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Failure mechanism of geosynthetic-encased stone columns in soft soils under embankment

Jian-Feng Chen [a], Liang-Yong Li [a], Jian-Feng Xue [a, b, *], Shou-Zhong Feng [a, c]

[a] Department of Geotechnical Engineering, School of Civil Engineering, Tongji University, 1239 Siping Road, Shanghai 200092, China
[b] Geotechnical and Hydrogeological Engineering Research Group (GHERG), Federation University Australia, Victoria 3842, Australia
[c] Wuhan Guangyi Transportation Science and Technology Co., Ltd, Wuhan 430074, China

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ABSTRACT

The reaction of geosynthetic-encased stone columns (GECs) in soft soils under embankment loading was modeled with an indoor physical model test and numerical models using three dimensional and two dimensional finite element methods. The experimental and three dimensional numerical modeling results showed that the failure of the GECs is caused by the bending of the columns rather than shear. Three dimensional finite element analysis showed that the distribution of unbalanced lateral loading acting on the columns is symmetric about a 'hinge point' above the plastic hinge, rather than triangle or uniform distribution. An equivalent shear resistance model of the GECs is proposed based on the distribution of the unbalanced lateral loadings on the wall. The stability of the embankment was analyzed in two dimensional finite element method by transforming the columns into equivalent soil walls using equivalent bending resistance and equivalent shear resistance methods. It was found that results from equivalent bending resistance method is closer to the estimations from the three dimensional analysis, which agrees with the bending failure mechanism of the GECs. It is suggested that one more row of such columns may be required to provide higher lateral resistance in the soils in front of the toe to improve the stability of the embankment.

1. Introduction

Stone columns have been more widely used as a cost and energy efficient, and environmental friendly method for soft soil treatment. For situations when the undrained shear strength of soil is too weak, stone columns may lose their effectiveness as the surrounding weak soils may not provide enough confinement to the columns, which may result in bulging or crushing failure of the columns at the upper section of the columns (Hughes et al., 1975). In that case, geosynthetic (i.e. geotextile or geogrid) encased stone columns (GECs) overcome the shortcomings and provide lateral confinement to the stone materials to improve the bearing capacity of the soils (Raithel et al., 2005; Yoo, 2010; Zhang et al., 2012; Dash and Bora, 2013; Elsawy, 2013; Wu and Hong, 2014).

The columns, such as sand compaction columns, stone columns, and deep mixed columns, can fail due to bending, sliding, rotation, shearing, tension, or a combination of the failure modes under embankment load (Kivelo and Broms, 1999; Han et al., 2005; Kitazume and Maruyama, 2006, 2007; Han, 2012; Zhang et al., 2014). Shear failure is the most common failure mode for sand compaction and stone columns (Abusharar and Han, 2011). Han et al. (2005) found that rotational failure of deep mixed columns is dominant under road embankment based on the findings from numerical analysis. Through centrifuge tests, Kitazume and Maruyama (2006, 2007) found that the deep mixed columns could fail under bending. They indicated that the area replacement ratio of deep mixed columns influences the bending failure significantly and sliding failure might happen to shorter columns. Based on numerical modeling results, Zheng et al. (2010) suggested that rigid columns (e.g. concrete piles) are more prone to bending failure rather than shear failure under embankment loading.

However, there is very limited literature on the failure mode of GECs under embankment load. This paper evaluates the stability of a road embankment built on GECs reinforced soils using laboratory testing, and three dimensional and two dimensional finite element analyses. The performance and failure mode of the encased stone columns under embankment loading are extensively studied.

* Corresponding author. Tel.: +61 3 51226448.
E-mail address: jianfen.xue@federation.edu.au (J.-F. Xue).

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2. Laboratory testing

The authors carried out a test on geotextile-encased stone column reinforced soft soils. The model was built in a 1200 mm × 400 mm × 800 mm (length × width × height) tank as shown in Fig. 1 to simulate a 3.5 m high embankment at a scale of 1:25 (model size to full size). The full size embankment will be modeled with finite element method in the later sections. Kaolin was mixed to the water content of 100%, which is well above the liquid limit (54.2%), to make the 400 mm thick of foundation soil. A standard medium sand from Pingtan island in the eastern China Sea, Fujian Province, China was used in the test for sand cushion. The sand has been widely used in China for research purpose (Zhou et al., 2012). The sand used in the test is well graded with mean size of 0.34 mm and coefficient of uniformity of 1.542. The sand was compacted to the density of 15.3 kN/m^3 to construct the 50 mm thick sand cushion between the embankment and the foundation soil. The friction angle of the sand was about 27.3° based on direct shear tests at the dry density of 15.6 kN/m^3. Silica sand with diameters ranging from 2 to 4 mm was compacted to the density of 17.2 kN/m^3 to construct the stone columns. The mean size of the silica sand was 2.64 mm, and the coefficient of uniformity was 1.861. The friction angle of the silica sand was 36.7°. Non-woven geotextile was used to encase the silica sand. The tensile strength of the geotextile was 0.42 kN/m based on tensile tests on six 20 mm × 20 mm samples. The stiffness of the geotextile was 4.0 kN/m at 5% tensile strain. Steel weights were used to construct the embankment. The weight was 380 g each, with the dimension of 5 cm × 5 cm × 2 cm (length × width × thickness).

The Kaolin slurry was poured into the tank and consolidated for two weeks under the self-weight with double side drainage, and the drains were remaining closed during the rest of the test. After consolidation, the undrained shear strength of the soil was obtained at 5.2 kPa using a miniature cone penetrometer developed by Chen et al. (2012). The layout of the GECs is shown in Fig. 1. To construct the sand columns, a 32 mm outer diameter steel pipe with 0.4 mm wall thick was driven into the soil and an auger extruder was used to remove the soil in the pipe. The non-woven geotextile was sewn to form a 32 mm diameter and 400 mm long tube, and placed into the steel pipe to form the geotextile casing. Silica sand was poured into the casing and compacted in layers of 50 mm to the designed density (17.2 kN/m^3). After constructing the sand column, the steel pipe was carefully pulled out. Two piezometers (K_1 and K_2) were installed below the middle section of the embankment to the depth of 200 mm as shown in Fig. 1(a) and (b) to monitor excess pore water pressure dissipation during construction.

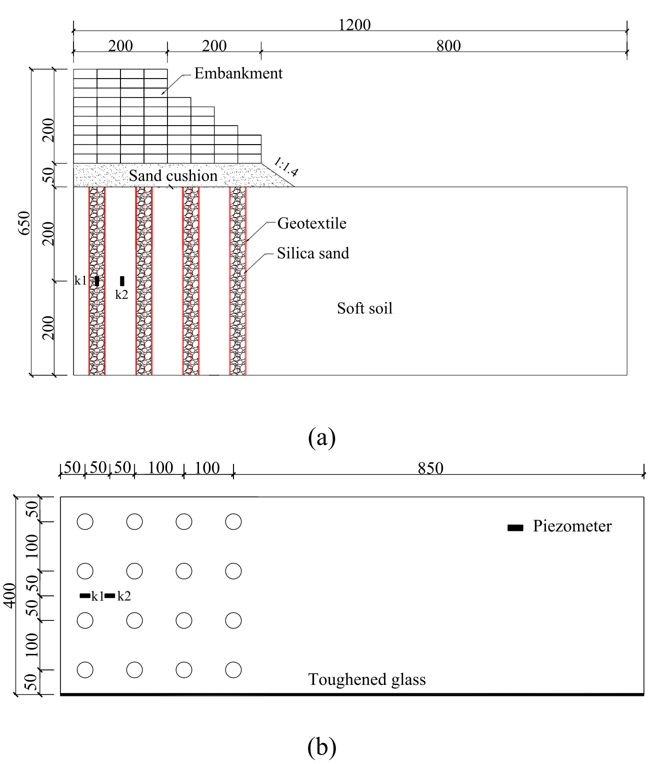

Fig. 1. Dimensions of the laboratory model embankment on GECs reinforced soft soils (units are in mm): (a) section view; (b) plan view.

The embankment was constructed using steel weights in three stages (stages 1–3) as shown in Fig. 2. The embankment was constructed in three stages using steel weights. Each loading stage was 10 min (time to place the weights) followed by a resting period for the dissipation of excess pore water pressure, which is about of 30–40 min based on the monitored data from piezometers shown in Fig. 3. After the construction of the embankment, sand bags with equivalent pressure of 21 kPa were applied on the embankment surface to fail the structure (stage 4 in Fig. 2). The deformation of the ground before the last loading stage is shown in Fig. 4. The shape of the deformed ground shows that large settlement has occurred below the embankment, with heave in the soils in front of the toe. The largest curvature of the deformed ground contour is located below the toe of the embankment, where is also the last column located.

After removing the weights, the soils were left for 2 weeks with the drains open to solidify the Kaolin for excavation. After the excavation, it was found that the columns were bended, with the largest deflection been observed in the column at the toe of the embankment as shown in Fig. 5, and the closer to the centerline of the embankment, the less the deflection in the columns. This agrees with the ground contour shown in Fig. 4. The deformation of the columns shows that bending failure occurred in the columns near the toe.

3. Three dimensional finite element model

The physical model tested in the laboratory is relatively small in size, therefore the stress level encountered in situ and the stress variations in the stone columns and geotextiles during the test cannot be fully studied. To further investigate the reaction of the GECs under road embankment, a 3.5 m high road embankment with 10 m wide on top was simulated with a three dimensional finite element code Z_Soil developed by the Swiss Federal Institute of Technology. The software has been used with great success in forensic analysis and designs (Truty et al., 2008). The side slope of the embankment is 35°. The thickness of the underneath soft soil is 10 m with the groundwater table located at ground surface. The encased stone columns are 10 m long and 0.8 m in diameter, which are installed at square pattern with center to center spacing of 2.5 m. A 1.25 m road section was simulated in the three dimensional model to consider the symmetrical structure as shown in Fig. 6. The material properties are shown in Table 1. The soils are modeled with Mohr Coulomb model and the geotextiles was simulated with isotropic linearly elastic perfectly plastic membrane materials embedded in the Z_Soil software. The geotextile has a tensile stiffness of 2000 kN/m, Poisson's ratio of 0.3 and tensile strength of 70 kN/m. The soft soil and geotextile interface was modeled with linear elastic-perfectly plastic interface model. The interface coefficient was 0.7 in the analysis.

Fig. 6 shows the finite element mesh used in the analysis. The mesh consists of 6132 elements. Impermeable boundary conditions