INFLUENCE OF THE SCALE EFFECT ON THE MECHANICAL PARAMETERS OF COARSE-GRAINED SOILS*

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Abstract– The mechanical parameters of coarse-grained soils are often obtained via indoor or field tests. In these tests, it is necessary to reduce the particle size of the original graded soils due to the size limitation of the testing apparatus. Therefore, several scale methods (e.g., the equivalent substitute method and parallel gradation method) have been proposed to reduce the size of the original graded soils to the proper testing size. However, the mechanical parameters will be different if different scale methods are adopted, a phenomenon that has been termed the “scale effect”. In this research, a group of large-scale oedometer tests were conducted with specimens that were downsized using different scale methods. The results show that (1) when adopting the same scale methods, the compression modulus increases with the increase in the nominal maximum particle size. (2) For the same nominal maximum particle size, samples that are downsized using the equivalent substitute method have a higher compression modulus and degree of particle breakage than those adopting the parallel gradation method. (3) Then Duncan-Chang $E-B$ model parameters were back calculated from oedometer tests using an immune genetic algorithm (IGA). These mechanical parameters were also used in a three-dimensional (3D) finite element method analysis of the Pankou Rock-fill Dam. The manner in which the scale method influences rock-fill dam deformation is also discussed.

Keywords– Oedometer test, scale effect, scale method, nominal maximum particle size, IGA

1. INTRODUCTION

In recent years, coarse-grained soils have enjoyed increasing use in dam construction, offshore structures, airports, and road foundations, among other applications. These soils are large, angular, and granular rock materials blasted from parent rock. The particle size of coarse-grained soils can reach 0.8-1.0 m, and can even exceed 1.2 m in some cases. In conventional tests, the maximum particle size allowed is only limited to 0.06 m due to the size limitation of the testing apparatus [1]. With the development of large-scale testing apparatuses, the allowed maximum particle size can reach 0.2-0.3 m; however, it is still not feasible to test the original graded coarse soils directly. Therefore, several scale methods (e.g., the equivalent substitution method, parallel gradation method, and combination method) have been proposed to reduce the size of the original graded soils to the proper testing size. However, the mechanical parameters will be different if different scale methods are adopted; this phenomenon has been termed the “scale effect”.

In the 1940s, Bishop and Henkel [2] began to address the scale effect in coarse-grained soils; Marachi et al. [3] studied the influence of the scale effect on the strength parameters of coarse-grained soils.
sourced from the Oroville Dam. Li et al. [4] studied the parallel gradation method and deduced a correction factor to extend the test results from downsized soils to the original graded soils.

In this study, the scale effect of coarse-grained soils was investigated by performing large-scale field oedometer tests. The tests were conducted during the construction of the Pankou Concrete-Faced Rock-fill Dam (CFRD). Specimens were prepared using different scale methods, and the test results obtained for these specimens were compared. An immune genetic algorithm (IGA) was adopted to back calculate the Duncan-Chang E-B model parameters from the oedometer tests. These parameters were also used in a three-dimensional (3D) finite element analysis of the Pankou Rock-fill Dam; within this context, the influence of the scale effect on the deformation of a rock-fill dam is discussed.

2. LARGE-SCALE FIELD OEDOMETER TEST

a) Brief introduction to basic scale methods

The grain size characteristics of coarse-grained soils are evaluated by sieve analysis. A nest of sieves is prepared by stacking test sieves on top of one another, with the largest opening at the top followed by sieves of successively smaller openings. A sample of dry soil is poured into the top sieve and shaken until each particle has dropped to a sieve with an opening too small to pass through, and the particles are retained. Then, the mass of soil retained on each sieve can be weighed; the percent retained in each sieve is calculated by dividing this mass by the total specimen weight.

Original graded coarse soils cannot be tested directly due to the limited apparatus size. Thus, several scale methods have been proposed to reduce the particle size of soils. The following are the three most commonly used scale methods [5].

Equivalent substitution method. The percent mass of a specimen retained on sieve $i$ is calculated using the following formula:

$$P_i = \frac{P_{oi}}{P_5 - P_{d_{max}}} \times P_5$$

where $P_i$ is the percent mass of a specimen retained on sieve $i$; $P_{oi}$ is the percent mass of the original graded soil retained on sieve $i$; $P_5$ is the cumulative percent mass of the original graded soil retained on 5-mm sieve; and $P_{d_{max}}$ is the percentage of particles coarser than the maximum permissible particle size in the original graded soil.

This method keeps the percentage of particles finer than 5 mm unchanged, although the shape of the particle-size distribution curve changes.

Parallel gradation method. The percentage of particles finer than $d/n$ in a specimen can be obtained using the following formula:

$$P_{d/n} = P_{do}$$

where $P_{do}$ is the percentage of particles finer than $d$ in the original graded soils and $n$ is the scale coefficient, defined by Eq. (3).

$$n = \frac{d_{maxo}}{d_{maxe}}$$

where $d_{maxo}$ is the maximum particle size of the original graded soils and $d_{maxe}$ is the maximum permissible particle size. Through this method, the particle-size distribution curves of the samples and original graded soils are exactly parallel, and the coefficient of uniformity $C_u$ and coefficient of curvature $C_c$ are constant.

Combination method. This method combines the equivalent substitution method and parallel gradation method. First, the parallel gradation method is used to reduce the percentage of oversized particles to below 40%, and then, the equivalent substitution method is used to reduce to the maximum permissible size.
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b) Physical characteristics of the testing soils

The coarse-grained soils used in the oedometer tests were sourced from the primary rock-fill of the Pankou CFRD. First, the coarse soils were roller compacted in a testing field, as shown in Fig. 1 (a) and (b); then, these coarse-grained soils were sieved into several fractions, as shown in Fig. 1 (c). Preliminary tests were carried out to identify the basic properties of the intact rock. The parent rock of the test rock-fill was silicalite, containing 75% quartz, 20% graphite, 2% muscovite, 2% iron objects, and 1% trace substances. The dry density of the parent rock was 2.60 g/cm³, and its uniaxial compressive strength was 117.0 MPa in the dry state.

Four specimens downsized using different scale methods were tested. The particle-size distribution curve of each sample is plotted in Fig. 2. The properties of each sample are shown in Table 1. The mechanical behaviors of the coarse soils are closely related to their dry densities [6]; in all tests, the initial dry density of each soil was 2.06 g/cm³.

![Testing field](image1) ![Coarse soils after roller compaction](image2) ![Sieving into several fractions](image3)

Fig. 1. Testing soils used in the oedometer tests

![Particle-size distribution curves of original graded soils and specimens](image4)

Fig. 2. Particle-size distribution curves of original graded soils and specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\rho_d$ (g/cm³)</th>
<th>$d_{\text{max}}$ (mm)</th>
<th>$e_0$</th>
<th>$C_u$</th>
<th>$C_c$</th>
<th>Scale method</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen 1</td>
<td>2.06</td>
<td>300.0</td>
<td>0.267</td>
<td>5.94</td>
<td>2.04</td>
<td><strong>Equivalent substitution</strong></td>
<td>$\rho_d$ = dry density</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>2.06</td>
<td>200.0</td>
<td>0.267</td>
<td>5.82</td>
<td>1.31</td>
<td><strong>Equivalent substitution</strong></td>
<td>$d_{\text{max}}$ = maximum particle size</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>2.06</td>
<td>300.0</td>
<td>0.267</td>
<td>6.19</td>
<td>1.01</td>
<td><strong>Parallel gradation</strong></td>
<td>$e_0$ = initial void ratio</td>
</tr>
<tr>
<td>Specimen 4</td>
<td>2.06</td>
<td>100.0</td>
<td>0.267</td>
<td>6.19</td>
<td>1.01</td>
<td><strong>Parallel gradation</strong></td>
<td>$C_u$ = coefficient of uniformity</td>
</tr>
<tr>
<td>Laboratory</td>
<td>2.06</td>
<td>60.0</td>
<td>0.267</td>
<td>5.17</td>
<td>1.20</td>
<td><strong>Combination method</strong></td>
<td>$C_c$ = coefficient of curvature</td>
</tr>
</tbody>
</table>

Table1. Properties of the specimens used in oedometer tests
c) Apparatus and procedures of the oedometer tests

The oedometer tests were conducted in a testing tunnel. The testing tunnel measured 12 m in length with a cross section of 2 m × 2 m (width × height). A cylindrical container measuring 0.65 m in height and 1.33 m in diameter was placed on the floor of the testing tunnel. The surroundings of the container were strengthened with reinforced concrete to prevent lateral deformation of the sample. Four hydraulic jacks were used to exert a vertical force. Displacement meters were installed to record the vertical displacement of the samples (Figs. 3 and 4). Coarse aggregates were compacted into the container so that the samples reached the desired dry density of 2.06 g/cm³.

(a) Filling of the container  (b) Testing specimen
Fig. 3. Oedometer test specimen
(a) Compression of soils  (b) Soils after testing
Fig. 4. Compression of the coarse-grained soils

Compression modulus

Curves of vertical stress versus settlement are shown in Fig. 5. The figure clearly shows that the settlement of specimen 4 is the largest, while that of specimen 1 is the smallest. By comparing specimen 1 with specimen 2 or specimen 3 with specimen 4, it can be seen that when the same scale method is adopted, settlement increases with the decrease in \( d_{\text{max}} \). Even when adopting the other scale methods, the settlement also tends to increase with the decrease in \( d_{\text{max}} \). As shown by the results, \( d_{\text{max}} \) plays an important role in the behavior of the coarse soils. By comparing specimen 3 with specimen 1, it can be seen that the settlement of the equivalent substitution method is smaller than that obtained using the parallel gradation method, but the difference is relatively small when \( d_{\text{max}} \) values of the specimen are both 0.3 m. Specimen #4 experiences relatively greater deformation than the other three groups, which implies that the soil properties are closely related to \( d_{\text{max}} \), particularly when the scale coefficient \( n \) is large.

Fig. 5. Relationship between vertical stress and settlement during the oedometer testing

<table>
<thead>
<tr>
<th>Specimen</th>
<th>0 - 0.7</th>
<th>0.7 - 1.4</th>
<th>1.4 - 2.1</th>
<th>2.1 - 2.8</th>
<th>2.8 - 3.5</th>
<th>Reloading modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen 1</td>
<td>211.2</td>
<td>140.8</td>
<td>126.7</td>
<td>181.0</td>
<td>140.8</td>
<td>833.8</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>140.8</td>
<td>158.4</td>
<td>115.2</td>
<td>115.2</td>
<td>126.7</td>
<td>586.0</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>211.2</td>
<td>140.8</td>
<td>126.7</td>
<td>158.4</td>
<td>140.8</td>
<td>785.4</td>
</tr>
<tr>
<td>Specimen 4</td>
<td>115.2</td>
<td>90.5</td>
<td>79.2</td>
<td>97.5</td>
<td>105.6</td>
<td>609.5</td>
</tr>
</tbody>
</table>
Influence of the scale effect on the...

For soils, the compression modulus $E_d$ determined by the oedometer test is defined as follows:

$$E_d = \frac{(1+e_i)(p_2-p_1)}{e_1-e_2}$$  \hspace{1cm} (4)

where $e_i$ is the initial void ratio corresponding to vertical stress $p_1$ and $e_2$ is the void ratio corresponding to vertical stress $p_2$ after an stress increment $\Delta p = p_2 - p_1$. Therefore, the compression modulus is stress level dependent. The compression modulus and reloading modulus of each sample are listed in Table 2.

As shown, the equivalent substitute method yields a higher compression modulus than the parallel gradation method for equal $d_{\text{max}}$ values. Thus, the equivalent substitute method may compensate for the decrease in particle size. From this perspective, samples downsized by the equivalent substitute method may exhibit mechanical behavior similar to that of the original graded soils. According to Table 2, the reloading modulus is also related to $d_{\text{max}}$; when using the parallel gradation method, the reloading modulus increases by 28.9% when $d_{\text{max}}$ increases from 0.1 m to 0.3 m. By comparing specimen 2 with specimen 3, it can be seen that although $d_{\text{max}}$ of specimen 2 is greater than that of specimen 4, its reloading modulus is smaller; therefore, the scale method may have a greater impact on the reloading modulus than $d_{\text{max}}$.

Particle breakage

Terzaghi [7] suggested that the deformation of rock-fill may be caused by the breakage of rock particles. Particle breakage typically occurs in the vicinity of high contact stress. This breakage could lead to the rearrangement of the granular structure of specimens. Particle breakage is determined by many factors, including particle shape, size, density, and strength of the parent rock. Marsal [8] and Hardin [9] proposed different methods to evaluate the degree of particle breakage. In the present study, Marsal’s method was used, and the particle breakage index $B_g$ was defined as the sum of the positive values of $W_k$ (Eq.5).

$$B_g = \sum \Delta W_k, \quad \Delta W_k = \begin{cases} W_k - W'_{k} & \text{if} \quad W_k - W'_{k} \geq 0 \\ 0 & \text{if} \quad W_k - W'_{k} < 0 \end{cases}$$  \hspace{1cm} (5)

where $W_k$ is the percent retained on sieve $k$ before testing and $W'_{k}$ is the percent retained on sieve $k$ after testing. The particle-size distribution curves are plotted in Fig. 6.
By comparing specimen 3 with specimen 4, we find that $B_g$ is strongly affected by the maximum particle size $d_{\text{max}}$ in the parallel gradation method. $B_g$ increases with the increase in $d_{\text{max}}$. When $d_{\text{max}}$ is 0.1 m, particle breakage is negligible. By comparing specimen 1 with specimen 2, it can be observed that $B_g$ is not strongly affected by $d_{\text{max}}$; $B_g$ increases by only 0.6% when $d_{\text{max}}$ increases from 0.2 m to 0.3 m.

As is known, particle breakage typically occurs in the vicinity of high stress contacts. Thus, the greater the number of contact points, the lower the degree of particle breakage. According to the particle-size distribution curves, the specimen downsized using the parallel gradation method exhibited a greater number of contact points than the specimen downsized using the equivalent substitution method; this phenomenon is more pronounced for small $d_{\text{max}}$ values, which explains why the equivalent substitution method induces a higher degree of particle breakage than the parallel gradation method.

### 3. FINITE ELEMENT ANALYSIS OF THE PANKOU DAM WITH BACK-ANALYZED PARAMETERS

#### a) Back analyzed Duncan-Chang E-B model parameters from oedometer tests

**Duncan-Chang E-B model**

In the Duncan-Chang E-B model [10], tangent modulus $E_t$, tangent bulk modulus $B_t$, reloading modulus $E_{ur}$ and internal friction angle $\phi$ are respectively expressed as follows:

\[
E_t = K_p a \left(\frac{\sigma_3}{p_a}\right)^n \left[1 - \frac{R_f (1 - \sin \phi) (\sigma_1 - \sigma_3)}{2C \cdot \cos \phi + 2\sigma_3 \sin \phi}\right]^2
\]  

\[
B_t = K_b p_a \left(\frac{\sigma_3}{p_a}\right)^m
\]

\[
E_{ur} = k_{ur} p_a \left(\frac{\sigma_3}{p_a}\right)^{\gamma_{ur}}
\]

\[
\phi = \phi_0 - \Delta \phi \log \left(\frac{\sigma_1}{p_a}\right)
\]

where $K$, $n$, $R_f$, $K_b$, $m$, $C$, $\phi$, $k_{ur}$, and $n_{ur}$ are material parameters; $\sigma_1$ and $\sigma_3$ are the first and third principal stresses, respectively, $p_a$ is the atmospheric pressure; and $\phi_0$ and $\Delta \phi$ are strength parameters.

**Brief introduction to IGA**

The IGA is a stochastic optimization method [11-12] that is suitable for a variety of nonlinear problems [13-14]. In the present study, the IGA method was used together with the finite element method to back calculate the E-B model parameters from the oedometer tests [15].

The following was selected as the objective function:

\[
F(X) = \sum_{i=1}^{m} \sum_{j=1}^{n} \omega_j (u_{ij}' / u_{ij}^m - 1.0)^2
\]

The objective function was used to measure the error between calculated displacement and displacement measured by the oedometer test. The IGA was used to search for a group of parameters to make the corresponding calculated and measured settlements best approximate to one another. In Eq. (7), $m$ is the number of load steps; $n$ is the number of monitoring points; $u_{ij}'$ is the calculated displacement of monitoring point $j$ at the $i$th load step; $u_{ij}^m$ is the measured displacement of monitoring point $j$ at the $i$th load
step; $\omega_j$ is the weight of monitoring point $j$ at the $i$th load step. The parameters were back analyzed in two steps. First, five loading parameters were back analyzed. Second, two reloading parameters were back analyzed. In each step, these unknown variables could be written as a row vector. The strength of the coarse soils was assumed to have not been affected by the scale method used, based on the results of Marachi et al [3].

$$\{X\} = \{K, n, R_f, K_s, m\} \quad \text{Variables in the first step}$$

$$\{X\} = \{k_{ur}, n_{ur}\} \quad \text{Variables in the second step}$$

(8)

**Back analyzed results and discussions**

The change in the parameters of specimen #1 during the optimization process is shown in Fig. 7. The measured and back-analyzed curves of specimen #1 are plotted in Fig. 8, the results show that the calculated and measured curves are rather consistent. Table 3 presents the $E-B$ model parameters that were back analyzed from the four specimens.

**Fig. 7. Change in parameters during optimization (specimen 1)**

**Fig. 8. Curves of the measured and back-analyzed displacement (specimen 1)**

**Table 3. Duncan-Chang E-B model parameters back analyzed from oedometer tests results**

<table>
<thead>
<tr>
<th></th>
<th>$\varphi_0(\degree)$</th>
<th>$\Delta \varphi(\degree)$</th>
<th>$R_f$</th>
<th>$K$</th>
<th>$n$</th>
<th>$K_s$</th>
<th>$m$</th>
<th>$k_{ur}$</th>
<th>$n_{ur}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen 1</td>
<td>51.0</td>
<td>8.1</td>
<td>0.830</td>
<td>1141</td>
<td>0.35</td>
<td>550</td>
<td>0.23</td>
<td>2736</td>
<td>0.69</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>51.0</td>
<td>8.1</td>
<td>0.770</td>
<td>951</td>
<td>0.25</td>
<td>419</td>
<td>0.13</td>
<td>2353</td>
<td>0.53</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>51.0</td>
<td>8.1</td>
<td>0.790</td>
<td>1079</td>
<td>0.39</td>
<td>486</td>
<td>0.13</td>
<td>2369</td>
<td>0.69</td>
</tr>
<tr>
<td>Specimen 4</td>
<td>51.0</td>
<td>8.1</td>
<td>0.760</td>
<td>902</td>
<td>0.33</td>
<td>356</td>
<td>0.17</td>
<td>2964</td>
<td>0.69</td>
</tr>
<tr>
<td>Indoor test</td>
<td>51.0</td>
<td>8.1</td>
<td>0.820</td>
<td>930</td>
<td>0.26</td>
<td>442</td>
<td>0.11</td>
<td>/</td>
<td>/</td>
</tr>
</tbody>
</table>
b) Three-dimensional finite element analysis of the Pankou CFRD

Brief introduction to Pankou CFRD

The Pankou Dam, located in Hubei Province in China, stands 109 m tall. The typical zoning of the Pankou CFRD is shown in Fig. 9. Finite element meshes of the dam are shown in Fig. 10. In this study, Duncan-Chang E-B model parameters of primary rock-fill were obtained from oedometer tests; the parameters of other materials are listed in Table 4.

<table>
<thead>
<tr>
<th>Bedding material</th>
<th>51.0</th>
<th>8.1</th>
<th>0.830</th>
<th>1141</th>
<th>0.35</th>
<th>550</th>
<th>0.22</th>
<th>2736</th>
<th>0.69</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transition material</td>
<td>51.0</td>
<td>8.1</td>
<td>0.770</td>
<td>951</td>
<td>0.25</td>
<td>419</td>
<td>0.13</td>
<td>2353</td>
<td>0.53</td>
</tr>
<tr>
<td>Secondary rock-fill</td>
<td>51.0</td>
<td>8.1</td>
<td>0.790</td>
<td>1079</td>
<td>0.39</td>
<td>486</td>
<td>0.13</td>
<td>2369</td>
<td>0.69</td>
</tr>
</tbody>
</table>
| Primary rock-fill parameters were back analyzed from the oedometer test results in Table 3

Displacement of Pankou dam using parameters back analyzed from oedometer test and indoor test

The displacement vectors and contours of the Pankou CFRD are shown in Figs. 11 and 12, respectively (primary rock-fill parameters obtained from specimen 1). Figs. 12a and b show the vertical and horizontal displacements upon dam completion, respectively. Figs. 12c, d and e show the vertical displacement, horizontal displacement, and deflection of the concrete slab during the full storage period, respectively. The displacement extrema of each group are listed in Table 5.

| a) At completion | b) During the full storage period
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Vertical displacement contours at completion</td>
<td>(b) Horizontal displacement contours at completion</td>
</tr>
<tr>
<td>(c) Vertical displacement contours during the storage period</td>
<td>(d) Horizontal displacement contours during the storage period</td>
</tr>
<tr>
<td>(d) Deflection contours of Concrete Face Slab during the storage period</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 11. Displacement vectors of the Pankou CFRD with the primary rock-fill parameters obtained from specimen 1

Fig. 12. Displacement contours of the Pankou CFRD with the primary rock-fill parameters obtained from specimen 1 (unit: cm)
Table 5. Displacement extrema of the Pankou CFRD with different groups of parameters

<table>
<thead>
<tr>
<th>Completion period</th>
<th>Storage period</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>V</td>
<td>LU</td>
<td>LD</td>
</tr>
<tr>
<td>Specimen 1</td>
<td>-48.6</td>
<td>15.8</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>-62.5</td>
<td>18.7</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>-53.5</td>
<td>14.7</td>
</tr>
<tr>
<td>Specimen 4</td>
<td>-69.4</td>
<td>18.5</td>
</tr>
<tr>
<td>Laboratory</td>
<td>-63.0</td>
<td>19.8</td>
</tr>
</tbody>
</table>

V = vertical displacement  
LU = upstream lateral displacement  
LD = downstream lateral displacement  
D = deflection of the concrete slab  
(unit: cm)

Discussion

According to Table 5, we conclude that (1) The fourth group of parameters shows the largest vertical displacement, while the first group shows the smallest, which is consistent with the fact that specimen 4 exhibits the greatest compression, while specimen 1 exhibits the least; (2) The vertical displacement of the second group of parameters is greater than that of the first group. This finding implies that higher \( d_{\text{max}} \) values induce less settlement. The same tendency can be observed by comparing the results of the third and fourth groups of parameters; (3) The first group of parameters shows less settlement than the third group, which implies that the equivalent substitution method leads to lower compression than the parallel gradation method in the rock-fill dam; (4) By comparing the field parameters with indoor parameters, it can be seen that the dam settlement associated with the field parameters is generally lower than that associated with the indoor parameters. However, the settlement indicated by the fourth group of parameters is even lower than that indicated by the indoor parameters. This finding suggests that when \( d_{\text{max}} \) is small, the parallel gradation method may overestimate dam settlement; (5) The lateral displacement of the indoor test parameters is the largest.

4. CONCLUSION

In this study, original graded coarse-grained soils were downsized using different scale methods. A series of large-scale oedometer tests were conducted to study the influence of the scale effect on coarse-soil behavior.

The oedometer test results show that (1) For the same scale method, the compression modulus increases with \( d_{\text{max}} \); (2) When the maximum particle sizes are equal, the compression modulus generated by the equivalent substitute method is larger than that generated by the parallel gradation method; (3) The particle breakage induced by the equivalent substitute method is greater than that induced by the parallel gradation method; (4) The reloading modulus is not directly related to the particle size or scale method used, which may be due to the effects of dry density and breakage; (5) The Duncan-Chang E-B model parameters were back analyzed from the results of the oedometer tests, which were also adopted in the 3D finite element analysis of the Pankou CFRD. The displacements calculated by these different groups of model parameters were compared. The results show that the vertical displacements calculated using the oedometer test parameters tend to be smaller than those calculated using the indoor test parameters. However, for small \( d_{\text{max}} \), the parallel gradation method may lead to rather large deformations.

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REFERENCES


