

MODELING OF GRAVELLY SOIL WITH MULTIPLE LITHOLOGIC COMPONENTS AND ITS APPLICATION

GA ZHANGⁱ⁾, JIAN-MIN ZHANGⁱⁱ⁾ and YILIN YUⁱⁱⁱ⁾

ABSTRACT

The behavior of a gravelly soil with multiple lithologic components was investigated using large-scale compression and triaxial tests based on the research scheme “separating before mixing”. A series of test results showed that the behavior of a gravelly soil with multiple lithologic components was significantly dependent on the identity and proportions of the lithologic components. A new constitutive model was established to capture the main behavior of a gravelly soil with arbitrary lithologic proportions using few parameters based on the presentation of an elasto-plasticity model and the deformation mechanism of a complex soil. The parameters of this model can be determined using triaxial tests of a simple soil with a single lithologic component. The numerical efficiency of the model was confirmed by successful application to several high rockfill dams. Three-dimensional finite element (FEM) stress-strain analysis results for the Jishixia concrete-faced rockfill dam were obtained as important references for the selection of rockfill and optimizing the design of the dam.

Key words: constitutive model, FEM, gravelly soil, mixture, rockfill dam (IGC: D6/E13/H4)

INTRODUCTION

The establishment of a constitutive model of soil is one of the key theoretical problems in soil mechanics. Significantly, models of different kinds of soil have become of great concern in recent years because of rapid developments in numerical techniques and increasing requirements for such a model from those involved in the design of large-scale construction projects.

The Cam clay model can be thought of as the starting point for models that can reasonably describe the behavior of an individual soil (Roscoe et al., 1963). Since then, hundreds of models based on either theoretical assumptions or test observations have been proposed to simulate the behavior of different kinds of soil. These soil models can be divided into two types: nonlinear elasticity models and elasto-plasticity models. Many nonlinear elasticity models have been proposed to describe the nonlinear behavior of a soil, including the Duncan-Chang model (Duncan and Chang, 1970), the Naylor model (Naylor, 1978), and the Gao model (Gao, 1997). These models are widely used in numerical analysis because of their simplicity with regard to physical concepts, mathematical formulations and determination of parameters. However, these models cannot capture adequately important components of soil behavior such as plastic deformation and volumetric changes. Thus, the develop-

ment of soil models has been focused on elasto-plasticity in recent years. A well-known example is the Cam clay model and its modified versions (Roscoe et al., 1963; Roscoe and Burland, 1968). Lade and Duncan (1975) proposed an elasto-plasticity model using a non-associated flow rule based on triaxial test results, and the dilatancy and effect of intermediate principal stress can be considered with this model. Many of the elasto-plasticity models have led to reasonable descriptions of different aspects of the behavior of soil. New ideas have evolved continuously in the development of the models; for example, bounding surface models have been developed to describe the cyclic behavior of a soil (Mroz, 1967; Hashiguchi and Ueno, 1977; Dafalias and Herrmann, 1980; Hashiguchi, 1980); damage concepts have been introduced to describe the change of physical state due to loading (e.g., Frantziskonis and Desai, 1987); and a superloading yield surface concept was proposed to describe the behavior of highly structured soils (Asaoka et al., 2000).

Most of the existing models are designed for sand or clay, and some models have been reported to describe the mechanical behavior of both sand and clay (e.g., Nakano et al., 2004). The behavior of a gravelly soil, which has a larger grain size than sand, has yet to be specifically considered, although this type of soil has been of great concern in practical projects such as the construction of high em-

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bankments, highways and high-speed railways. Particularly, a gravelly soil with multiple lithologic components, a mixture of different kinds of gravelly soils, has been used as the main fill material in many large-scale projects. For example, a great quantity of such a complex gravelly soil was used in the 185.5 meter high Sanbanxi concrete-faced rockfill dam (CFRD) and the 100 meter high Jishixia CFRD in China, the 205 meter high Bakun CFRD in Malaysia, and the Beijing-Tianjin high-speed railway. For these projects, millions of cubic meters of complex gravel with different lithologic components were used as the main fill materials. However, the proportions of the lithologic components cannot be determined via geologic investigation due to the large amount of filling material and the complex distribution of rock strata. Indeed, even an approximate range of proportions is difficult to estimate. Thus, the stress-strain response of these dams is predicted considering different possible lithologic proportions of rockfill using numerical simulation. A constitutive model is needed to describe the behavior of gravelly soils with different lithologic proportions in the stress-strain analysis of dams.

In China, two models are usually used for gravelly soil in the stress-strain analysis of a practical project: the E-B model (Duncan et al., 1980) and Shen elasto-plasticity model (Shen, 1990). The E-B model, which is often preferred for its simplicity, cannot capture dilatancy behavior or plastic deformation of a gravelly soil. Shen elasto-plasticity model can capture the elasto-plasticity behavior; however, it has been shown to predict fairly larger dilative volumetric strain response. It has become necessary to develop a model with a more accurate description of the behavior of a gravelly soil. In addition, the existing models are unable to provide a unified description of the behavior of a gravelly soil with multiple lithologic components. In other words, a possible lithologic proportions should be regarded as a material to determine model parameters if these models are used. Therefore, a model is needed for a unified description of behavior of the gravelly soil with different lithologic proportions.

This study is focused on a simplified constitutive model with an accurate, unified description of the behavior of a gravelly soil with multiple lithologic components by presenting the research scheme “separating before mixing”. Firstly, the behavior of a gravelly soil with multiple lithologic components is investigated using a series of large-scale tests. Then, a two yield surfaces model is proposed for a reasonable description of the behavior of a gravelly soil with a single lithologic component. A multiple lithologic components model is consequently established by extending the single lithologic component model to capture the behavior of a gravelly soil with multiple lithologic components based on the deformation mechanism analysis. Finally, as a practical application example, the multiple lithologic components model is used for the stress-strain analysis of a high CFRD and some results are obtained as references for the design of this dam.

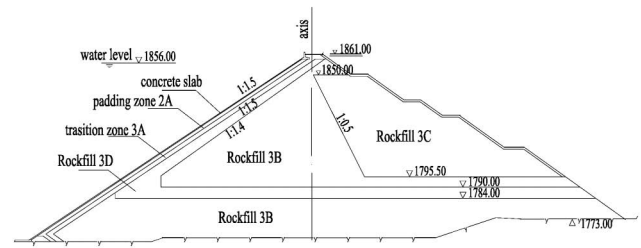


Fig. 1. Standard section of Jishixia CFRD

BACKGROUND

The Jishixia CFRD is located in the Huanghe River, Qinghai Province, China. Its maximum height is 100 m (Fig. 1). The main body of the dam (rockfill 3B and 3C) is constructed using 2,800,000 m³ of complex gravels with three lithologic components; namely, conglomerate, fine sandstone, and mudstone. The proportions of the three lithologic components of the rockfill cannot be determined exactly because of the massive amount of rockfill and the complex distribution of rock strata. The design engineers require an estimate of the stress and deformation of the dam within the possible range of proportions of the lithologic components. Thus, a unified model of gravelly soil with multiple lithologic components was used for the complex gravels comprising the main body of this dam.

TEST OBSERVATIONS

Large-scale unidimensional compression and drained triaxial compression tests were conducted to investigate the behavior of the gravel used as the rockfill for the Jishixia CFRD. According to the research scheme “separating before mixing”, two steps were included: (1) three groups of tests were conducted on simple gravels with a single lithologic component; namely, conglomerate gravel, fine sandstone gravel, and mudstone gravel; (2) two groups of tests were conducted on complex gravels with different proportions of lithologic components; namely, type I gravel (conglomerate/fine sandstone/mudstone = 5:3:2, weight/weight) and type II gravel (conglomerate/fine sandstone/mudstone = 2:5:3, weight/weight). Type I gravel was recommended by the design institute of the project.

The specific gravity is 2.70, 2.76, and 2.75 for the conglomerate gravel, fine sandstone gravel, and mudstone gravel, respectively. The grain size distributions of the different gravels are the same in the tests (Fig. 2). In all tests, the gravel was compacted by layer to the designed dry density of 2.08 g/cm³ and saturated before the test.

The unidimensional compression tests were conducted with a TH-20t CSASSI large-scale test apparatus, which can be used for large-scale monotonic and cyclic shear and compression tests of a soil and the soil-structure interface (Zhang and Zhang, 2006a). The test results show that compressibility of all the gravels, including the

simple and complex gravels, decreases with increasing normal stress, and compressibility due to re-loading and unloading is significantly less than that due to monotonic loading. Moreover, the compressibility of the two complex gravels was within that of the three simple gravels under the same normal stress condition.

The large-scale triaxial test apparatus can be used for monotonic and cyclic triaxial tests with a large load capacity of 2000 kN in both axial and radial directions (Fig. 3). The test sample was 700 mm in height and 300 mm in diameter. The loading and data acquisition of stresses and strains were both automated. The loading rate was 0.2 mm/min for all tests.

The drained triaxial test results indicate that the three simple gravels exhibit different stress-strain relationship responses (Fig. 4). For the relationship between the deviation principal stress, $\sigma_1 - \sigma_3$, and axial strain, ε_1 , the responses of three gravels are qualitatively similar. The maximum $(\sigma_1 - \sigma_3)$ value and initial slope of the $(\sigma_1 - \sigma_3) \sim \varepsilon_1$

relationship curve increases with increasing confining pressure, σ_3 . However, the volumetric strain responses of the three gravels differ significantly. If the confining pressure is small (e.g., 0.1 MPa), the conglomerate gravel dilates significantly after a little compression, while the mudstone gravel exhibits continuous contraction with increasing axial strain. The fine sandstone gravel exhibits a small dilation, far less than that of the conglomerate gravel. If the confining pressure is large (e.g., 1.5 MPa), a continuous contraction can be found for all three simple gravels. The extent of contraction of the mudstone gravel is larger than those of the others under the same conditions. In other words, the main behavior of volumetric strain of the three simple gravels under triaxial test conditions can be summarized as: (1) the volumetric strain of

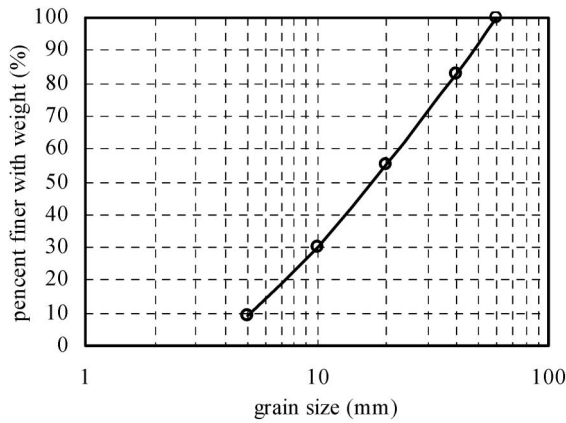


Fig. 2. Grain size distributions of a gravel

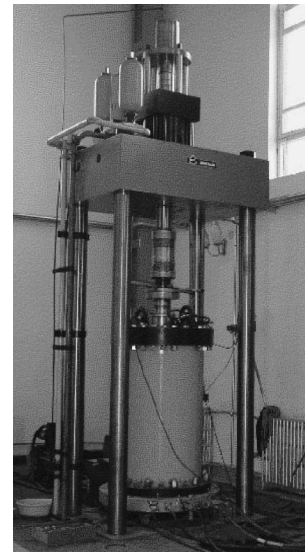


Fig. 3. A large-scale triaxial test apparatus

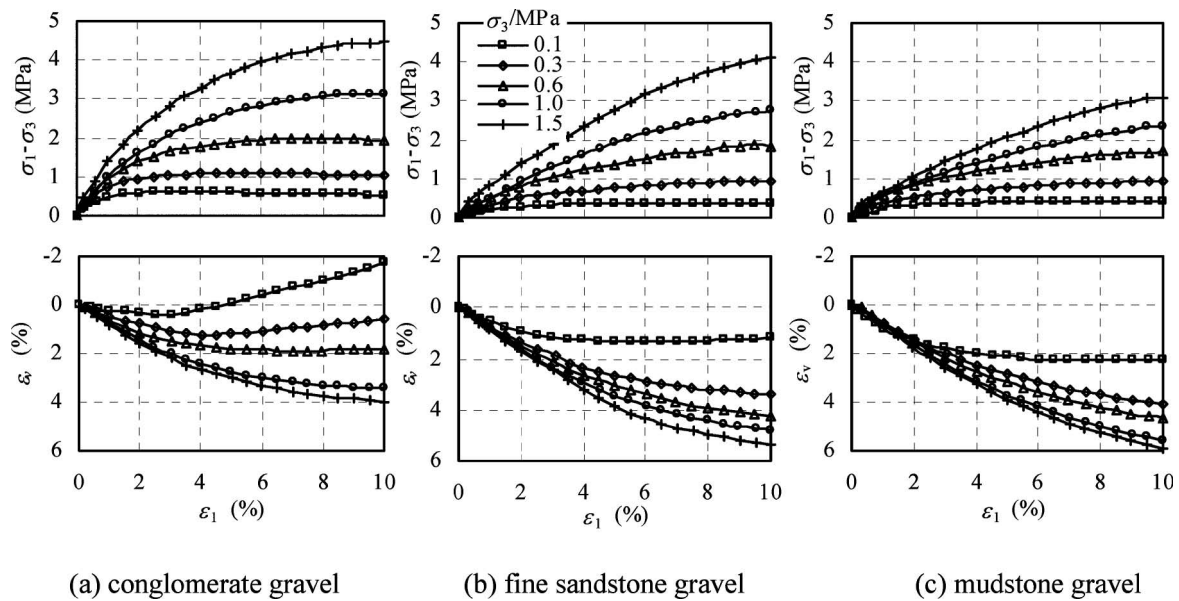


Fig. 4. Triaxial test results of simplex gravels with a single lithologic component (compression is defined as positive): σ_1 , major principal stress; σ_3 , minor principal stress; ε_1 , axial strain; ε_v , volumetric strain

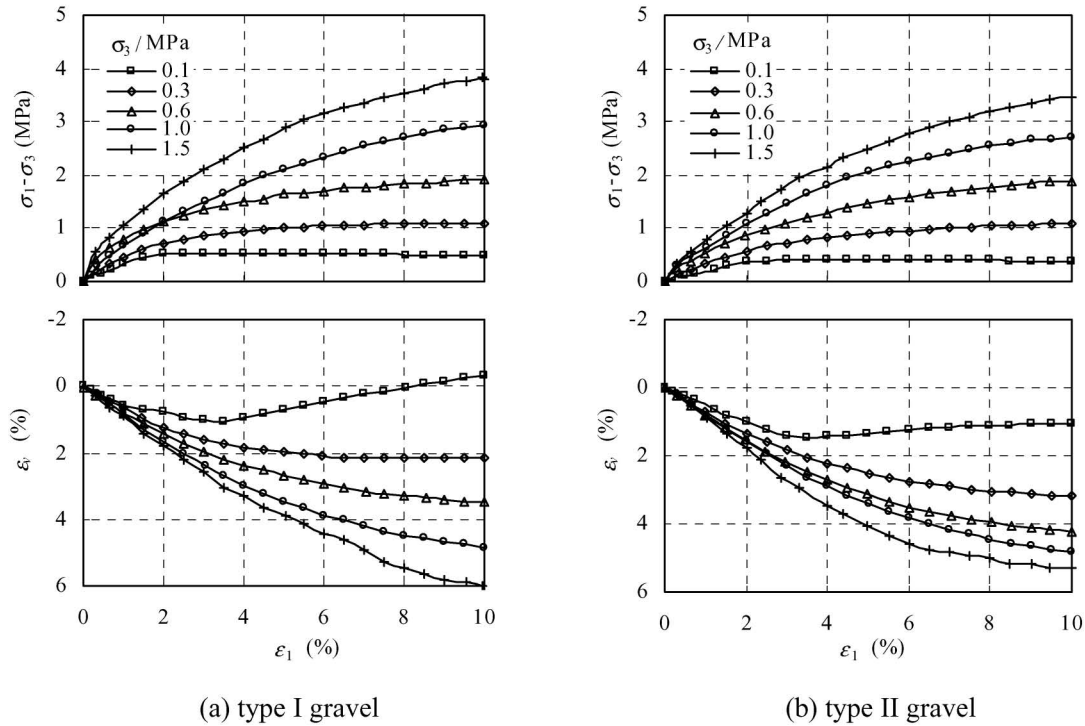


Fig. 5. Triaxial test results of complex gravels with multiple lithologic components (compression is defined as positive): σ_1 , major principal stress; σ_3 , minor principal stress; ε_1 , axial strain; ε_v , volumetric strain

the conglomerate gravel changes from dilation to contraction with increasing confining pressure; (2) the volumetric strain of the mudstone gravel exhibits a continuous contraction and the extent of contraction increases with increasing confining pressure; and (3) the volumetric strain of the fine sandstone is between those of the conglomerate gravel and the mudstone gravel. This demonstrates that an effective constitutive model of gravelly soil should capture the comprehensive volumetric strain responses, including dilation, contraction, and the change from dilation to contraction.

Figure 5 shows the triaxial test results of the two complex gravels. It can be seen from Figs. 4 and 5 that the stress-strain relationships of the complex gravels are within those of the three simple gravels. Moreover, the lithologic proportions have a significant effect on the response of a complex gravel. For example, the type I gravel exhibits significant dilation in the volumetric strain if the confining pressure is fairly small, because it involves predominant conglomerate component. In addition, the volumetric strain response of the type II gravel is similar to that of the fine sandstone gravel because the content of fine sandstone is dominant.

According to the test results, the strength of a gravelly soil can be formulated using the Mohr-Coulomb criterion. That is:

$$\sin \phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} \quad (1)$$

The friction angle, ϕ , is nonlinear with confining pressure, and is expressed as:

Table 1. Strength parameters of gravels

gravel	conglomerate	sandstone	mudstone	type I	type II
ϕ_0 (°)	47	43	42.5	45.5	43
$\Delta\phi$ (°)	9	6	7	8	6

$$\phi = \phi_0 - \Delta\phi \lg \left(\frac{\sigma_3}{p_a} \right) \quad (2)$$

where ϕ_0 and $\Delta\phi$ are the strength parameters and p_a is the standard atmosphere (101325 Pa). Table 1 shows the strength parameters of five types of gravels. It can be seen that the strength parameters of the conglomerate gravel are highest and those of the mudstone gravel are lowest. It should be noted that the strength parameters of the complex gravels are within those of the simple gravels. The proportions of lithologic components have a significant effect on the strength parameters of the complex gravel.

It can be concluded that the behavior of a complex gravel is significantly dependent on the identity and proportions of its lithologic components. A change in volumetric behavior is of concern for modeling the behavior of a complex gravelly soil because it is fairly comprehensively dependent on the proportions of its lithologic components. In other words, an effective model should capture volumetric strain behavior including the dilation, contraction, and change from dilation to contraction with increasing confining pressure.

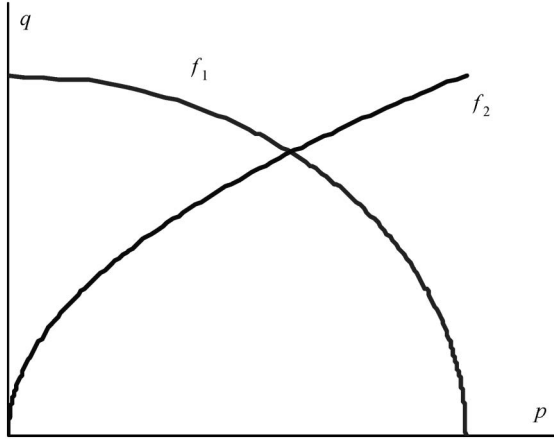


Fig. 6. Two yield surfaces of the model: p , average stress; q , principal shear stress

SINGLE LITHOLOGIC COMPONENT MODEL

Formulations

An elasto-plasticity model, referred to as a single lithologic component model (SL model), is proposed to capture the main behavior of a gravelly soil with a single lithologic component, including the comprehensive volumetric change behavior. This model serves as the basis of the model of behavior of a gravelly soil with multiple lithologic components.

The following assumption on the two yield surfaces (Fig. 6) (Shen, 1990) is used for the SL model:

$$\begin{cases} f_1 = p^2 + 4q^2 \\ f_2 = \frac{q^2}{p} \end{cases} \quad (3)$$

where p is average stress and q is principal shear stress. Define:

$$\begin{cases} p = \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3) \\ q = \frac{1}{\sqrt{2}} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \end{cases} \quad (4)$$

where σ_1 , σ_2 , and σ_3 , are major, intermediate, and minor principal stresses, respectively. The assumption on these yield surfaces has been confirmed to capture the elasto-plasticity behavior of a gravelly soil by many test results (e.g., Shen, 1990) and experiences from numerical analysis of high embankments.

The concept of a partial plastic strain is introduced to describe the plastic strain corresponding to a yield surface. The two yield surfaces are used to judge whether plastic strain occurs. By definition, a partial plastic strain occurs if $f_1 > (f_1)_{\max}$ or $f_2 > (f_2)_{\max}$. The associated flow rule is adopted to determine the direction of plastic strain. Thus, the elasto-plasticity modulus matrix can be derived as follows:

$$\begin{aligned} [\mathbf{D}]^{ep} = & [\mathbf{D}] - \frac{1}{D_b} \left\{ A_1 [\mathbf{D}] \{n_1\} \left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T + A_2 [\mathbf{D}] \{n_2\} \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T \right. \\ & + A_1 A_2 [\mathbf{D}] \left(\{n_1\} \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_2\} \left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T \right. \\ & - \{n_1\} \left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_2\} \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T \\ & + \{n_2\} \left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_1\} \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T \\ & \left. \left. - \{n_2\} \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_1\} \left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T \right) \right\} [\mathbf{D}] \end{aligned} \quad (5)$$

where

$$\begin{aligned} D_b = & 1 + A_1 \left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_1\} + A_2 \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_2\} \\ & + A_1 A_2 \left(\left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_1\} \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_2\} \right. \\ & \left. - \left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_2\} \left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T [\mathbf{D}] \{n_1\} \right) \end{aligned} \quad (6)$$

$$\{n_1\} = \frac{\left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T}{\sqrt{\left\{ \frac{\partial f_1}{\partial \sigma} \right\}^T \left\{ \frac{\partial f_1}{\partial \sigma} \right\}}}, \quad \{n_2\} = \frac{\left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T}{\sqrt{\left\{ \frac{\partial f_2}{\partial \sigma} \right\}^T \left\{ \frac{\partial f_2}{\partial \sigma} \right\}}} \quad (7)$$

where σ is a stress tensor, and A_1 and A_2 are plasticity coefficients corresponding to a yield surface, respectively. It should be noted that $A_1 = 0$ if $f_1 < (f_1)_{\max}$ and $A_2 = 0$ if $f_2 < (f_2)_{\max}$, where f_1 and f_2 are current values of the yield surface functions and $(f_1)_{\max}$ and $(f_2)_{\max}$ are maximum historical values of the yield surfaces, respectively. The elastic modulus matrix, $[\mathbf{D}]$, can be determined using two elastic parameters; namely, elastic modulus, E_{ur} , and elastic Poisson's ratio, μ_{ur} , according to classical elastic theory.

The plasticity coefficients, A_1 and A_2 , are derived using equivalent stress under triaxial test conditions (Shen, 1990). In this way, two increment equations, including the relationship between the deviation principal stress and axial strain, and the relationship between the volumetric strain and axial strain, can be obtained according to Eq. (5). Thus, the plasticity coefficients can be solved using tangential slopes of the deviation principal stress-axial strain relationship and volumetric-axial strain relationship under triaxial test condition, E_t and μ_t , as follows:

$$\begin{cases} A_1 = \frac{3\eta \left(\frac{3}{E_t} - \frac{\mu_t}{E_t} - \frac{2(1+\nu_{ur})}{E_{ur}} \right) + 12 \left(\frac{\mu_t}{E_t} - \frac{1-2\nu_{ur}}{E_{ur}} \right)}{4(1+12\eta)(1+2\eta^2)} \\ A_2 = \frac{3 \left(\frac{3}{E_t} - \frac{\mu_t}{E_t} - \frac{2(1+\nu_{ur})}{E_{ur}} \right) + 24\eta \left(\frac{\mu_t}{E_t} - \frac{1-2\nu_{ur}}{E_{ur}} \right)}{4(6-\eta)(1+2\eta^2)} \end{cases} \quad (8)$$

where η is the stress ratio and defined as:

$$\eta = \frac{q}{p} \quad (9)$$

E_t is assumed to be described using the following equation (Duncan and Chang, 1970):

$$E_t = \frac{\Delta\sigma_1}{\Delta\varepsilon_1} = K p_a \left(\frac{\sigma_3}{p_a} \right)^n (1 - R_f S_1)^2 \quad (10)$$

where K , n , and R_f are model parameters. Here, the elastic modulus, E_{ur} , is assumed to be $1.2E_t$. The elastic Poisson's ratio, μ_{ur} is assumed to be $1/3$. S_1 is defined as the stress mobilized level using the following equation:

$$S_1 = \frac{(\sigma_1 - \sigma_3)(1 - \sin \phi)}{2\sigma_3 \sin \phi} \quad (11)$$

where the friction angle, ϕ , has been expressed using Eq. (2).

As stated, the volumetric behavior needs a reasonable description. Thus, a new, simple formulation of the tangential bulk modulus, μ_t , is proposed on the basis of the test results reported here. That is:

$$\mu_t = \frac{\Delta\varepsilon_v}{\Delta\varepsilon_1} = 1 - 2 \left(G - F \lg \left(\frac{\sigma_3}{p_a} \right) + \frac{D(\sigma_1 - \sigma_3)(1 - R_f S_1)}{E_t} \right) \quad (12)$$

where G , F , and D are model parameters.

Parameters

The elastic parameters, E_{ur} and μ_{ur} , have been determined in the foregoing formulation. Thus, the SL model has only eight parameters for a gravelly soil: ϕ_0 , $\Delta\phi$, K , n , R_f , G , F , and D . All the parameters can be determined using a group of traditional triaxial tests. The strength parameters, ϕ_0 and $\Delta\phi$, are determined according to the relationship between the strength and confining pressure using Eqs. (1) and (2). The parameters K , n , and R_f are determined according to the relationship between the deviator principal stress and axial strain using the same procedures as those in the Duncan-Chang model (Duncan and Chang, 1970); so the detailed procedures are given

only briefly here.

The parameters G , F , and D are determined according to the relationship between the volumetric strain and axial strain of triaxial test results. The determination steps are recommended as follows. (1) To fit the relationship between the radial strain, ε_3 , and axial strain, ε_1 , using a parabola under a constant confining pressure condition (Fig. 7 (a)); (2) To determine the initial strain increment ratio, μ_i , and D_i corresponding to a confining pressure using the differentiation of the parabola:

$$-\frac{d\varepsilon_3}{d\varepsilon_1} = \mu_i + D_i \varepsilon_1 \quad (13)$$

Thus, the parameter D is obtained using the average of all the D_i values under different confining pressure conditions. The parameters G and F are determined by fitting the relationship between μ_i and the logarithm of the ratio of the confining pressure divided by the standard atmosphere using the following equation (Fig. 7 (b)):

$$\mu_i = G - F \lg \left(\frac{\sigma_3}{p_a} \right) \quad (14)$$

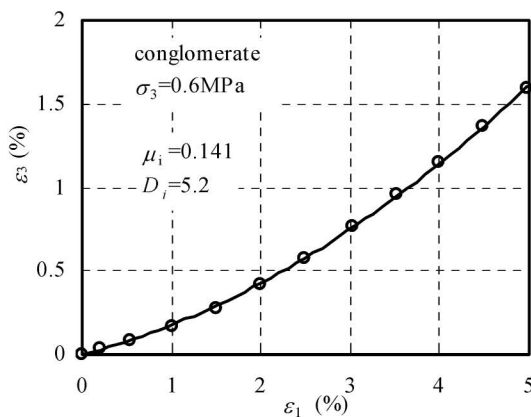
Verifications

Table 2 shows the SL model parameters of the three simple gravels that are determined from the triaxial test results.

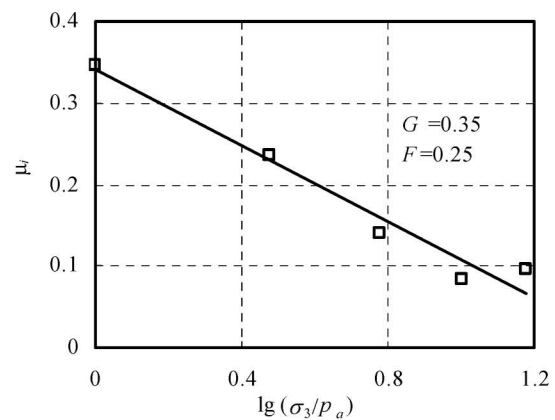
To verify the effectiveness of the SL model, we compare the model predictions of the stress-strain relationship of the gravels with the triaxial test results (Fig. 8). The predicted curve shows a good fit to the test result. For

Table 2. Parameters of simplex gravels

gravel	ϕ_0 (°)	$\Delta\phi$ (°)	R_f	K	n	G	F	D
conglomerate	47	9	0.82	1100	0.21	0.35	0.25	5.1
sandstone	43	6	0.76	500	0.20	0.20	0.19	5.1
mudstone	42.5	7	0.79	450	0.15	0.16	0.14	4.3



(a)



(b)

Fig. 7. Determination of volumetric parameters: point, test data; line, fit line; ε_1 , axial strain; ε_3 , radial strain; σ_3 , confining pressure; μ_i , initial strain ratio; p_a , standard atmosphere

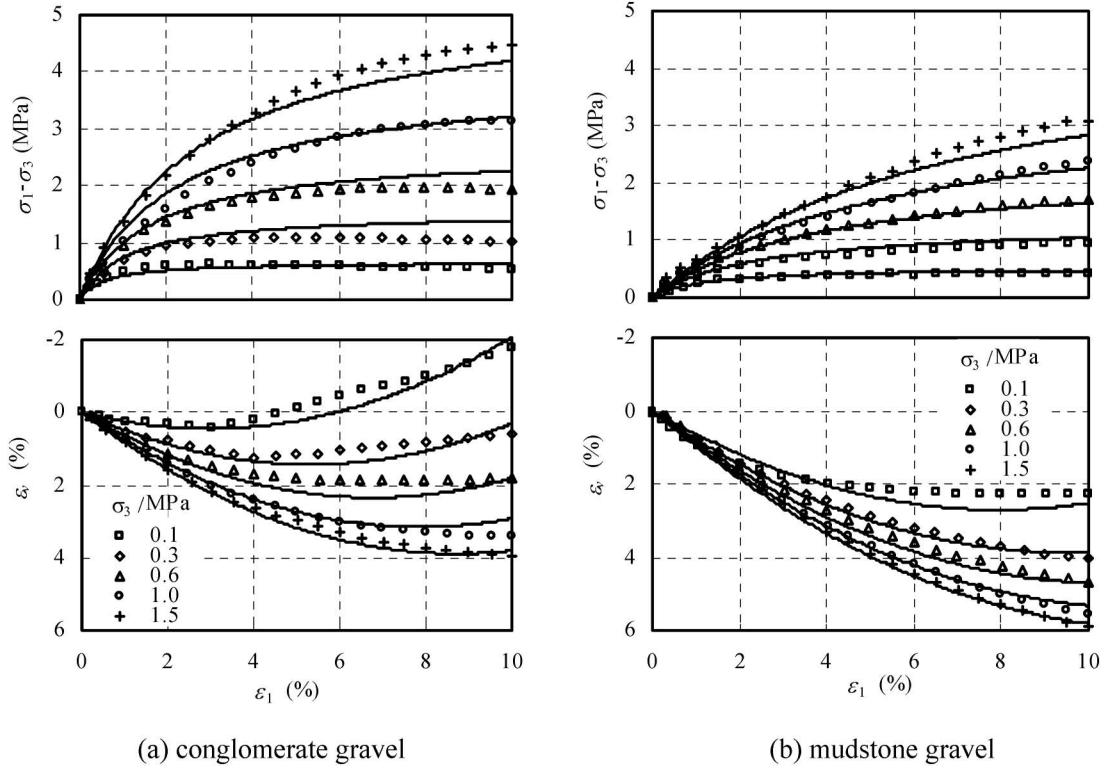


Fig. 8. SL model predictions and test results under triaxial test conditions: points, test data; lines, model predictions; σ_1 , major principal stress; σ_3 , minor principal stress; ε_1 , axial strain; ε_v , volumetric strain

the volumetric strain, the SL model can describe the continuous contraction at all the confining pressures of the mudstone gravel, and capture the dilation if the confining stress is small and its change to contraction when increasing the confining stress of the conglomerate gravel. This demonstrates that the model can capture the main behavior of the gravelly soil, including the comprehensive volumetric change during a large strain.

MULTIPLE LITHOLOGIC COMPONENTS MODEL

A complex gravel with three lithologic components is used as an example to establish a multiple lithologic components model (ML model). The proportions of the conglomerate, fine sandstone, and mudstone are defined as m , n , and l , respectively; obviously, $m + n + l = 1$.

The ML model is established on the basis of the SL model. Thus, the key problem is how to determine the parameters of the SL model for a complex soil according to the lithologic proportions. We analyzed the effect of parameters of the SL model on the stress-strain relationship response. It was shown that the parameter K of the SL model, which can be thought of as a stiffness of the soil, has a significant effect on the deformation behavior. Thus, the deformation mechanism was considered under a unidimensional condition for determination of the parameter K . We consider two extreme cases in which the three lithologic components are not mixed together. In other words, the three components are placed in horizontal layers or in vertical layers (Fig. 9). For the components

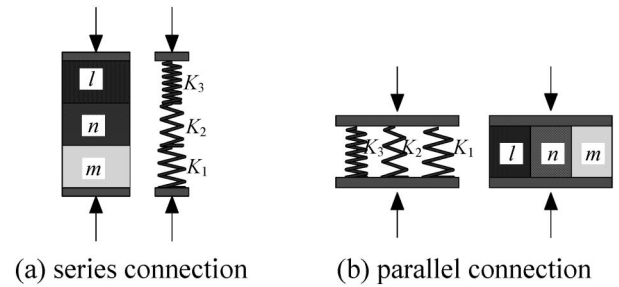


Fig. 9. Deformation mechanism of a complex gravel

that are placed horizontally, the soil can be simplified to a group of springs via a series connection (Fig. 9 (a)). Thus, the parameter K should be K' using the following equation:

$$\frac{1}{K'} = \frac{m}{K_1} + \frac{n}{K_2} + \frac{l}{K_3} \quad (15)$$

where K_j ($j = 1-3$) represents the value of K of the three simple gravels. For the components that are placed vertically, the soil can be simplified to a group of springs via a parallel connection (Fig. 9 (b)). Thus, the parameter K should be K'' using the following equation:

$$K'' = m \cdot K_1 + n \cdot K_2 + l \cdot K_3 \quad (16)$$

The deformation of a complex gravel should lie within those of the two kinds of special soils. Thus, the parameter K of the complex gravel can be formulated us-

ing the following equation by introducing a weighting factor, ω :

$$K = \omega K' + (1 - \omega) K'' \quad (17)$$

Obviously, $K = K''$ if $\omega = 0$ and $K = K'$ if $\omega = 1$. The value of ω is proposed to be 0.5 because the gravels are mixed uniformly. As a result, the value of the parameter K can be determined according to those of the three simple gravels.

Other parameters of the ML model are assumed to be obtained using the average of corresponding parameters of the three simple gravels weighted by their lithologic proportions. In other words, an arbitrary parameter except K , of the ML model for a gravel with multiple lithologic components can be derived using the following equation:

$$d = m \cdot d_1 + n \cdot d_2 + l \cdot d_3 \quad (18)$$

where d represents one of the parameters, except K , of the ML model (e.g., ϕ , n , G , etc.). d_j ($j = 1-3$) represents a corresponding SL model parameter of the three simple gravels. Therefore, all the parameters of the ML model can be determined using Eqs. (17) and (18) according to the lithologic proportions and the parameters of three simple gravels (Table 2). The behavior of the complex gravel with arbitrary lithologic proportions can be described using the ML model. Only three groups of triaxial tests on the simple gravels with a single lithologic component are needed to determine all the parameters.

To verify the effectiveness of the ML model, we com-

pare the model predictions of the stress-strain relationship of two complex gravels with different lithologic proportions with the triaxial test results (Fig. 10). These comparisons demonstrate that the model prediction curves are in good agreements with the test results for the complex gravel with multiple lithologic components. Therefore, we can conclude that the ML model is effective in capturing the primary behavior of the gravelly soil with multiple lithologic components, including strength and comprehensive volumetric change behavior.

The ML model can be extended easily to describe the behavior of a gravelly soil with four or more kinds of lithologic components. It can be realized by simply extending Eqs. (15)–(18) to the number of lithologic components. Obviously, the ML model can be reduced to the SL model if only one lithologic component is involved in a gravelly soil.

NUMERICAL ANALYSIS

The ML model has been programmed into a three-dimensional finite element (FEM) software that is widely used for stress-strain analysis of high embankments in China. The numerical efficiency of the model is confirmed by successful application to a few high rockfill dams. As an example, the analysis results for the Jishixia CFRD are presented here.

With the three-dimensional FEM analysis, we simulate the practical process of construction and water storage of the Jishixia CFRD, including: (1) the dam body is con-

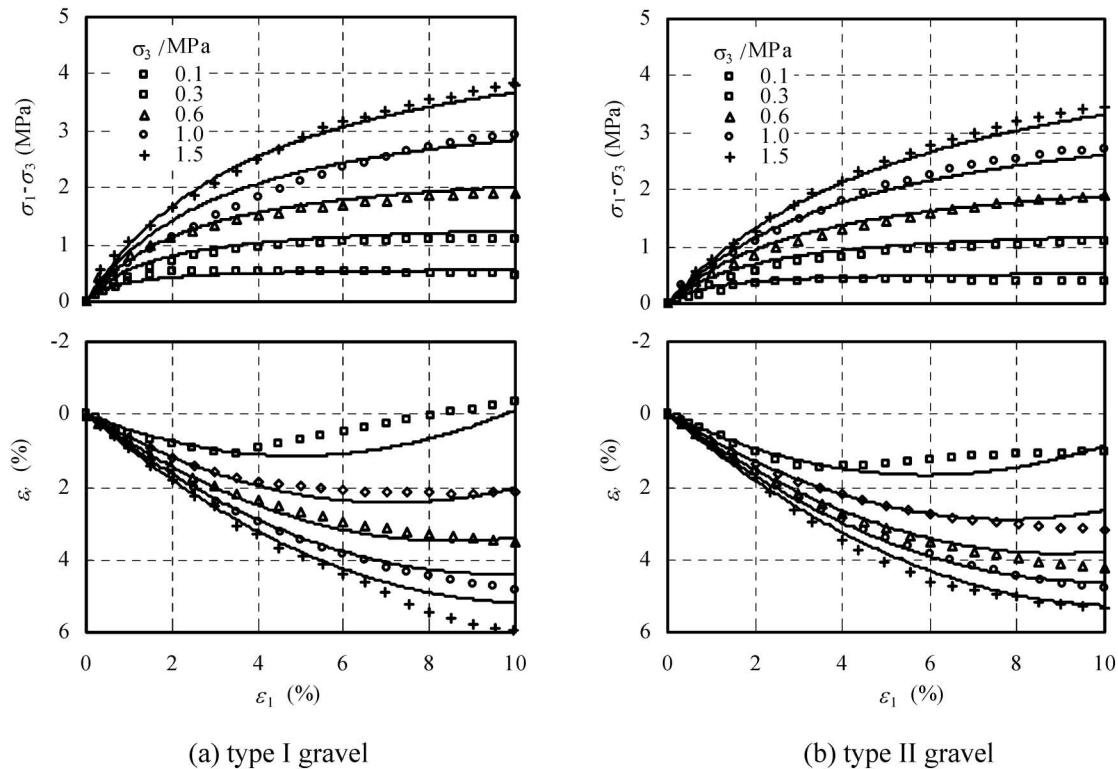


Fig. 10. ML model predictions and test results under triaxial test conditions: points, test data; lines, model predictions; σ_1 , major principal stress; σ_3 , minor principal stress; ε_1 , axial strain; ε_v , volumetric strain

structed from the bedrock to the top by horizontal layers; (2) the concrete slab is built after the body is completed; (3) the water is stored to the design level, 1856 m, after the accomplishment of the slab. A three-dimensional FE mesh was made on the basis of the practical distributions of the bedrock (Fig. 11).

The gravelly soil in the zones of rockfill 3B and rockfill 3C are characterized using the ML model. The model parameters are given in Table 2 and the effect of the proportions of lithologic components is discussed here. Other types of gravels, including the padding zone 2A, the transition zone 3A, and the rockfill 3D, are characterized using the SL model, because they are composed of a single lithologic component. Table 3 gives the parameters that are determined via the triaxial test results.

The concrete slab is characterized using a linear elastic model with the elastic module of 30 GPa and Poisson's ratio of 0.167. The interface between the concrete slab and the bedding zone is characterized using an elasto-

plasticity damage model (Zhang and Zhang, 2005). The formulations of the interface model are derived on the basis of a new model framework with concepts and assumptions based on systematic test results (Zhang and Zhang, 2006b). The model parameters are determined easily from a series of cyclic shear tests and a confining compression test. The comparisons of model predictions with the results for a series of tests under various loading conditions demonstrate that the model is effective in providing a unified description of monotonic and cyclic behavior of the interface between a structure and gravelly soil (Zhang and Zhang, 2005).

According to the recommendation of the design institute of the dam, the type I gravel is firstly selected as the rockfill 3B and 3C in the FEM analysis to determine the basic stress-strain response of the dam (Figs. 12–14). The calculated distributions of stress agree with the experience of this kind of dam; this demonstrates that the present model is effective in the numerical simulation (Fig. 12). The stress fields of the dam are affected significantly by the water storage.

The calculated contour lines of settlement show that maximum settlement occurs in the middle of the dam (Figs. 13(a) and (b)), and the settlement increases a little due to water storage. During the construction period, horizontal displacements point from the dam axis to both sides (Fig. 13(c)). An increment to downstream of the horizontal displacement is made by water storage (Fig.

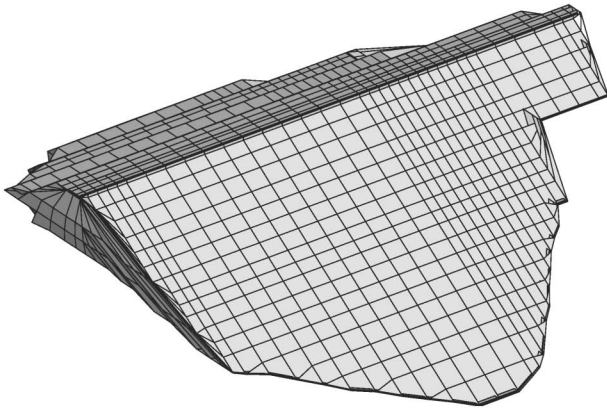


Fig. 11. Schematic view of 3D FE mesh of Jishixia CFRD

Table 3. Parameters of gravelly soil of Jishixia CFRD

zone	ϕ_0 (°)	$\Delta\phi$ (°)	R_f	K	n	G	F	D
2A	49	9	0.92	950	0.63	0.35	0.36	5.0
3A	50	9	0.91	1000	0.53	0.34	0.35	5.0
3D	50	7	0.81	1000	0.35	0.33	0.32	5.0

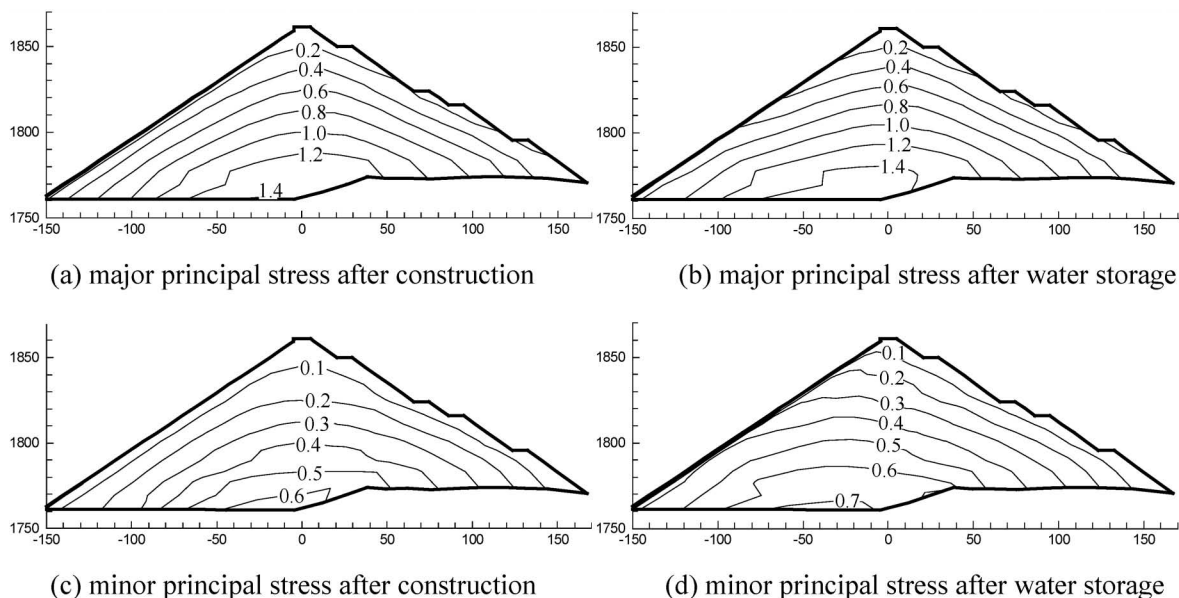


Fig. 12. Contour lines of stress in a standard section of type I gravel used as the rockfill: unit, MPa; compression is defined as positive

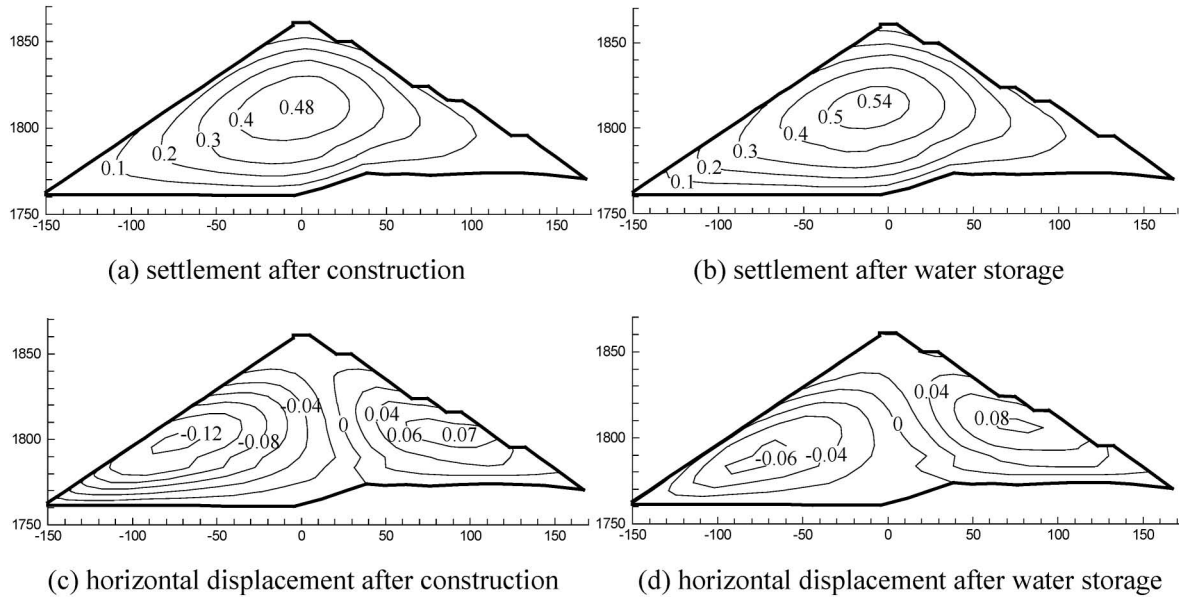


Fig. 13. Contour lines of deformation in a standard section of type I gravel used as the rockfill: unit, m

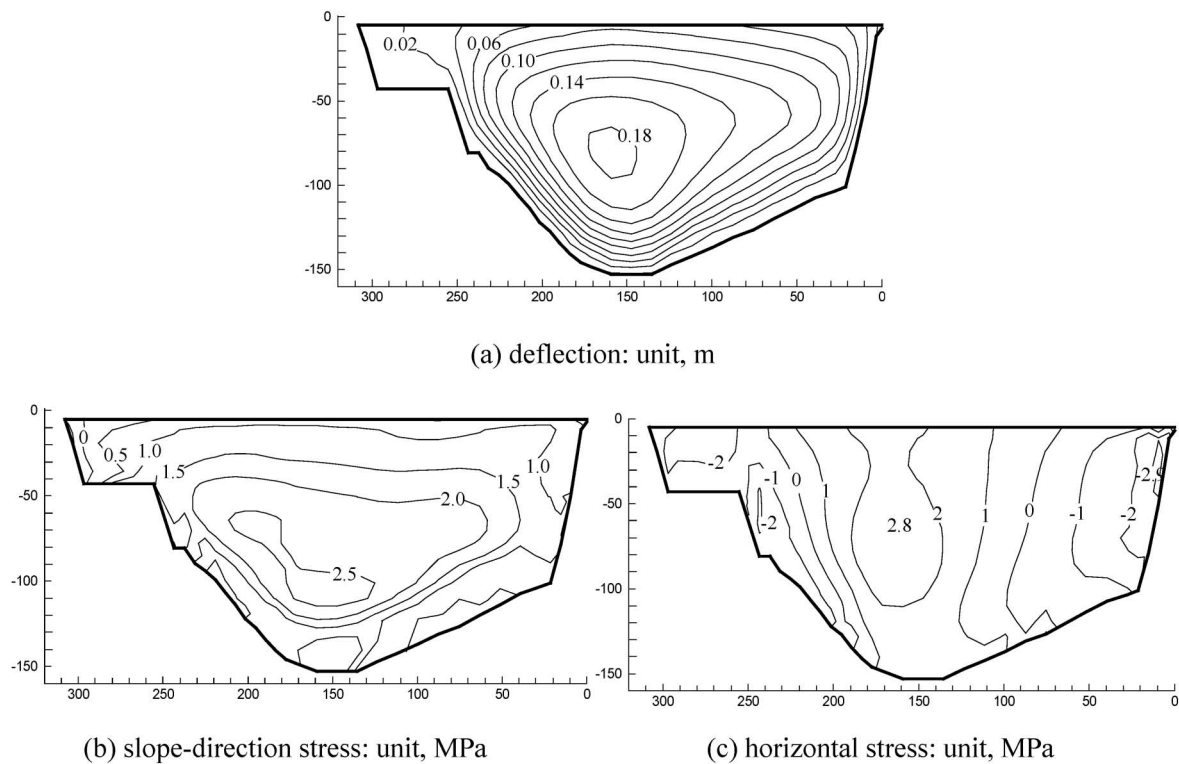


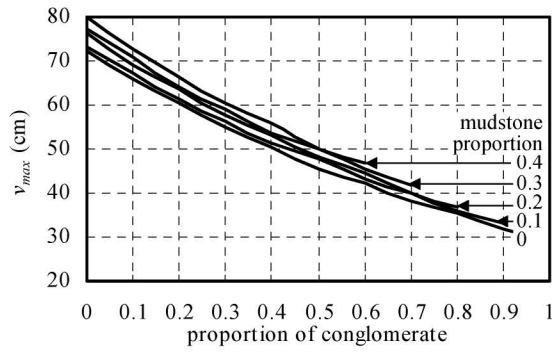
Fig. 14. Contour lines of stress and deformation of slab of type I gravel used as the rockfill: unit, m; compression is defined as positive for the stress

13(d)).

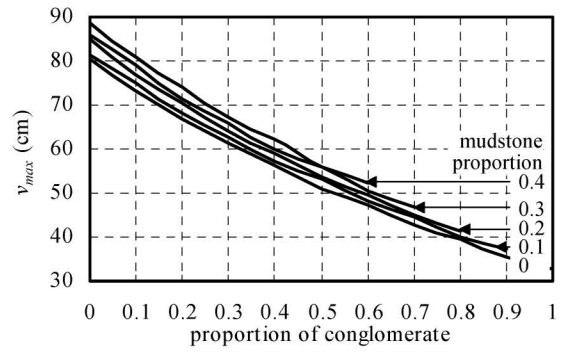
The calculated contour lines of response of the slab show that maximum response, including the deflection and stress, occurs in the middle of the slab (Fig. 14). In the direction parallel with the slope, there are compression stresses in most areas of the slab; the tensile stresses exist only near the edges of the slab (Fig. 14(b)). Horizontal stress exhibits compression behavior in the middle of

the slab and tensile behavior in both sides (Fig. 14(c)). The tensile stress is not large and can be accepted for the project.

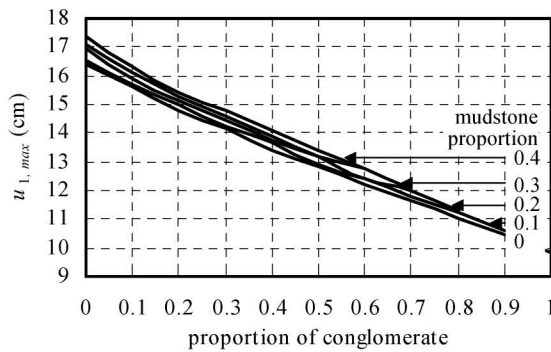
As mentioned above, the proportions of the lithologic components of a complex gravel cannot be known exactly. With the ML model, we can obtain the stress-strain response of a rockfill with arbitrary lithologic proportions. Therefore, the response of the dam was analyzed



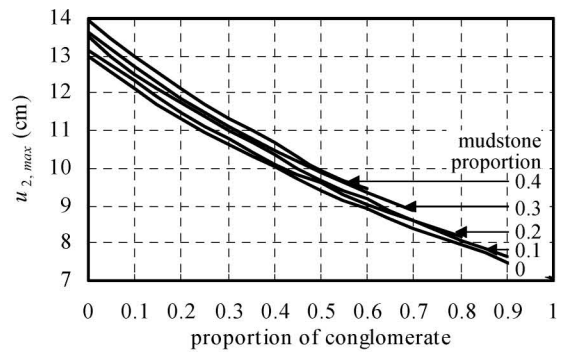
(a) settlement after construction



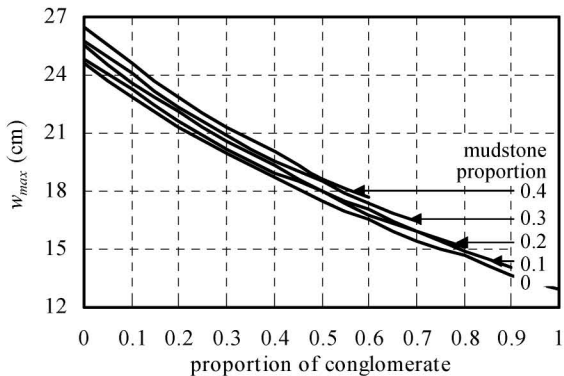
(b) settlement after water storage



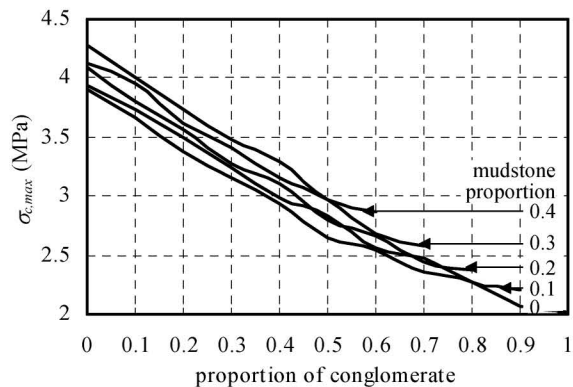
(c) horizontal disp. to upstream after construction



(d) horizontal disp. to downstream after construction



(e) deflection of the slab



(f) slope-direction stress of the slab

Fig. 15. Maximum response of the dam according to lithologic proportions: v_{\max} , maximum settlement; $u_{1, \max}$, maximum horizontal displacement to upstream; $u_{2, \max}$, maximum horizontal displacement to downstream; w_{\max} , maximum deflection of the slab; $\sigma_{c, \max}$, maximum compression stress of the slab in the direction parallel with the slope

when different lithologic proportions are used for the rockfill; thus we obtained the relationship between the maximum stress-strain response of the dam and the lithologic proportions (Fig. 15). With these relationships, we can consider the maximum response according to any proportions we choose. In addition, we can estimate the distribution of response by referring to Figs. 12–14. For example, if we consider a complex gravel with the lithologic proportions of a type II, we can find a maximum settlement after construction and during water storage of 67 cm and 72 cm, respectively. The distribution of settle-

ments can be estimated by scaling Figs. 13(a) and (b) using the ratio of corresponding maximum values. These results provide important references for the design of the dam; they have been used for the selection of rockfill and optimizing the design of the dam.

The FEM analysis of practical CFRDs shows that the present ML model yields a unified prediction of the response of the dam using the complex gravel with a very wide range of possible lithologic proportions under the condition that only a few groups of conventional triaxial tests are needed to determine the model parameters.

CONCLUSIONS

The behavior of a gravelly soil with multiple lithologic components was investigated using large-scale unidimensional compression and triaxial tests based on the research scheme “separating before mixing”. A new constitutive model was established to describe the behavior of a gravelly soil with multiple lithologic components. The model and FEM software were used to describe the three-dimensional response of the Jishixia CFRD. The main summaries and conclusions are as follows:

(1) The behavior of a gravelly soil with multiple lithologic components is significantly dependent on identity and proportions of the lithologic components. The description of the volumetric behavior of such a gravelly soil is fairly comprehensive, including dilation, contraction, and change from dilation to contraction with increasing confining pressure.

(2) An elasto-plasticity model (SL model) is proposed, using two yield surfaces based on the observations of test results. The validity of this model to provide a reasonable characterization of the main behavior of the stress-strain relationship of a gravelly soil during a large strain is confirmed by the test results.

(3) On the basis of the SL model and the deformation mechanism of a complex soil, we establish a new constitutive model of a gravelly soil with multiple lithologic components (ML model). This model is capable of capturing the main behavior of the gravelly soil with arbitrary lithologic proportions. All the model parameters can be determined using triaxial tests of the simple soil with a single lithologic component.

(4) The numerical efficiency of the model is confirmed by successful application to several high CFRDs. A series of three-dimensional stress-strain analysis results of the Jishixia CFRD provide important references for the selection of rockfill and optimizing the design of the dam.

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