Geomechanics and Engineering, *Vol. 13, No. 6 (2017) 929-946*
DOI: https://doi.org/10.12989/gae.2017.13.6.929

Calculation models and stability of composite foundation treated with compaction piles

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(Received June 26, 2017, Revised February 17, 2017, Accepted May 22, 2017)

Abstract. Composite foundation treated with compaction piles can eliminate collapsibility and improve the bearing capacity of foundation in loess area. However, the large number of piles in the composite foundation leads to difficulties in the analysis of such type of engineering works. This paper proposes two simplified methods to quantify the stability of composite foundation treated with a large number of compaction piles. The first method is based on the principle of making the area replacement ratios of the simplified model as the same time as the practical engineering situation. Then, discrete piles arranged in a triangular shape can be simplified in the model where the annular piles and compacted soil are arranged alternately. The second method implements equivalent continuous treatment in the pile-soil area and makes the whole treated region equivalent to a type of composite material. Both methods have been verified using treated foundation of an oil storage tank. The results have shown that the differences in the settlement values obtained from the water filled test in the field and those calculated by the two simplified methods are negligible. Using stability analysis, the difference ratios of the static and dynamic safety factors of the composite foundation treated with compaction piles calculated by these two simplified methods are found to be 3.56% and 5.32%, respectively. At the same time, both static and dynamic safety factors are larger than the general safety factor, which should be greater than or equal to 2.0 according to the provisions in civil engineering. This indicates that after being treated with compaction piles, the bearing capacity of the composite foundation is effectively improved and the foundation has enough safety reserve.

Keywords: composite foundation; compaction piles; stability; safety factor; soil-structure interaction

1. Introduction

Because of the reliability and economic implications, composite foundation treated with compaction piles are widely used in the reinforcement treatments of fill soil, cohesive soil, loose sand, and collapsible loess. The reinforcement mechanism of the composite foundation depends on the principle that both the compaction piles and the compacted soil bear the load of the upper

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http://www.techno-press.org/?journal=gae&subpage=7 ISSN: 2005-307X (Print), 2092-6219 (Online)

structure up to a certain proportion. Moreover, the soil beneath the foundation is compacted, and its bearing capacity is significantly improved. The area around the compacted soil can constrain the piles in return; then, the compaction piles and compacted soil work together to bear the load from the foundation.

Currently, some progress has been made in terms of composite foundation research, such as the work mechanism of composite foundation treated with compaction piles, the roles of pile and soil in composite foundation, the interaction characteristics between pile and soil, and factors that affect the bearing capacity and deformation of the composite foundation. Based on Biot consolidation theory, Chen *et al.* (2001) used the finite element method to simulate the behavior of composite foundation. Ying *et al.* (2005) verified the feasibility of composite foundation of a large oil storage tank treated with compaction piles using field test, and discussed the change rules of foundation deformation, pile-soil stress and pore water pressure. Cao *et al.* (2006) analyzed the stresses of lime-soil compaction pile based on the unified strength theory and discussed the stress of the soil around the hole in the compacted and the hole formation process of lime-soil compaction pile. *Tan et al.* (2011) studied the factors affecting the bearing performances of the composite foundation with single-flexible-pile or multi-flexible-pile using ANSYS program. Sarkar and Maheshwari (2011) found that separation and sliding between the soil and pile had a significant effect on the complex behavior of pile groups under dynamic conditions, and the behavior of the soil medium surrounding the piles was nonlinear during strong excitations because of the separation between the soil and pile. Hasan and Mehrnaz (2011) proposed a simple analytical solution to study pile-soil-pile interaction in pile groups under dynamic loads. Mi and Yang (2012) evaluated the effect of the compaction pile in collapsible loess foundation by considering different factors such as distance between the centers of two piles, processing depth, processing area, and pile hole filling. Kahyaoglu *et al.* (2012) used numerical analysis to study pile-soil interaction under relative movement between the piles and the moving soil. Zhao *et al.* (2013) compared the bearing capacity and the pile-soil stress ratio of the composite foundation treated with different forms and recommended that the effect of pile group on the composite foundation should be considered. Fattah *et al.* (2013) used the finite element technique to analyze pile-soil systems in undrained conditions; the pile was modeled as an elastic-plastic material, and the soil was assumed to follow the modified Cam clay model. Cui *et al.* (2013) derived a formula describing the relationship between the settlement of composite foundation and the radius of the compacted zone. Based on the unified strength theory and applying the conditions of compatible deformation for both soil and piles. Yasser and Ahmed (2014) used a numerical method to study pile-soil interaction subjected to an axial or lateral load. A parametric study was conducted to study the effect of crucial design parameters, such as the soil's modulus of elasticity and the radius of the soil surrounding the pile. Ghazavi *et al.* (2014) studied the pile-soil interaction by considering batter pile group with several other parameters that affect the pile-soil-pile interaction, such as pile-pile distance, group geometry, and length of piles. Ma *et al.* (2016) evaluated the bearing capacity of the composite foundation using the static load test of a single pile and composition foundation, and implemented 3-D model of the composite foundation treated with compaction piles using the finite difference equation method.

In contrast, there have been only a few studies on the simplification model and stability analysis of the composite foundation treated with compaction piles using FEM method. Yang and Yin (1998) used the composite constitutive finite element method to disperse the piles of composite foundation into the soil around the piles and verified the validity of the method. He *et al.* have (2012) noted that the problem regarding the boundary conditions of a dynamic analysis for the composite foundation should be paid attention. In addition, He *et al.* (2012) have studied the dynamic stability of the composite foundation using the strength reduction method. Liu *et al.* (2014) considered that the compaction piles in practical engineering were arranged in an equilateral triangle; thus, it cannot be accurately modeled in a two-dimensional software, and an equivalent treatment was conducted for the problem.

In summary, the main research methods for composite foundation are practical tests and numerical simulations, and the numerical simulation is usually based on a certain simplification such as simplifying the 3-D model as a plane problem or studying a part of the compaction piles. The number of piles in the composite foundation is usually very large, therefore, the principal work to simplify the composite foundation is reasonable. In addition, the number of studies on the strength reduction method on the stability of composite foundation is relatively low. This paper puts forward two new methods to simplify the composite foundation, in order to overcome the difficulty in numerical simulation due to the large number of piles. Using simplified methods for the composite foundation, this paper investigates the foundation treatment of an oil storage tank. To closely model and study the actual behavior and stability of the composite foundation of the oil storage tank assuming specific characteristics for the oil storage tank, large-scale spatial numerical simulation models are established taking in consider both soil-structure and fluid-structure interactions. Two simplified methods are used to simulate the water filling test on the site and compare the calculation results of the two simplified methods where the two methods can be validated to a certain extent. Lastly, the safety of the composite foundation is evaluated on the basis of static and dynamic stability analysis. The two simplified methods proposed in this paper can provide new approaches for the safety evaluation of similar types of engineering projects in the future.

2. Simplified models of composite foundation treated with compaction piles

The compaction piles and compacted soil for the composite foundation have interactions, and the number of piles is very large and their shapes are slender. Thus, the simplification of the compaction piles and the compacted soil to study the composite foundation is very essential. The pile-soil region of the composite foundation can be defined as a cylindrical body under the concrete foundation, with a radius of which is greater than or equal to the radius of the concrete foundation and a height extended from the composite foundation surface to the bottom of the pile.

As shown in Fig. 1, hundreds of piles are arranged in real practical engineering and all of the piles can be modeled separately using the finite element method. However, this can satisfy the need of engineering practice, but the convergence and calculation efficiency are very difficult to control due to the large number of contacts and elements for the slender piles. Therefore, in this paper, the pile-soil region is reasonably simplified, namely the scattered triangular piles in practical engineering are treated by an equivalent continuous method.

2.1 The first simplified method of composite foundation

Based on the principle of making the area replacement ratios of the simplified model and practical engineering the same (as much as possible), an equivalent continuous treatment is carried out for the piles arranged in a normal triangle in practical engineering (Fig. 1). The pile-soil region for the composite foundation is simplified as a model where the annular piles and the compacted

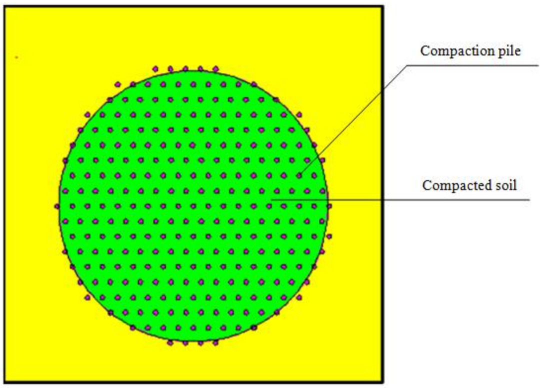
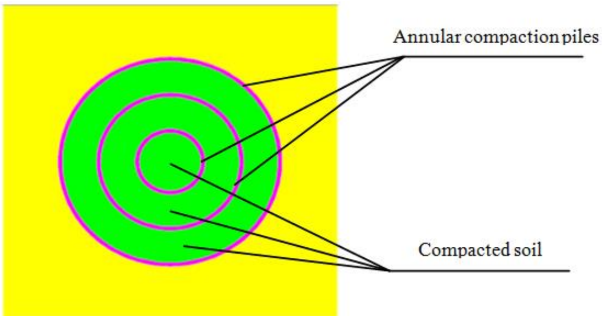
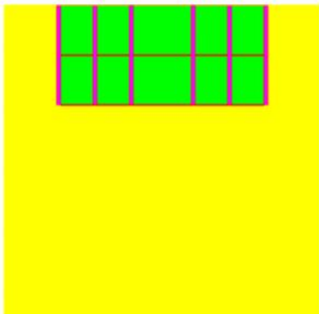


Fig. 1 Pile arrangement

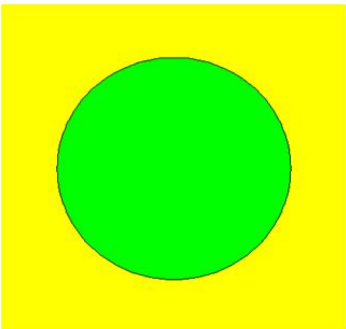


(a) Plan of the first method

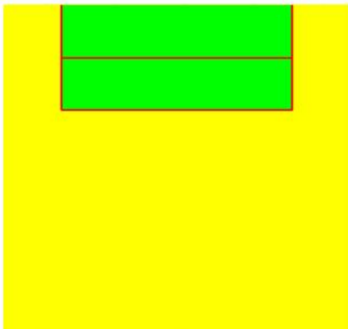


(b) Profile of the first method

Fig. 2 Simplification of composite foundation by the first simplified method



(a) Plan of the second method



(b) Profile of the second method

Fig. 3 Simplification of composite foundation by the second simplified method

soil are arranged alternately, as shown in Fig. 2. Eqs. (1)-(2) are used to calculate the area replacement ratio of practical engineering

$$m = \frac{d^2}{d_e^2} = \frac{nA_p}{A} \tag{1}$$

For the piles that are arranged in a regular triangle

$$d_e = 1.05s \tag{2}$$

where m is the area replacement ratio; d is the average diameter of the pile; d_e is the equivalent diameter of a pile that shares the foundation treatment area; n is the number of piles; A_p is the area of a single pile; A is the total area of the pile-soil region; and s is the pile spacing.

2.2 The second simplified method of composite foundation

The second method is to carry out an equivalent continuous treatment in the whole pile-soil region (Fig. 3), namely the piles and the compacted soil are simplified as a type of composite material. As a result, the number of materials and elements can be reduced, which greatly improves the calculation efficiency and reduces the difficulty of calculation and convergence. On the basis of the second simplified method, the next important step is to determine the parameters of the composite material.

According to the existing literature, the composite modulus E_{sp} and the foundation bearing capacity $f_{sp,k}$ of the composite foundation can be calculated as follows

$$f_{sp,k} = mf_{p,k} + (1-m)\, f_{s,k} \tag{3}$$

$$E_{sp} = mE_p + (1-m)E_s \tag{4}$$

The composite density ρ_{sp}, composite cohesive strength c_{sp}, composite friction angle φ_{sp} and composite Poisson ratio v_{sp} of the foundation after being treated can be expressed as Eqs. (5)-(8)

$$\rho_{sp} = m\rho_p + (1-m)\rho_s \tag{5}$$

$$c_{sp} = mc_p + (1-m)c_s \tag{6}$$

$$\varphi_{sp} = m\varphi_p + (1-m)\varphi_s \tag{7}$$

$$\upsilon_{sp} = m\upsilon_p + (1-m)\upsilon_s \tag{8}$$

where ρ_{sp}, c_{sp}, φ_{sp} and v_{sp} are the density, cohesion, friction angle and Poisson's ratio of the composite foundation, respectively; ρ_p, c_p, φ_p and v_p are the density, cohesion, friction angle and Poisson's ratio of the pile, respectively; ρ_s, c_s, φ_s and v_s are the density, cohesion, friction angle and Poisson's ratio of the compacted soil, respectively.

2.3 Simplification of the surrounding loess region

In the real engineering situation, the peripheral region around the treated pile-soil region is infinite, but in the finite element calculation, it is required to cut out a reasonable finite region instead of having an infinite region. According to the results of the pile test (Cook 1980, Shi 1983), the radius that shear deformation can be neglected in the surrounding region is 12D, where D is the diameter of the pile.

2.4 The treatment of boundary condition

The numerical simulation model replaces the infinite region with finite boundary conditions. Some differences are produced when compared with the true transmission of seismic waves, but through the treatment of boundary conditions, these effects can be reduced to a certain extent. Existing research investigations have shown that under the premise of meeting certain boundary conditions, constraint types of the bottom and side boundaries of the composite foundation have little effect on the calculation results (Hu *et al.* 2009). Three displacement freedom degrees are restricted on both four sides and the bottom of the foundation in this paper.

3. Stability of composite foundation

3.1 Transient dynamic equation and its solution

The transient dynamic analysis method can be used to determine the response of the structure under dynamic action. The response of the structure can be obtained by solving the dynamic equation of the structure, such as displacement, velocity, and acceleration.

The basic motion equation of the composite foundation under earthquake action can be expressed as the following (Wang and Shao 2000, Cheng and Zheng 2011)

$$\mathbf{M}\ddot{\mathbf{u}}(t)+\mathbf{C}\dot{\mathbf{u}}(t)+\mathbf{K}\mathbf{u}(t)=-\mathbf{M}\ddot{\mathbf{u}}_g(t) \tag{9}$$

where $\ddot{\boldsymbol{u}}(t)$, $\dot{\mathbf{u}}(t)$ and $\boldsymbol{u}(t)$ are the acceleration array, velocity array and displacement array of the nodes of the isolated body respectively; **M**, **C** and **K** are the mass matrix, damping matrix and stiffness matrix of the isolation body respectively; $\ddot{\boldsymbol{u}}_{\mathrm{g}}(t)$ is the earthquake acceleration. The Rayleigh damping matrix is used as the damping matrix of the isolation body

$$\mathbf{C}=\alpha\mathbf{M}+\beta\mathbf{K} \tag{10}$$

where α is the mass damping coefficient and β is the stiffness damping coefficient. They can be calculated by Eq. (11)

$$\alpha=\frac{2\omega_i\omega_j}{\omega_i+\omega_j}\xi,\quad \beta=\frac{2}{\omega_i+\omega_j}\xi \tag{11}$$

where ξ is the damping ratio of the i or j mode (approximately, $\xi_i=\xi_j$), and ω_i and ω_j are the natural frequencies of vibration (their values can be obtained by a modal analysis of the isolation body).

To obtain the top maximum displacement of the composite foundation under seismic action, we suppose that the duration of the seismic wave is T_d. The time history curve of displacement can be obtained by the time history analysis of the isolated body. The Newmark integral method is used to solve Eq. (9)

$$\mathbf{u}_{t+\Delta t}=\mathbf{u}_t+\Delta t\dot{\mathbf{u}}_t+(0.5\text{-}\delta)\Delta t^2\ddot{\mathbf{u}}_t+\delta\Delta t^2\ddot{\mathbf{u}}_{t+\Delta t} \tag{12}$$

$$\dot{\mathbf{u}}_{t+\Delta t}=\dot{\mathbf{u}}_t+(1\text{-}\gamma)\Delta t\ddot{\mathbf{u}}_t+\gamma\Delta t\ddot{\mathbf{u}}_{t+\Delta t} \tag{13}$$

where γ and δ are constants.

Then, the differential equation of motion at $t+\Delta t$ can be expressed as

$$\mathbf{M}\ddot{\mathbf{u}}_{t+\Delta t}+\mathbf{C}\dot{\mathbf{u}}_{t+\Delta t}+\mathbf{K}\mathbf{u}_{t+\Delta t}=-\mathbf{M}\ddot{\mathbf{u}}_{g(t+\Delta t)} \tag{14}$$

If we take γ=0.5, δ=*0.25* and $\Delta t\leq 0.01T_{\max}$ ($T_{\max}$ is the maximum natural vibration period of the isolation body), then the Newmark method will be unconditionally stable, and it can make the results reach a certain accuracy.

Substituting Eqs. (12) and (13) into Eqs. (14) and (15) can be obtained

$$(\mathbf{M}+\frac{\Delta t}{2}\mathbf{C})\ddot{\mathbf{u}}_{t+\Delta t}+\mathbf{C}(\dot{\mathbf{u}}_t+\frac{\Delta t}{2}\ddot{\mathbf{u}}_t)+\mathbf{K}\mathbf{u}_{t+\Delta t}=-\mathbf{M}\ddot{\mathbf{u}}_{g(t+\Delta t)} \tag{15}$$

From Eq. (12), we can obtain the following equation

$$\ddot{\mathbf{u}}_{t+\Delta t}=\frac{4}{\Delta t^2}(\mathbf{u}_{t+\Delta t}-\mathbf{u}_t)-\frac{4}{\Delta t}\dot{\mathbf{u}}_t-\ddot{\mathbf{u}}_t \tag{16}$$

Substituting Eq. (16) into Eq. (15), we obtain

$$(\mathbf{K}+\frac{2}{\Delta t}\mathbf{C}+\frac{4}{\Delta t^2}\mathbf{M})\mathbf{u}_{t+\Delta t}=\mathbf{C}(\frac{2}{\Delta t}\mathbf{u}_t+\dot{\mathbf{u}}_t)+(\frac{4}{\Delta t^2}\mathbf{u}_t+\frac{4}{\Delta t}\dot{\mathbf{u}}_t+\dot{\mathbf{u}}_t)-\mathbf{M}\dot{\mathbf{u}}_{g(t+\Delta t)} \tag{17}$$

After obtaining $\mathbf{u}_{t+\Delta t}$ by Eq. (17), $\ddot{\mathbf{u}}_{t+\Delta t}$ and $\dot{\mathbf{u}}_{t+\Delta t}$ can be obtained by Eqs. (16) and (13), respectively. Through the above solution, we can obtain the time T' when the maximum horizontal displacement of the composite foundation appears. For a geometric invariant system, the solution of the positive and negative problem is unique, so the composite foundation is in the most unfavorable condition at T'.

3.2 Strength reduction method

The so-called strength reduction method (Cheng *et al.* 2016, Cheng and Zheng 2011, Zhao and Zheng 2003, Matsul and San 1992, Smith and Griffiths 1988) describes the reduction of the soil shear strength parameters c and $\tan\varphi$ by η. The original geotechnical shear strength parameters c and φ are replaced with the virtual shear strength index c' and φ', until the composite foundation reaches the limit failure state. Then, the reduction coefficient η of the soil shear strength parameters is the safety factor

$$c'=\frac{c}{\eta},\quad \varphi'=\arctan\left(\frac{\tan\varphi}{\eta}\right) \tag{18}$$

and

$$\tau=c'+\sigma\tan\varphi'=\frac{\mathrm{c}}{\eta}+\sigma\tan\varphi' \tag{19}$$

where c is the cohesive strength and φ is the friction angle.

Fig. 4 Full view of the oil depot

Fig. 5 The holing construction of compaction piles

4. Numerical example

4.1 Engineering background

Lintao oil depot of PetroChina Gansu branch is an expansion project as shown in Fig. 4. Six steel storage tanks with a 5000 cubic meter capacity were built, and composite foundation treated with compaction piles was utilized as shown in Fig. 5. The pile diameter is 0.4 m; the pile spacing is 1 m; the piles are arranged in a regular triangle; the pile length is 16 m. The pile hole is filled with 2:8 lime soil in the upper 8 m section, and the lower 8 m section is filled with plain soil. The design requirement for the bearing capacity for the composite foundation is greater than 220 kPa.

4.2 Finite element model

The element Solid 45 is used for the soil and concrete foundation, which has 8 nodes and can reflect plasticity, creep, expansion, stress stiffening, large deformation, and large strain. The element Shell 181 is used for the tank, which has 4 nodes and 6 six degrees of freedom in each node; Shell 181 is suitable for shell structures with thin to medium thickness and has the function of stress stiffening and large deformation, so it can reflect the nonlinear characteristic. The element Fluid 80 is used to simulate liquid inside the liquid storage tank, Fluid 80 is suitable to simulate the flow of liquid in the container without net flow rate, hydrostatic pressure calculation, fluid-solid interaction calculation; Fluid80 has 8 nodes and 3 degrees of freedom in each node (namely, translation in the direction of x, y and z). To reflect the real behaviors, two contact pairs are set between the oil storage tank and the concrete foundation and between the concrete foundation and the composite foundation. To improve the computational efficiency and convergence, specific parameters of contact pairs are set, through continuous trials. The contact stiffness factor and the permeability coefficient are set as 0.01 (Liang 2013). The direct coupling method is used for tank-fluid coupling analysis, namely degrees of node freedom are coupled to treat the tank-liquid interaction in ANSYS.

4.2.1 Materials

Because the focus of this research focus is the treatment effect of the composite foundation of the oil storage tank, the D-P model is used for the composite foundation. Elastic material is used for the concrete foundation, the oil storage tank, and fluid material where ANSYS is used to simulate the liquid. The material parameters are shown in Table 1 to Table 3. The soil material parameters of the first method are obtained from the actual situation of the project, and the soil material parameters of the second method (composite parameters) can be calculated by Eqs. (5)-(8).

Table 1 Parameters of soil materials (the first method)

Materials	Density ρ/(kg·m^{-3})	Elastic modulus E/(MPa)	Cohesion c/(kPa)	Friction angle φ/(°)	Poisson's ratio v	Friction coefficient μ
Pure soil	1925	30.3	90	31.30	0.3	0.25
Lime-soil	1904	100	450	37.34	0.3	0.25
Loess	1650	20	30.31	25.10	0.38	0.25
Compacted soil	1843	28.7	37.55	27.20	0.35	0.25

Table 2 Parameters of soil materials (the second method)

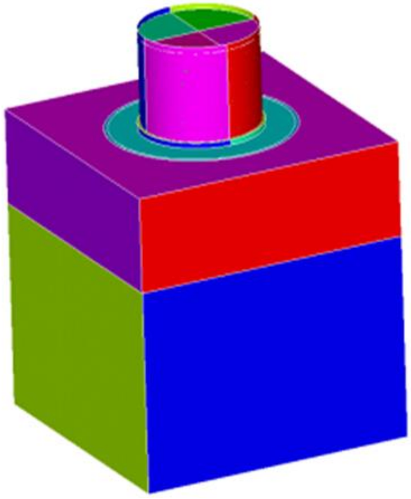
Materials	Density ρ/(kg/m^3)	Elastic modulus E/(MPa)	Cohesion c/(kPa)	Friction angle φ/(°)	Poisson's ratio v	Friction coefficient μ
Composite material (0-8 m)	1852	39.40	99.42	28.72	0.34	0.25
Composite material (8-16 m)	1855	28.94	45.42	27.82	0.34	0.25
Loess	1650	20	30.31	25.10	0.38	0.25

Table 3 Material parameters of foundation, oil storage tank and fluid

Materials	Density ρ/(kg·m^{-3})	Elastic modulus E /(MPa)	Bulk modulus K/(MPa)	Poisson's ratio v	Viscous coefficient μ/(N·S/m)
Concrete	2500	3×10^4	--	0.2	--

Table 3 Continued

Materials	Density ρ/(kg·m^{-3})	Elastic modulus E/(MPa)	Bulk modulus K/(MPa)	Poisson's ratio ν	Viscous coefficient μ/(N·S/m)
Steel	7800	2×10^5	--	0.3	--
Fluid	800	--	3×10^4	--	0.00224


(a) Geometric model

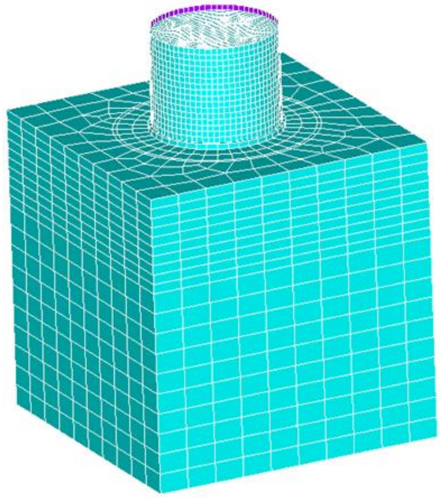

(b) Finite element model

Fig. 6 Calculation model

4.2.2 Model

After simplifying the composite foundation by the first and the second methods, the geometric and finite element models, which consider the soil-structure interaction and fluid-structure interaction, as shown in Figs. 6(a)-6(b). A cube with a dimension of 50 m×50 m×50 m is cut out as the foundation; the pile length is 16 m; the height of the soil that is under the pile bottom is 34 m; the diameter and height of the concrete foundation are 22 m and 0.9 m, respectively; diameter of the tank is 21 m, its thickness is 0.02 m, and its height is 16 m; the liquid height is 15 m. In addition, for the first method, the area replacement ratio calculated by Eqs. (1)-(2) is 14.5%. By adjusting the annular pile widths and their spacing where annular piles width is 0.65 m, the radius of the center compacted soil is 4.6 m, and the width of the other compacted soil is 5.1 m. To meet the requirements for the equal area replacement ratio, the total area of the annular pile is 41.6325π m^2, the total area of the pile-soil area of the composite foundation is 280.5625π m^2, and the ratio of the annular piles area and the total pile-soil area is 14.84%. The area replacement ratio of the first simplified method is closer to the ratio 14.5% of the actual project.

4.3 Model validation

To validate the simplified methods, the water filling test in the field is simulated by the simplified methods where the layout of measuring points is shown in Fig. 7. The settlement values calculated by the two simplified methods are compared with the data recorded from the water filling test in the field, as shown in Table 4.

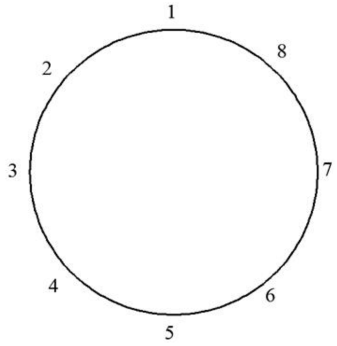


Fig. 7 Water filling test observation point

Table 4 Settlement data comparison (mm)

Points	Data of water filled test			First method	Second method
	2012.08.04	2012.08.05	2012.08.06		
1	45	48	52	67.74	67.33
2	45	48	51	67.83	67.07
3	45	49	53	66.71	66.34
4	46	49	53	68.20	67.14
5	44	47	51	70.20	67.62
6	45	48	52	68.66	66.73
7	44	48	51	66.84	66.59
8	44	47	51	67.11	67.55

From Table 4, it can be seen that the settlement values of each observation point of the water filling test are gradually increased over the time. However, because the testing time in the field is short, a further increase of the settlement value caused by consolidation could not be measured. As a result, there are some gaps between the settlement values calculated by the two numerical simplified methods and the data measured in the field, but the settlement values calculated by the two simplified methods of the measuring points are very close. Thus, the two simplified methods are validated up to a certain degree. According to an available reference (API-650 2007), the settlement should be less than 150 mm; thus, after being treated with compaction piles, the composite foundation settlement can meet the requirement, and its bearing capacity can be improved effectively.

The difference between the actual and predicted settlements can be explained by calculating consolidation settlement of the soil, the total settlement of composite foundation considering consolidation can be calculated by Eq. (20) (GB5007 2010)

$$S = \psi_s \sum_{i=1}^{n} \frac{P_0}{E_i} (Z_i \bar{\alpha}_i - Z_{i-1} \bar{\alpha}_{i-1}) \tag{20}$$

where ψ_s is the modified coefficient of settlement calculation, it is equal to 1.0 (Ding *et al.* 1996);

n is the soil layers; P_0 is the additional pressure at the bottom of the foundation when considering quasi permanent combination of loads, which depends on the weight of the concrete foundation, liquid and tank, the value is about 159173Pa; E_i is elastic modulus, as shown in Table 3; Z_i and Z_{i-1} are the distances from the bottom surface of the foundation to the ith and the i-1th layers of soil; $\overline{\alpha}_i$ and $\overline{\alpha}_{i-1}$ are the average additional stress coefficients from the bottom surface of the foundation to the ith and the i-1th layers bottom.

The final settlement of composite foundation considering consolidation calculated by Eq. (20) is 73.89 mm, the differences between the settlement calculated by Eq. (20) and the two simplified methods are small.

4.4 Static stability

The finite element strength reduction method can be used to evaluate the limit state and safety reserve of the composite foundation. This method has been used widely in the field of geotechnical engineering to calculate the safety factors. For the composite foundation of an oil storage tank in collapsible loess region, stability problem is worthy to study, which can provide a theoretical basis for the safety assessment of the project. Eq. (18) is used to calculate new values of c' and φ' based on the known reference values of c and φ. The divergence of the finite element calculation is taken as the failure criterion to determine whether the composite foundation reaches the limit state or not. Then, the safety factors can be obtained. The plastic zones and the safety factors of the static stability obtained by the two simplified methods are shown in Figs. 8 and 9 respectively.

Fig. 8 Static plastic strain of the first method (η=2.499)

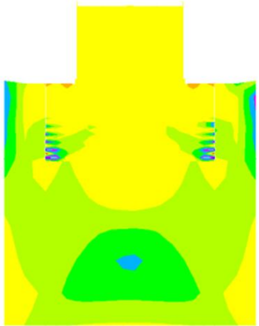


Fig. 9 Static plastic strain of the second method (η=2.410)

The static safety factors calculated by the two simplified methods are 2.499 and 2.410, respectively, and the difference ratio is 3.56%. As observed from Figs. 8 and 9, the plastic zone of the second method is wider than that of the first method. The figures also show some common features, namely the plastic zones are mainly focused in the treatment region under the oil storage tank, and the plastic deformations are even greater near the interface between the treated and untreated region because the material parameters of the two sides have large differences.

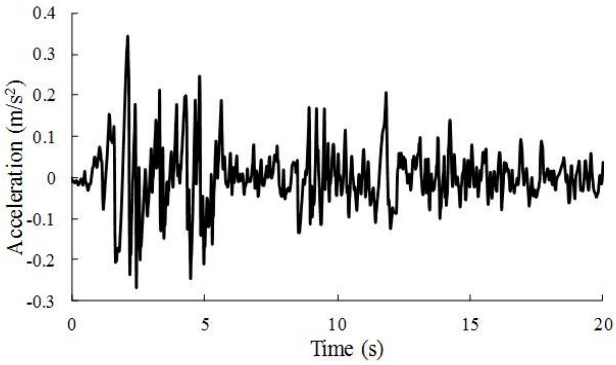


Fig. 10 El-Centro wave

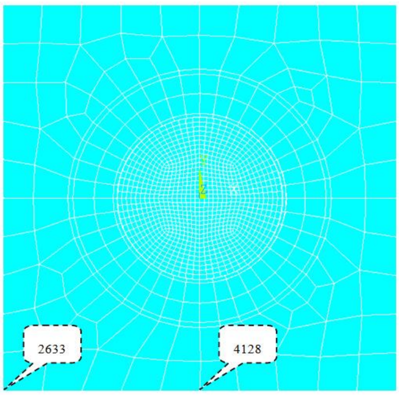


Fig. 11 The corresponding node of maximum displacement

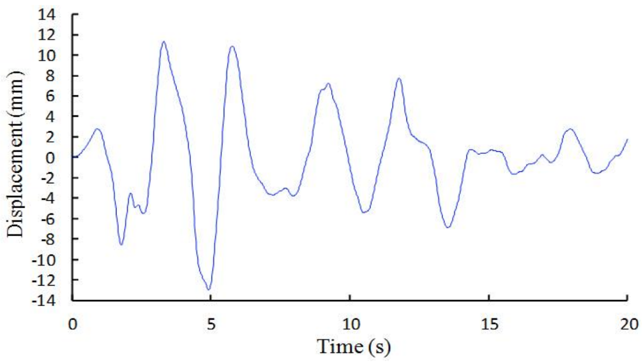


Fig. 12 Displacement time history curve of node 2633

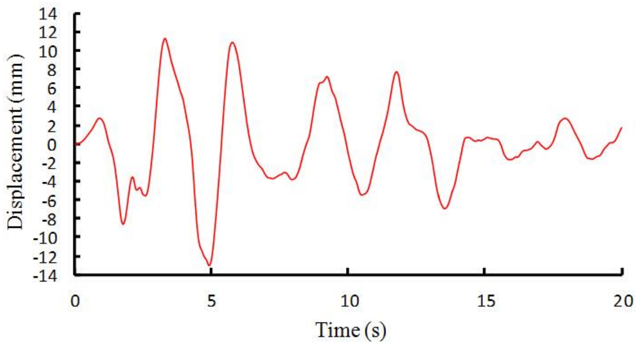


Fig. 13 Displacement time history curve of node 4128

Table 5 Nodes displacement

Node 2633		Node 4128	
Time (s)	Displacement (mm)	Time (s)	Displacement (mm)
4.88	-12.828	4.88	-12.873
4.90	-12.936	4.90	-12.981
4.92	-12.958	4.92	-13.002
4.94	-12.901	4.94	-12.945
4.96	-12.777	4.96	-12.820
4.98	-125927	4.98	-12.634
5.00	-12.332	5.00	-12.371

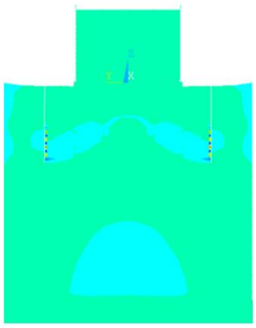


Fig. 14 Dynamic plastic zone of the first method (η=2.554)

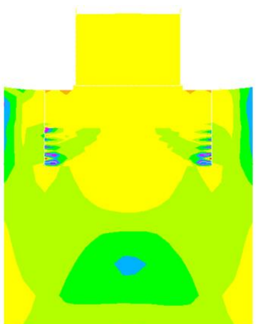


Fig. 15 Dynamic plastic zone of the second method (η=2.425)

4.5 Dynamic stability

The steps of dynamic stability analysis are as follows: (a) Remove the horizontal displacement constraint *x* of the four sides of the static model and input the seismic wave to conduct history analysis; (b) extract the maximum node horizontal displacement of the *x* direction of the four sides of the composite foundation; (c) exert the horizontal displacement obtained from step (b) on the static model as the initial displacement; and (d) use the strength reduction method to calculate the dynamic safety factors.

(1) Seismic wave

The seismic problem of the underground space is more complex because the amount of seismic effect on the upper structure is large. Therefore, this paper is based on the method of seismic analysis of the upper structure. The method of direct input acceleration is used to conduct history analysis, which is simple to conduct. The El-Centro wave as shown in Fig. 10 is selected from Pacific Earthquake Engineering Research Center (PEER), and the duration of the seismic wave is 20s.

(2) Solution of dynamic stability

Displacement constraints of the four sides of the composite foundation in the *x* direction are removed. The seismic wave shown in Fig. 10 is inputted into the model in the *x* direction. The maximum horizontal displacements corresponding to the first and second simplified methods appeared on nodes 2633 and 4128 respectively as shown in Fig. 11. The time history curves of the maximum displacements of the corresponding nodes are shown in Figs. 12 and 13 respectively.

From the time history curves of the node displacement in Figs. 12 and 13, it can be seen that the maximum displacements corresponding to the top nodes 2633 (the first method) and 4128 (the second method) for the composite foundation appear around 5 s. Then, the horizontal displacements for the corresponding nodes around 5s are extracted as shown in Table 5. The maximum displacements corresponding to the nodes 2633 and 4128 are 12.958 mm and 13.002 mm, respectively, at a corresponding time of 4.92 s.

Using the above analysis, the nodes displacement of the *x* direction of the model is extracted at 4.92 s. By removing the displacement constraints in the *x* direction for the four sides of the static model, the node displacement of the *x* direction at 4.92 s is exerted on the static model. Finally, the strength reduction method is used to solve the dynamic stability of the models that have exerted initial displacements. The results of the dynamic plastic strains and safety factors corresponding to the first method and the second method are shown in Figs. 14 and 15 respectively.

The dynamic safety factors calculated by the dynamic stability analysis using the two simplified methods are 2.554 and 2.425, as shown in Figs. 14 and 15 respectively, and the difference ratio is 5.32%. By observing the static plastic strains in Figs. 8 and 9 and the dynamic plastic strains in Figs. 14 and 15, it can be seen that the distribution of the plastic zone under the action of ground motion is wider than the static model. Moreover, both the static and dynamic safety factors of the composite foundation treated with compaction piles are larger than the general safety factor, which should be greater than or equal to 2.0 according to the provisions in civil engineering.

5. Conclusions

- Using the two simplified methods to simulate the water filling test, it was shown that the difference between the calculated values and the experimental values was small, and the validations of both methods are verified up to a certain degree. The static safety factors of the composite foundation calculated by the first and the second methods were 2.499 and 2.410, the dynamic safety factors were 2.554 and 2.425, and the difference ratios of the two types of safety factors were 3.56% and 5.32%, respectively. The static and dynamic safety factors were greater than the general safety factor, which should be greater than or equal to 2.0 according to the provisions in civil engineering. This showed that the composite foundation treated with compaction piles had enough safety reserve.

• There were some differences in the plastic zone distribution for the two methods, but the common features were that the plastic zone obtained by the static stability was more concentrated in the region treated with piles and the plastic deformation was larger, especially near the interface between the treated and untreated regions. To reduce this type of influence, the distance between the edge of the outermost piles of the composite foundation and the edge of the concrete foundation of the oil storage tank should be large enough. By comparing the results of the static and dynamic stability analyses, the plastic zones of the dynamic stability were wider than that of the static stability.

• The first method can consider the pile-soil contact and the pile-soil stress ratio up to a certain extent, but in the second method, and because of simplifying the pile-soil region as a composite material, it was not able to reflect these features.

• For the second method, it was easy to control the convergence, and the computational efficiency was higher. It exhibited advantages using the overall index such as settlement, static and dynamic safety factors to evaluate the safety of the composite foundation.

• By using the two new simplified methods, the complexity of the problem can be reduced, the computational efficiency can be improved and the accuracy of the calculation ensured at the same time. Moreover, these methods will be helpful in engineering applications.

Conflict of interests

The authors declare that there are no conflicts of interest regarding the publication of this paper.

Acknowledgements

This research is supported in part by the National Natural Science Foundation of China (Grant number: 51478212) and the Education Ministry Doctoral Tutor Foundation of China (Grant number: 20136201110003) and is a part of the Support Project of Science and Technology in Gansu province (Grant number: 144GKCA032).

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