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Simulation of collapse settlement in rockfill material due to saturation

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Abstract

In the first impounding of rockfill dams, additional settlements occur in upstream side in saturated rockfills due to collapse phenomenon; even high rainy seasons can cause additional deformation in the dumped rockfills. Unfortunately these displacements are not taken into account in the conventional numerical models which are currently used to predict embankment dam behavior during impounding. In this paper to estimate these displacements, strain hardening-strain softening model in Flac is modified based on the laboratory tests, in which same impounding process in such dams is considered. Main feature of the model is reproduction of nonlinear behavior of rockfill material via mobilized shear strength parameters and using collapse coefficient to display induced settlement due to inundation. This mobilization of shear strength parameters associated with some functions for dilatancy behavior of rockfill are used in a finite difference code for both dry and wet condition of material. Collapse coefficient is defined as a stress dependent function to show stress release in the material owing to saturation. To demonstrate how the model works, simulation of some large scale triaxial tests of rockfill material in Gotvand embankment dam is presented and results are compared with those from laboratory tests, which are in good agreement. The technique could be used with any suitable constitutive law in other coarse-grained material to identify collapse settlements due to saturation.

Keywords: Collapse settlement, wetting, rockfill, numerical modeling, nonlinear behavior

1. Introduction

Collapse settlement is particularly important in upstream side of clay core rockfill dams during first impounding. Significant collapse settlement in such dams commonly occurs when the reservoir is filled. This can be up to 1% or more of the dam height giving total maximum settlements sometimes in excess of 2%. The amount of collapse settlement depends on the quality of the rockfill material and its compaction. With increasing use of poor grade rockfills, significant settlement due to this factor is becoming more common, which can cause instability in body of dam, damage to rigid structures on dam crest and loss of freeboard. Collapse settlement may also be important in road embankment if they become inundated or in reclaimed land on which buildings are to be constructed. There is a remarkable complexity in characterizing these fills due to their inelastic, nonlinear, and highly stress dependant behavior. Also, great difficulties are encountered in performing significant laboratory tests on such material. In practice, several relations permit a fairly accurate representation of this important feature of rockfill material. A satisfactory technique for its prediction is clearly important. Nobari and Duncan [1] pioneered the modeling of collapse settlement; they developed a technique closely tied to the hyperbolic model of Duncan and Chang [2] which made direct use of triaxial test results. Its essential features are described in Chapter 12 of Maranha das Neves [3]. Alawaji [4] carried out sets of oedometer and direct shear test on loess soils in which the initial dry density and the normal pressure at wetting and shearing were each varied in turn. Oedometer results indicate that collapse potential decreases with density and increases logarithmically with normal pressure.

An update version of hyperbolic wetting simulation procedure was formulated by Escuder et al. [5] and their methodology was applied to an embankment with 100 m height. Silvani and Bonelli [6] investigated the effects of buoyancy forces and the decrease in the coefficient of friction in a rockfill column maintained by vertical rigid walls which was progressively filled with water. This simulation was based on discrete element approach, and each individual particle

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was modeled separately. The simulation brought out the occurrence of local destabilizations which give rise to significant rearrangement in rockfill.

In this paper some simulations are conducted to model collapse settlement and non-linear stress-strain behavior of rockfill material observed in tested specimens. For simulation of collapse settlement, a technique is presented in which initial stresses in each element are decreased by stress release coefficient and also mechanical parameters of rockfill is changed from dry to wet parameters. Stress dependant coefficient for stress reduction is extracted from laboratory tests and shows how initial stresses decrease with wetting of rockfill material. Some other valid equations are also used for definition of mechanical parameters in Strain-Hardening/Softening model of software FLAC [7]. For instance, Vermeer and de Borst's relation [8] for frictional hardening is modified and implemented by use of a new coefficient.

2. Tested material

These tests were carried out on conglomerate rockfill samples from borrow area of the Gotvand embankment dam with 176 m height to investigate its mechanical properties [9]. The samples had 32 inches diameter and 32 inches height with three different gradation curves which are shown in Fig. 1. Specimens had maximum grain size 6 inches with various fine content (percent passing NO. 200 sieve) from 3% to 16%.

Two sets of test were carried out in this investigation; in the first set of tests, saturated specimens were consolidated under isotropic stress and then drained strain-controlled axial loading was applied, and each test was continued until the axial strain at the end of loading reached to 10%. This kind of tests was done on three different gradation of rockfill shown in Fig. 1. In other set of tests, initially dry samples were loaded under isotropic stress and then axial loading was applied until axial strain reached to 3%, then samples were saturated and deviatoric loading continued. This kind of test was only carried out on rockfill type B. Testes were performed with 3 different cell pressure σ3=0.1, 1 and 2 MPa for determination the effect of confining pressure on rockfill behavior. Results of performed tests and their simulations are presented in subsequent sections of this paper.

3. Model improvement

An appropriate model for rockfill material would be the model, which is able to predict peak shear strength, hardening behavior, stress dependant function as well as collapse phenomenon accurately. In fact, the model should precisely simulate stress and strain fields either in dry condition or after inundation. In this study, strain hardening-softening (SS) model in FLAC is used with some modification; For definition of collapse settlement and nonlinear behavior of rockfill material, some implementation is developed in the modeling, which its basic features includes stress dependent modulus, hardening and dilatancy behavior and relevant collapse parameters which are described in the following sections. This formulation is used with two sets of material parameters, one for dry material and another for wet material after saturation.

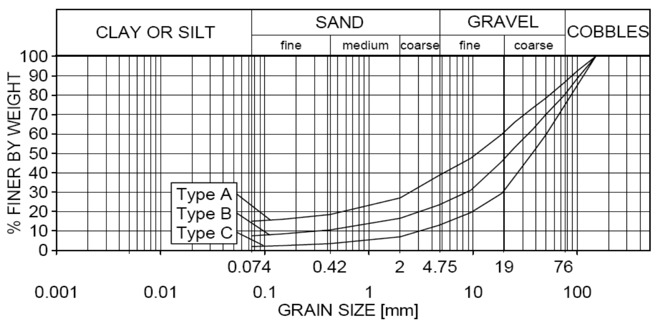


Fig. 1. Grain size distribution curve of tested rockfill [9].

3.1. Stress dependent elastic modulus

Dependency of elastic modulus on confining pressure is obvious in almost all geotechnical material. Hence, equation (1) which was proposed in hyperbolic (nonlinear elastic) model by Duncan and Chang [2] is applied in model for definition of this dependency. Many researchers have found the validity of hyperbolic stress dependant modulus relationship for various kinds of soil and rock under various test and practical conditions [5].

$$E = KP_a\left(\frac{\sigma_3}{P_a}\right)^n \tag{1}$$

Where E is Young's modulus, K is modulus number, Pa is atmospheric pressure and n is the exponent for stress dependent modulus.

Poisson's ratio (ν) is considered as a constant value, which associated with above mentioned elasticity parameters results in appropriate response in the elastic domain.

3.2. Failure envelopes and potential functions

The SS model allows representation of non-linear material hardening based on prescribed variations of the Mohr-Coulomb model properties as the function of the deviatoric plastic strain. The failure envelopes in the model are defined by following relations:

$$f^s = \sigma_1 - \sigma_3(1+\sin\varphi_m)/(1-\sin\varphi_m) + 2c_m\sqrt{(1+\sin\varphi_m)/(1-\sin\varphi_m)} \tag{2}$$

$$f^t = \sigma_m{}^t - \sigma_3 \tag{3}$$

Where f^s and f^t are, shear failure and tensile failure, respectively; σ_1 and σ_3 are major and minor principal stresses, and c_m, φ_m and σ_m^t are mobilized cohesion, mobilized friction angle and mobilized tensile strength.

The model has non-associated flow rule in shear and associated flow rule in tension with following plastic potential functions, respectively:

$$Q^s = \sigma_1 - \sigma_3(1+\sin\psi_m)/(1-\sin\psi_m) \tag{4}$$

$$Q^t = -\sigma_3 \tag{5}$$

Where ψ_m is mobilized dilation angle.

Thus, with above mentioned equations both hardening and softening behavior can be modeled depends on the increase or decrease in the mobilized parameters, respectively.

For rockfill material, which is the subject of study in this paper, frictional hardening and dilatancy behavior are defined based on mobilized friction and dilation angles, which are explained in the following sections.

3.3. Frictional hardening

Vermeer and de Borst [8] proposed Equation (6) for frictional hardening behavior of geotechnical material, in which mobilized friction angle (φ_m) depends on plastic strain (ε_p) and gradually increases to reach the peak friction angle:

$$\sin\varphi_m = 2\frac{\sqrt{\varepsilon_p \times \varepsilon_f}}{\varepsilon_p + \varepsilon_f}\sin\varphi_p \tag{6}$$

Where, ε_f is plastic strain at peak friction angle φ_p.

In this paper, the above equation is modified to equation (7) using a new coefficient, m and adding initial friction angel, φ_0:

$$\begin{aligned} \sin\varphi_m &= \sin\varphi_0 + 2\frac{\sqrt{\varepsilon_p{}^m \times \varepsilon_f{}^{2-m}}}{\varepsilon_p + \varepsilon_f}(\sin\varphi_p - \sin\varphi_0) \quad for\, \varepsilon_p \le \varepsilon_f \\ \sin\varphi_m &= \sin\varphi_p \qquad for\, \varepsilon_p > \varepsilon_f \end{aligned} \tag{7}$$

This coefficient and initial friction angle can be obtained from laboratory triaxial tests. Variation of mobilized friction angle with plastic strain for different value of m is shown in Fig. 2. Mobilized friction angle increases from initial value φ_0 to peak value, φ_p. For higher value of m, the mobilized friction angle increases less respect to plastic strain. When value of m is less than 1, both hardening and softening phenomena can be simulated with only one equation. It is notable that only stress path coefficient with value m>0 is acceptable for equation (7).

3.4. Dilatancy behavior

An equation for presenting the variable dilation angle was put forward by Rowe [10] so-called stress dilatancy equation. This is the desired equation between mobilized dilatation angle and the plastic strain as follows:

$$\sin\psi_m = \frac{\sin\varphi_m - \sin\varphi_{cv}}{1 - \sin\varphi_{cv}\sin\varphi_m} \tag{8}$$

$$\sin\varphi_{cv} = \frac{\sin\varphi_p - \sin\psi_p}{1 - \sin\varphi_p \sin\psi_p} \tag{9}$$

Where ψ_m and ψ_p are mobilized dilation angle and peak dilation angle, respectively. φ_{cv} is the critical state friction angle or friction angle of constant volume.

The mobilized dilatation angle is initially negative and increases with increase of plastic strain. To prevent this high value of negative dilation angle in small strains, following equation was presented by Søreide [11], which also used in this paper simultaneously with Rowe equation in modeling dilation behavior of rockfill material.

$$\sin\psi_m{}^* = \sin\psi_m\left(\frac{\sin\varphi_m}{\sin\varphi_p}\right)^P \tag{10}$$

Where, P is constant value and controls the shape of the curve. Variation of mobilized dilation angle with plastic strain is indicated in Fig. 3, for different values of power P. Contrary to the original Rowe formulation (P=0), the modified version gives dilation angle near zero for no friction mobilization. With increasing value of P, initially negative mobilized dilatation decreases in small plastic strains, even though all curves converge to peak dilation angle in higher mobilization levels.

3.5. Stress dependent peak friction and dilation angle

Due to this fact that maximum friction and dilation angles depend upon confining pressure, which also observed in the laboratory tests, equations (11) and (12) are implemented in the model. These stress dependent friction and dilation angles are used as maximum friction and dilation angle in equation (7) and (9), respectively:

$$\varphi_p = \varphi_s - \Delta\varphi\log\left(\frac{\sigma_3}{P_a}\right) \tag{11}$$

$$\psi_p = \psi_0\left(\frac{\sigma_3}{P_a}\right)^r \tag{12}$$

In the above equations, φ_p is maximum friction angle used in

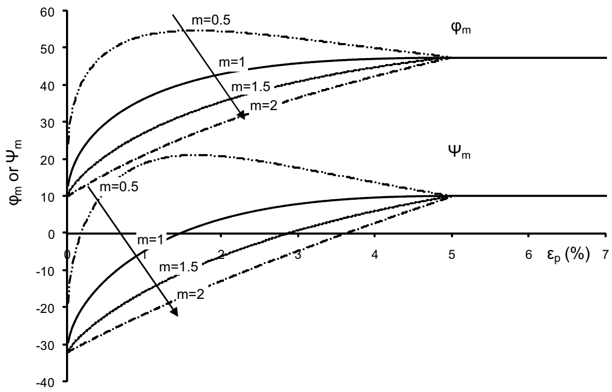


Fig. 2. Variation of mobilized friction and dilatation angle with variation of coefficient m

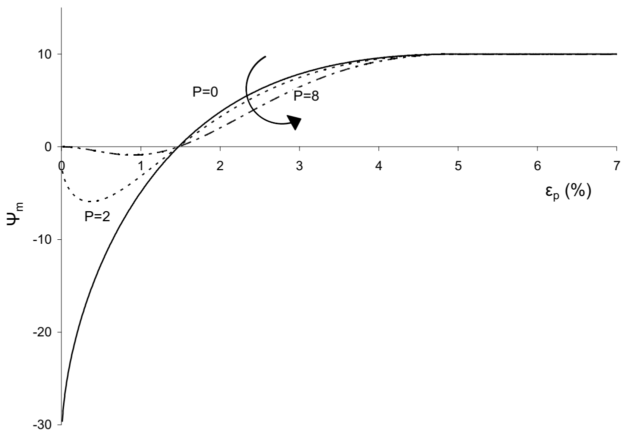


Fig. 3. Variation of mobilized dilation angle with variation of P

equation (7), σ_3 is the minor principal stress, φ_s is the angle of internal friction at σ_3= 100 KPa, Pa is atmospheric pressure, $\Delta\varphi$ is the reduction in friction angle for a 10 time increase in σ_3, ψ_p is maximum dilation angle used in equation (9), ψ_0 is dilation angle at σ_3=100 KPa and r is an exponent which indicates rate of dilation variation.

4. Modeling the collapse behavior

The functions introduced in the previous section have been applied for modeling stress-strain behavior in both dry and wet samples. However, to simulate wetting effects on dry samples, considering the collapse phenomenon in the model is a must. In the procedures like Nobari and Duncan's scheme [1], two separate analyses for dry and wet conditions of geomaterial are performed and then, differences of stresses in these two conditions are applied as nodal forces to each element of the dry analysis, which cause further deformation called as collapse settlement. In contrast, in the method proposed in this paper, only one dry analysis is required and the process of inundation is modeled afterward using a stress release coefficient, which results in precise change in the stress path.

Flooding rockfill specimens subjected to overburden pressures leads to a sudden stress release and settlement (collapse) attributed also to the breakage of particles due to rock weakening induced by wetting [12]; in other word, this drop in the stress state and developed settlement is due to sliding of soil grains, their rearrangement and even their breakage during saturation. In this paper, for recreation of collapse settlement, a stress release coefficient is defined to decrease stress components due to wetting. Indeed, according to Fig. 4 which is hypothetical stress path, when a dry rockfill media is submerged, internal stress decreased vertically from its initial value σ_t to collapse stress σ_c, therefore collapse coefficient is regarded as equation (13):

$$Csr = \sigma_c / \sigma_t \tag{13}$$

In the triaxial tests, which are investigated in this paper, collapse coefficient depends on confine pressure.

Dependency of collapse phenomenon to confine pressure has been observed experimentally and numerically by authors and other researchers [13, 14, 15]. In this research, this dependency is considered in the coefficient itself; for instance,

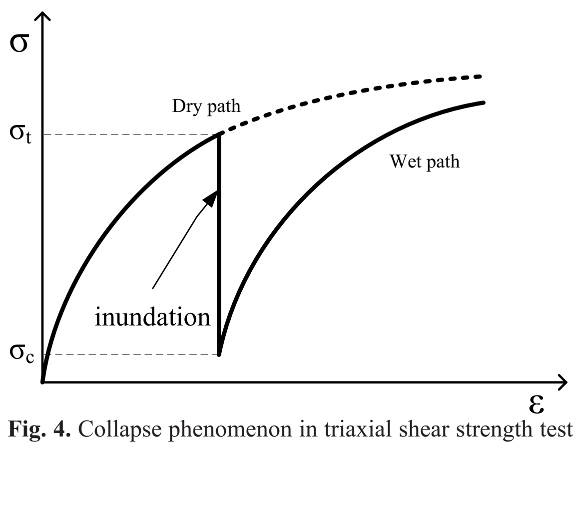


Fig. 4. Collapse phenomenon in triaxial shear strength test

Csr is defined as a stress dependent function, which can be extracted from laboratory tests. Thus, equation (14) is applied in numerical modeling, which multiplied by the current dry stresses value and brings new stress state:

$$Csr = \alpha + \beta\left(\frac{\sigma_3}{P_a}\right) \tag{14}$$

Where α and β are constant parameters which can be determined from laboratory tests.

For practical application of collapse coefficient, first of all, stress state of the geomaterial is calculated from numerical analysis in the dry state; then collapse coefficient is applied to the current stress state of those elements which are inundated now and strength and deformation parameters changes from dry to wet state and calculations continues to reach a final stress state corresponding to applied load

5. Numerical modeling

Numerical modeling of triaxial tests has been performed using code "FLAC" with the above mentioned modifications in strain hardening-softening model. FLAC is a bi-dimensional finite difference code (explicit scheme) which permits simulation of the behavior of soils, rocks, etc. The program is based on the Lagrangian calculation scheme, and each element behaves according to a prescribed stress–strain law as a response to applied forces and boundary restraints. Model correction in this study is done via internal programming language (Fish) which permits definition of functions, controlling calculation schemes and the nature of calculations (i.e., user-defined constitutive relationship). This feature makes FLAC very powerful for research purposes.

Triaxial test results are simulated by single zone axisymmetric formulation and appropriate boundary condition. As it was mentioned, SS model was used for simulation, which its great advantage is the possibility that the cohesion, friction, dilation and tensile strength may harden or soften after the onset of plastic yield. In the Mohr-Coulomb model, those properties are assumed to remain constant, whereas in SS model it is possible to define shear strength parameters as piecewise-linear functions of a hardening parameter, e.g. plastic shear strain. The code measures the total plastic shear strains by incrementing the hardening parameter at each time step and causes the model properties to conform to the user-defined functions. All presented equations are implemented in model using some functions in the fish environment.

Four sets of triaxial tests were carried out to identify stress-strain behavior of the rockfill material, three of them on wet samples and the last one on dry sample which was inundated during deviatoric loading. Model parameters for all samples are prepared and shown in Table 1. The parameters in this table were introduced in section 3 and 4 of this paper except c and ν which are cohesion and poison's ratio, respectively. As depicted in table 1, coefficient β is positive, which means that with higher confine pressure higher collapse coefficient exist in these tests. Also it can be seen from triaxial test results that in higher confine pressure, higher stress release occurs in the stress states.

In simulation of collapse test, first, initial stresses and confining pressure are applied to axisymmetric single zone

Table 1. Model parameters for simulation

Test type	Ordinary tests			Collapse tests	
Rockfill type	A	B	C	Dry	Wet
Density(gr/cm^3)	2.15	2.16	2.12	2	2.2
k	700	700	700	650	600
n	0.5	0.6	0.6	0.5	0.5
ν	0.15	0.15	0.15	0.15	0.15
C(KN/m^2)	60	50	40	70	30
ε_f(%)	5	5	6	6	8
m	1	1	1	1	1
φ_0 (˚)	10	10	10	10	15
φ_s (˚)	46	47.5	48	53	45
$\Delta\varphi$ (˚)	4.5	5	5	7	3
P	0.1	0.1	0.1	0.1	0.1
ψ_0 (˚)	5.9	8.7	13.8	14.2	6.7
r	-0.05	-0.45	-0.65	-1.58	-0.34
α	-	-	-	0.11	0.11
β	-	-	-	0.003	0.003

model. After exerting velocity in y direction as the axial loading, the model is permitted to be solved until axial strain reaches to 3%. At this time, collapse simulation is performed in the model; in fact, stress release coefficient is used to reduce all stress components of the element currently saturated and then for simulation of saturated media, wet parameters are used and simulations continues until axial strain reaches to 10%. This procedure is exactly similar to triaxial collapse test procedure in the laboratory.

6. Results of simulation and discussions

Deviatoric stress and volumetric strain versus axial strain curves obtained from laboratory tests for different gradation of rockfill material in various test conditions are compared with those from numerical modeling. Results of simulation on wet samples are shown in Fig. 5 to 7 and Fig. 8 shows simulation of collapse phenomenon. Figures show how the analytical curves of collapse settlement tests are similar to experimental results and the nonlinearity of stress-strain constitutive law is accurately followed by the rockfill material in laboratory tests.

6.1. Stress - strain behavior

The stress strain behavior of tested and modeled rockfill specimens are illustrated in Figs. 5.a to 8.a. Generally, it can be seen from these figures that with a decrease in fine content of rockfill, the peak stress and corresponding friction angle increase in modeling similar to those from laboratory tests. For example, at σ_3=2 MPa sample C exhibits a maximum deviatoric stress of 4.1 MPa at final axial strain of 10 % and corresponding friction angle is φ_s=48 degree, whereas in similar condition at sample B and A, maximum deviatoric stresses are 3.99 and 3.77 MPa and friction angles are 47.5 and 46 degree, respectively. It is obvious in the collapse tests (Fig. 8) that mobilized shear strengths before and after inundation are approximately like experimental results. Thus, the modified model is able to simulate stress field of the whole samples properly.

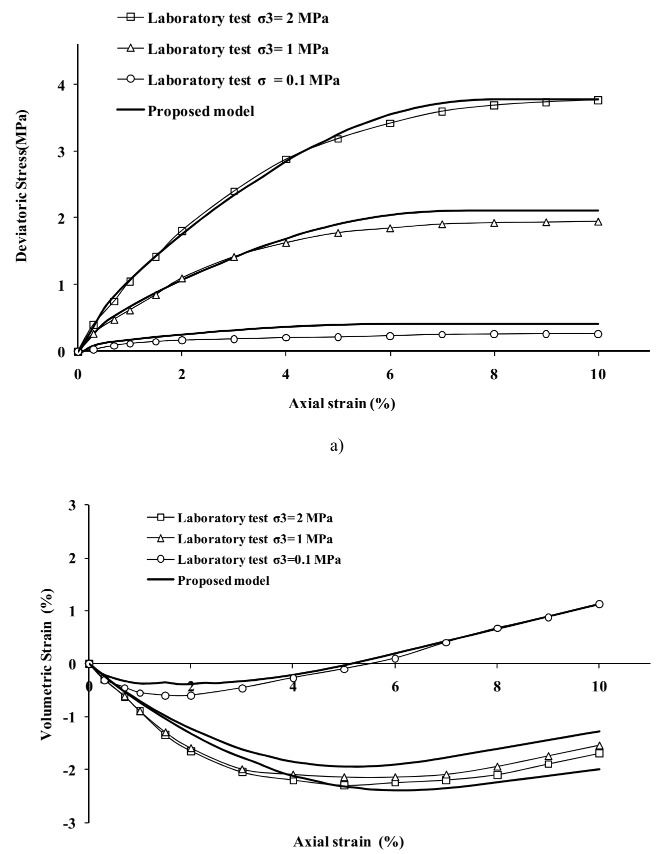


Fig. 5. Comparison of modeled and observed triaxial test results in rockfill type A

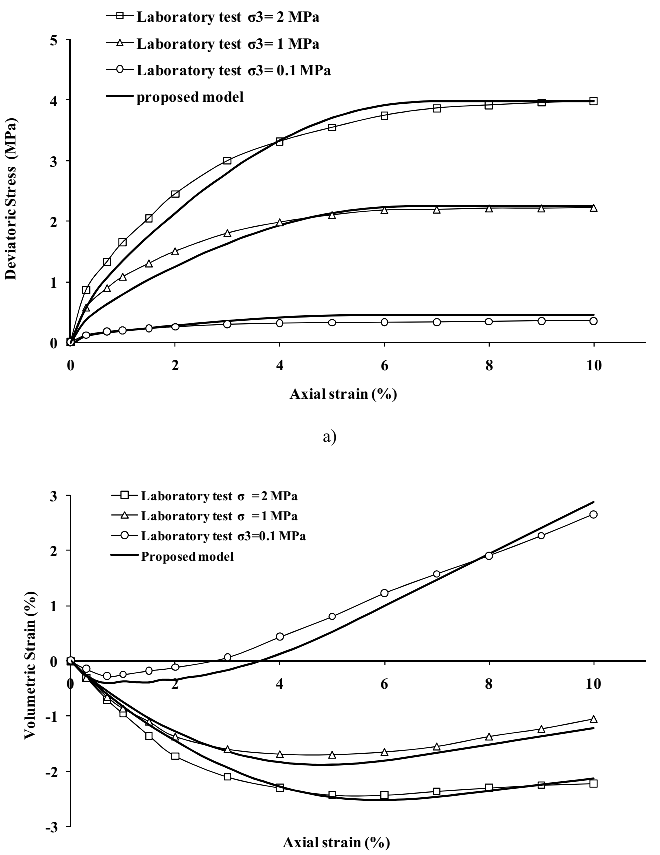
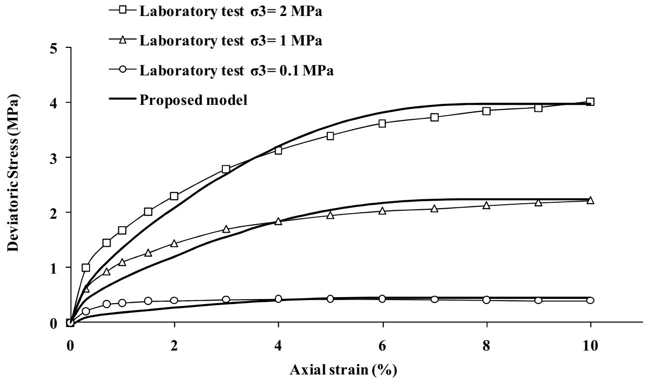


Fig. 6. Comparison of modeled and observed triaxial test results in rockfill type B



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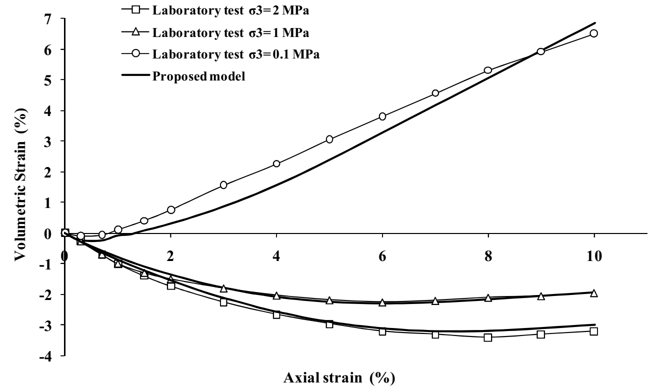
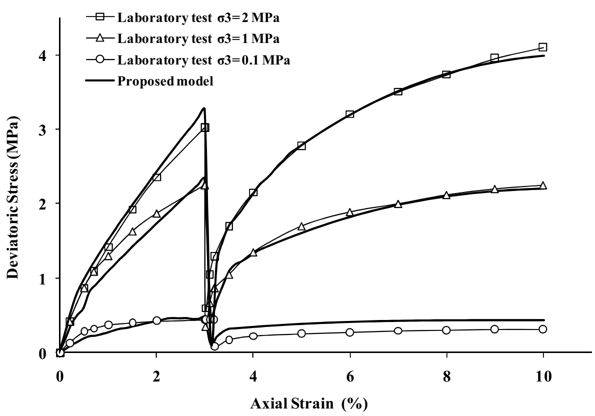


Fig. 7. Comparison of modeled and observed triaxial test results in rockfill type C



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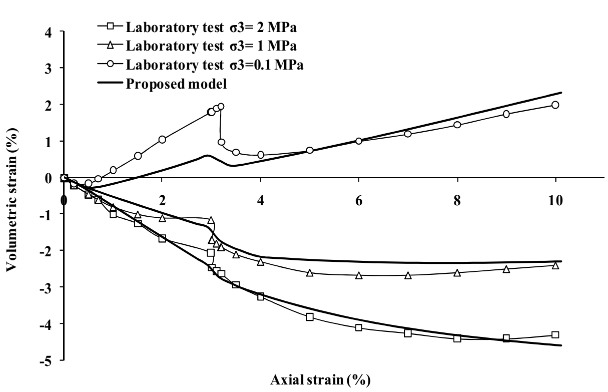

Fig. 8. Collapse phenomena in modeling compared with measurements from triaxial tests

6.2. Volumetric strain-axial strain behavior

The variations of volumetric strain versus axial strain are also presented in Fig. 5.b to 7.b for regular tests and Fig. 8.b for collapse test. The study of these figures indicate that as expected in lower confining pressure σ_3=0.1 MPa during the initial strains, the volume of specimens decreases slightly (the positive sign of volumetric strain indicates dilatancy behavior), while with further shearing, the opposite behavior is notable and specimens show an increase in volume. It is obvious from test results that increase in confining pressure limits the volumetric expansion of specimens. This propensity is captured by numerical modeling using confining stress dependent dilation angle by equation 12. It is clear from collapse test results that dilatancy behavior is being restricted through the specimen inundation; as a matter of fact, wet specimens show less dilation angle respect to dry samples. As can be seen in figures 5.b to 8.b, with using proposed equations in SS model, good agreement exists between laboratory results and simulations. Another outcome from Fig. 8.b is that although collapse coefficient is applied to the stress field, strain field is affected in such a way that negative or compressive volumetric strain occurs in the element.

7. Conclusion

The paper has attempted to provide an insight into modeling the various aspects of the engineering properties of rockfill material, with focus in collapse settlement phenomenon. This study allows accessing the phenomena that take place at the time of inundation in rockfill materials and therefore enables detailed study of collapse settlement in such materials. This kind of study is of immediate interest for dam engineers who wish to assess the settlements induced by the filling. The results, therefore, are expected to be useful in estimating nonlinear behavior and collapse settlements in the upstream shells of earth-core dams or road embankments which are likely to be submerged.

In order to model mentioned phenomenon, some features are applied to strain hardening-strain softening model in software Flac to simulate stress dependency, hardening behavior and collapse phenomenon in rockfill material. Results of modeling are compared with those from physical tests; in general, it is shown that results of the simulations are in good agreement with laboratory tests in all stages of loading both in strength and dilation behavior. From this adjustment this conclusion can be drawn that this modified model is appropriate to simulate the behavior of rockfill material both in dry condition and after inundation.

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