

Applicability of UH model to coarse-grained soil

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Abstract. Large-scale triaxial, K0 compression and constant stress ratio path tests were carried out on two kinds of coarse-grained soils. Stress-strain relationships of these tests were simulated by using UH model for sands to study the applicability of the model to different stress paths of coarse-grained soil, and the results were compared with those of Duncan's E-B model. It is shown that the mechanical behaviors of coarse-grained soil can be well described by the UH model, and overall its capacity to reflect stress-strain relationship under different stress paths is better than that of Duncan's E-B model. Finally, the UH model and Duncan's E-B model were used for stress-deformation analysis of a core rockfill dam. The applicability of UH model for sands to coarse-grained soil was verified.

1. Introduction

With the challenges of climate change and for reducing the emission of greenhouse gas, the renewable energy, such as the hydropower, has been rapidly developed in China [1]. Numerous high earth rockfill dams have been or under constructions in western China. The safety of the hydraulic centre, as well as the dams, the residents and the infrastructures in the downstream area is of major concerns for the hydropower exploitation. A good understanding of the mechanical and/or hydro-mechanical behaviour of the dam is necessary. In fact, for the rockfill dams, the most volumetric part is composed of the so-called rockfill, with an average diameter larger than that we frequently encountered in the laboratories. The classic laboratory test equipment is not suitable for this type of soil. So, the rockfill material is usually scaled into smaller sized material called coarse-grained soil for laboratory test. To avoid the error caused by laboratory test, some in-situ measurements are also developed, but the in-situ measurements have also some difficulties [2-3]. Nowadays, the model parameters used for analysis are still mainly gained by laboratory test [4]. In this context, a series of research on the coarse-grained soil was carried out, with the laboratory test and the numerical approaches. Based on special laboratory tests, a constitutive model has been verified for this type of soil, and then the model has been applied to a rockfill dam. The current article is organised as following, after a short introduction, the model is briefly introduced, then the

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experimental research is presented, an application on a rockfill dam is given for the illustration. The discussions and conclusions are given at the end.

At present, many soil constitutive models have been put forward, including nonlinear elastic models [5-8] and elastoplastic models [9-13]. However, most of the models were proposed for clay or sand, and few constitutive models for coarse-grained soil.

Yao et al. [14-15] proposed the three-dimensional unified hardening model for overconsolidated clays (the UH model), based on the relationship between the current yield surface and reference yield surface. The UH model can model many characteristics of overconsolidated clays well, including stress-strain relationships, shear dilatancy, strain-hardening and softening, and stress path dependence behaviour.

In order to describe the similarities and differences between clays and sands in a unified constitutive model, Yao et al. [16] developed the UH model for clays and sands, called UH model for sands.

2. UH model for sands

The UH model for sands, developed by Yao et al. [16], is a constitutive model based on the unified hardening (UH) model [15].

2.1 Shear yielding of sands and its description

A new parameter is introduced into the yield function of the modified Cam-clay model, and is expressed as

$$f = \frac{(1+\chi)q^2}{M^2p^2 - \chi q^2} + 1 - \frac{p_x}{p} = 0 \quad (1)$$

where χ is defined as the critical state parameter since it can adjust the vertical distance between NCL and CSL, and it is non-negative and smaller than 1; M is critical state stress ratio; p is mean effective stress, kPa; q is deviatoric stress, kPa; and p_x is intersection of the current yield surface with the p -axis, kPa.

2.2 Dilatancy of sands and its description

The state when the plastic deformation turns from contraction to dilatancy, is a characteristic state used to describe the dilatancy of sands. Here, the equation of the phase transformation line is adopted as that of the characteristic state stress ratio M_c .

$$M_c = M \cdot \exp(-m \cdot \xi) \quad (2)$$

where m is a dilatancy parameter; ξ is state variable describing the current density.

3. The large-scale triaxial test and K0 test

3.1 Test material and test method

Soils used for tests are one sandy pebble and one gravel. The sandy pebble (denoted by LS) was taken from Dashixia Concrete Faced Sand-gravel Dam, China, while the gravel (denoted by SS) was taken from Maji Rockfill Dam, China. Grain gradations of the two soils are shown in Table 1.

Table 1 Particle size distribution of soil samples

Material	No.	Mass percentage (%) of particle groups (mm)				
		60~40	40~20	20~10	10~5	<5
Sandy pebble	LS	17.6	23.8	18.4	11.7	28.5
Gravel	SS	16.9	22.9	19.6	16.1	24.5

Large-scale consolidated-drained (CD) triaxial tests were conducted on soils of LS and SS with specimen densities 2.04 and 2.20 g/cm³. The void ratios of LS and of SS are 0.34 and 0.23. The specimen size is 300 mm in diameter and 600 mm in height, and the confining pressures in tests for LS are 600, 1200, 1800 and 2400 kPa, and those for SS are 400, 800, 1600 and 2400 kPa, respectively. The tests were conducted by a large-scale triaxial apparatus, which was in the key laboratory of ministry of education for geomechanics and embankment engineering, Hohai University (Fig. 1(a)). Besides, the constant stress ratio path tests were also carried out by this apparatus, and the stress ratio (σ_1/σ_3) was kept to 2.5 during loading.

In the CD triaxial test, a negative pressure was maintained in the samples for 15 mins before saturation. Then the samples were saturated by allowing water to pass through the base of the triaxial cell and using a top drainage system for removing air voids. The samples were first subjected to the required consolidation pressure and were then sheared by applying axial loading to achieve a rate of loading of 1 mm/min. The shear procedure was terminated when the axial strain reached 15%. In the constant stress ratio path test, the only difference between the CD test is that the σ_1/σ_3 was kept to 2.5 during shearing. The rate of loading was 1 mm/min, and the initial confining pressure σ_3 was 30 kPa in consolidation phase, which would rise with the increase of in shearing.



(a) Large-scale triaxial apparatus



(b) Large-scale K0 apparatus

Fig. 1. Apparatus for laboratory tests

K0 compression tests were performed on LS soil. The apparatus is a large sized oedometer, newly developed by Hohai University [17] (Fig. 1(b)). The maximum axial force can be 800 kN, with an axial stress up to 6.4 MPa. The soil sample was dry, with 400 mm in diameter and 300 mm in height. Stress control was adopted during the test, with a rate of loading of 10 kPa/min.

3.2 Analysis of laboratory test results

Triaxial test results are shown as scattered points in Fig. 2. The test results were also predicted by UH model for sands (hereinafter called UH model) and Duncan's E-B model (hereinafter called E-B model) with parameters given in Table 2 and 3, which are shown with dashed and solid lines, respectively.

Table 2 Parameters of Duncan’s E-B model

Material	R_f	K	n	K_b	m	$\varphi_0/(^\circ)$	$\Delta\varphi/(^\circ)$
LS	0.77	530	0.56	364	0.16	52.27	6.77
SS	0.73	1361	0.36	527	0.29	55.48	9.55

Table 3 Parameters of UH model

Material	M	ν	κ	λ	N	Z	χ	m
LS	1.92	0.21	0.07	0.15	1.86	0.57	0.87	0.1
SS	1.69	0.18	0.04	0.07	0.95	0.39	0.72	0.1

Fig. 2(a) shows that LS presents strain hardening and shear contraction. There is a good agreement between the model simulations and test results for the two models. Fig. 2(b) shows that SS is strain softening and shear dilatancy. The UH model is obviously superior to E-B model.

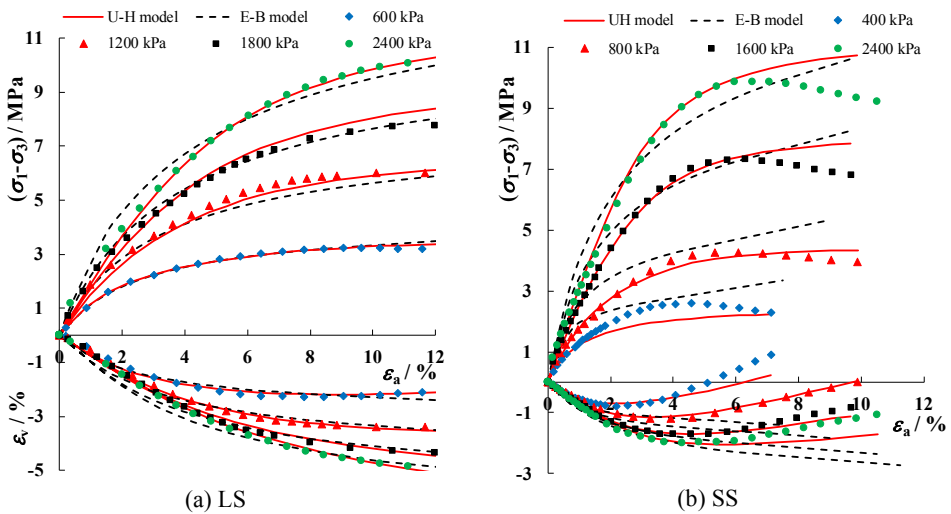


Fig. 2. Comparison between triaxial CD test results and model predictions

Fig. 3 shows the predictions of K0 compression test results on LS by the two models. The model parameters used are the same as the triaxial test, given in Table 2 and 3. Apparently, the predictions of UH model is closer to the test data than that of E-B model,

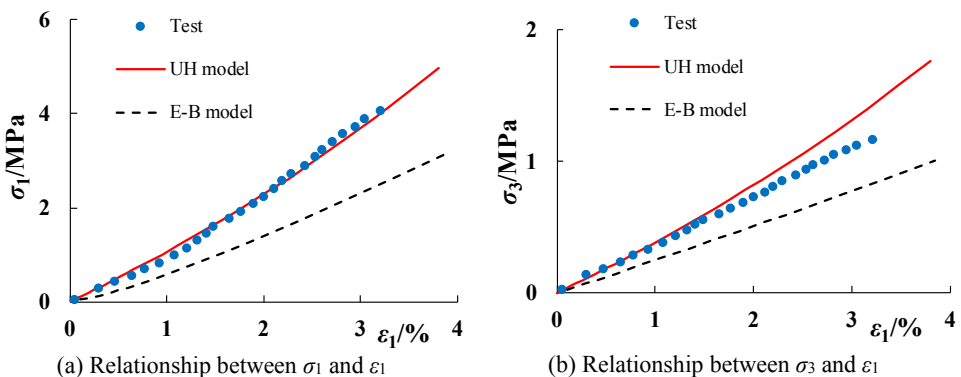


Fig. 3. Comparison between K0 compression test results and model predictions on LS sample

Fig. 4 gives the predictions of the constant stress ratio path test results on SS, with the same model parameters (given in Table 2 and 3). The predictions of UH model agrees well with the test results, and there is an evident deviation between the prediction of E-B model and test data for the p - ε_v relationship. For the q - ε_a relationship, both the two models cannot well predict the test results, but the predictions of UH model is still better.

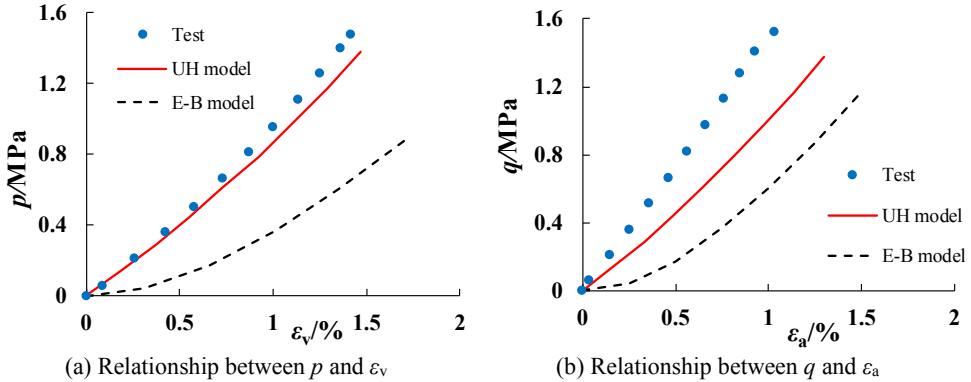


Fig. 4. Comparison between constant stress ratio test results and model predictions on SS sample

4. Application of stress-deformation analysis in Rockfill dam

In this section, the stress-deformation analysis of a core rockfill dam was performed by UH model and E-B model, the calculation results were then compared.

4.1 Overview of the dam and finite element model

The earth rockfill dam is 240 m high and the main material zones of the dam are rockfill, transition layer, inverted filter, clay core, ground soil, etc (Fig. 5).

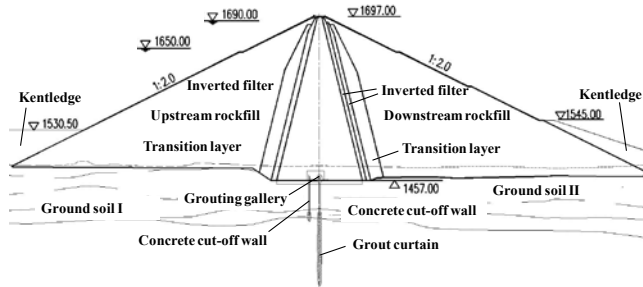


Fig. 5. Typical cross section of core rockfill dam

The model parameters for UH model and E-B model are listed in Table 4 and 5, which were determined with laboratory triaxial test. The concrete material part, such as cut-off wall, was assumed to be linear elastic, with elastic modulus $E=30$ GPa and Poisson ratio $\nu=0.17$.

Table 4 Parameters of Duncan's E-B model

Material	ρ /(g·cm ⁻³)	φ_0 /(°)	$\Delta\varphi$ /(°)	c /kPa	R_f	K	n	K_b	m
Rockfill	2.18	52.9	8.6	0	0.83	1155	0.49	456	0.25
Transition layer	2.25	52.2	7.3	0	0.81	1017	0.32	353	0.21
Inverted filter	2.17	51.2	7.8	0	0.74	1363	0.27	460	0.19
Clay core	2.21	30.4	0	96	0.79	402	0.44	204	0.42
Ground soil I	1.45	48.0	7	0	0.80	1100	0.34	360	0.32
Ground soil II	1.31	46.9	6.5	0	0.77	1075	0.33	343	0.27

Table 5 Parameters of UH model

Material	M	ν	κ	λ	N	Z	χ	m
Rockfill	1.700	0.11	0.05	0.169	1.887	0.567	0.28	0.25
Transition layer	1.699	0.20	0.05	0.192	1.900	0.272	0.43	1.37
Inverted filter	1.700	0.19	0.02	0.100	1.155	0.462	0.23	0.11
Clay core	1.487	0.40	0.01	0.100	1.170	0.593	0.44	0.55

There are 17780 nodes and 19363 elements in the finite element model. Most of the elements are 8-node hexahedrons, with a few others, such as 6-node pentahedrons and 4-node tetrahedrons for transition. Fig. 6 shows the three-dimensional finite element mesh of the dam.

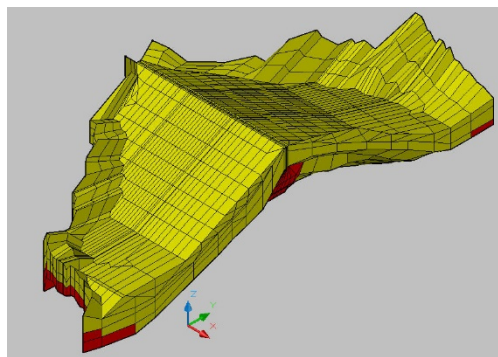


Fig. 6. Three-dimensional finite element mesh of dam body

4.2 Calculation results comparison between models

The settlement of rockfill dam is usually about 1%~2% of the dam height, and displacement ratio (maximum horizontal displacement along river to maximum settlement) is less than 0.25 [18]. In this paper, the settlement from E-B model accounts for 1.3 % of the dam height (including the ground soil, 50 m), with the displacement ratio 0.18, while the settlement ratio from UH model is 0.9%, with the displacement ratio 0.19, which are all within the reasonable range.

Fig. 7 shows the contour lines of major principal stress in the maximum cross section. Due to the so-called arch effect, the major principal stress in core wall is smaller than that in transition layer at the same elevation. The arch effect calculated by E-B model is stronger than that by UH model.

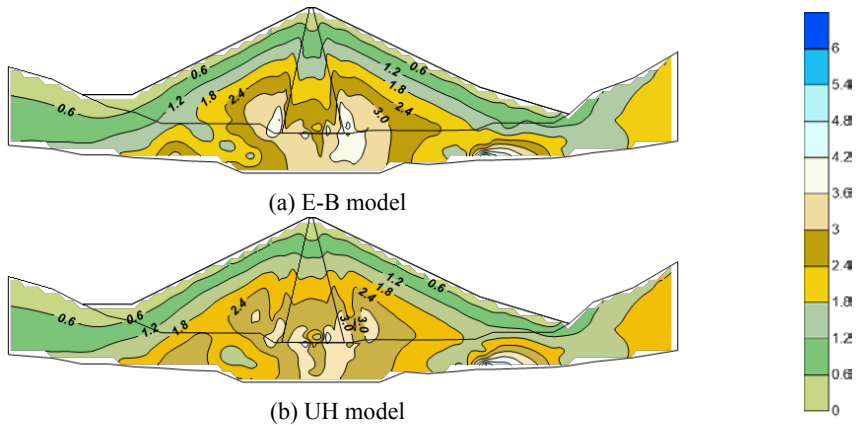


Fig. 7. Contour lines of major principal stress at completion (unit: MPa)

5. Discussion and Conclusions

In this paper, large-scale triaxial test and K0 compression test were conducted on two coarse-grained soils. The test results were predicted by UH model and by E-B model. By comparison, the UH model was better than E-B model in the prediction of these test data. Furthermore, the UH model can be well applied in the stress-deformation analysis of rockfill dam, as shown in section 4. The following conclusions are drawn:

(1) The stress-strain relationship of triaxial CD test can be relatively well described by both UH model and E-B model. However, for the coarse-grained soil with strain softening and shear dilatancy, E-B model has a poor ability of simulation, and it cannot reflect these behaviours while UH model can describe these properly.

(2) Simulations of K0 compression test and constant stress ratio test show that UH model can well predict stress-strain relations under complex stress paths, and the predictions are better than that of E-B model.

(3) Finite element analysis indicates that UH model can be applied in the calculation of earth rockfill dam.

Acknowledgements

The authors gratefully acknowledge the financial support from the National Key Research and Development Program of China (Grant No. 2017YFC0404801), the National Natural Science Foundation of China (Grant No. 51479052) and the Chinese Scholarship Council.

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