Experimental Study on Constitutive Property of Cementitious Rockfill Material

Xianfeng He, Li Zhou, Na Li

ABSTRACT

With a new round of hydropower development in China, a large number of water conservancy and hydropower projects have been started, and many of them are rockfill dam. Rockfill dam and cofferdam used granular rockfill roller compaction construction, which has low cost, convenient construction, however, with the dam height increase, dam body (cofferdam) section will constantly increase, not only human, material and financial will be spent large, but also construction time will be extend, because of small friction angle and lower or none cohesion of rockfill. If we can improve the cohesion and friction angle of the rockfill, it will greatly reduce the dam section size and project cost. Although the roller compaction construction has the characteristics of convenient construction and high material strength, but as a kind of concrete materials with low water cement ratio, the colloid material proportion is still relatively large, high cost. Obviously, added only trace amounts of colloidal material without the change of construction process, that can improve the mechanical indexes of rockfill is the ideal choice. However, there is a lack of research results in effects of the rockfill filling trace amounts of colloidal material. In this paper, Study on the mechanical properties of the rockfill filling trace amounts of colloidal material has been done through large-scale triaxial shear test, to obtained forced deformation laws and constitutive model of the new material.

INTRODUCTION

Cementitious rockfill dam has a wide application prospect, as it is convenient in construction, saving in materials, easy in construction diversion, fits in soft foundation and protects the environment. But, this new material which only started its limited applications in 1990s\textsuperscript{[1-5]}, has not yet developed a complete fundamental
theory and designed techniques when compares with widely-applied roller compacted concrete dam and concrete faced rockfill dam. Among all the design techniques, the mechanical property of cementitious rockfill materials is one of the essential. As there is no special requirements to the source materials, there are therefore large difference in content of fine particles and moisture content, both of which usually affect largely the mechanical property and lead to the complexity. In this context, this paper describes the experimental study in mechanical property of the new materials and the related constitutive models.

STUDY PROGRAMS

Experiment

Through large-scale consolidated drained tri-axial shear test under multi-confining pressures, the stress property of the materials is observed which is the basic data for constitutive relations. The confining pressure applied is 400kPa, 600kPa, 800kPa respectively, and the consolidation ratio is 1.0. The dry density of the soil sample is 2.05 g/cm³, with the dimensions Φ300mm×700mm.

Experiment Materials

1) Cementitious materials include cement, coal ash and water. The cement is ordinary Portland cement P.O42.5 produced by China Tianrui Group Cement Company Limited, and its technical specialty shown in Table 1. The coal ash used is class I produced by Zhengzhou Jin Longyuan Coal Ash Co. Ltd, with the technical specialty in Table 2. The water used is from tap.

<table>
<thead>
<tr>
<th>fineness</th>
<th>Water requirement for standard consistent (%)</th>
<th>Setting time(min)</th>
<th>Strength/MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific surface area (m²/kg)</td>
<td>Initial setting time</td>
<td>Final setting time</td>
<td>Stability</td>
</tr>
<tr>
<td>369</td>
<td>27.4</td>
<td>170</td>
<td>210</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>qualification</th>
<th>fineness</th>
<th>Water content ratio(%)</th>
<th>Moisture content (%)</th>
<th>Loss of ignition (%)</th>
<th>SO3(%)</th>
<th>Free calcium oxide(%)</th>
<th>Total Alkali (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>The measured values</td>
<td>11.3</td>
<td>80</td>
<td>0.2</td>
<td>4.29</td>
<td>1.75</td>
<td>0.41</td>
<td>1.21</td>
</tr>
<tr>
<td>Class I</td>
<td>≤12.0</td>
<td>≤95</td>
<td>≤1.0</td>
<td>≤5.0</td>
<td>≤3.0</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

2) Construction aggregate

The aggregate in the experiment is the filling materials used in a cofferdam. The aggregate stuff in the field was firstly screened and the upper and lower envelope curves were determined. The sample was made by grading with the lower envelope curve. As there are particles in the raw materials which are beyond the grain size limitation of the equipment, this kind of particles were replaced by a ratio
while keeps the mass same\cite{7}. The grading of the particle size after replacement is shown in Table 3.

<table>
<thead>
<tr>
<th>grading</th>
<th>Particle composition(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>60~40 (mm)</td>
</tr>
<tr>
<td>Lower limitation</td>
<td>8.75</td>
</tr>
</tbody>
</table>

**Experiment Equipment**

The experiment employed 1000kN electric hydraulic servo to conduct coarse-grained soil static and dynamic tri-axial test. The machine has following technical specialty (Table 4).

<table>
<thead>
<tr>
<th>parameters</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical dimension</td>
<td>$30\times75$cm(diameter\times hight)</td>
</tr>
<tr>
<td>Consolidation axial load</td>
<td>0~500kN</td>
</tr>
<tr>
<td>Piston stroke</td>
<td>0~250mm</td>
</tr>
<tr>
<td>dead axial load</td>
<td>0~1000kN</td>
</tr>
<tr>
<td>Confining pressure</td>
<td>0~2000kPa</td>
</tr>
<tr>
<td>Control mode</td>
<td>auto</td>
</tr>
<tr>
<td>live axial load</td>
<td>0~±300kN</td>
</tr>
<tr>
<td>Vibration frequency</td>
<td>0.01~5Hz</td>
</tr>
<tr>
<td>Sampling mode</td>
<td>auto</td>
</tr>
</tbody>
</table>

**EXPERIMENT PROCEDURES AND RESULTS**

**Sample and Experiment**

The materials was firstly weighted to meet the design requirements and mixed well, the mixture was then filled and compacted into the sample preparation tube layer by layer (5 layers totally). Within the 60-day molding maintaining, the sample preparation procedure employed movable base connected to the preparation tube. Specifically, a 0.2mm-thickness thin iron sheet was put inside the molding tube. When the sample was compacted and molded, the tube was taken away while leave the iron sheet with the molded sample. The iron sheet was usually fixed with the rubber band and sealed with plastic film when the sample was maintained in the normal temperature. After 60-day maintaining, the sample was moved to the testbed, removed from the iron sheet and plastic film, loaded to the rubber membrane, ready for consolidated drained (CD) shear test.

According to related study\cite{7}, the test was conducted by several steps. The test needs vacuum saturation firstly. To be specific, air was extracted through the pipe connected to the permeable plate on the top of the sample, which results in a negative pressure in the sample body leading to water passing through the body from the bottom to the top. This saturation procedure lasted 20 minutes and stop. Secondly, confining pressure was then imposed to the saturated sample and consolidation started by ratio. After that, drained tri-axial shearing started at the loading speed of 0.02mm/s, during which real-time data about axial pressure, axial displacement, drainage and pore pressure were recorded. Peak intensity was set to be breaking standard and the test ended when axial strain reached 20%.
Output

Stress-strain

Stress-strain curve is shown in Figure 1. It can be seen that stress-strain in cementitious rockfill materials consists of 3 stages, namely approximate straight line, upward curve, downward curve. For the stage of approximate straight line, when the axial stress is relatively low (lower than 65%~80%), the curve is almost straight line, similar to that of elastic material, and the slope of the line grows when confining pressure higher. The terminal point in this stage was marked as elastic limit. For the stage of upward curve, as the plastic deformation rate grew, the stress-strain graph changed gradually from approximate straight to curve and deflected to axial. When the material approached straight limit, plastic deformation grew rapidly and so did the curve transition. Shear failure occurred inside the sample when the stress reached the break point. For the stage of downward curve, plastic deformation was swift after axial stress reached the strength limit, and stress decreased rapidly as strain grew. The strain grew continuously when stress reached the residual strength, which agree to the outputs of other research [8-10].

When the test finished, the physical form after deformation usually showed arch in the middle. The arch could be in the upper part when the confining pressure was relative low, and moved down when the confining pressure grew.

Figure 1. Tri-axial stress-strain curve (60d).

Figure 2. Axial volume-strain change curve (60d).
Volume change

It was observed that the volume of the sample was changing as the axial stain varied. The volume change was plotted along the strain grew under different confining pressure, shown in Figure 2. Combining the stress-strain curve and volume-strain curve, we found the sample was in the state of contraction when the test started and reached the extreme when the strain was strength limit. After that, as shear failure occurred in the sample, stress gradually decreased and plastic deformation grew rapidly, resulting in the contraction died away and finally dilation developed. It can be seen from Figure 2 that the dilation point move backward along the axis of the strain.

The phenomenon above lies in the colloidal content is relative low in the cementitious rockfill materials and cementation among particles is relative weak, leading to shear deformation and volume deformation when there is load imposing. Weather the sample dilates or contracts, it depends on stress imposing on the material.

CONSTITUTIVE MODEL

As the curves show in Figure 1 and Figure 2, cementatious rockfill material has a feature of non-linear. This study tried to use bi-modulus $K-G$ model to describe the consititutive relation. bi-modulus $K-G$ model couldn’t show dilation or contraction of the material, but the strain used to determine the model parameters consists of dilation and its volume change caused by normal stress increment. The constitutive model in this paper involves two parts. The curve before peak strength uses $K-G$ model considering volume change, and softening part after the peak strength uses data mining model.

$$K-G \text{ (model1) Tangent Bulk Modulus } K_i$$

In isotropic consolidation test, according to Hook’s law, there is:

$$K_i = \frac{\Delta p}{\Delta \varepsilon_v}.$$ (1)

This equation reflects that the tangent slope of $p \sim \varepsilon_v$ curve can express the physical meaning of $K_i$. Therefore, based on the relation between void ratio $e$ and pressure $p$ ($e \sim p$ curve), $dp/d\varepsilon_v = K_i$ can be deduced. With the experiment data, there is the following $e \sim p$ equation

$$e = e_0 - \lambda \ln p.$$ (2)

And tangent bulk modulus $K_i$ is furthermore deduced as

$$K_i = \frac{dp}{d\varepsilon_v} = \frac{1 + e_0}{\lambda} \ p.$$ (3)

Where $e_0$ is initial void ratio, $\lambda$ is the constant. In this test, $e_0 = 5.896$, $\lambda = 1.258$

2) Tangent shear modulus $G_i$
Domaschuk use large-scale tri-axial test where is $p$ constant to determine $G_t$. This experiment didn’t use that and their outputs because of the different test conditions. As the curve $(q/3) \sim \varepsilon$ (Figure 3) of cementitious rockfill material in this test is observed to be approximate to hyperbolic curve, Duncan hyperbolic therefore is used to describe $(q/3) \sim \varepsilon$ relation.

![Figure 3. $(q/3) \sim \varepsilon$ curve of cementitious rockfill material (60d).](image)

With Hooke’s law, further analysis found

$$G_t = \frac{\Delta \frac{q}{3}}{\Delta \varepsilon} \frac{\partial (\frac{q}{3})}{\partial \varepsilon}.$$  \hspace{1cm} (4)

And for a certain shear force $q/3$, $(q/3) \sim \varepsilon$ relation can be expressed as following equation

$$\frac{q}{3} = \frac{\varepsilon}{a + b\varepsilon} \text{ or } \frac{\varepsilon}{(q/3)} = a + b\varepsilon.$$ \hspace{1cm} (5)

Where $a, b$ are constant.

If set $\varepsilon/(q/3)$ as vertical axial and $\varepsilon$ as the horizontal, hyperbolic curve can be change to a line. Similar to the way that determine parameter $a, b$ in $E-B$ model[12], $a, b$ were obtained here by drawing a straight line between the point that stress reaches 70% and the point reaches 80%. $a$ is the intercept when the straight line cross over the vertical axial and $b$ is the slope of the line. With $a$ and $b$, the initial tangent shear modulus $G_t$ is determined and approximate value to stress difference $(q/3)_{ult}$ is as well.

Based on equation (5), $\varepsilon$ can be determined as

$$\varepsilon = \frac{a}{\frac{q}{3} - b}.$$ \hspace{1cm} (6)

With the combination of equation (4) and (5), there is

$$G_t = \frac{a}{(a + b\varepsilon)^2}.$$ \hspace{1cm} (7)

When combining equation (6) and (7), there is

$$G_t = \frac{1}{a} [1 - b(\frac{q}{3})]^2.$$ \hspace{1cm} (8)

When $\varepsilon \rightarrow \infty$, 

497
\[ b = \frac{1}{(q/3)_{\text{ult}}} = \frac{1}{(q/3)_{\text{f}}}. \]  

(9)

Break ratio \( R_f \) is defined as:

\[ R_f = \frac{\left(\frac{q}{3}\right)_f}{\left(\frac{q}{3}\right)_{\text{ult}}}. \]  

(10)

Where \( \left(\frac{q}{3}\right)_f \) is the stress difference when sample breaks up.

There is

\[ b = \frac{1}{(q/3)_{\text{ult}}} = R_f \frac{1}{\left(\frac{q}{3}\right)_f}. \]  

(11)

And

\[ a = \frac{1}{G_i}. \]  

(12)

When input equation (11) and (12) into equation (8), tangent shear modulus can be deduced as:

\[ G_i = G_i [1 - R_f \frac{\left(\frac{q}{3}\right)_f}{\left(\frac{q}{3}\right)_{\text{ult}}}]^2. \]  

(13)

\[ \text{① } G_i \text{ and } \sigma_3 \]

When stress reaches its 70% and 80%, there are corresponding \( \bar{\varepsilon} \) and \( \bar{\varepsilon} \) respectively. If a straight line is drew between the two points, the parameters \( a, \ b, \ G_i \) and \( (q/3)_{\text{ult}} \) under confining pressure scenarios can be determined, the value in this test were shown in table 5

<table>
<thead>
<tr>
<th>Confining pressure</th>
<th>( a )</th>
<th>( b )</th>
<th>( G_i ) (kPa)</th>
<th>( (q/3)_{\text{ult}} ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>0.0003147</td>
<td>0.0011658</td>
<td>3177.63</td>
<td>857.78</td>
</tr>
<tr>
<td>600</td>
<td>0.0001886</td>
<td>0.0008019</td>
<td>5302.23</td>
<td>1247.04</td>
</tr>
<tr>
<td>800</td>
<td>0.0001220</td>
<td>0.0006298</td>
<td>8196.72</td>
<td>1587.81</td>
</tr>
</tbody>
</table>

Table 5. PARAMETERS UNDER DIFFERENT CONFINING PRESSURE AT 60d.

Treat \( G_i \) and \( \sigma_3 \) with dimensionless method, \( (G_i/P_a) \sim (\sigma_3/P_a) \) curve is determined as Figure 4, which is further regressed to get the following \( (G_i/P_a) \sim (\sigma_3/P_a) \) relation

\[ \frac{G_i}{P_a} = k_i e^{n(\frac{\sigma_3}{P_a})}. \]  

(14)
In this way, the initial tangent shear modulus $G_t$ is expressed as

$$G_t = k_1 P_a e^{n \frac{q}{P_a}}. \quad (15)$$

Where $k_1, n(k_1 > 0, n > 0)$ is constant. This output agrees to other researches\[7, 10\].

2. $(q_f / P_a)$ and $\sigma_3$

According to the breaking strength of the sample under different confining pressure, the relation curve between $(q_f / P_a) \times \left(\frac{\sigma_3}{P_a}\right)$ and $(\frac{\sigma_3}{P_a})$ (dimensionless) can be plotted as in Figure 5. Further analysis found following equation

$$\left(\frac{q_f}{P_a}\right) \times \left(\frac{\sigma_3}{P_a}\right) = k_2 \left(\frac{\sigma_3}{P_a}\right)^m. \quad (16)$$

Which can be rewritten as

$$\frac{q_f}{P_a} = k_2 P_a \left(\frac{\sigma_3}{P_a}\right)^{m-1}. \quad (17)$$

Where $k_2, m(k_2 > 0, m > 0)$ are all constant. This equation, combining with Figure 5, shows breaking strength $(q_f / P_a)$ grows as the confining pressure increases.

3. $R_f$

Based on the experiment data, break strength $(q_f / P_a)$ and its asymptotic value $(q_{ult} / P_a)$ can be determined. Using equation (10), the corresponding break ratio $R_f$ is further determined.

If input (15) and (17) equations into equation (13), there is

$$G_t = k_1 P_a e^{n \frac{q}{P_a}} [1 - R_f \left(\frac{q_f}{P_a}\right) \left(\frac{\sigma_3}{P_a}\right)^{m-1}]^2. \quad (18)$$

The equation above shows $G_t$ reduces as stress increases and grows as confining pressure increase. This trend of equation (18) agrees to that in Duncan model but more succinct than the latter as it only has 4 dimensionless parameters $k_1, k_2, m, n$. The values of the parameters, based on this test, are shown in Table 6.
Constitutive Model after the Peak Strength

Firstly, regress method was applied to $q / 3 - \varepsilon$ curve. Optimization was then conducted, and the model was rewritten as

$$\left(\frac{q}{3}\right) = Ae^{-\varepsilon/t_1} + k_0.$$  \hfill (19)

Where $A$, $t_1$, $k_0$ are regression constant, shown in table 7

<table>
<thead>
<tr>
<th>$k_1$</th>
<th>$k_2$</th>
<th>$m$</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.47</td>
<td>2.964</td>
<td>1.745</td>
<td>0.236</td>
</tr>
</tbody>
</table>

As shown in table 7, there is internal relations between confining pressure and the parameters $A$, $t_1$, $k_0$. In order to study further these relations, the curves between $\sigma_3$ and parameter $A$, $t_1$, $k_0$ were plotted respectively (Figure 7-figure 9). Among these curves, $\sigma_3$, after treated with dimensionless method, was set as the horizontal axial. $A$ was firstly transformed and $A(\sigma_3^2 P_a)$ was then set as the vertical axial. After regression analysis to the data above, the relation curve between confining pressure $\sigma_3$ and $A$, $t_1$, $k_0$ were drawn out in Figure 7-figure 9.)
\[ A(\sigma_3) = \frac{A_1 \ln(\frac{\sigma_3}{P_a}) + A_2}{(\frac{\sigma_3}{P_a})^2}. \]  

(20)

\[ t_1(\sigma_3) = t_2(\frac{\sigma_3}{P_a}) + t_3. \]  

(21)

\[ k_0(\sigma_3) = k_0 e^{\frac{a(\sigma_3)}{P_a}}. \]  

(22)

Where \( A_1, A_2, t_2, t_3, k_{01}, a(\text{if} A_2>0, \text{and} t_2>0, t_3>0, k_{01}>0, a>0) \) are regression coefficients.

If the equation(20)~equation(22) were put into equation(19), the numerical relation between shear strength \( q/3 \) and shear strain \( \varepsilon \) after the peak strength is as followed:

\[ q = \frac{3}{P_a} \left[ \frac{A_1 \ln(\frac{\sigma_3}{P_a}) + A_2}{(\frac{\sigma_3}{P_a})^2} e^{-\frac{\varepsilon}{t_2(\frac{\sigma_3}{P_a})+t_3} + k_{01} e^{a(\frac{\sigma_3}{P_a})}} \right]. \]  

(23)

With the similar way, the other six dimensionless parameters here \( A_1, A_2, t_2, t_3, k_{01}, a \) can also be determined by using output of the cementitious rockfill material tri-axial experiment. The values of these parameters in this test are shown in table 8.

Table 8. PARAMETERS FOR THE STAGE AFTER THE PEAK STRENGTH AT 60d.

<table>
<thead>
<tr>
<th>parameters</th>
<th>( A_1 )</th>
<th>( A_2 )</th>
<th>( t_2 )</th>
<th>( t_3 )</th>
<th>( k_{01} )</th>
<th>( a )</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>363.2</td>
<td>-444.2</td>
<td>0.977</td>
<td>6.684</td>
<td>2.944</td>
<td>0.141</td>
</tr>
</tbody>
</table>

SUMMARY

Two-stage constitutive model was developed for cementitious rockfill material based on large-scale tri-axial test. Bi-module \( K-G \) non-linear model was developed for the stage before the break strength and regress analysis model was setup for the softening stage after the break strength. The outputs show the model setup in the test can solve the deformation of the material occurred before and after shear failure, which can help for related studies and applications.

ACKNOWLEDGEMENTS

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Research Special Funds For Projects, grant 201401022, 201301061; MWR Key Science and Technology Promotion Projects, grant 1261420162562.

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