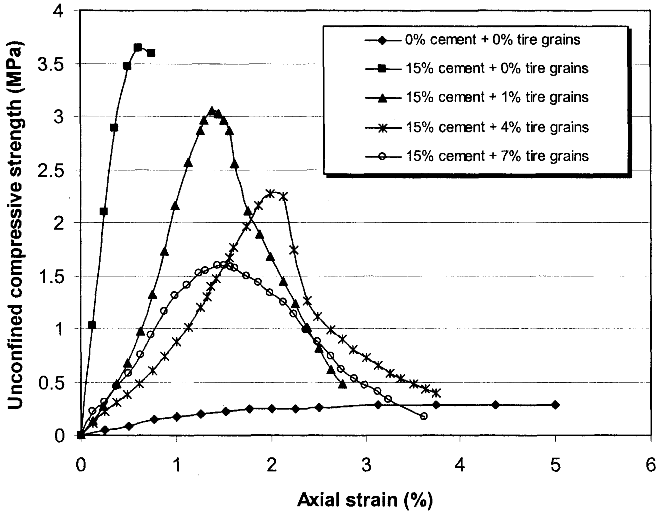

FIG. 3. Results of unconfined compression tests of cement-stabilized soft clay with 4 mm tire grains.

It can be seen from Figures 2 and 3 that the addition of cement alone significantly increases the ultimate strength and stiffness (modulus of elasticity) of clay soil, and decreases the deformation at failure. However, it can also be seen that the addition of cement alone exhibits brittle failure with significant loss of post-peak shear strength. On the other hand, it is shown that the addition of shredded rubber tires to cement-stabilized clay increases soil ductility regardless of the amount of shredded tires added. This is attributed to the reinforcement impact of shredded tires, which enhance soil ability to restrain cracking.

Figures 2 and 3 also show that stiffness and ultimate compressive strength of cement-stabilized clay seems to decrease by the addition of shredded tires. This is because shredded rubber tires carry small amount of load due to their low elastic modulus relative to the surrounding stabilized soil, leading to reduction in the compressive strength and stiffness of the soil matrix. By comparing the results of Figures 2 and 3, it can also be seen that cement-stabilized clay with tire grains seems to be more prone to the reduction in compressive strength than that of cement-stabilized clay with tire powder. For example, the ultimate compressive strength of cement-stabilized clay is decreased by 3% when 1% tire powder was used, whereas the compressive strength of cement-stabilized clay is decreased by 20% when 1% tire grains were added. This may be attributed to the fact that larger tire sizes create larger weak planes in certain areas in soil matrix due to the loss of bonding between tire grains and surrounding stabilized soil, leading to the development of failure paths when load is applied. This phenomenon was also recognized by Turatsinze et al. (2005) who studied the micrograph of bonding between cement paste and rubber shreds. Based on the above results, it is recommended that 1% tire powder is added to cement-stabilized clay so that the detrimental impact of shredded tire in terms of compressive strength and stiffness is reduced to minimal while keeping better ductility.

Tensile strength tests

Based on the results obtained earlier from the compressive strength tests, it was decided to conduct the indirect tensile strength test only at 1% tire powder. The results of the test are shown in Figure 4, which clearly indicate that the tensile strength of soft clay is significantly increased by the addition of cement. However, it is also obviously shown that the tensile behavior of cement-stabilized clay exhibits brittle failure. As shown in Figure 4, the addition of 1% tire powder significantly enhances the ductility of cement-stabilized clay, even though ultimate tensile strength decreases. Again, this is attributed to the reinforcement impact of rubber tires in increasing the ability of cement-stabilized clay to restrain cracking. It can also be seen from Figure 4 that, on the contrary to the compressive strength tests, the addition of rubber tire to cement-stabilized clay does not reduce the tensile strength modulus of elasticity.

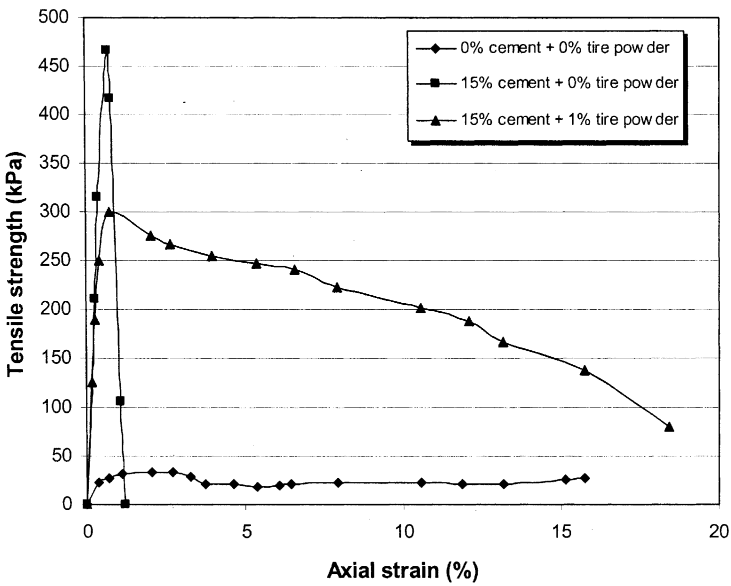

FIG. 4. Results of indirect tensile strength test of cement-stabilized soft clay with 440 μm tire powder.

SUMMARY AND CONCLUSIONS

A series of laboratory tests were conducted to investigate the effects of shredded rubber tires on the strength and stiffness of cement-stabilized soft clay. The tests conducted include the unconfined compression strength and indirect tensile strength. Cement was added by an amount of 15% (by weight of dry soil) and two different sizes of shredded tires, i.e. 440 μm tire powder and 4 mm tire grains, were added in order of 1%, 4% and 7% by weight of dry soil. All experiments were carried out after curing time of 7 days. The study has yielded the following conclusions:

1. The addition of shredded tires increases the compressive strength ductility of cement-stabilized clay regardless of the amount of tire added, but seems to decrease the ultimate compressive strength and compressive modulus of elasticity.
2. The addition of 1% tire powder (by weight of dry soil) seems to reduce the detrimental impact of shredded tires in decreasing the compressive strength and stiffness of cement-stabilized clay, while keeping better ductility.
3. The addition of 1% tire powder increases the tensile strength ductility of cement-stabilized clay and seems not to decrease the tensile strength modulus of elasticity, even though it decreases the ultimate tensile strength.

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Analysis on Load Transfer for Single Pile Composite Foundation Under Embankments Based on Elastic Theory

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Abstract

A method to analyzing load transfer characteristics is presented for single pile composite foundation under embankment by means of elastic theory coupling with load transfer method. The pile and its above fill can be taken as a column with varied section area. The settlement of soil adjacent to pile caused by frictional resistance is calculated by Mindlin solution, and that induced by the filling load on the ground surface is calculated by Boussinesq solution. Non-linear springs are taken into consideration to simulate the character of pile-soil contact surface. A simplified calculation model for single pile composite foundation under embankment is then established. The pile-soil-cap-embankment interaction can be considered conveniently in this method. The correctness of this method was validated by pile efficacy of model test. The distributions of axial force, frictional resistance and displacement of pile and soil are then compared with those of piles under large area embankment.

INTRODUCTION

Piled embankment can effectively limit the differential settlements, large lateral deformation, overall or local instability, so it is applied in worldwide highway on soft clay or widening of existing roads, as well as embankment adjacent the bridge abutment.

Many scholars have researched the mechanisms of composite foundation of piled embankment.

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Low (1993), Hewlett (1988), Han (2002) and Chen (2004) have researched the soil arching effect separately through model test, numerical or theoretical analysis. Xu (2005) and Chen (2005) build an approximately closed solution of pile axial force, displacement of pile and soil by load transfer method, in which the load transfer function assumed idealized elastic-plastic model, soil uniform, pile efficacy pre-calculated by approximate method. The distribution of negative friction was also discussed. Lou (2009) presented a more effective method for analyzing the interaction of pile-soil-embankment, where the pile and its above fill, the soil adjacent to pile and its above fill were both taken into as a column. Each column was analyzed one-dimensionally by load transfer method, in which the load transfer function can assumed non-linear model, soil layered, pile efficacy calculated during analyzing.

For all these studies, only the center piles and its surrounding soil under large area embankment were considered, but not the difference between the center piles and corner or edge piles, because of its inherent disregard for the continuity of the soil mass. It is necessary to establish an analytical model based on elastic theory as Poulos 1980) to analyze the settlement of single or group piles.

The main difficulty for analysis of piled embankment is the unknown proportion of vertical load carried by the pile or cap. Based on the research of Lou (2009), the embankment fill are also separated into two parts in this study, the fill above the pile (inside fill) and the fill above the soil (outside fill). The pile and the inside fill are taken into a column, simulated by one dimension compression member, as well as the outside fill. The settlement of soil surrounding pile caused by frictional resistance is calculated by Mindlin solution, and that caused by the load of outside fill is calculated by Boussinesq solution. Springs are taken into consideration to simulate the character of pile-soil interface. A simplified calculation model for single pile composite foundation under embankment is then established, which can take into account the pile efficacy, frictional resistance, and displacement.

ANALYTICAL METHOD

Fundamental assumption. To simplify the analysis, piles are arranged in square, and the fill is considered as having an equivalent domain treated by single pile, with the height of fill assumed to be same. At the pile head elevation, the total load carried by the pile and its surrounding soil is equal to the weight of the fill. The fill and piles are isotropic elastic medium. There are interaction between pile and soil, inside fill and outside fill, as shown in Figure 1. At the same elevation, the settlement of inside fill is considered to be equal, as well as outside fill.

The frictional resistance between pile and soil, inside and outside fill, and the tip resistance of pile are all simulated by springs with the nonlinear or elastic-plastic load transfer model. The elements of pile and springs are shown in Figure 2.

The load carried by pile is transferred into surrounding soil through frictional resistance. The settlement of soil adjacent to pile caused by frictional and tip resistance is calculated by Mindlin solution, and that caused by the load of fill between piled caps is calculated by Boussinesq solution, as shown in Figure 3.

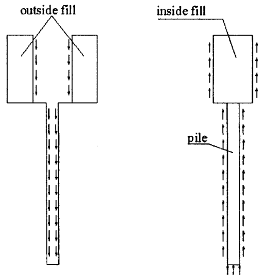

Figure 1. Assumption of pile-soil stress

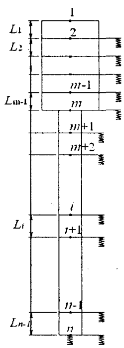

Figure 2. Pile element and spring surface

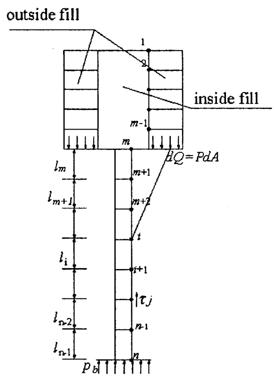

Figure 3. Settlement calculation of soil surrounding pile

Basic equation. The pile and the upper fill on the cap are divided into a series of elements as a generalized pile, The number of node of upper fill is m, and that of pile is u. The total number of node is n, while the element is n-1.

The equations of the pile axial force and settlement can be expressed as follows:

$$[K_p]\{s_p\} = \{Q_p\} - \{R\} \tag{1}$$

Where $[K_p]$=total stiffness matrix of generalized pile, including pile and its above fill, n×n.

$\{s_p\}$= vector of displacement of all nodes of generalized pile, n×1,

$\{s_p\} = \{\{s_p\}_1 , \{s_p\}_2\}$, while:

$\{s_p\}_1 = \{s_{p1} , s_{p2} , \dots , s_{pm-1} , s_{pm}\}^T$,

$\{s_p\}_2 = \{s_{pm+1}\ ,\ s_{pm+2}\ ,\ \cdots\ ,\ s_{pn-1}\ ,\ s_{pn}\}^T$

$\{Q_p\}$= vector of load carried by generalized pile, which is the weight of fill above the pile, $n \times 1$, $\{Q_p\} = \{Q_{p1}\ ,\ Q_{p2}\ ,\ \cdots\ ,\ Q_{pm}\ ,\ 0\ ,\ \cdots\ ,\ 0\}^T$.

$\{R\}$=vector of frictional resistance of generalized pile, n×1. $\{R\} = \{\{R\}_1\ ,\ \{R\}_2\}$,

and: $\{R\}_1=\{R_1\ ,\ R_2\ ,\ ,\ \cdots\ ,\ R_{m-1}\ ,\ R_m\}^T$,$\{R\}_2 = \{R_{m+1}\ ,\ R_{m+2}\ ,\ ,\ \cdots\ ,\ R_{n-1}\ ,\ R_n\}^T$.

The outside fill is simulated by one dimension compression member, and the equation of the axial force and displacement can also be expressed as follows:

$$[K_s]\{s_s\}_1 = \{Q_s\}_0 + \{R\}_1 \tag{2}$$

Where $[K_s]$=total stiffness matrix of outside fill, m×m.

$\{s_s\}_1$= vector of displacement; $\{s_s\}_1 = \{s_{s1}\ ,\ s_{s2}\ ,\ \cdots\ ,\ s_{sm}\}^T$.

$\{Q_s\}_0$=vector of load carried by outside fill, which is the deadweight of outside fill, $\{Q_s\}_0 = \{Q_{s1}\ ,\ Q_{s2}\ ,\ \cdots\ ,\ Q_{sm}\}^T$.

$\{R\}_1$= the frictional resistance between inside fill and outside fill, m×1,

$\{R\}_1 = \{R_1\ ,\ R_2\ ,\ \cdots\ ,\ R_m\}^T$.

The settlement of soil surrounding pile is composed of two parts, and one part caused by frictional resistance can be calculated by Mindlin solution, another caused by the load of outside fill can be calculated by Boussinesq solution.

The settlement caused by frictional resistance can be shown as:

$$\{s_s\}_{2M} = [I_s]\{R\}_2 \tag{3}$$

Where $\{s_s\}_{2M}$ =settlement of an arbitrary point in the soil surrounding pile caused by frictional and tip resistance, $\{s_s\}_{2M} = \{s_{sm+1}\ ,\ s_{sm+2}\ ,\ \cdots\ ,\ s_{sn-1}\ ,\ s_{sn}\}^T$.

$[I_s]$=flexibility matrix of settlement calculated by Mindlin integral formula, u×u.

$\{R\}_2$=column vector of frictional resistance, u×1, $\{R\}_2 = \{R_{m+1}\ ,\ R_{m+2}\ ,\ \cdots\ ,\ R_{n-1}\ ,\ R_n\}^T$.

The settlement of soil surrounding pile caused by outside fill can be shown as:

$$\{s_s\}_{2B} = [B_s]\{P\} \tag{4}$$

Where

$\{s_s\}_{2B}$ =settlement of an arbitrary point in the soil surrounding pile caused by the weight of outside fill, and the column vector form is the same as that of $\{s_s\}_{2M}$.

$[B_s]$=flexibility matrix of settlement of soil element surrounding pile, which is calculated by

Boussinesq solution, u×m. The settlement is caused by the deadweight of outer fill column and the frictional resistance between inside and outside fill.

$\{P\}$=the sum load of deadweight of outside fill and the frictional resistance at the pile head elevation, and in order to express the equation conveniently, it can be shown as the form of column vector, $\{P\} = (\{Q_s\}_0+\{R\}_1)^T$.

Combining Eq.(3) and (4), the settlement of soil surrounding pile can be given by

$$\{s_s\}_2 = \{s_s\}_{2M} + \{s_s\}_{2B} = [I_s]\{R\}_2 + [B_s](\{Q_s\}_0 + \{R\}_1) \tag{5}$$

Where $\{s_s\}_2$=the total vector of settlement adjacent to pile,

$\{s_s\}_2 = \{s_{sm+1}, s_{sm+2}, \dots, s_{sn-1}, s_{sn}\}^T$.

Combining Eqs.(2) and (5), the equations of settlement can be shown as

$$\begin{bmatrix} [K_s] & 0 \\ 0 & [E_2] \end{bmatrix}\{s_s\} = \begin{bmatrix} [E_1] & 0 \\ [B_s] & 0 \end{bmatrix}\{Q_s\} + \begin{bmatrix} [E_1] & 0 \\ [B_s] & [I_s] \end{bmatrix}\{R\} \tag{6}$$

Where $[E_1]$=unit matrix, m×m; $[E_2]$=unit matrix, u×u.

$\{Q_s\}$=column vector, n×1, $\{Q_s\} = \{\{Q_s\}_0, 0, 0, \dots, 0, 0\}^T$.

$\{s_s\}$=total column vector of settlement, n×1, $\{s_s\} = \{\{s_s\}_1, \{s_s\}_2\}^T$.

The calculation of frictional resistance. The frictional resistance between generalized pile and soil (including outside fill) can be calculated through the relative settlement:

$$\{R\} = [K_w]\{w\} \tag{7}$$

Where

$[K_w]$= stiffness matrix of interaction between generalized pile and soil, calculated by load transfer function, n×n.

$\{w\}$=relative settlement between generalized pile and soil,

$$\{w\} = \{s_p\} - \{s_s\} \tag{8}$$

The last equations. Substituting Eq.(7) and (8) into Eq.(1), the matrix equation of generalized pile can be obtained.

$$([K_P]+[K_w])\{s_p\} = \{Q_p\} + [K_w]\{s_s\} \tag{9}$$

Substituting Eq.(7) and (8) into Eq.(6), the settlement matrix equation of outside fill and the elastic half-space ground is expressed as follows:

$$([K_{sf}]+[BI][K_w])\{s_s\} = [EB]\{Q_s\} + [BI][K_w]\{s_p\} \tag{10}$$

Where $[K_{sf}] = \begin{bmatrix} [K_s] & 0 \\ 0 & [E_2] \end{bmatrix}$, $[BI] = \begin{bmatrix} [E_1] & 0 \\ [B_s] & [I_s] \end{bmatrix}$, $[EB] = \begin{bmatrix} [E_1] & 0 \\ [B_s] & 0 \end{bmatrix}$.

Solution of the equations and load transfer function. Iterative method is used to solve equations (9) and (10). The solving procedure is same as the depiction by Lou (2009).

The characteristic of pile-soil interface can be simulated by general load transfer function, such as the linear elastic-plastic model, exponential cuve model, hyperbolic model and so on. It can also be referred to the paper by Lou (2009).

CASE STUDIES

Case 1. The model test by Xu (2004) is taken to carry out the contrastive analysis. Table 1 shows the parameters for the case.

Table1. Parameters for Control Case

Material	Parameters
Embankment fill	fine sand, h=0.45m, γ=15.6kN/m^3, compressive modulus=12MPa, φ_u=38°
Model pile	aluminum alloy pipe, diameter=32.3mm, length=9m, wall thickness=1.5mm, E_p=71.4GPa
Pile cap	square armor plate, 90mm×90mm×5mm
Foundation soil	fine sand, γ=15.6kN/m^3, compressive modulus=12MPa, frictional coefficient k=0.30, ultimate end bearing capacity=4000kPa

The efficacy of centre pile calculated under large area embankment is in agreement with that from model test, as shown in Figure 4. The efficacy of center pile is obviously bigger than that of corner pile, which means that the more influence of adjacent piles, the more settlement of soil, and the more embankment load carried by centre pile. Figure 4 also shows that the efficacy of single pile calculated here is a little less than the efficacy of corner pile measured in the model test, because the corner pile measured in the model test is still influenced by the adjacent piles, but the single pile is influenced by no pile. It also can be seen that the calculated pile efficacy decreases with pile spacing increasing, which is similar with that from the model test.

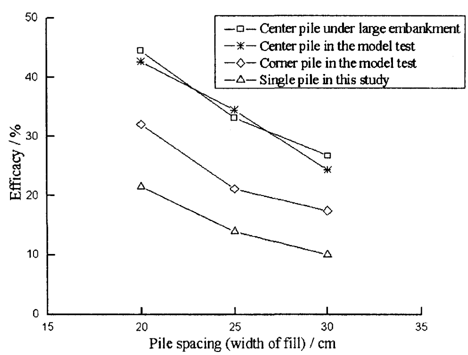

Figure 4. Comparison of pile efficacy

Case 2. In the literature by Lou (2009), the frictional resistance distribution and pile axial force of centre pile under large area embankment are analyzed through load transfer method. In order to contrast with the result in this paper, the same parameters are adopted, which are shown in Table 2. The difference is that the model of the soil adjacent to pile is assumed a half space instead of one dimensional compression column. For the two-layer soil in the Table 2, approximate treatment is referred to the book by Poulos (1980).

Table 2. Parameters for Control Case

Material	Parameters
Embankment fill	Height=4m, 2.5m×2.5m, γ=18kN/m^3, modulus of compressibility=15MPa, c=10kPa.
Pile	Diameter = 0.4m, length = 15m, E_p = 35GPa, pile spacing = 2.5m.
Pile cap	Square, 1m×1m.
First layer of soil	Thickness = 10m, γ = 18kN/m^3, compressive modulus = 3MPa, frictional coefficient k = 0.20, Ultimate frictional resistance=40kPa,relevant relative displacement=0.01m.
Second layer of soil	Thickness =10m,γ = 18kN/m^3, compressive modulus = 6MPa, frictional coefficient k = 0.30, ultimate frictional resistance = 60kPa, ultimate tip resistance = 1000kPa, relevant tip displacement=0.01m.

The main calculated results are shown in Table 3. The efficacy of single pile calculated by this method is obviously less than that of centre pile under large embankment. Because the dispersion of additional stress is not considered under the large area embankment, the compressive deformation of soil surrounding pile and the efficacy of center pile are both the maximum value, while the efficacy of single pile calculate in this study is the minimum value because of the dispersion of superimposed stress and the absent influence of adjacent piles. It also can be seen that the elevation of neutral point of single pile is higher than that of central pile under large embankment, because the different settlement between pile and soil decreases with the consideration of dispersion of additional stress.

Table 3. Main calculation results

	Pile efficacy (%)	Neutral point of pile z_0/L
Central pile under large embankment	69.80	0.50
Single pile under rectangular embankment	39.66	0.29

Figure 5 presents the profile of vertical stress in the outside fill. It can be observed that there is a critical height in the fill, upper which the vertical stress in the outside fill matches with that in the natural ground, while a reduction of the vertical stress in the outside fill exists instead of the