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Modeling Stress-Strain Behavior of Sand-EPS Beads Lightweight Fills Based on Cambridge Models

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ABSTRACT: A lightweight fill was produced by blending expanded polystyrene (EPS) beads and sands in proportions. Such formed granular geomaterials, known as sand-EPS lightweight fills, have potentials of being lightweight compared to traditional fills, thus are suitable for many infrastructure works where less overburdens are expected, e.g., utilities trench backfills. Consolidated drained triaxial compression (TC) tests were conducted on the lightweight fills to observe materials' stress-strain relationships, specifically, the stress-strain variations associated with the mixing ratios of EPS beads. EPS beads were incorporated into the mixtures based on their mass ratios over sands, i.e., 5‰, 15‰ and 25‰. It was found that confining pressures and mixing proportions played important roles in affecting the stress-strain behaviors of lightweight fills. The materials underwent shear contraction, which is related to the inclusion of EPS beads. The constitutive law of the lightweight fills was derived based on Cambridge model and revised Cambridge model, and verified by the test results.

INTRODUCTION

Lightweight fills posses an attractive advantage over general soil fills, i.e., low unit weight. Such property immediately offsets some overburdens, and thus mitigates the settlement or deformation of filled grounds. Potential end applications of the materials include embankments over soft ground or being widened, slope and retaining works, and many backfilling works. Furthermore, many lightweight fills are able to isolate or mitigate vibrations. As such, more and more researches are drawn onto the lightweight fills. At the very beginning, Horvath (1994, 1997) and Duškov (1997) applied expanded polystyrene (EPS) block geofoams to embankments and bridge approach embankments to reduce overburdens, isolate vibrations and resist frost attacks. The Japanese researchers initiated laboratory and pilot scale studies on lightweight soils containing EPS scraps or beads, geofoam materials, or foaming agents (Tsuchida 2001). Recently, Chinese researchers investigated the strength and deformation behavior (Liu et al. 2006) and dynamic behavior (Wang and Gao 2007) of lightweight fills containing silty soils, EPS beads and Portland cement. Sediment or dredge slurry was also used to form lightweight soils, which not only reclaims the slurry, but also stabilizes the pollutants in the slurry (Zhu et al. 2005). Besides the EPS materials and geofoams, rubber or tire scraps are also used to make lightweight geomaterials.

In this paper, EPS beads and construction sands were blended homogeneously to form nonstructural granular lightweight fills. The non-cementitious lightweight fills not only save the use of cement, but also are suitable for works where low strength and instant excavation are expected. The focus of the research is on the constitutive law reflecting the stress-strain behavior of the granular fills. Laboratory consolidation-drained triaxial compression (TC) tests were implemented on fills of different mixing proportions. The stress-strain relationships of the materials were observed and analyzed. Based on Cambridge and revised Cambridge models, a proportion-based constitutive model was derived and verified.

EXPERIMENTAL PROGRAM

Materials and Specimens

Materials include construction fine sands and EPS beads. The water content of sand is 5%. Its specific gravity is 2.62. The gradation curve of sands is shown in Fig. 1. It is classified as well-graded. EPS is a super light polymer, foamed from polystyrene resin and pre-puffed at 35-40 folds. The EPS beads are even and spherical, sizing around 2 mm (Fig. 2), with bulk density and specific gravity being 0.015g/cm^3 and 0.03, respectively.

The specimens were formed by incoporating EPS beads into sands at a mass ratio η of EPS beads over sands. Selected ratios are 5‰, 15‰ and 25‰. Scaled materials were mixed completely through air-mixing methods. The specimens were prepared in accordance with Chinese Standard for Soil Test Method (GB/T50123-1999). Mixtures were loaded into the rubber membrane hooped by a splittable mold. The relative density of mixtures were controlled at 0.5. Adverse pressures were loaded to saturate the specimens.

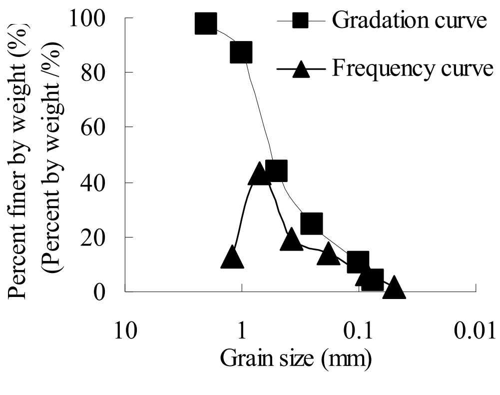

FIG. 1. Gradation curves of sands.

FIG. 2. Sand-EPS beads mixture.

Test Methods

Consolidation-drained TC tests were implemented in accordance with the test standard. The confining pressures σ_3 were 100, 200, 300 and 400 kPa. The compression velocity was 0.015 mm/min. Observations include deviatoric stress q, axial strain ε_a and volumetric strain ε_v.

DISCUSSION OF CONSTITUTIVE LAW

Establishment of Constitutive Law

Many constitutive models were proposed with regards to the stress-strain behavior of soils (Huang 1980, Yin 1988, Shen 1989, Li 2006). These models are largely divided into two types: elastic models and elasto-plastic models. The latter is more robust than the former to represent the soil behavior, e.g., soil hardening or softening characteristics, shear dilatancy and stress paths, and thus is more prevailing (Qu 1987, Yin 1988, Qian and Yin 1996, Li 2006). One of the classical elasto-plastic model is Cambridge model, which contains relatively less parameters, and is easy to be verified. Eqs. 1-2 present the strain differential equations of Cambridge model (Qu 1987).

$$d\varepsilon_v = \frac{-de}{1+e} = \frac{1}{1+e}\left[\frac{\lambda-k}{mp}\left(dq-\frac{qdp}{p}\right)+\lambda\frac{dp}{p}\right] \tag{1}$$

$$d\varepsilon_s = \frac{\lambda-k}{(1+e)mp}\left(\frac{dq}{m-q/p}+dp\right) \tag{2}$$

$$d\varepsilon_a = d\varepsilon_s + \frac{1}{3}d\varepsilon_v \tag{3}$$

where, ε_v denoting volumetric strain, ε_a denoting axial strain, ε_s denoting shear strain, *e* denoting initial void ratio, λ denoting the slope of normal consolidation line $v-\ln p$, *k* denoting the slope of over-consolidation line $v-\ln p$, *m* denoting lateral confinement modulus, *p* denoting average normal stress, *q* denoting general stress.

Eqs. 1-3 are combined to produce Eq. 4:

$$d\varepsilon_a = \frac{\lambda-k}{(1+e)mp}\left(\frac{dq}{m-q/p}+dp\right)+\frac{1}{3(1+e)}\left[\frac{\lambda-k}{mp}\left(dq-\frac{qdp}{p}\right)+\lambda\frac{dp}{p}\right] \tag{4}$$

Similarly, Eqs. 5-7 can be derived in accordance with revised Cambridge model (Qu 1987).

$$d\varepsilon_v = \frac{1}{1+e}\left[(\lambda-k)\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2+\left(\frac{q}{p}\right)^2}+\lambda\frac{dp}{p}\right] \tag{5}$$

$$d\varepsilon_s = \frac{\lambda - k}{1+e}\left[\frac{2\left(\frac{q}{p}\right)}{m^2 - \left(\frac{q}{p}\right)^2}\right]\left[\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2 + \left(\frac{q}{p}\right)^2} + \frac{dp}{p}\right] \tag{6}$$

$$d\varepsilon_a = \frac{\lambda - k}{1+e}\left[\frac{2\left(\frac{q}{p}\right)}{m^2 - \left(\frac{q}{p}\right)^2}\right]\left[\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2 + \left(\frac{q}{p}\right)^2} + \frac{dp}{p}\right] + \frac{1}{3(1+e)}\left[(\lambda - k)\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2 + \left(\frac{q}{p}\right)^2} + \lambda\frac{dp}{p}\right] \tag{7}$$

Sand-EPS bead lightweight mixtures are more complex than soils, which complicates the constitutive law of the lightweight mixtures. The classical soil constitutive models may not suit the stress-strain behavior of sand-EPS mixtures. For instance, as shown in Fig. 3, neither Cambridge model nor revised Cambridge model is able to reflect the stress-strain relationship of a specimen. It is seen that the strain is either over-estimated or under-estimated for Cambridge model and revised Cambridge model, respectively.

Nevertheless, it was noticed that the higher the EPS proportion η, the closer to the Cambridge model the curve; the less the EPS proportion η, the closer to the revised Cambridge model the curve. The stress-strain curve basically fluctuates within these two models depending upon the EPS proportions. It was thus targeted to derive a model comprising Cambridge and revised Cambridge models to simulate the observations in Fig. 3.

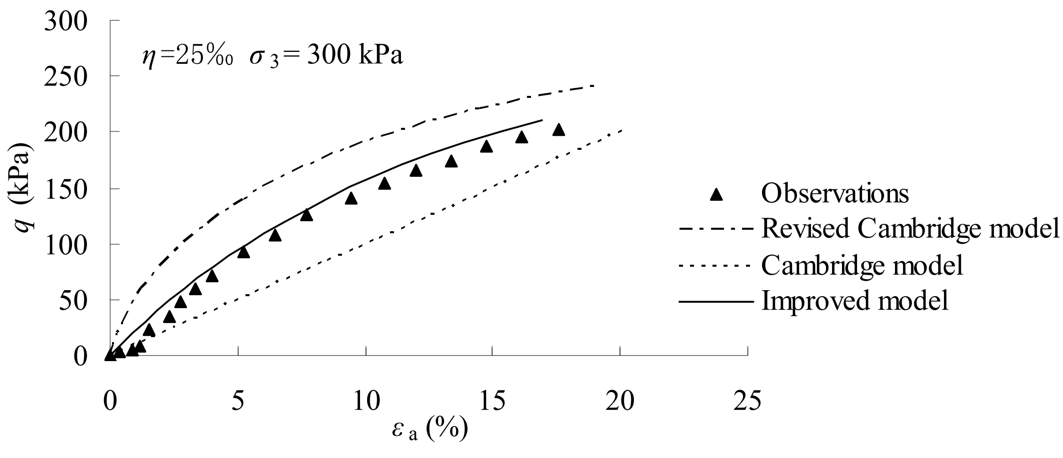

FIG. 3. Deviatoric stress-axial strain simulations.

A lightweight fill unit was divided into two parts, denoted by $\zeta(\eta)$ and $(1-\zeta(\eta))$. Stress-strain increments of the former part was depicted using revised Cambridge model, i.e., Eqs. 5-7; the latter part was depicted using Cambridge model, i.e., Eqs. 1-2 and 4. It was assumed that the boundary conditions are stabilized in shears. Thus, the total strain increments ($d\varepsilon_v$, $d\varepsilon_s$, $d\varepsilon_a$) are equal to the sum of strain increments of two parts, i.e., $d\varepsilon_v^A$, $d\varepsilon_s^A$, $d\varepsilon_a^A$ for part $(1-\zeta(\eta))$, and $d\varepsilon_v^B$, $d\varepsilon_s^B$, $d\varepsilon_a^B$ for part $\zeta(\eta)$, as described in Eqs. 8-10.

$$d\varepsilon_v = (1-\zeta(\eta))d\varepsilon_v^A + \zeta(\eta)d\varepsilon_v^B \tag{8}$$

$$d\varepsilon_s = (1-\zeta(\eta))d\varepsilon_s^A + \zeta(\eta)d\varepsilon_s^B \tag{9}$$

$$d\varepsilon_a = (1-\zeta(\eta))d\varepsilon_a^A + \zeta(\eta)d\varepsilon_a^B \tag{10}$$

The model described in Eqs. 8-10 comprises Cambridge and revised Cambridge models, and is able to depict the complex stress-strain behavior by taking into account the EPS proportions.

Merge Eqs. 1, 2 and 4 and Eqs. 5-7 into Eqs. 8-10, the following strain increments are obtained.

$$d\varepsilon_v = [1-\zeta(\eta)]\frac{1}{1+e}\left[\frac{\lambda-k}{mp}\left(dq-\frac{qdp}{p}\right)+\lambda\frac{dp}{p}\right]+\zeta(\eta)\frac{1}{1+e}\left[(\lambda-k)\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2+\left(\frac{q}{p}\right)^2}+\lambda\frac{dp}{p}\right] \tag{11}$$

$$d\varepsilon_s = [1-\zeta(\eta)]\frac{\lambda-k}{(1+e)mp}\left(\frac{dq}{m-q/p}+dp\right)+\zeta(\eta)\frac{\lambda-k}{1+e}\left[\frac{2\left(\frac{q}{p}\right)}{m^2-\left(\frac{q}{p}\right)^2}\right]\left[\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2+\left(\frac{q}{p}\right)^2}+\frac{dp}{p}\right] \tag{12}$$

$$\begin{aligned} d\varepsilon_a = {} & [1-\zeta(\eta)]\left\{\frac{\lambda-k}{(1+e)mp}\left(\frac{dq}{m-q/p}+dp\right)+\frac{1}{3(1+e)}\left[\frac{\lambda-k}{mp}\left(dq-\frac{qdp}{p}\right)+\lambda\frac{dp}{p}\right]\right\}+ \\ & \zeta(\eta)\left\{\frac{\lambda-k}{1+e}\left(\frac{2\left(\frac{q}{p}\right)}{m^2-\left(\frac{q}{p}\right)^2}\right)\left(\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2+\left(\frac{q}{p}\right)^2}+\frac{dp}{p}\right)+\frac{1}{3(1+e)}\left[(\lambda-k)\frac{2\left(\frac{q}{p}\right)d\left(\frac{q}{p}\right)}{m^2+\left(\frac{q}{p}\right)^2}+\lambda\frac{dp}{p}\right]\right\} \end{aligned} \tag{13}$$

where, $\zeta(\eta)$ denoting the ratio of material volume relying on revised Cambridge model over the entire volume, which is associated with the EPS proportion η. The more the EPS contents, the less the $\zeta(\eta)$ value. According to the TC results, the model parameters are figured out in Table 1.

Table 1. Parameters of Constitutive Model.

η /‰	m	k	λ	$\zeta(\eta)$
5	1.4723	0.0353	0.0869	0.8100
15	1.1107	0.0918	0.2386	0.6500
25	0.9084	0.1399	0.3952	0.5200

The solid line in Fig. 3 represents a model curve plotted in terms of Eq. 13 and parameters in Table 1. It is seen that the solid line is superior to the Cambridge models to simulate the observations.

Verification of Model

Deviatoric Stress-Axial Strain Relationships

Verifications were implemented by plotting strain increments (Eqs. 11-13) and test observations in a chart, as shown in Figs. 4 and 5. The solid lines represent the model curves, which basically simulate the observations of specimens subject to test stress conditions. Under high confining pressures, the model curves deviate a bit from the observations, slightly under-estimating strains. Nevertheless, the improved strain increments are suitable for simulating deviatoric stress-axial strain behavior of sand-EPS beads lightweight fills.

Volumetric Strain-Axial Strain Relationships

The volumetric contraction is presumed negative. As shown in Figs. 4 and 5, sand-EPS beads lightweight fills underwent shear contraction throughout the TC tests. When the confining pressures σ_3 are 100 and 400 kPa, the model curves well simulate the observations. When the EPS proportions are relatively low, the model curves are also robust to simulate the observations. When the confining pressures σ_3 are 200 and 300 kPa, the volumetric strain simulations are weak. This may source from the bias occurred to the observation of volumetric strain of EPS beads. Anyhow, the simulations are superior to that directly given by Cambridge models, and are basically acceptable for describing the volumetric strain-axial strain relationships.

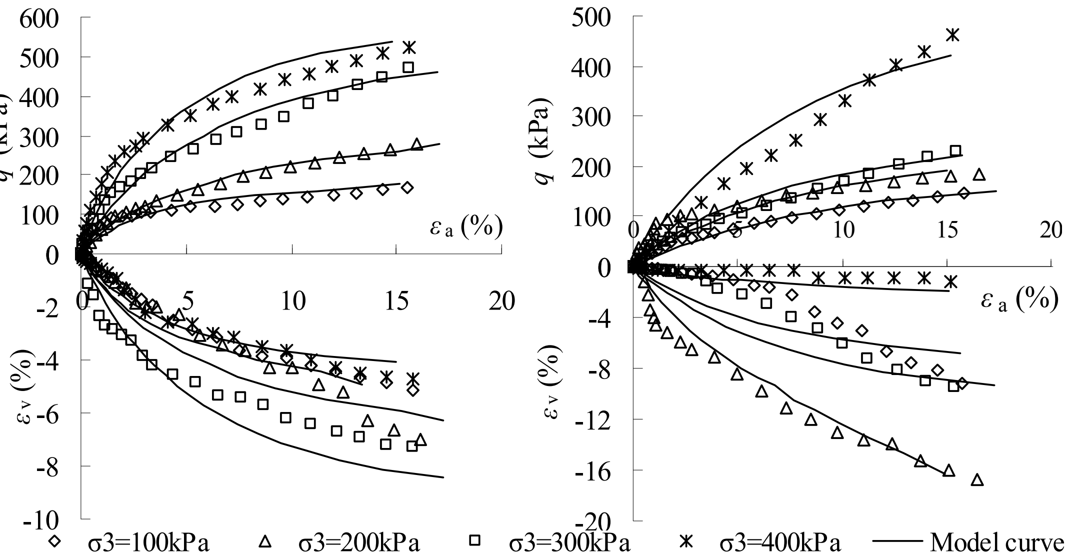

FIG. 4. q - ε_a - ε_v simulations (η=5‰). **FIG. 5. q - ε_a - ε_v simulations (η=15‰).**

Combine the above stress-strain simulations, the proposed model was verified suitable for depicting the constitutive laws of sand-EPS bead lightweight fills, and thus usable for estimating the stress, strain, settlement and strength of practical works.

CONCLUSIONS

Through consolidation-drained TC tests, sand-EPS bead lightweight fills of three mixing proportions were researched about their deviatoric stress-volumetric strain-axial strain relationships. Based on TC test results and analyses, a proportion-based model was derived depicting the constitutive laws of lightweight fills in terms of Cambridge and revised Cambridge models. The model was verified according to the TC observations.

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