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Laboratory tests on geosynthetic-encapsulated sand columns

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ABSTRACT

Granular columns have been introduced into engineering practice to improve the bearing capacity and reduce settlement of sand column in a weak or soft soil. The improvement can be enhanced by encapsulating the column with tensile resistant material. The improvement depends on the confinement offered by the surrounding soil, the reinforcing material and the granular column material. In this study, the extent of improvement for a sand column subjected to constant confining pressures is studied through laboratory experiments. A series of triaxial compression tests were carried out in laboratory to investigate the response of sand columns encapsulated by geotextiles. The tests consisted of triaxial compression tests on sand columns with two different densities and encapsulated by sleeves fabricated from three different geotextiles. The increase in deviatoric stress, the reductions in volumetric and radial strains, and the increase in confining pressure generated by the encapsulating reinforcement were measured and analyzed. The mobilized pseudo-cohesion and friction angle corresponding to various axial strains are analyzed to interpret the reinforcing effect. The experimental results are compared with data obtained from analytical method reported in the literature.

1. Introduction

Although construction on soft soils is most commonly undertaken using basal reinforcement, with or without prefabricated vertical drains (Rowe and Li, 2005; Briançon and Villard, 2008; Ghazavi and Lavasan, 2008; Li and Rowe, 2008; Rowe and Taechakumthorn, 2008; Chen et al., 2008; Bergado and Teerawattanasuk, 2008; Basudhar et al., 2008; Kazimierowicz-Frankowska, 2007) there is growing interest in the use of granular columns constructed by filling a cylindrical column with granular material. This technique has been introduced into engineering practice to improve the bearing capacity and reduce settlement of sand column foundations resting on the weak soil (Bergado et al., 1991, 1992; Raithel et al., 2002). The improvements on bearing capacity via granular columns are achieved through the inclusion of a stronger granular material. In response to a vertical load, an expanded granular column will squeeze the native soil, and result in an additional confining pressure onto the column. That leads to an increase in the stiffness and strength of granular column. However, insufficient lateral support at shallow column depth (top portion) frequently causes bulging failure at the top portion of the column (Hughes and Withers, 1974; Madhav and Miura, 1994). Therefore, reinforcement on granular columns, especially over the top few meters, is needed to provide lateral support to enhance the lateral confinement of column. The reinforcement can be achieved by enveloping a granular column with a flexible fabric or by placing horizontally laminated reinforcing sheets on the granular column either in full or partial height (Rao and Bhandari, 1977; Alamgir, 1989; Ayadat and Hanna, 1991; Cai and Li, 1994; Madhav et al., 1994; Broms, 1995; Nods, 2002; Raithel et al., 2002; Kempfert, 2003; Sharma et al., 2004). Geosynthetic-encased sand columns were successfully used to found a dike in very soft soil for land reclamation (Raithel et al., 2002).

The reinforcing effects on sand columns have been verified by laboratory triaxial compression tests performed on sand columns reinforced with horizontal disks or external encapsulation (Broms, 1977; Gray and Al-Refai, 1986; Chandrasekaran et al., 1989; Al-Joulani, 1995; Ashmawy and Bourrdeau, 1998; Haeri et al., 2000; Ayadat and Hanna, 2005; Sivakumar et al., 2004). Test results show that the reinforcement on sand columns increases the peak strength and the axial strain at failure, and reduces the loss of post-peak column strength. However, there is a dearth of literature on the analytical study of reinforced granular columns. Murugesan and Rajagopal (2006) presented numerical analysis results based on finite element technique that investigated the effect of external encapsulation on the confining pressure on a sand column embedded in soft clay. Raithel and Kempfert (2000) proposed analytical and numerical methods to study the performance of geosynthetic-encased sand columns. Kempfert (2003) studied the effectiveness of the encased columns for several projects. He

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reported on the improvement of column strength for encased granular column over stone column. He also showed a significant improvement on the column strength that increases with the geotextile stiffness.

Wu and Hong (2008) and Wu et al. (in press) reported an analytical method that investigated the stress–strain relation of granular columns reinforced with horizontal disks or external encapsulation. In this paper, we will report the results obtained from a series of laboratory triaxial compression tests conducted on sand columns encapsulated by geotextiles. The effect of encapsulating sleeve on the deviatoric stress increase and the volumetric reduction were investigated. The contributions of the sleeve toward the confining pressure were analyzed. The mobilized pseudo-cohesion and friction angle corresponding to various axial strains were studied to interpret the reinforcing effect. Finally, the experimental results are compared with analytical results reported in the literature (Wu et al., in press).

2. Experimental program

2.1. Materials used for testing

The experimental program consisted of performing triaxial compression tests on 140 mm high × 70 mm diameter dry sand samples encapsulated in geotextile sleeves. The soil used was uniformly graded angular quartz sand with a specific gravity (G_s) of 2.65, effective size (D_{10}) of 0.7 mm, uniformity coefficient (C_u) of 1.23 and coefficient of gradation (C_c) of 0.84. According to the Unified Soil Classification System this soil can be classified as SP. The maximum and minimum dry unit weights of the soil are 16.48 and 13.73 kN/m^3, respectively. The triaxial compression tests were carried out on sand with 60% and 80% relative densities.

A reinforcing sleeve was fabricated by sewing a piece of geotextile sheet 140 mm × 240 mm into a sleeve 140 mm in height and 70 mm in diameter. A sewn overlap seam 20 mm wide using a polyester thread with the stitch pattern shown in Fig. 1 was adopted for this study to create the sleeve. Sand columns encapsulated in geotextile sleeves sewn in this manner produced relatively uniform results when loaded under compression. The seam did not produce localized failure at the seam nor generate very stiff seam that affects the sleeve stiffness in the vertical direction.

Three different types of geotextile were used to make the reinforcing sleeves designated as GT1, GT2 and GT3. The tensile load–strain behavior of the test geotextiles was determined by performing wide width tension test on 200 mm wide and 100 mm long specimens. To include the sewing effect on the extension behavior of the sleeve in the triaxial compression test, the tensile test specimen was fabricated by sewing two pieces of geotextile, each of 200 mm wide and 60 mm long, into a 200 mm × 100 mm specimen as shown in Fig. 1. The tensile test was carried out using a strain rate of 0.24 mm/min. This strain rate is much slower than that used in the ASTM specification (10 ± 3%/min), but approximates the circumferential strain rate of the geotextile sleeves in the triaxial compression tests.

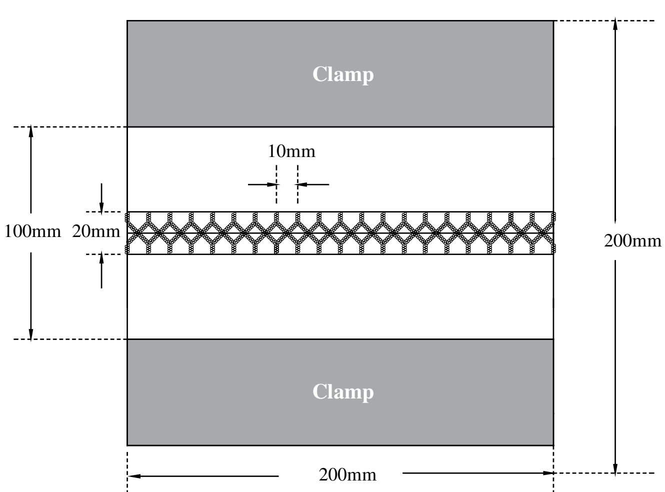

Fig. 1. The stitch pattern of the sewn geotextile.

2.2. Test procedure

A triaxial test apparatus that can accommodate 7 mm diameter samples was used for all the tests. All the sand columns, reinforced and unreinforced, were prepared in dry condition within a split cylinder mould. A 0.3 mm thick rubber membrane was used for chamber pressure application. The reinforced sand column was prepared by inserting a geotextile sleeve into the cylinder mould and the rubber membrane. The sand was filled in the cylinder mould or geotextile sleeve by pluviation from a tube to make unreinforced and reinforced columns. Constant pluviation heights were pre-calibrated to produce desired relative densities. The sand placement was divided into 5 layers, and the uniformity of the deposited sand was checked at each layer. The tests were carried out at five different confining pressures of 20, 50, 100, 200 and 500 kPa. The axial load was applied on the sample through a proving ring at an axial strain rate of 0.3%/min. The change in volume of the specimen was obtained by measuring the amount of water expelled or entering the pressure chamber.

3. Experimental results

The experimental results for test materials (geotextiles and soil), unreinforced and reinforced sand columns with two relative densities are presented in this section.

3.1. Test geotextiles and soil

The tensile force–strain relation of the three test geotextiles is presented in Fig. 2. The three geotextiles showed significant tensile degradation at large strains. The tensile force–strain curves for the seamed specimens deviated from those obtained from plain geotextiles as the strain increased. The secant stiffness at 1, 5 and 10% strains for the three seamed geotextiles GT1, GT2 and GT3 are 39, 31 and 22 kN/m; 52, 35 and 28 kN/m; and 83, 51 and 44 kN/m, respectively. The tensile strengths of the three seamed geotextiles are 3.84, 6.20 and 8.77 kN/m, respectively.

The triaxial compression test results for the unreinforced sand specimens are depicted in Fig. 3. Sample bulging was observed after sand columns exhibited some axial deformations; maximum horizontal or lateral strain occurred at the mid-height of the samples. Sand column contracts more and expands less while subjected to higher confining pressure. A higher confining pressure also extends the contraction behavior of a column to a greater axial strain. The deviatoric stress–strain curves showed no significant residual strength, which may be attributed to the angular shape of the test sand particles. The soil specimens exhibited continuous increasing in volumetric strain and bulging deformation with the increase in axial strain.

Assuming that the test sand has no cohesion, the line tangent to stress Mohr's circle and passing through the origin of the $\sigma - \tau$ coordinate gives the friction angle of the sand. The peak friction angle evaluated by using this method varies with the confining pressure. The peak friction angle for 60% relative density sand decreases from 38.5° for a confining pressure of 20 kPa to 36.6° for a confining pressure of 500 kPa. For 80% relative density sand the