that the distribution of vertical stress along the depth in embankment fills, which is helpful to understand the behavior of piled embankments.

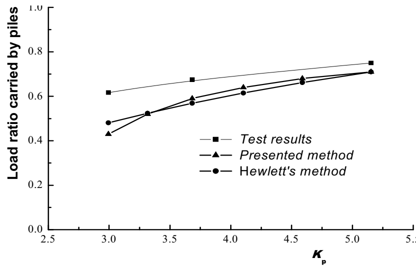

FIG. 3. Comparisons between measured and calculated results.

ACKNOWLEDGMENTS

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Dynamic Finite Element Analysis of Micropile Foundation in Subgrade

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ABSTRACT: The effect of micropiles on soil reinforcement under static and dynamic loadings was studied. Embankment on untreated soil and treated soil by micropiles were modeled using the finite element code PLAXIS. The displacement caused by embankment static loading and acceleration of the embankment caused by seismic loading were calculated and compared. It was found that micropiles treated soil can greatly reduce the settlement of the embankment and mitigate seismic response of the embankment. The results of this study provide valuable information about the design and application of micropiles.

INTRODUCTION

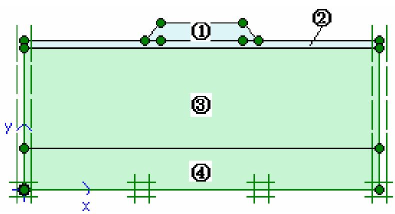
Micropiles are small diameter grouted piles that are traditionally used in foundation retrofit. Micropiles can also be installed in almost any ground condition. Experimental evidence has indicated that micropiles behave well under seismic loadings due to their high flexibility. Moreover, observations in the 1995 Kobe Earthquake demonstrated the good performance of friction piles under seismic loading (Wong, J.C.,2004). However, the seismic behavior of micropiles is not fully understood due to limited number of tests both in full and model scale, as well as the limited amount of numerical modeling studies for micropiles. Some researchers such as Yamane,T. et al. (2000), Wong, J.C.(2004), Shanzhi Shu.(2005) and Wang, Liyan et al.(2005) have studied on mechanical characteristics of micropiles. The dynamic analysis of soil-pile-structure interaction is a very complex problem (Nogami,T.et al.,1992; Wu,guoxi et al.,1997; Boulanger, Ross W. et al.,1999; Lok,M.H.,1999; Ousta,R.et al.,2001; and Surendran Balendra, 2005). The Finite Element (FE) method provides a tool to understand the seismic behavior of micropiles. The finite element code PLAXIS with dynamic analysis function has been used to model the soil and micropiles system, and analyze the dynamic characteristics of unreinforced soil and micropile reinforced foundation.

THE STATIC ANALYSIS OF MICROPILES

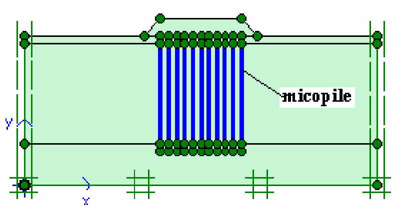
The finite element model for insitu four soil layers strata was generated. The soil including fill was treated as perfectly elastic-plastic materials in the simulation. Parameters of soil used in the simulation are listed in the Table 1.

Table 1. Soil Properties for the Micropile Foundation Analysis

Soil Type	Unsaturated Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	Cohesion (kN/m^2)	Friction Angle (degrees)	Young's Modulus (kN/m^2)	Possi-on's Ratio
Fill	16	20	1	30	8000	0.3
Clay Silt	16	18	5	25	10000	0.35
Soft soil	17	18.5	7	20	5000	0.35
Coarse Sand	17	20	1	34	30000	0.3



a. Untreated by micropiles



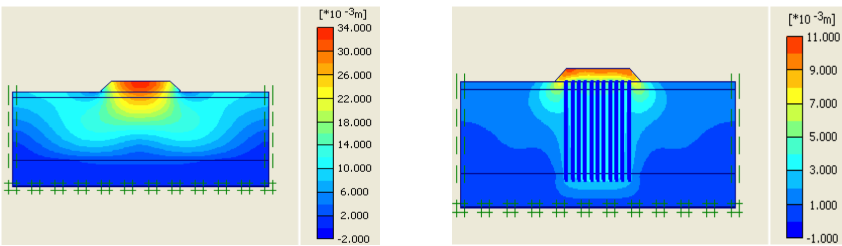
b. Treated by micropiles

FIG. 1. Problem Setting and the FEM Model.

The micropiles with diameter of 0.2 m and length of 14 m were modeled using the 5-node beam element. The interactions between the soil and the piles were taken into account by setting interface elements between the soil and the pile. In formulating the stress-strain behavior at the soil- pile interface, the thickness of interface was assumed as 0.1 to 0.01 times the length of corresponding interface element. The corresponding strength reduction factors were selected based on both the material properties of the soil and the pile.

The boundary conditions were set as the standard boundary conditions. The displacements in all directions at the bottom boundary were fixed; the boundaries at both sides can only move in the vertical direction. When the geometry and boundary condition setting was complete, the finite element mesh was generated. In order to consider the possible large stress gradient, the meshes are refined at the soil and pile interface.

The total deformation of the embankment is shown in Figure 2. Figure 2(a) shows that the total displacement profile for the whole geometry in the last times step; the maximum displacement at this time step is 33.24 $\times 10^{-3}$ m. Figure 2(b) shows the maximum displacement in the last time step is 10.35 $\times 10^{-3}$ m. The displacement plot shows the maximum displacement at the top of embankment which is untreated is much higher than the displacement at the top of embankment which is reinforced by micropiles. The result shows that micropiles treated soil can greatly reduce the settlement of the embankment.



a. Untreated by micropiles **b. Treated by micropiles**

FIG. 2. FEM plot of extreme total displacement.

THE DYNAMIC ANALYSIS OF MICROPILES

The dynamic analysis uses the same geometry as that of the static analysis. The triaxial unloading stiffness E_{ur} is set as 3 times as the triaxial loading stiffness E_{ref} and the oedometer loading stiffness E_{oed} is set the same as the triaxial loading stiffness E_{ref}. The corresponding shear modulus G and p-wave velocity V_p and s-wave velocity V_s are determined based on theories of continuum mechanics and wave propagation:

Table 2. P-wave and S-wave Velocity of Soil

Soil Type	$V_p(m/s)$	$V_s(m/s)$
Fill	81.22	43.41
Clay Silt	99.15	47.63
Soft soil	68.01	32.67
Coarse Sand	152.6	81.56

$$V_p = \sqrt{\frac{E_{oed}}{\rho}} \quad \text{where} \quad E_{ode} = \frac{(1-\nu)E}{(1+\nu)(1-2\nu)} \qquad (1)$$

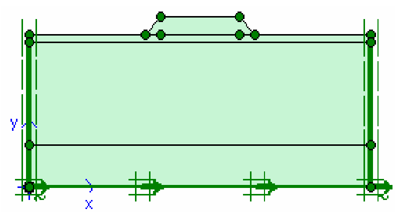
$$V_s = \sqrt{\frac{G}{\rho}} \quad \text{where} \quad G = \frac{E}{2(1+\nu)} \qquad (2)$$

The boundary conditions were changed to the standard earthquake boundary that was automatically generated as absorbent boundaries (Plaxis, 2002). The left-hand and right-hand boundaries have prescribed displacement u_x = 0.01 m and the bottom boundary has prescribed displacement u_y =0.00 m.

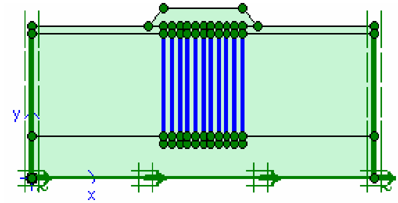
The material damping was considered for the soil. In PLAXIS, a global material damping term (the Rayleigh damping) is assumed, which is proportional to the mass and stiffness of the system. The damping matrix can be written as a combination of the mass matrix and the stiffness matrix:

$$[C]=\alpha[M]+\beta[K] \tag{3}$$

where $[C]$ is damping matrix $[M]$ is mass matrix $[K]$ is stiffness matrix



a. Untreated by micopiles



b. Treated by micopiles

FIG. 3. FEM Dynamic Model.

The Rayleigh coefficients $\alpha$and $\beta$are difficult to determine from soil tests. Based on previous literatures, they can be estimated from the fundamental frequency of the pile-soil system and the hysteretic damping ratio. However, this information was not available for such purpose in this example. The Rayleigh coefficients were assumed as α= 0.01 for mass matrix and β= 0.01 for the stiffness matrix. Since there is lack of recorded strong ground motions, the default ground motion file named 225a.smc (see FIG. 4.) in PLAXIS is set as the rock base ground motion(Plaxis, 2002). The calculation was carried out to 250 time steps with a time interval 10 seconds.

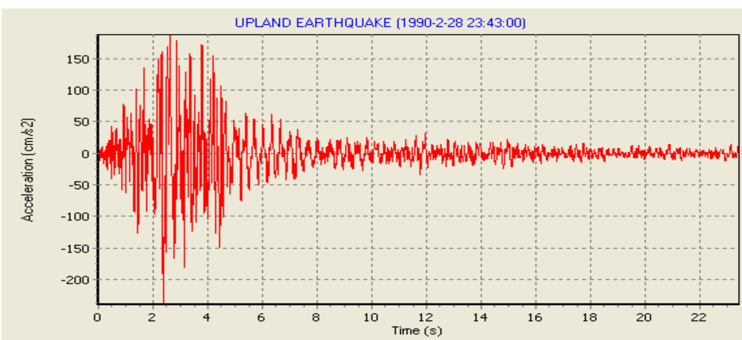


FIG. 4. Record of upland earthquake.

Figure 5 and Figure 6 show time-acceleration curves of point in the middle of the embankment. Figure 5 shows that the maximum acceleration at point E (22, 9) is 1.466 m/s^2 for embankment on untreated soils and 1.065m/s^2 for embankment treated by micropiles.

By comparing Figure 5 and Figure 6, it is found that the acceleration at point E (22, 9) of the untreated embankment is much higher than the treated embankment. The result shows that the piles are effective in increasing the earthquake resistance of soil.

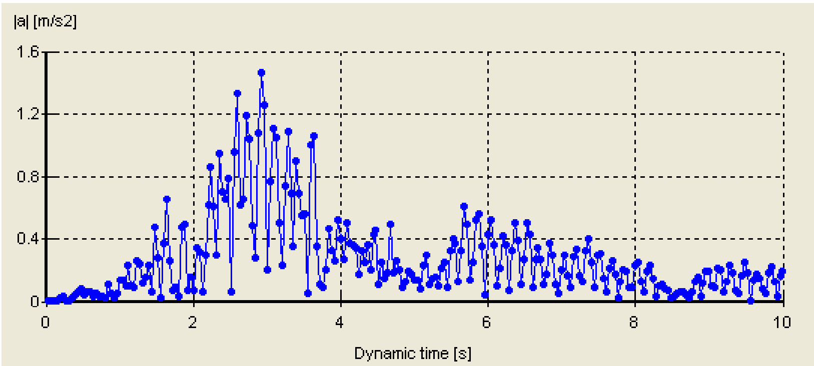


FIG. 5. Time-acceleration curves of E Point before the soil treatment.

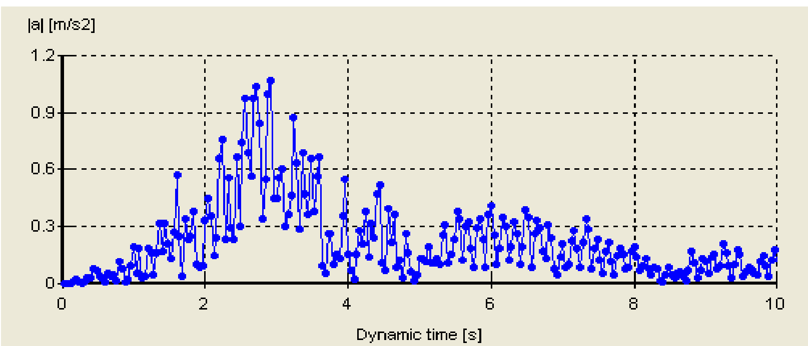


FIG. 6. Time-acceleration curves of E Point after the soil treatment.

CONCLUSIONS

The displacements and dynamic characteristics of pure soil foundation and micropiles reinforced foundation were analyzed using finite element code PLAXIS. The displacement caused by embankment static loading and acceleration of the embankment caused by seismic loading were calculated and compared. It is found that micropiles treated soil can greatly reduce the settlement of the embankment and mitigate seismic response of the embankment. The results of this study provide valuable information about the design and application of micropiles.

ACKNOWLEDGMENTS

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Theoretical and Experimental Study on the Vertical Bearing Behavior of Super-Long Filling Piles in Soft Soil Area

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ABSTRACT: Based on the vertical static load test results on a test pile, located in the soft soil, the load transfer mechanism and bearing behavior of super-long filling piles is first discussed. Then, a method of predicting the axial bearing capacity of super-long piles by the pile top settlement is advised. By introducing the linear elastic-fully plastic model and tri-broken-line model to fit the development of shaft resistance and tip resistance respectively, analytical solutions of axial load capacity is derived under elastic stage of the subsoil. Finally, the present method is applied to analyze the data from the test pile, from which good agreement between the predicted and measured pile top load-settlement curves is found. The results also show that buckling stability analysis and deformation checks are necessary for super-long filling piles, and the axial bearing capacity of super-long piles should be controlled by the allowable settlement on the pile top.

INTRODUCTION

The specific definition of super-long filling piles is not clear so far, but it generally refers to piles whose length is more than 50 m or length to diameter ratio (l/d) is 100 (Lin et al., 1999), and the bearing capacity is mainly due to shaft resistance. Due to the super length, this kind of piles behaves quite differently. Under normal working load, the tip resistance of super-long pile is only a small portion of the total capacity and can be neglected. In addition, studies by Zhu et al (2003) shows that: (i) for a given pile vertical load, if all other factors remain constant, the pile top settlement does not increase with the pile length; (ii) the state of maximum pile shaft resistance or failure of pile shaft structure can be regarded as the ultimate bearing state of the super-long pile. To determine the axial bearing capacity of the super-long pile top settlement is very important (Lu 1981).

In recent years, super-long filling piles are widely used. Nevertheless, due to the complex load transfer mechanism, relevant design methods have not been presented (Poulos 1989; Yang 1998). Therefore, traditional theories have to be adopted in design and calculation. However, for super-long filling piles, the allowable pile top settlement is usually determined by the serviceability criteria of the superstructure and will control the axial load capacity (Zhao 2000; Wang et al. 1999). A large number of pile tests (Shi and Liang 1994) show that, when the *l*/*d* exceeds a certain value, the tip resistance mobilization will decrease or even not mobilize any more. Thus, the validity of calculating the axial load capacity of super-long piles by traditional methods remains to be discussed. Because of these reasons, the piles are generally lengthened to satisfy the bearing capacity in design, but this will lead to unnecessary waste. Fortunately, the load transfer mechanism of super-long piles have already been studied by some researchers (Kraft 1981; Cao 1986).

To discuss the load transfer mechanism and more suitable analysis methods for super-long piles, a field load test was first done on a test pile in the typical soft soil area of Dongting Lakes in China. Then, a method to determine the vertical bearing capacity of super-long piles by the pile top settlement is proposed in this paper.

EXPERIMENTAL STUDY

Test Design

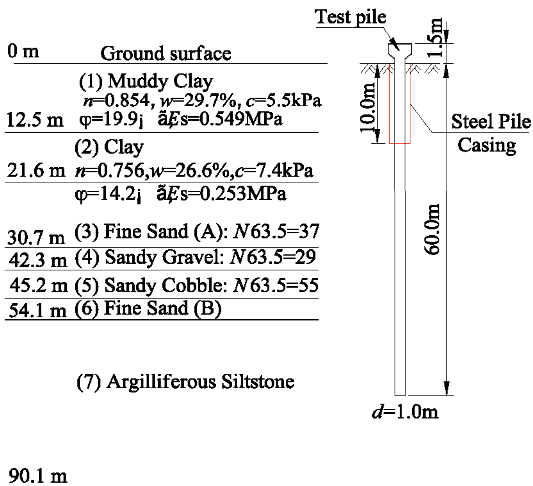

Fig. 1. Soil profile around the test pile.

The soil around the test pile is shown in Fig. 1. The test pile with a design diameter of 1.0 m and length of 60 m was formed by stabilized liquid method. A steel pipe of 1.3 m outside diameter, 7 mm wall thickness and 10 m length was used to support the upper unstable segment of the pile bore. The pipe was driven 9.2 m into the ground. The measured thickness of sediment under the pile tip was 50 mm. Anchorage of the counter-force system was provided by a pair of 1.8 m diameter tension piles each with an uplift capacity of about 9 MN (Fig. 2).

Four dial gages were placed symmetrically to measure the pile top settlement. Another 2 dial gages are placed around the steel drive pipe on pile top to observe the settlement of the pipe. In order to determine the distribution of axial force accurately, two kinds of strain gauges, i.e., steel and concrete, were used. In total 34 groups of 136 fully airtight concrete strain gauges were embedded in the test pile.

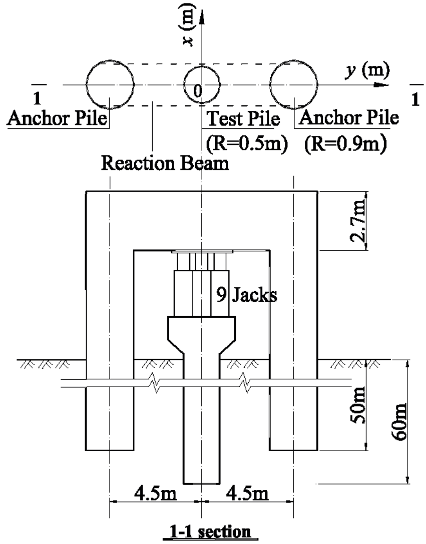

Fig. 2. Sketch of the test equipment.

Test Results

The final steady load on the test pile was 16.72 MN and corresponding top settlement was 35.31 mm. In the loading process, when the load was increased to 17.28 MN, the pile top settlement increased sharply, and exceeded the maximum range of the indicating gauge and could not keep stable. At the same time, the load only kept stable for 3 minutes and then a blare sent out down from the ground. After that, obvious offset of the pile top was observed, and the load decreased from 17.28 MN to 11.00 MN quickly. Furthermore, crack appeared in the ground surface around the pile top. From these, buckling failure of the pile shaft was determined.

The measured pile top load and settlement is shown in Fig. 3. The calculated axial

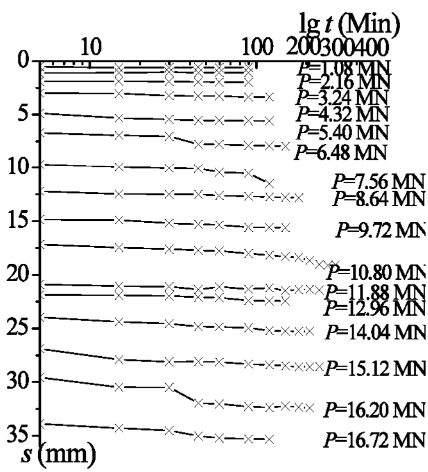

Fig. 3. The s~lgt curve.

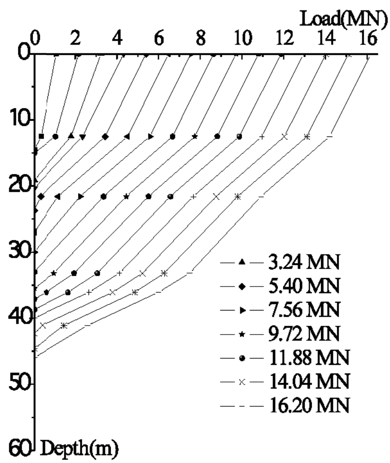

Fig. 4. Pile shaft axial force distribution.