | 468.7| 0.44| 0.71  | 0.40       | 0.83| 0.70  |
| Lower envelope   | 1.97      | 45.7            | 5.6             | 635.3| 0.40| 0.74  | /          | /   | /     |
|                  | Drying    | 47.0            | 6.4             | 714.8| 0.39| 0.72  | /          | /   | /     |
|                  | Saturated | 45.2            | 5.5             | 535.6| 0.42| 0.68  | 0.27       | 0.94| 0.66  |

3.2. Triaxial compression test

A large-scale compression test was conducted on four groups of samples of filter I of class I and filter I of class II. These samples had the same relative density of 0.80, and were prepared according to the same method that was adopted for the large-scale triaxial test. The large-scale compression test used six successively-applied vertical load grades, i.e., 100, 200, 400, 800, 1,600, and 3,200 kPa. See the compression parameters under saturated conditions in Table 4.

According to test results, under the same relative density, the compression resistance declined with increasing fine particle content. For filter I of the same class, compression resistance uniformly increased with increasing relative density. Under the same relative density, the compression resistance declined with increasing fine particle content.

Table 4. Parameters of the compression test on filter I

| Vertical stress (kPa) | 0.0  | 100 | 200 | 400 | 800 | 1600 | 3200 |
|----------------------|------|-----|-----|-----|-----|------|------|
| \( e \)              | 0.371| 0.367| 0.364| 0.359| 0.351| 0.338| 0.316|
| \( \alpha_c \) (MPa×10^{-3}) | I    | 3.492| 3.014| 2.528| 2.012| 1.654| 1.327|
|                      | II   | 4.035| 3.345| 2.830| 2.286| 1.796| 1.407|
| \( E_v \) (MPa×10^{2}) | I    | 0.392| 0.455| 0.542| 0.681| 0.829| 1.033|
|                      | II   | 0.340| 0.410| 0.484| 0.599| 0.763| 0.974|
| \( m_r \) (MPa^{3/2})  | I    | 2.548| 2.199| 1.844| 1.468| 1.207| 0.968|
|                      | II   | 2.944| 2.441| 2.065| 1.668| 1.310| 1.026|

3.3. Indoor permeability test

The indoor permeability coefficient followed the hydrostatic head method [7], and the seepage direction was from bottom up (sample size: \( \Phi \) 300 × 300 mm; seepage path: 300 mm; test water temperature: 20 °C). See the test results of permeability coefficient and permeability gradient presented in Table 5.

The test indicated that the permeability coefficients of both classes of filter I were \( 1 \sim 5 \times 10^{-3} \) cm/s, which indicates that the critical and failure gradients of filter I of class II both exceeded 10, and that the forms of permeability failure were uniformly soil flow. For filter I of the same class, the permeability coefficient decreased with increasing relative density, while critical and failure gradients both increased with increasing relative density. Under the same relative density, both the permeability coefficient and critical and failure gradients decreased with increasing fine particle content.
Table 5. Results of permeability test

| Filter | Relative density | Gradation   | permeability coefficients ($10^{-3}$cm/s) | critical gradients | failure gradients |
|--------|------------------|-------------|------------------------------------------|--------------------|------------------|
| I      | 0.80             | Upper envelope | 2.34 | 1.76 | 1.89 |
|        |                  | Lower envelope | 3.15 | /    | /    |
|        | 0.94             | Upper envelope | 1.32 | 1.80 | 2.02 |
|        |                  | Lower envelope | 1.81 | /    | /    |
| II     | 0.80             | Upper envelope | 1.35 | 1.70 | 1.81 |
|        |                  | Lower envelope | 2.31 | /    | /    |
|        | 0.85             | Upper envelope | 1.08 | 1.75 | 1.88 |
|        |                  | Lower envelope | 1.82 | /    | /    |

4. Dynamic Characteristic Test

4.1. Test contents and conditions

Filter I of class II whose content of particles < 0.075 mm in size was 12% (upper envelope gradation, with a relative density of 0.8) was selected as the test material to test dynamic elastic modulus, damping ratio, dynamic residual deformation, and dynamic strength under various grades of confining pressure and consolidation stress ratio. In the test, confining pressure was applied by four grades, i.e., 300 kPa, 800 kPa, 1,200 kPa, and 1,600 kPa. The consolidation stress ratio $K_c$ was set to 1.5 and 2.0. Axial dynamic stress was applied by two grades. When the consolidation stress ratio was 1.5, it was $\pm 0.3\sigma_3$ and $\pm 0.8\sigma_3$, respectively. When the consolidation stress ratio was 2.0, it was $\pm 0.4\sigma_3$ and $\pm 1.0\sigma_3$, respectively. Each grade of the axial dynamic stress was applied by a vibration number of 30, and at a frequency of 0.1 Hz.

4.2. Test results

According to Shen Zhujiang’s dynamic constitutive model[8], the parameters of the dynamic characteristic model are listed in Table 6.

Table 6. Parameters of the dynamic characteristic model

| Filter | $k_c$ | $k_1'$ | $n$ | $k_2$ | $e_1'$ | $k_1$ | $e_2'$/ |
|--------|-------|-------|-----|-------|-------|-------|-------|
| II     | 1.5   | 2229  | 0.545 | 838  | 16.9  | 12.7  | 0.26  |
|        | 2.0   | 2506  | 0.535 | 942  | 19.4  | 14.6  | 0.25  |

The parameters of the dynamic residual deformation model are listed in Table 7. Figure 5 shows the relationship curve between dynamic residual strain and cycling vibration number.

Table 7. Parameters of the dynamic residual deformation model

| Filter | $c_1$ (%) | $c_2$ | $c_4$ (%) | $c_5$ (%) |
|--------|-----------|-------|-----------|-----------|
| II     | 0.98      | 0.90  | 19.8      | 0.71      |

Note: $c_3 = 0$
Figure 5. Relationship curve between dynamic residual strain and cycling vibration number

5. Dynamic Finite-element Analysis

5.1. Calculation model and mesh generation
The gravelly earth core rockfill dam has a height of 131.3 m, a maximum corewall height of 127.8 m, a corewall top width of 4.0 m, and an upper/lower slope ratio of 1:0.25. Filter I of class I and filter I of class II were mounted at both the upper and lower sides of corewall, with the same slope ratio of 1:0.25 (horizontal width of filters at the upper side: 3 m; horizontal width of filters at the lower side: 4 m). Figure 6 shows the three-dimensional (3D) finite-element mesh generation map of the gravelly
earth core rockfill dam. In mesh generation, solid elements are eight-node hexahedral isoparametric elements.

Figure 6. 3D finite-element mesh generation map of a gravelly earth core rockfill dam

5.2. Calculation of parameter seismic input

For this calculation, multi-stage loading was adopted to simulate the filling of dam body and upper/lower-side spoil sites and the impoundment of the reservoir. Dam construction and reservoir impoundment were conducted according to the actual filling order of the dam.

In finite element calculation, static analysis used Shen Zhujie’s double-yield-surface elastoplastic model [9], and the dynamic calculation introduced the equivalent viscoelastic model [10]. See the dynamic calculation parameters of various dam materials in Table 8.

| Materials       | $k_2$ | $k_1$ | $\lambda_{\text{max}}$ | $c_1$(%) | $c_2$ | $c_4$(%) | $c_5$ |
|-----------------|-------|-------|------------------------|----------|-------|----------|-------|
| Core material   | 680   | 15    | 0.23                   | 0.71     | 0.95  | 6.0      | 1.18  |
| Contact clay    | 200   | 6.0   | 0.31                   | 0.83     | 0.95  | 8.9      | 1.15  |
| Filter I        | 942   | 14.6  | 0.25                   | 0.98     | 0.90  | 19.8     | 0.71  |
| Filter II       | 1367  | 28.3  | 0.27                   | 1.15     | 0.94  | 14       | 0.45  |
| Transition material | 1580 | 28.5  | 0.25                   | 1.18     | 0.90  | 20       | 0.47  |
| Rockfill I      | 1285  | 25.0  | 0.25                   | 1.2      | 0.65  | 22       | 0.54  |
| Rockfill II     | 1400  | 24.0  | 0.25                   | 1.1      | 0.70  | 20       | 0.55  |

Table 9 provides the ground motion parameters of the site. Ground motion adopted the artificially-synthesized seismic time-history curve using the site-specific response spectrum as target spectrum, as shown in Figure 7. In dynamic analysis, ground motion adopted three-direction input, where the vertical ground motion level was set to 2/3 of the horizontal ground motion level.

| Probability level | PGA (gal) | Seismic coefficient | Amplification factor ($\beta_{\text{max}}$) | Characteristic period ($T_g$) | Attenuation coefficient ($c$) |
|-------------------|-----------|---------------------|--------------------------------------------|-----------------------------|-------------------------------|
| $P_{100}^{0.01}$   | 332.6     | 0.339               | 2.5                                        | 0.50                        | 0.82                          |

Table 8. Dynamic calculation parameters

Table 9. Ground motion parameters of the dam bedrock site
5.3. Dynamic characteristic analysis on filter I

Under seismic action, the maximum dynamic shear stress of filter I was 0.26 MPa, and the maximum equivalent dynamic shear stress ratio was 0.68. Figure 8 shows the distribution of the maximum dynamic shear stress $\sigma_{\text{d,max}}$ and equivalent dynamic shear stress ratio $\tau_d/\sigma_0'$ of filter I at the upper side of the corewall, respectively.

![Figure 8. Distribution map of dynamic shear stress of filter I at the upper side of the corewall (MPa)](image)

(a) dynamic shear stress $\sigma_{\text{d,max}}$  
(b) dynamic shear stress ratio $\tau_d/\sigma_0'$

5.4. Liquefaction analysis on filter I

The estimate of total stress adopts the method proposed by Seed et al. [11-12]. Based on experience, an equivalent vibration number is related to the earthquake magnitude. Table 10 provides the dynamic strength values of filter I at different vibration numbers in the vibration test. According to the relationship curve between the equivalent vibration number and the earthquake magnitude was provided by Seed et al. (Figure 9). Since the dam site area has a maximum earthquake magnitude of 8, the equivalent vibration number of filter I was set to 20.

| $\sigma_3$(kPa) | $K_c$ | $\tau_d/\sigma_0'$ | 12   | 20   | 30   |
|----------------|-------|-------------------|------|------|------|
| 300            | 0.5137| 0.4691            | 0.4411|
| 800            | 0.4735| 0.4305            | 0.4154|
| 1200           | 0.4412| 0.4231            | 0.4099|
| 1600           | 0.4325| 0.4161            | 0.4043|

Figure 7. Time-history curve of the horizontal designed ground motion acceleration of the dam bedrock site
The relational expressions of anti-liquefaction dynamic shear stress ratio $\sigma_d/\sigma_0'$ with confining pressure $\sigma_0'$ and consolidation stress ratio $K_c$ at different vibration numbers can be obtained through fitting according to Table 10. When the equivalent vibration number $N = 20$, the anti-liquefaction dynamic shear stress ratio can be calculated according to the following equation:

$$\frac{\sigma_d}{\sigma_0'} = 0.5377 \left(\frac{\sigma_0}{\sigma_0'}\right)^{-0.00627} \cdot K_c^{-0.0081}$$

Figure 10 shows the liquefaction degree distribution map of filter I at the upper side of the corewall. Clearly, the liquefaction degree had a maximum value of 0.58 (below 0.8), and the anti-liquefaction safety coefficient had a minimum value of 1.72, which would not trigger the liquefaction of filter I under seismic action.

Figure 9. Relationship between equivalent vibration number and earthquake magnitude

Figure 10. Liquefaction degree distribution map of filter I at the upper side of the corewall
6. Conclusions

(1) Relying on the field measured gradation envelope of filter I of a gravelly earth core rockfill dam project in the filling process, this study prepared two classes of filter I (i.e., filter I of class I with a fine particle content of 8%, and filter I of class II with a fine particle content of 12%). The relative densities of filter I of class I were set to 0.8 and 0.94 for sample preparation, corresponding to test densities of 1.97 and 2.05 g/cm³; those of filter I of class II were set to 0.8 and 0.85.

(2) According to the large-scale triaxial test, the stress-strain test curves of both classes of filter I presented nonlinearity, compressive hardening, elastoplasticity, and other general laws. Under the same relative density, both strength index and deformation resistance declined with increasing fine particle content. For filter I of the same class, both the strength index and deformation resistance increased with increasing relative density. Under the same relative density, both strength index and deformation resistance declined with increasing fine particle content.

(3) According to the permeability test, the permeability coefficients of both classes of filter I were 1-5×10⁻³ cm/s. The critical and failure gradients of filter I of class II both exceeded 10, and the forms of permeability failure were uniformly soil flow. Under the same relative density, both permeability coefficient and critical and failure gradients decreased with increasing fine particle content. For filter I of the same class, the permeability coefficient decreased with increasing relative density, while both critical and failure gradients increased with increasing relative density.

(4) The dynamic triaxial test was conducted on filter I of class II with content of particles < 0.075 mm in size of 12% and with relative density of 0.8, thus obtaining the parameters of the dynamic characteristic model and the dynamic residual deformation model for filter I. Based on this, dynamic finite-element anti-seismic liquefaction analysis was conducted on a gravelly earth core rockfill dam. The results showed that: Under seismic action, the maximum dynamic shear stress of filter I at the upper side of the corewall was 0.26 MPa, and the maximum equivalent dynamic shear stress ratio was 0.68. According to the total stress method, under seismic action, the liquefaction degree had a maximum value of 0.58 (which is below 0.8), and the anti-liquefaction safety coefficient had a minimum value of 1.72, which does not trigger the liquefaction of filter I.

(5) The results of experimental study and computational analysis presented in this study offer relevant references for the exploration of the gradation design of filter I of earth core rockfill dams and the related engineering influence.

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