Stability of geosynthetic-encased stone columns under embankment load

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ABSTRACT: Stone columns have been widely used as a cost and environmental friendly method for soft soil treatment. For situations when the undrained shear strength (c_u) of soil is too weak, stone columns may lose their effectiveness as the surrounding weak soils may not provide enough confinement to the columns. 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. GECs have been successfully used for road embankment or dike foundations. However, the stability of GECs in soft soil under embankment is still far from clear. In this study, centrifuge model tests were performed on embankments supported by GECs with different reinforcement stiffness and encasement lengths. The test results show that, for the ordinary stone columns (OSCs) composite foundation, columns under embankment are apt to bulge with no shear slip trend to incur significant settlement because the stones are squeezed into the soft soil. For the GECs with half-length encasement composite foundation, columns at the centerline of the embankment are mainly compressed and incur obvious bulging deformation at the junction of the encased and un-encased portions. Meanwhile columns under embankment slope and near slope shoulder can tilt and bend largely due to insufficient bending stiffness of columns. The above two factors lead to the greatest settlement of the GECs with half-length encasement composite foundation. For the GECs with full-length encasement composite foundation, columns at the centerline of the embankment suffer vertical compression deformation while those under the embankment slope bend outwards. The bending deformation and settlement decrease with an increase of encasement stiffness. Full length encasement with high stiffness is required for the GECs composite foundation embankment to reduce the settlement and to ensure stability in practical application.

Keywords: embankment; geosynthetic-encased stone column; centrifuge model test; soft soil

1 INTRODUCTION

Stone columns have been widely used as a cost and environmental friendly method for soft soil treatment. When the undrained shear strength c_u of soil is too weak ($c_u < 15$kPa), the stone columns may deform laterally into the surrounding soft soil due to the insufficient lateral support of soil (Raithel *et al.*, 2004). In that case, geosynthetic can be used to encase the stone columns to provide additional lateral confinement and improve the bearing capacity of stone columns.

Currently, many researchers (Ali *et al.*, 2012; Ali *et al.*, 2014; Murugesan and Rajagopal, 2009; Dash *et al.*, 2013; Gu *et al.*, 2015; Yoo *et al.*, 2015) get insight into the failure mechanism of geosynthetic encased stone columns (GECs) through scaled-down physical model tests. However, the above researches are mainly on the rigid load condition, and very little information is available on the study of GECs under the embankment load. Chen *et al.* (2015a, 2015b) performed indoor model tests and numerical analyses to study the stability of GECs under embankment load and found that the GECs fail in bending, but the results are lack of validation of in situ test.

In the present study, centrifuge model tests were performed on embankments supported by GECs with different reinforcement stiffness and encasement lengths to evaluate the stability of GECs under the in situ stress condition.

2 CENTRIFUGE MODEL TESTS

2.1 *Model design*

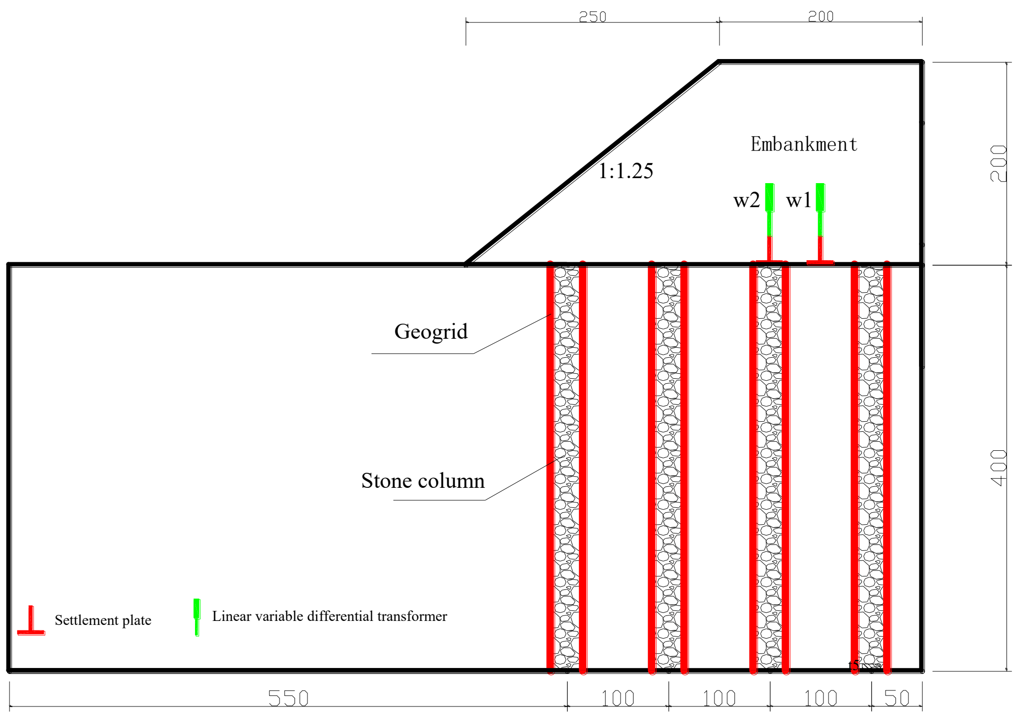
The prototype embankment, 5 m high with side slope of 1:1.25, was constructed over a 10 m thick soft clay. The encased stone columns are 10 m long and 0.8 m in diameter, which are installed at square pattern with center to center spacing of 2.5 m.

The centrifuge tests were performed on Tongji University's geotechnical centrifuge machine with a capacity of 150 g.t and a 3 m radius arm. A strong box with internal dimension of 900 mm length, 700 mm width and 700 mm depth was used in the present study. The scale factor (N) applied in the tests was 25. Figure 1 shows the cross-section of the centrifuge model. Four groups of centrifuge model tests were conducted shown in Table 1.

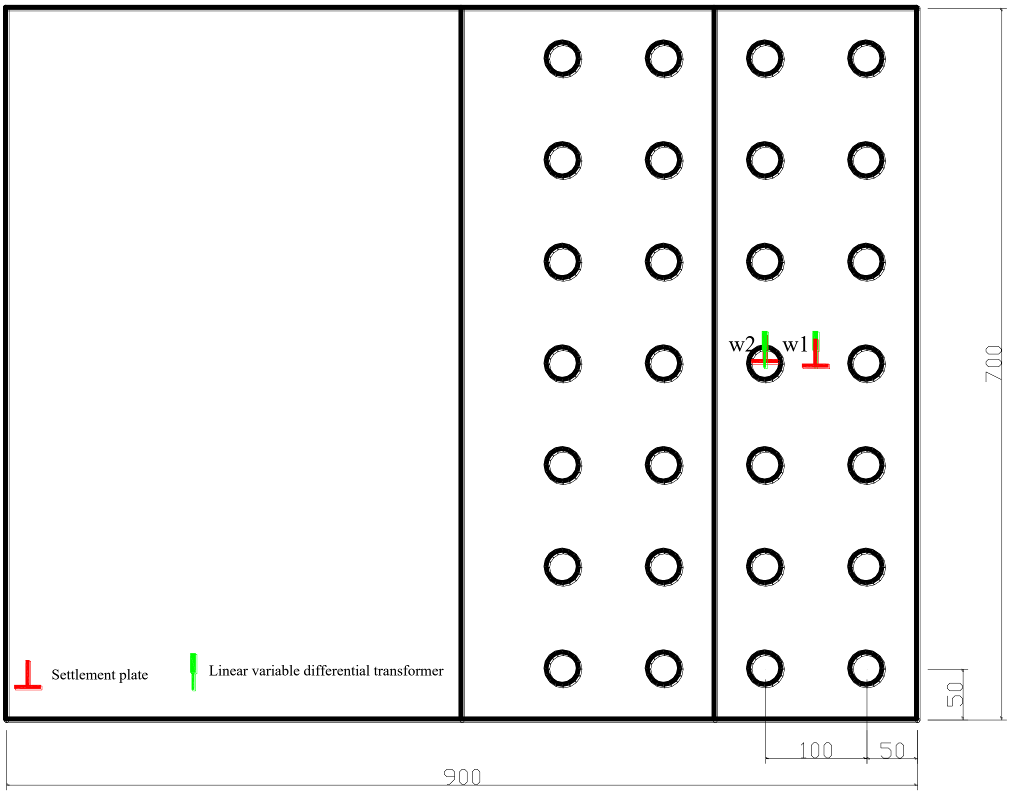
Table 1. Summary of model tests

Test No.	Test description	Column length, L (mm)	Column diameter, D (mm)	Reinforcement length, x (mm)	Geosynthetic
T1	OSCs	400	32	0	/
T2	GECs-G1F	400	32	400	G1
T3	GECs-G2F	400	32	400	G2
T4	GECs-G2H	400	32	200	G2

OSCs, ordinary stone columns with no encasement; GECs-G1F, geosynthetic stone columns encased with full length G1; GECs-G2F, geosynthetic stone columns encased with full length G2; GECs-G2H, geosynthetic stone columns encased with half-length G2.



(a) Section view



(b) Plan view

Figure 1. Dimensions of the centrifuge model embankment on GECs reinforced soft soils (units: mm)

2.2 *Materials and preparation*

The soft soil bed was made up of fully saturated remoulded kaolin clay. The liquid limit and plastic limit of the kaolin clay are 54.2% and 34.3% respectively. The effective internal friction angle of the kaolin clay is 27.7°, as determined by the consolidation undrained test.

The sand used for embankment with particle sizes in the range of 0.5-2 mm. The maximum and minimum dry densities of the sand are 2.4 and 1.8 g/cm^3 respectively. The friction angle of the sand was 35°, obtained through direct shear test.

Silica sand with particle sizes ranging from 2.5 to 3 mm was compacted to the density of 1.75g/cm^3 to construct the stone columns. The maximum and minimum dry densities of the sand are 1.85 and 1.60 g/cm^3 respectively. The mean size of the silica sand was 2.64 mm, and the coefficient of uniformity was 1.891. The friction angle of the silica sand was 38°, obtained through direct shear test.

Two kinds of commercially available polymers were used to encase the stone columns in the present study, namely G1 and G2. The tensile strength properties of G1 and G2 determined from the wide-width tensile tests. The ultimate tensile strength and stiffness of G1 at 5% strain are 13.6 and 132kN/m respectively and the values corresponding to prototype dimensions at 25 g are 340 and 3300kN/m respectively. The ultimate tensile strength and stiffness of G2 at 5% strain are 2.5 and 18kN/m respectively and the values corresponding to prototype dimensions at 25 g are 62.5 and 340kN/m respectively. Furthermore, the ultimate tensile strength and stiffness values of the prototype corresponding to the two polymers fall in the range of products used in practical application.

2.3 *Test procedure*

A thin layer of petroleum grease was applied to the box walls, followed by two thin polythene sheet strips to reduce the friction between the strong box and the soil. Then the fully saturated remoulded kaolin clay slurry was poured into the strong box and consolidated on the centrifuge machine for three hours. After consolidation, the undrained shear strength of the soil was obtained at about 5.6kPa by a miniature cone penetrometer developed by Chen et al. (2012). After the preparation of bed, a hollow stainless steel pipe with 32 mm outer diameter and 0.4 mm wall thick was driven into the clay and an auger extruder was used to remove the clay in the pipe. Then the geosynthetic encasing with the same diameter was put into the pipe. Silica sand was poured into the casing and compacted in layers of 50 mm to the designed density (1.75g/cm^3). After placing each layer of silica sand, the casing pipe was lifted up gently to a height such that a minimum overlap of 15mm between the bottom of the casing pipe and the silica sand within the casing pipe was maintained. The procedure was repeated until the entire height of the stone column was formed.

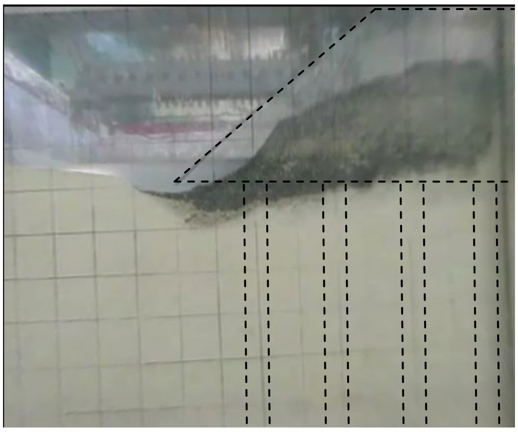
Then, as shown in Fig. 1, Two linear variable differential transformers (LVDTs) were installed to measure the settlement on soft soil (w1) and column (w2).

The embankment fill was constructed to 200 mm height in one time and the acceleration is increased to 25g in 5 min and lasted for 10 min.

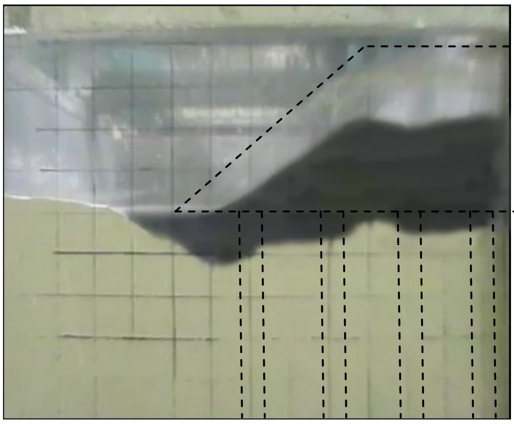
3 MODEL TEST RESULTS

3.1 *Settlement*

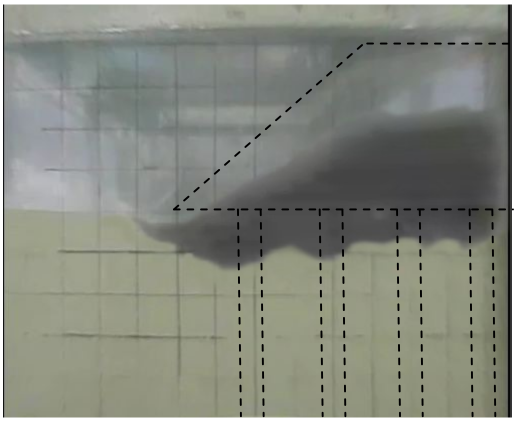
Figure 2 presents the deformation of embankments at the end of 25g acceleration. The dashed lines sketch out the contours of original embankments and columns. It can be seen that settlement of the GECs-G2H composite foundation embankment is the greatest. The embankment is almost all sank into the foundation soil. This is attributed to the tilting and bending of the columns under the embankment slope and near the slope shoulder. The actual settlement of the OSCs composite foundation embankment is great, but the kaolin clay slurry near the box wall covered the bottom of the embankment fill making the embankment fill looks inclined. Settlements of the GECs-G1F and GECs-G2F composite foundation embankment are small relatively and the settlement decreases with the increasing of stiffness of the encasement.



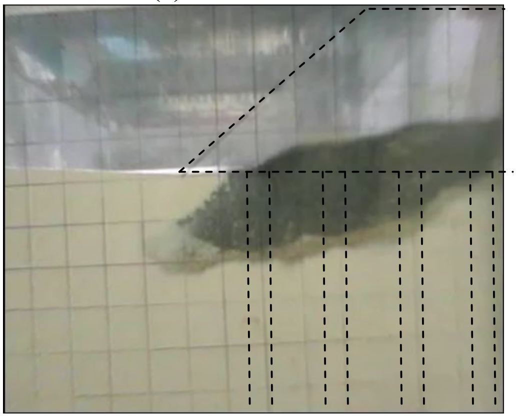
(a) OSCs



(b) GECs-G1F



(c) GECs-G2F



(d) GECs-G2H

Figure 2. Sketches of embankment deformation

Settlement on the top of columns and soil measured by the linear variable differential transformers are shown in Figure 3. It can be observed that settlement on the top of columns is smaller than that on the soil and decreases with the increasing of stiffness of encasement. Furthermore, the settlement of GECs-G2H composite foundation is the largest. As shown in Table 2, at the end of rest period, the settlement on the top of GECs-G2H is about 79mm. However, the settlement on the top of OSCs, GECs-G1F and GECs-G2F are about 45mm, 12mm and 25mm respectively, which are 57%, 15% and 32% to that of GECs-G2H. Meanwhile, the different settlement between soil and column of the GECs-G2H composite foundation is the smallest and is the same with the OSCs composite foundation. The different settlement of OSCs, GECs-G2H, GECs-G2F and GECs-G1F are about 7mm, 7mm, 21mm and 26mm respectively at the end of rest period.

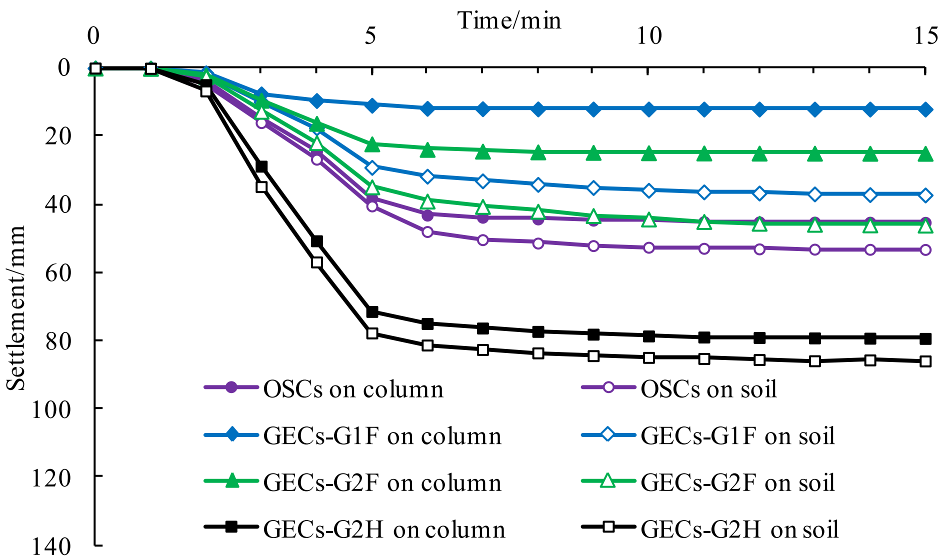


Figure 3. Settlement on the top of columns and soil

Table 2. Settlement on the top of columns and soil

Type	Settlement on soil/mm		Settlement on column/mm		Different settlement/mm	
	At the end of construction period	At the end of rest period	At the end of construction period	At the end of rest period	At the end of construction period	At the end of rest period
OSCs	45	52	40	45	5	7
GECs-G1F	28	38	11	12	17	26
GECs-G2F	38	46	23	25	15	21
GECs-G2H	78	86	71	79	7	7

3.2 *Failure mode*

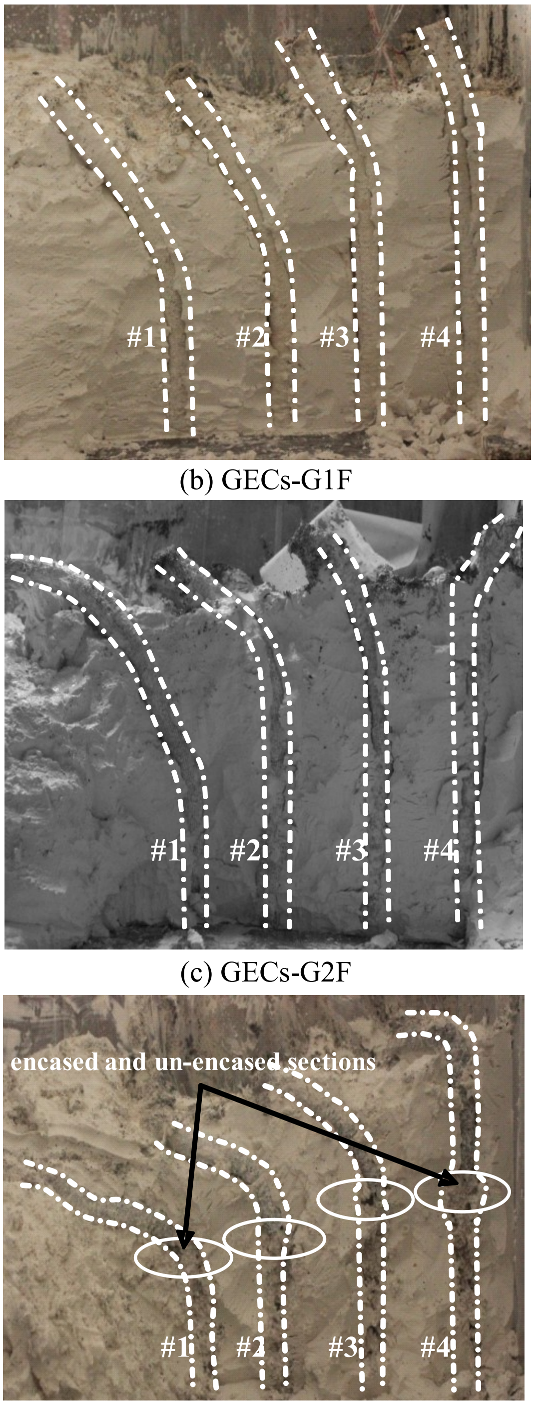
As the settlement of GECs-G1F and GECs-G2F are much smaller than that of OSCs and GECs-G2H at 25g condition, to investigate the stability of GECs-G1F and GECs-G2F further, increased the acceleration to 50g and lasted for 10 min. Figure 4 shows the deformation of OSCs, GECs-G1F, GECs-G2F and GECs-G2H after removing the soil around the columns. The dashed lines sketch out the contours of deformed columns. Figure 4(a) shows that, as the stones squeezed into soft soil which cannot provide sufficient lateral confining pressure, OSCs under embankment are apt to bulge to incur significant settlement. Bending can be observed at the upper portion of the column 1 under the embankment slope, but there is no shear slip trend of the composite foundation. Although the bending stiffness of OSCs is low, when the OSCs bulging, the lateral load provided by the embankment decreasing, the OSCs composite foundation not necessarily failed by shear sliding.

Figure 4(b) and Figure 4(c) show that, for the GECs-G1F and GECs-G2F composite foundation, the column 4 at the centerline of embankment is mainly compressed and bends outwards slightly, column 3 bends outwards obvious relatively, while column 1 and column 2 under the embankment slope bend outwards significantly. Furthermore, the bending deformation of columns decreases with the increasing of stiffness of the encasement. The above phenomena are consistent with the findings of authors (Chen *et al*., 2015; Chen *et al*., 2015) by indoor model tests and numerical analyses.

Figure 4(d) shows that, for the GECs-G2H composite foundation, the column 4 at the centerline of embankment is compressed and incurs obvious bulging deformation at the sections of the encased and un-encased portions. The reason is that the load on the top of columns is transferred to the lower portion of columns so that the bulging within the un-encased section increased. Meanwhile, columns under embankment slope and near slope shoulder can tilt and bend outwards largely due to insufficient bending stiffness of columns and lateral load provided by the embankment. The above two factors lead to the greatest settlement and deformation of the GECs-G2F composite foundation.

As described above, encasement stiffness and length have influence on the settlement and stability of the composite foundation under embankment. Columns encased with high stiffness geosynthetic are better than that with low stiffness obviously. However, as the lateral load of embankment and insufficient bending stiffness of columns, columns reinforced with half-length encasement are even worse than that with no encasement. As a result, full length encasement with high stiffness is required for the GECs composite foundation embankment to reduce the settlement and to ensure stability in practical application.

(a) OSCs



(b) GECs-G1F

(c) GECs-G2F

(d) GECs-G2H

Figure 4. Deformation of columns

4 CONCLUSIONS

Four groups of centrifuge model tests were performed on embankments supported by GECs with different reinforcement stiffness and encasement lengths to evaluate the stability of GECs. It can be concluded that:

(1) For the OSCs composite foundation, columns under embankment are apt to bulge with no shear slip trend to incur significant settlement because the stones are squeezed into the soft soil.

(2) For the GECs with half-length encasement composite foundation, columns at the centerline of the embankment are mainly compressed and incur obvious bulging deformation at the junction of the encased and un-encased portions. Meanwhile columns under embankment slope and near slope shoulder can tilt and bend largely due to insufficient bending stiffness of columns. The above two factors lead to the greatest settlement of the GECs with half-length encasement composite foundation.

(3) For the GECs with full-length encasement composite foundation, columns at the centerline of the

embankment suffer vertical compression deformation while those under the embankment slope bend outwards. Furthermore, the bending deformation and settlement decrease with an increase of encasement stiffness.

(4) Full length encasement with high stiffness is required for the GECs composite foundation embankment to reduce the settlement and to ensure stability in practical application.

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