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Time effect of pile-soil-geogrid-cushion interaction of rigid pile composite foundations under high-speed railway embankments

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Abstract. Centrifuge model tests were used to simulate pile-raft composite foundation and pile-geogrid composite foundation with different pile spacing for researching the time effect of negative skin friction of rigid piles in high-speed railways. The research results show that the negative skin friction has a significant impact on the bearing capacity of composite foundation. Pile-raft composite foundation has higher bearing capacity compared to pile-geogrid composite foundation to reduce the effect of negative skin friction on piles. Both the foundation settlement and negative skin friction have significant time effect. The distribution of skin friction can be simplified as a triangle along the pile. The neutral point position moves deeper in the post-construction stage at larger pile spacing. For pile-geogrid composite foundation, the setting of pile-cap affects the position of neutral point in the post-construction stage. Reinforced cushion with geotextile may promote the better performance of cushion for transmitting the loads to piles and surrounding soils. Arching effect in the cushion of the composite foundation is a progressive process. The compression of the rigid piles contributes less than 20% to 25% of the total settlement while the penetration of the piles and the compression of the bearing stratum below the pile tips contribute more than 70% of the total settlement. Some effective measures to reduce the settlement of soils need to be taken into consideration to improve the bearing capacity of pile foundation.

Keywords: embankment engineering; negative skin friction; rigid pile composite foundation; pile-cushion interaction; centrifuge model test; time effect

1. Introduction

The post-construction settlement of foundation plays a significant role in the embankment design and safe operation of high-speed railways (Wang *et al.* 2014 and Wang *et al.* 2015). Since the requirements of post-construction settlement of high-speed railway embankment are generally much stricter than those in highways and constructional engineering (British Standard 8006 2010, Railway Technology Research Institute 2001, Nordic Geotechnical Society 2004, Marchi *et al.* 2006 and Ministry of Railways of People's Republic of China, Code for design of high-speed railway 2014), various measures for foundation reinforcement, such as rigid pile composite foundation (RPCF), are adopted to reduce the settlement of weak foundation (Abushara *et al.* 2009, Chen *et al.* 2010, Wang *et al.* 2015 and Wang *et al.* 2016). At the same time, it is not a trivial task to calculate the RPCF settlement (Fenton *et al.* 2002, Han *et al.* 2002, Toshihiro *et al.* 2005, Shideh *et al.* 2010 and Mutsumi *et al.* 2011), which is significantly affected by the negative skin friction of piles.

The negative skin friction on piles is the result of interaction between a pile and surrounding soils that have more settlement than the pile. Negative skin friction induces additional vertical loading on the foundation soil underneath the pile and may lead to structural failure of embankment under severe conditions (Lee *et al.* 2004, White *et al.* 2004 and Comodromos *et al.* 2005). Many studies have been carried out to analyze the behavior of piles subject to negative skin friction. However, the understanding of the mechanism is still limited. As a result, empirical methods are adopted in most existing design methods. In-situ tests have been carried out to examine the behavior of piles subject to negative skin friction (Cho *et al.* 2002, Gavin *et al.* 2007 and Ng *et al.* 2008). These in-situ tests provide valuable information about negative skin friction. However, the number of in-situ studies is limited due to the huge cost. In addition, the development of negative skin friction in an in-situ test usually takes long time, during which testing conditions may change and hence affects the quality of test data inevitably. Recently, physical model studies have been conducted to investigate the negative skin friction (Leung *et al.* 2004 and Samui 2008). All these researches (Cho *et al.* 2002, Leung *et al.* 2004, Ng *et al.* 2008, Gavin *et al.* 2007 and Samui 2008) examine the effect of negative skin friction on pile behavior only under conditions that are different from working conditions of piles in composite foundation.

Geotextile reinforcement cushion has been successfully incorporated in pile foundations as an integrated system to reduce embankment settlements, minimize soil yielding above the pile-cap, and enhance the efficiency of loads

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transmission (Abusharar *et al.* 2009, LeHello and Villard 2009, Jenck *et al*. 2009, Chevalier *et al.* 2011 and Blanc *et al.* 2013). Arching effect in cushion may transfer loads to the piles more effectively (Chen *et al.* 2008, Eskisar *et al.* 2012, Guo and Zhou *et al.* 2013). However, due to the complex working condition of the geotextile reinforced cushion, the evolution of cushion performance with pile penetration into the cushion is not clear yet during the whole process of construction and post-construction stages.

Pile-soil-geogrid-cushion is a key structure in Rigid Pile Composite Foundations which is named as bearing the loads with pile and pile soils together. In this system, loads sharing ratio is a critical parameter for embankment design and construction. Furthermore, the applications and benefit of using Pile-soil-geogrid-cushion can effectively control deformation of foundations with reasonable project costs.

Comparing with the reduced-scale model tests and in-situ large scale tests, the centrifuge model test has the advantage to simulate the gravity property of prototype soil and its effect on the deformation process of geotechnical structures within a short-period of time (McCullough *et al.* 2007, Peiris *et al.* 2008, Hudacsek *et al.* 2009 and Hossain *et al.* 2010). Centrifuge tests can be used to replace the large scale in-situ tests. In a centrifuge test, the model with 1/N ratio is put into a specified centrifuge to conduct the test in a space with centrifugal acceleration of N·g. The inertial force is equivalent to the gravity in the prototype.

In this study, a series of centrifugal model tests were carried out to investigate the time effect of pile-soil-geogrid-cushion interaction in rigid pile composite foundation (RPCF). Two structures of RPCF were examined, which include regular group piles, pile-caps, raft plate with geogrid reinforced gravel cushion or geogrid reinforced lime treated soil cushion, etc. The influence of pile spacing on the negative skin friction of piles, the variation of neutral point position and pile-cushion interaction with time were analyzed.

2. Centrifuge model testing

2.1 Experimental apparatus

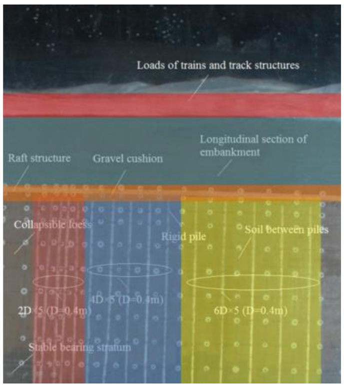
The centrifugal apparatus used in this study has the maximum capacity of 20 g-ton, with 200 g centrifugal acceleration. The effective rotating radius is 1.55 m, and inner size of model box is 415 mm×370 mm×230 mm. Percolation holes were made at the bottom of model box to allow water flow into or out of the testing soil bed during testing to simulate the wetting and drying process.

2.2 Physical model design

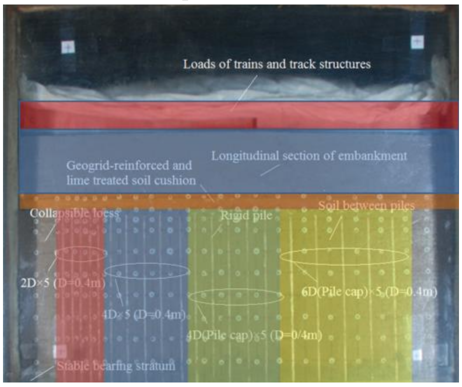
The physical models were made 1:100 of the prototype, with 100 g centrifugal acceleration being used. The soil (collapsible loess) was obtained from the third level terrace of the Yellow River in Lanzhou, China, and its physical properties are summarized in Table 1. Fig. 1 shows the RPCF models, in which steel tubes (the same material as the prototype pile) were used as rigid model piles. The length, external diameter and the thickness of the prototype pile are 16 m (prototype value, similarly hereinafter), 40 cm, and 10 cm, respectively. The soil layers were created by tamping in layers and then were completed the consolidation in centrifugal machine before model-making. The closed piles were driven 15 m into collapsible loess layer and 1 m into the bearing stratum (2 m thick pebble bed) underneath the loess layer. The materials of gravel cushion and reinforced concrete slab are sand and aluminium alloy plate, respectively. Fig. 1 shows the prototype embankment, loads of trains and track structures with respect to the design code of high-speed railways (Ministry of Railways of People's Republic of China, Code for design of high-speed railway 2014).

Table 1 Main controlling indexes of physical model

Soil	Density ρ (kg/m^3)	Void Ratio e	Water Content ω (%)	Degree of Saturation S_r (%)	Cohesion $c(kPa)$	Internal Friction Angle ϕ (°)
Collapsible Loess Layer	1600	1.02	19.0	50.7	16.6	24.3
Embankment Filling	2080	0.55	19.0	93.5	35.1	30.3
Gravel cushion	2340	0.52	7.2	32.4	2.0	42.0
Lime treated soil cushion	2010	0.39	17.0	87.6	37.2	32.6
Stable bearing stratum	2500	0.53	19.0	89.6	2.5	43.0



(a) Pile-raft composite foundation (PRCF)



(b) Pile-geogrid composite foundation (PGCF)

Fig. 1 Test models of rigid pile composite foundations

2.3 Test procedures and deformation determination

Two types of tests were performed: model 1 central longitudinal section test on pile-raft composite foundation (PRCF, shown in Fig. 1(a)) and model 2 central longitudinal section test on pile-geogrid composite foundation (PGCF, shown in Fig. 1(b)). In these tests, the height of railway embankment was 6.0 m, and the thickness of soil layer was 15 m on top of a bearing stratum. The loads of trains and track structures were simulated by an equivalent soil column load (3.0 m high and 3.4 m wide with the unit weight of 18.0 kN/m^3). The pile spacings were selected as *S*=2D, 4D and 6D with D being pile external diameter. In model 2, a 40 cm thick gravel cushion with a layer of geogrid was constructed on top of the piles, and geogrid with the fiber spacing of 21.8 mm was placed in the middle height of the gravel cushion. The size of pile cap is 1.0 m (length)×1.0 m(width)×0.3 m(thickness). For the raft structure (model 1), a 45 cm thick reinforced concrete slab (simulated using aluminium alloy slab) was constructed on the top of the gravel cushion. In model 1, a 1 m thick lime-soil cushion with a layer of geogrid (the same as that in model 2) was constructed on top of the foundation. In model 1 (Fig. 1(a)) 3 trials were made with three different spacings (*S*=2D,4D and 6D). Model 2 (Fig. 1(b)) was made with 4 trials with 3 different spacings, one of the spacings (S=4D) with and without pile cap. Pile groups of various pile spacing were installed in different regions of the physical models in order to reduce the number of centrifuge tests shown in Fig. 1 for models 1 and 2, respectively. In order to reduce friction at the boundaries in the centrifuge model tests, a thin layer of vaseline is applied on the inner side of the model box walls. Plastic particles were pinned along the model as observation monuments (as shown in Fig. 1). A fixed-point high-speed camera was used to take high-resolution photos during the flight for displacement field analysis. Professional digital SLR camera (Canon EOS 5D Mark III) used in these tests with 23.3 million pixels and up to 5760×3840 resolution can captures a high quality photo with the function of bulb exposure of camera and hall sensor unit in centrifugal machine. The effectiveness and feasibility of these centrifuge model tests have been validated in a previous study (Wang *et al.* 2015, 2016).

The effect of water in collapsible loess has been discussed in previous research work (Wang *et al.* 2014). In that article, the acceptability of using remolded loess to simulate the behavior of undisturbed collapsible loess in wetting test was examined including the most sensitive to water contents. And then the time-history of collapse deformation of remolded and undisturbed collapsible loess can be divided into three stages has been analyzed.

The multiple stages of embankment construction were simulated by adjusting the centrifugal acceleration (Chen *et al.* 2010 and Yapage *et al.* 2012). The advantage of this method is convenient for making model and experiment operating, meanwhile, it may cause some value error in prototype changing process. But for experiments that main pay more attention to post-construction stage (centrifugal acceleration reach the design value), the value error is within permission. The displacement of foundation soils was monitored at each stage. After completion of the embankment (when the acceleration reached 100 g in tests), the post-construction settlements during 200 days and 1980 days (5 years) were monitored.

It should be specially explained that dynamic loading in high speed railway engineering is main considered when the height of embankment is less than 3 m. We call it low embankment because the ratio of dynamic loading and dead loading is more than 0.1. In that case, we must consider the influence of dynamic loading. In tests of this paper, the height of embankment is 6 m and the ratio of dynamic loading and dead loading is much less than 0.1, so that we neglected the influence of dynamic loading and considered it as a part of dead loading.

3. Test results and analysis

3.1 Negative skin friction on piles during the post-construction stage (PCS)

When analyzing negative skin friction, the piles are assumed to be rigid. The relative displacement (the observation locations set in different depth) between piles and the surrounding soils was used to determine the negative skin friction. The neutral point is the position that the relative displacement between piles and the surrounding soils is zero. Above the neutral point, the pile displacement (vertical downward) is less than that of the surrounding soils, which results in negative skin friction (downward) on the pile surface.

Figs. 2 and 3 shows the variation of the neutral point, at which the relative displacement between the pile and surrounding soil vanishes, for piles with the pile spacing of *S*=2D. As the depth increase, the relative displacement decreases and the negative skin friction tends to decrease. At the neutral point, the relative displacement and the skin friction become zero. Below the neutral point, the friction is upward and contributes to the bearing capacity of the piles. For PRCF, the neutral point is at -1.2 m at the end of the construction stage (ECS). During the post-construction stage (PCS), the position of neutral point changes from -1.2 m (which corresponds to the ratio between the negative skin friction zone and the length of pile, or the pile length ratio, of 0.075) to the range of -3.5 m (the pile length ratio of 0.22) gradually, which implies that the negative skin friction on piles increases slightly and the bearing capacity decreases with time. For PGCF at ECS, the neutral point is at -9.0 m. In PCS, the position of neutral point moves upward from -9.0 m (the pile length ratio of 0.56) to a range of -6.0 m to -7.0 m (the pile length ratio of 0.37 to 0.43) gradually, which implies that the negative skin friction on piles decreases significantly and the bearing capacity increases with time. The maximum negative skin friction is expected to take place at the top of the pile. From distribution of the negative skin friction zone, PRCF has higher bearing capacity than PGCF.

The variation of the relative pile-soil displacement when the pile spacing is increased to *S*=4D is presented in Figs. 3 and 4. For PRCF at ECS (the end of construction stage), the position of neutral point is at the depth of -11.0 m (the pile length ratio of 0.69). The neutral point gradually moves upward to -9.5 m (the pile length ratio of 0.59) and becomes stable in PCS, and the range of variation is 1.5 m. At ECS, for PGCF without pile-cap, the soil vertical displacement (downward) is higher than that of the pile, resulting in negative skin friction along the entire length of the pile.

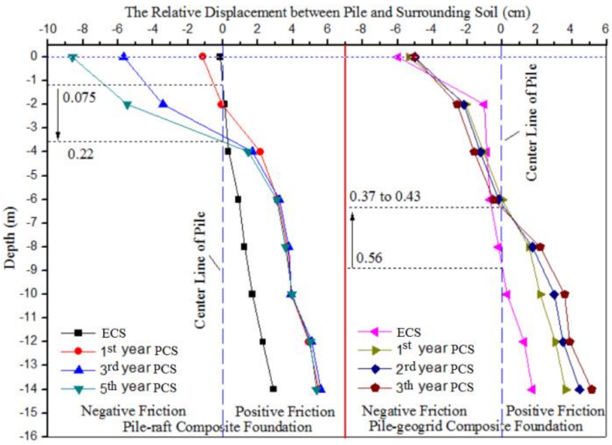


Fig. 2 Relative displacement of RPCF in S=2D

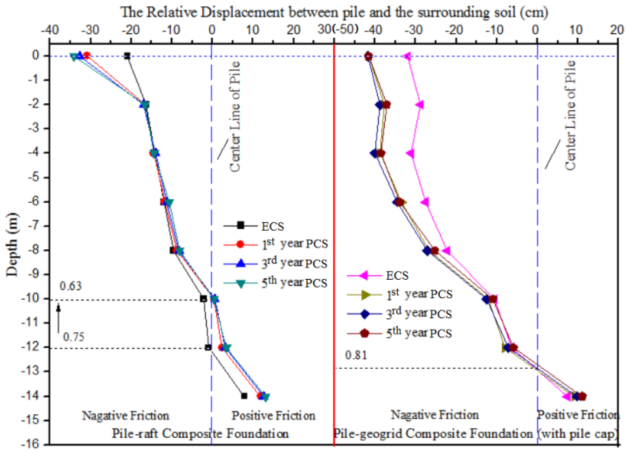
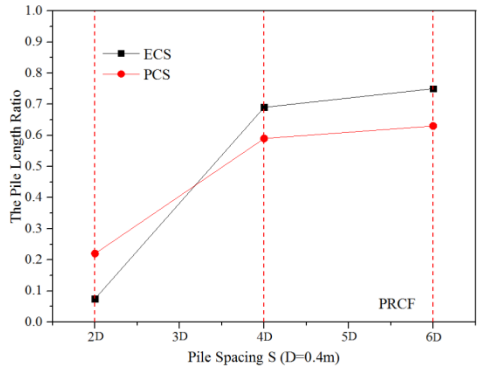
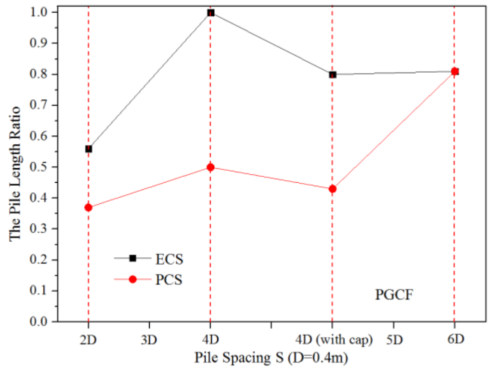


Fig. 5 Relative displacement of RPCF in S=6D



(a) PRCF



(b) PGCF

Fig. 3 The pile length ratio with pile spacing of RPCF

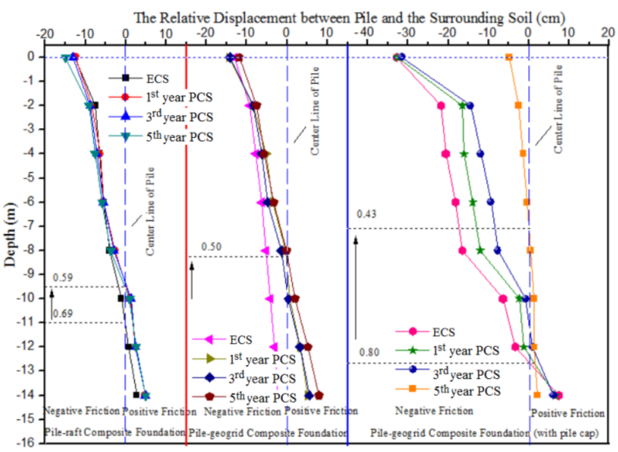


Fig. 4 Relative displacement of RPCF in S=4D

During PCS, the position of neutral point moves up to -8.0 m (corresponding to the pile length ratio of 0.50). Pile-caps has some influence on the relative displacement between the pile and the surrounding soils. When the pile-caps are used, the caps can transfer more loads to the piles, and the position of neutral point is at -12.8 m (the pile length ratio of 0.80) at ECS, accompanied by an increase in the pile-soil relative displacement. During PCS, the neutral point gradually shifts upward to -7.0 m (the pile length ratio of 0.43). Smaller relative displacement is observed in PCS, which increases slightly the negative skin friction on piles and reduces the bearing capacity.

While the pile spacing was increased to *S*=6D (see Figs. 2 and 5), for PRCF, the neutral point position is at -12.0 m (the pile length ratio of 0.75) at ECS. In PCS, the neutral point position moves upward to the depth of -10.0 m (the pile length ratio of 0.63). For PGCF, the neutral point position is at -13.0 m (the pile length ratio of 0.81) at ECS. In PCS, the neutral point position stays the same at this depth. These results show that the pile spacing has direct impact on the location of neutral points at ECS. However, it does not seem to affect the depth of neutral points when the soil displacement becomes stabilized in PCS.

In summary, for PRCF, the position of neutral point moves deeper in PCS as the pile spacing increases. For the pile spacing in the range of *S*=2D to 6D, the pile length ratio varies from 0.22 to 0.63. For PGCF in PCS, the positions of neutral point for piles with and without caps both tend to move upward or keep stable. The pile length ratio varies in the range of 0.37 to 0.81 for the pile spacing of *S*=2D to 6D. The maximum negative skin friction takes place at the top zone of the piles. However, the value of maximum pile skin friction varies with time during PCS.

3.2 Time-dependent development of maximum relative displacement in PCS

Fig. 6 presents the time history of maximum relative displacement measured at the top zones of piles and surrounding soils for pile spacing *S*=2D. At ECS for PRCF, the relative displacements between the pile and the surrounding soils of PRCF and PGCF is -1.6 mm and -60.0 mm (minus sign means that the displacement of surrounding soils is larger than that of piles), respectively. During the PCS, the relative displacement of PRCF and

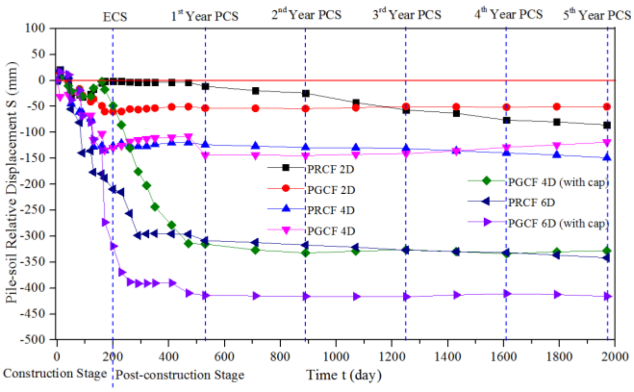


Fig. 6 Maximum relative displacement of RPCF

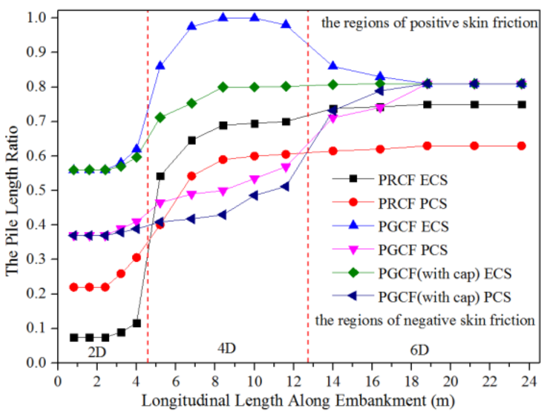


Fig. 7 Variation of the neutral point positions of PRCF

PGCF was -84.3 mm and 9.3 mm, respectively. The maximum value of relative displacement rate of PRCF and PGCF is -32.4 mm/year and 6.6 mm/year.

The time history of the maximum relative displacement for pile spacing *d*=4D is presented in Figure 6. For RPCF, the relative displacements between pile and the surrounding soils of PRCF, PGCF and PGCF (with cap) are -126.5 mm, -130.0 mm and -48.2 mm respectively. During the PCS, the relative displacement of PRCF, PGCF and PGCF (with cap) are -22.6 mm, 11.0 mm and 279.9 mm respectively, with the maximum value of the relative displacement rate of -09.3 mm/year, -14.0 mm/year and -266.7 mm/year, respectively. The time history of the maximum relative displacement for pile spacing *S*=6D (presented in Fig. 6). At ECS for RPCF, the relative displacements between pile and the surrounding soils of PRCF and PGCF are -209.5 mm and -319.2 mm, respectively. During the PCS, the post-relative-displacement of PRCF and PGCF is -132.2 mm and -96.7 mm, respectively. The maximum value of relative displacement rate of PRCF and PGCF are -99.0 mm/year and -94.9 mm/year.

From the above analyses, we can draw conclusion that the maximum value the relative-displacement between piles and surrounding soils have significant time dependency and usually occur at the top of piles. The value of maximum negative skin friction (estimated by maximum value the relative-displacement between piles and surrounding soils) and the position of neutral point are two important parameters for simplified friction distribution along the pile. The maximum value of n relative displacement generally occurs during the construction stage and its value changes with the pile spacing. During PCS the value of relative displacement is less than that in the construction stage and with less time effect. However, the maximum value of relative displacement rate indicates that the relative-displacement between piles and surrounding soils in PCS may still cause additional settlement and negative skin friction to increase loads on piles. More attention should be paid to the time effect of relative displacement (negative skin friction) on piles, especially during the PCS, in case of unexpected larger settlement or differential settlement that may reduce the safety and durability of railway embankments.

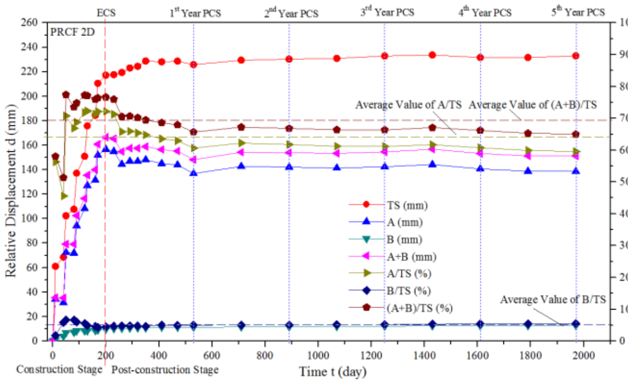
3.3 Time-dependent different distribution of the neutral point positions

Different distribution of neutral point position is also important for PRCF. Significant different distribution of the positions of neutral point may cause additional load on the surface of pile along the longitudinal direction of a railway embankment, thereby increasing the differential settlement. This section mainly discusses the different distribution of the of neutral point position with time.

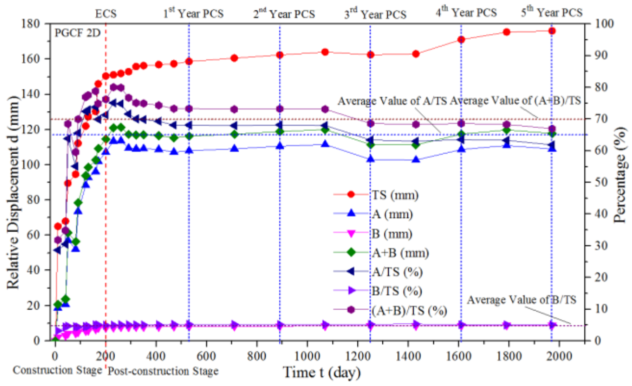
When using different pile spacing in different zones along the longitudinal direction of a railway embankment, significant variation of neutral point positions may take place between these segments. Fig. 7 presents the evolution of neutral point positions underneath the longitudinal central line of embankments in adjacent zones was treated using different pile structures (see Fig. 1). For RPCF at ECG, the values of pile length ratio between two segments with pile spacing of *S*=4D and *S*=2D are 0.426 (PRCF), 0.24 (PGCF) and 0.115 (PGCF with cap in 4D) respectively. In the 5th year of PCS, the difference between pile length ratios of these segments are 0.095 (PRCF), 0.055 (PGCF) and 0.02 (PGCF with cap in 4D) respectively. Between the segments of *S*=2D and *S*=4D, PRCF reduces the negative skin friction on the pile surface, however, the difference between pile length ratios is larger than that in PGCF. It indicates that the distribution of negative skin friction changes fast in PRCF between this segment, which may cause significant differences of loads on adjacent piles in ECS.

In the ECS, the different values of pile length ratio in two segments with pile spacing of *S*=4D and *S*=6D are 0.038 (PRCF), -0.12 (PGCF) and 0.005 (PGCF with cap) respectively. In the 5th year of PCS, the difference of pile length ratio values between two segments are 0.01 (PRCF), 0.142 (PGCF) and 0.22 (PGCF with cap). On the contrary, between the segments of *S*=4D and *S*=6D, the difference of pile length ratio value of PRCF is smaller than that in PGCF, due to the better ability to reduce negative skin friction on pile surface. Pile-caps setting can reduce the difference in pile length ratios between two segments.

To reduce negative skin friction of RPCF during PCS, measures such as proper drainage system, and waterproof to reduce the deformation of soils should be considered. In addition, setting displacement pile into soils between rigid piles above the neutral point should be taken into account in collapsible loess foundation, especially in the transition zone between segments using different treatments. Details of these studies need to be further researched.

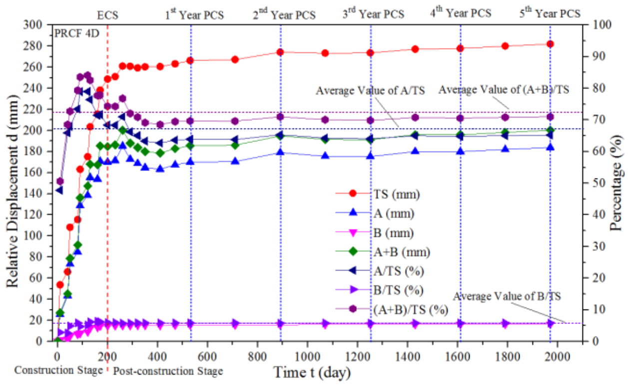


(a) PRCF

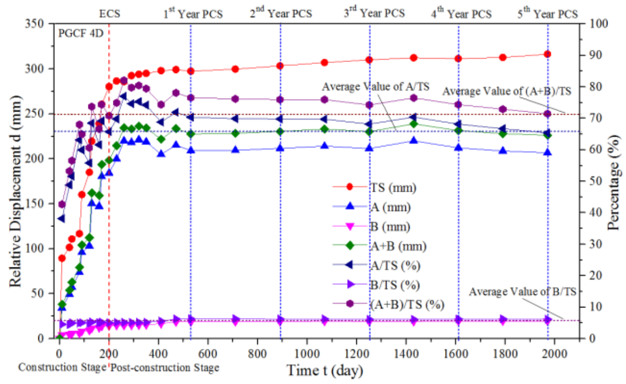


(b) PGCF

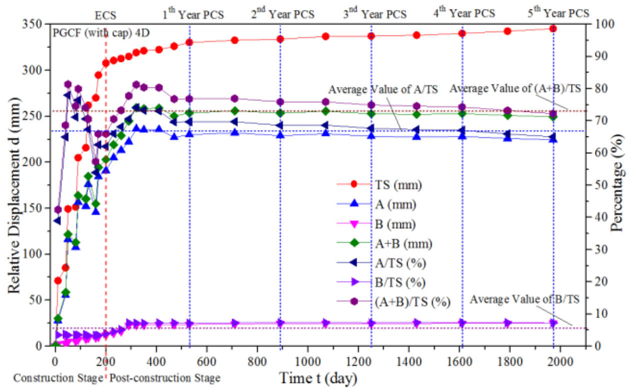
Fig. 8 Time-dependent development of pile-cushion interaction in d=2D



(a) PRCF



(b) PGCF

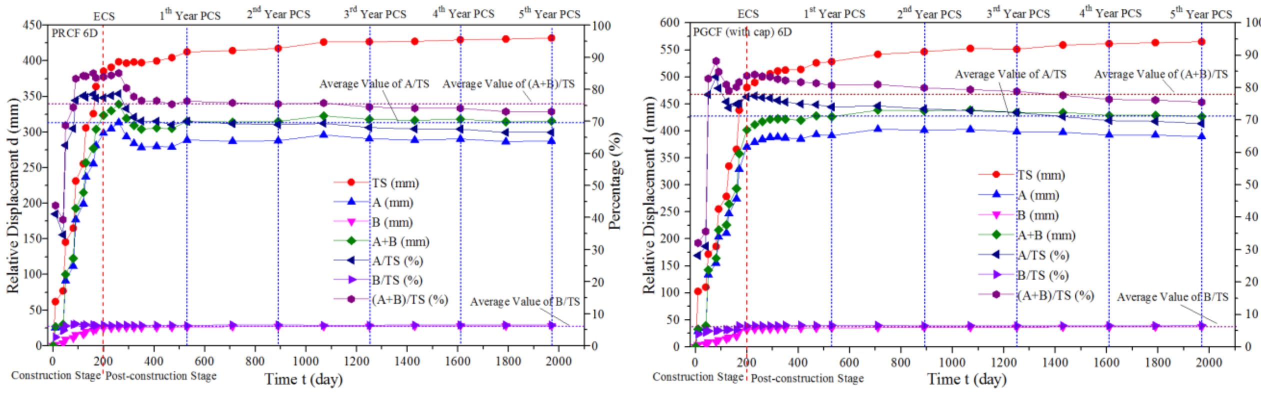


(c) PGCF (with cap)

Fig. 9 Time-dependent development of pile-cushion interaction in d=4D

3.4 Time-dependent development of pile-cushion interaction

This section discusses the development of pile-cushion interaction with time. The main effects of the cushion with a layer of geogrid are: (1) to ensure the share of load between piles and the surrounding soils and adjust the ratio of the stresses shared by the piles and soils; (2) to reduce stress concentration at the bottom of foundations. In this study, the end-bearing piles in these RPCF tests transfer load directly to the stiff stratum underneath via end bearing. Pile capacity may be limited to the structural strength of the pile. Considering the structures of RPCF, the total settlement of the composite foundations is mainly from four sources: (1) compression of the piles and soils (compression of reinforced area); (2) penetration of the piles into the cushion and the bearing stratum; (3) compression of cushion and (4) compression of the soil below the pile tips. To get insights into the mechanism of load sharing between piles and the surrounding soils, it is necessary to examine the relative displacement between a pile and the cushion layer above it. The pile penetration into the cushion can be used to estimate the load transfer mechanism on the pile and the foundation settlement. Lager value that the pile penetration into the cushion could weaken the soil-arching effect in cushion so that the loads on pile will get larger and larger. This phenomenon that not given more attention in design is to the disadvantage of settlement controlling and



(a) PRCF (b) PGCF

Fig. 10 Time-dependent development of pile-cushion interaction in d=6D.

Fig. 11 The larger pile-cushion interaction in PGCF

embankments stability.

Figs. 8(a) and 8(b) present the variation of pile-cushion interaction with time of *S*=2D of PRCF and PGCF and show the total settlement (TS) and the value of penetration of pile into the cushion with compression of cushion (A) and the value of penetration of pile into the bearing stratum (B) as well as the proportion of A, B and (A+B) over TS.

From the analyses of experimental data, the penetration of pile into the cushion and compression of cushion plays a critical role in the foundation settlement (about 65%). The compression of the piles and soils (compression of reinforced area as C) is less than 35% (PRCF) and 30% (PGCF) in TS proportion. The value of penetration of pile into the bearing stratum is the smallest one in these components due to bearing capacity of bearing stratum.

When the pile spacing is increased to *S*=4D, the variation of pile-cushion interaction with time in RPCF is presented in Figs. 9(a)-9(c). Similar to the case with in *S*=2D, the value of penetration of pile into the cushion with compression of cushion (A) is the largest part in TS (about 67%). The proportions of A and B both increase slightly than those in *S*=2D. Furthermore, the proportion of the compression of piles and soils (compression of reinforced area as C) is less than 28%. With an increase in pile spacing, more loads were transmitted to piles.

Figs. 10(a) and 10(b) present that the variations of pile-cushion interaction with time of RPCF as the pile spacing increases to *S*=6D. Similar to the status in *d*=2D and 4D, the value of penetration of pile into the cushion with compression of cushion (A) is also the largest part in TS (about 69% in PRCF and 71% PGCF). The proportions of A and B are both slightly higher than those in *S*=4D. Thereby, the estimated proportion of the compression of piles and soils (compression of reinforced area as C) is less than 25% in PRCF and 23% in PGCF. With the increase of pile spacing, piles bear more loads from the superstructure.

The test results can be validated by field measurements in the railway embankment project. According to the actually measured result, the compression of the rigid piles contributed less than 21% of the total settlement while the penetration of the piles and the compression of the soft soils below the pile tips contributed nearly 80% of the total settlement (Zheng *et al.* 2011). In RPCF, especially for the pile spacing increasing, The compression of the rigid piles contributed less than 21% (field measurement, *S*=4D PRCF) and 28% (centrifugal test, *S*=4D PRCF) of the total settlement while the penetration of the piles and the compression of the bearing stratum below the pile tips contributed nearly 80% of the total settlement (field measurement, *S*=4D PRCF) and 72.5% (centrifugal test, *S*=4D PRCF). Furthermore, in accordance with the analysis of test data, arching effect in the cushion of the composite foundation is a progressive process. With the increase of pile spacing, the arching effect decreases and may even disappear (e.g., large penetration of pile into the cushion) or change to other bearing pattern (e.g., inverted cone or wedge distribution of stresses). This change may lead to the deduction of pile-soil stress ratio and increase settlement or even instability of embankments.

The experimental results is validated in actual high-speed railway project built on deep soft soil foundation in southeast of China. Fig. 11 show the larger pile-cushion interaction in PGCF which has caused railway embankment instability and larger settlement in PCS. In this railway construction project, the average embankment height is about 5.5 m with 250 km/h maximum design speed. PGCF (20 m pile length, 1 m×1 m× 0.3 m pile cap, 1 layer geogrid and *S*=2.5D, D=0.4m) was used to reinforce foundation which is 18 m soft soils. According to preliminary estimate, the penetration value of the piles (with cap) into cushion is 19 cm in average and 31 cm in maximum. For average 50 cm foundation settlement, the proportion of penetration value of pile into the cushion with total settlement is more than 38% in average and 62% in maximum.

As a result, design parameters of cushion (e.g., thickness, particle size etc.) should be adjusted corresponding to the pile spacing. Reinforced cushion with geotextile may promote the better performance of cushion for transmitting the loads to piles and surrounding soils.

4. Conclusions

(1) Using centrifugal model test to simulate the skin friction distribution of pile is feasible. Based on a comparision of relative-displacement between piles and surrounding soils zone in different composite foundations, it is found that PRCF can effectively reduce the relative-displacement between piles and surrounding soils on piles and increase the bearing capacity. The foundation settlement and relative-displacement between piles and surrounding soils are time-dependent. The distribution of skin friction can be simplified as a triangle along the pile. During the PCS, the position of neutral point moves deeper as pile spacing is increased.

(2) For PGCF, the setting of pile-cap affects the position of neutral point in the PCS. For piles without caps, the position of neutral point tends to move upward. For piles with caps, the neutral point ascends gradually before it became stabilized.

(3) The maximum value of relative-displacement between piles and surrounding soils mostly occurs during construction stage and its value changes with the pile spacing. However, the relative-displacement between piles and surrounding soils in PCS may causes obviously negative skin friction on piles.

(4) To reduce negative skin friction of RPCF during PCS, measures such as proper drainage system, and waterproof to reduce the deformation of soils should be considered. Setting displacement pile above neutral point between rigid piles should also be taken into account in collapsible loess foundation, especially in the transition zone between segments using different treatments.

(5) Arching effect in the cushion of the composite foundation is a progressive process. The compression of the rigid piles contributes less than 20% to 25% of the total settlement while the penetration of the piles and the compression of the bearing stratum below the pile tips contribute more than 70% of the total settlement. Two actual projects have validated the experimental results should been paid more attention. It is suggested that the design parameters of cushion be adjusted with the pile spacing, and reinforced cushion with geotextile should be taken into account.

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