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SETTLEMENT ANALYSIS OF GRANULAR FILL ON SOFT SOIL UNDER CIRCULAR LOAD

LIANG Xu (梁 旭)[1], WU Wei-feng(吴伟丰)[2], CAI Yuan-qiang(蔡袁强)[1], PAN Xiao-dong(潘晓东)[1]

([1] *Geotechnical Engineering Research Institute, Dept. of Civil Engineering,*
[2] *Office of Construction, Zhejiang University, Hangzhou 310027, China*)

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Abstract: This paper presents a theoretical model and new idea for designing granular fill oil-tanks foundation. The finite difference method was used to analyze the influence of the oil-tanks radius, the modulus, and thickness of the granular fill on the maximum and differential settlement of a granular fill foundation of oil-tanks. Comparison of the modelled results with those of traditional method and observation showed that the new method could be used efficiently in practice.

Key words: circular load, granular fill, shear layer, maximum settlement, differential settlement
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INTRODUCTION

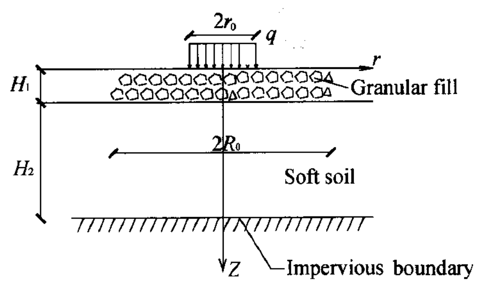
Soft soil is widely distributed over coastal and riverside regions, where because of their special locations, oil-tanks are often constructed. Large oil-tanks with heavy load are sensitive to differential settlement. Engineers are usually more concerned with the differential settlement, rather than the general settlement in the design of a foundation. On the other hand, estimation of the differential settlement is much more difficult than estimation of the maximum settlement. Therefore, when the foundation of oil-tanks is designed, the critical problem is to prevent the differential settlement from being so large as to be dangerous to the oil-tanks.

Granular fill is an effective method to improve the soft soil of oil-tanks foundations.

The functions of granular fill may be generalized as follows: (1) to improve the bearing capacity of shallow foundations; (2) to reduce the settlement of foundations; (3) to reduce differential settlement between the center and edge of load region; (4) to accelerate drainage and consolidation of soft soil. The settlement problem of oil-tank foundations becomes more complex because of the presence of granular fill. The study results of this problem has practical value in engineering, especially for application to oil-tanks.

MODEL AND DEDUCTED EQUATIONS

Sanjay Kumar and Sarvesh Chandra (1995) put forward a granular fill foundation model under strip load, which was developed from the Winkler model. The granular fill in this model was idealized into a Pasternak shear layer (1954) and the soft soil was idealized into a spring-dashpot system. The spring-constant adopted was that of the Winkler model (1876). Settlement under plane-strain situation formed by strip load applied to granular fill was analyzed, considering the primary consolidation of soft soil only. Yin (1998) put forward a one-dimensional foundation model reinforced by geosynthetic considering nonlinear spring support. However, with respect to oil-tank foundations, it is difficult to obtain results conforming satisfactorily to engineering practice when the Winkler model is used to calculate foundation settlement. This paper presents a new settlement-analysis model for granular fill-soft soil system as shown in Fig. 1. The model considers granular fill as a Pasternak shear layer and represents saturated soft soil with Terzaghi's (1943) one dimensional consolidation model. The spring represents the soil skeleton and the dashpot simulates dissipation of pore water pressure in the soil. Axial symmetrical load is applied onto granular fill. The equation controlling the response of the model at any instant of time can be obtained by considering the equilibrium of forces.



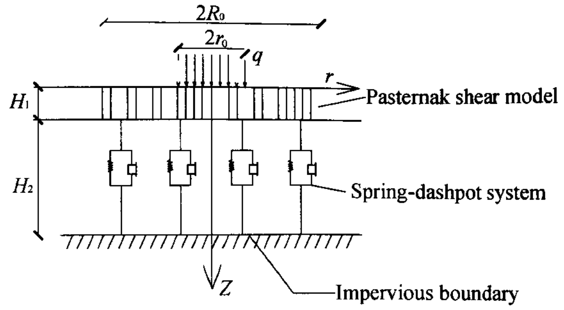


Fig.1 Conceptual diagram and suggested foundation model

With the shear layer element shown in Fig.2, the resultant force along Z coordinate axis being zero, the equation can be written as:

$$2\pi rq\mathrm{d}r - 2\pi rq_s\mathrm{d}r - 2\pi rH_1\tau_{rz} + (\tau_{rz} + \frac{\partial \tau_{rz}}{\partial r}\mathrm{d}r) \times 2\pi(r + \mathrm{d}r)H_1 = 0 \quad (1)$$

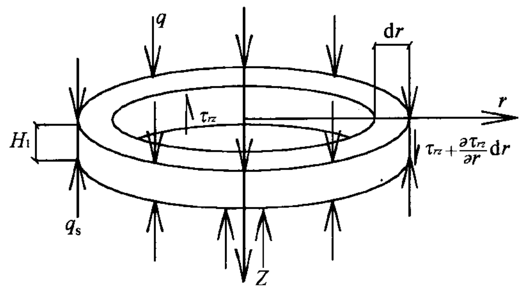


Fig.2 Forces on the shear layer element

Eq.(1) can also be simplified to:

$$qr - q_s r + \tau_{rz}H_1 + \frac{\partial \tau_{rz}}{\partial r}rH_1 = 0 \quad (2)$$

where q is the circular uniform load intensity, q_s is the vertical stress at the interface of the shear layer and the soft soil. Since $\tau_{rz} = G\frac{\partial w(r)}{\partial r}$, Eq.(1) can be written in the following form:

$$q - q_s + GH_1\frac{\partial^2 w(r)}{\partial r^2} + \frac{GH_1}{r}\frac{\partial w(r)}{\partial r} = 0 \quad (3)$$

where G and τ_{rz} are respectively the shear modulus and the shear stress of the layer. H_1 is the thickness, $w(r)$ is the vertical surface displacement, r is the distance measured from the center of the load region.

As to the underlying soft soil layer which is considered as a spring, the strain state of the elastic layer in the $r - z$ plane was studied with Vlazov model (Selvadurai, 1979; see Fig.3). The vertical force equilibrium equation can be obtained through Langrage's principle of virtual work.

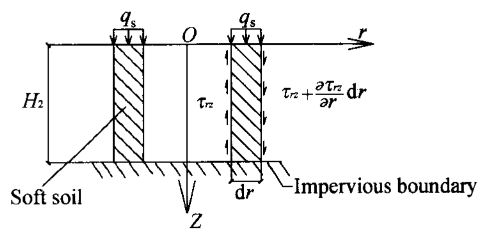


Fig.3 Vlazov model

$$\sigma' = k_s w(r) - 2t\left[\frac{\partial^2 w(r)}{\partial r^2} + \frac{1}{r}\frac{\partial w(r)}{\partial r}\right] \quad (4)$$

where $k_s = \frac{E_0}{H_2(1 - v_0^2)}$, $t = \frac{E_0 H_2}{12(1 + v_0)}$, and σ' is the effective stress. E_0 and v_0 are respectively the elastic modulus and Possion's ratio of soil skeleton. H_2 is the thickness of the soft soil. Based on Terzaghi's one-dimensional consolidation theory, the average pore water pressure(u_a) in soil can be written as(Das, 1983; William et al., 1979):

$$u_a = \sum_{m=1}^{m=\infty}\frac{2u_0}{M^2}e^{-M^2 T_V} \quad (5)$$

where $M^2 = \frac{m^2\pi^2}{4}$ and m is a positive odd integer(1,3,5 ⋯). T_V ($= \frac{C_V t}{H_2^2}$) is the time factor for the primary consolidation and u_0 is the initial pore water pressure.

According to the effective stress theory, the total stress is the sum of u_a and σ', and can be written as:

$$q_s = \sigma' + \sum_{m=1}^{m=\infty} \frac{2u_0}{M^2} e^{-M^2 T_v} \quad (6)$$

With $u_0 = q_s$, Eq.(6) can be written as:

$$q_s = \sigma' + \sum_{m=1}^{m=\infty} \frac{2q_s}{M^2} e^{-M^2 T_v} \quad (7)$$

Then the following equation can also be obtained:

$$\sigma' = q_s\left(1 - \sum_{m=1}^{m=\infty} \frac{2}{M^2} e^{-M^2 T_v}\right) \quad (8)$$

U (the average degree of consolidation in one-dimensional theory) equals $\left(1 - \sum_{m=1}^{m=\infty} \frac{2}{M^2} e^{-M^2 T_v}\right)$ Eq.(8)can be written as:

$$\sigma' = q_s U \quad (9)$$

Substituing Eq.(9) into Eq.(4) yields the following equation:

$$q_s = \frac{k_s w(r)}{U} - \frac{2t}{U}\left[\frac{\partial^2 w(r)}{\partial r^2} + \frac{1}{r}\frac{\partial w(r)}{\partial r}\right] \quad (10)$$

Substituting Eq. (10) into Eq. (3), the equation can be written as:

$$q = \frac{k_s w(r)}{U} - \left[GH_1 + \frac{\partial t}{U}\right]\frac{\partial^2 w(t)}{\partial r^2} - \frac{1}{r}\frac{\partial w(r)}{\partial r}\left[GH_1 + \frac{\partial t}{U}\right] \quad (11)$$

Eq.(12) can also be applied for characteristics analysis of granular fill on soft soil improved by prefabricated band drains or sand drains, whose main difference is the degree of consolidation at a particular instant of time.

The following normalization may be used to make Eq.(11) dimensionless:

$$\Psi = r/r_0, W = w(r)/r_0, G^* = \frac{\frac{2t}{U} + GH_1}{k_s r_0^2},$$

$$q^* = \frac{q}{k_s r_0} \quad (12)$$

Using the above normalization, Eq.(11) becomes:

$$q^* = \frac{W}{U} - G^* \frac{\partial^2 W}{\partial^2 \Psi} - \frac{G^*}{\Psi}\frac{\partial W}{\partial \Psi} \quad (13)$$

FINITE DIFFERENCE EQUATIONS

The following central finite difference-scheme is used:

$$W(\Psi_i) = \frac{W(\Psi_{i+1}) - W(\Psi_{i-1})}{2h} \quad (14)$$

$$W''(\Psi_i) = \frac{W(\Psi_{i+1}) - 2W(\Psi_i) + W(\Psi_{i-1})}{h^2} \quad (15)$$

$(i = 1,2,\cdots,n)$

Eq.(13) can be written as:

$$q_i^* = \frac{W_i}{U} - G^*\left[\frac{W_{i+1} - 2W_i + W_{i-1}}{h^2}\right] - \frac{G^*}{\Psi_i}\left[\frac{W_{i+1} - W_{i-1}}{2h}\right] \quad (16)$$

The degree of consolidation is taken as a function of time.

LOADING AND BOUNDARY CONDITIONS

q_0^* is a uniform non-dimensional load acting over a radius($2r_0$). Due to symmetry about the center of the load region, only radius (r_0) needs to be calculated and the slope of settlement-distance profile at the center of load region is considered to be zero (i. e. $\frac{dw}{d\Psi} = 0$ at $\Psi = 0$). Since the edge of granular fill is assumed not to be under shear stress, the slope is also considered to be zero (i.e. $\frac{dw}{d\Psi}$ at $\Psi = R_0/r_0$).

Some equations can be obtained with Eq. (16) and boundary conditions as follows:

$$-\left[\frac{W_{i+1} - 2W_i + W_{i-1}}{h^2}\right] - \frac{1}{\Psi_i}\left[\frac{W_{i+1} - W_{i-1}}{2h}\right] + \frac{W_i}{UG^*} = \frac{q_i^*}{G^*} \quad (17)$$

$$\frac{-W_2 + 4W_1 - 3W_0}{2h} = 0 \quad (18)$$

$$\frac{3W_n - 4W_{n-1} + W_{n-2}}{2h} = 0 \quad (19)$$

$(i = 1,2,\cdots,n-1)$

W_0 and W_n can be substituted in Eq.(17). After iteration, Eq.(17) can be written in a matrix form. W_1, W_2, $\cdots$, W_{n-1} and then W_0 and W_n can be obtained through Eqs. (18) and (19).

CALCULATION AND ANALYSIS

Fig. 1 shows oil-tank soft foundation soil (H_2 = 20 m) improved by granular fill. The load radius (r_0) is 10 m while the granular fill radius (R_0) is 12 m. Uniform pressure q_0 amounts to 150 kPa, and the elastic modulus (E_0) of soft soil is 3 MPa. v_0 is 0.4 and H_1 is 3 m.

The following results can be easily obtained: k_s = 0.1786 MN/m^3; t = 3.57 MN/m; q^* = 0.084

Table 1 The effects of G and U on G^*

G^*	U = 10%	U = 50%	U = 100%
G = 20 MPa	7.357	4.159	3.759
G = 10 MPa	5.677	2.479	2.08
G = 5 MPa	4.838	1.639	1.24

Fig. 4 below shows the settlement-distance profiles when G is 20 MPa, 10 MPa and 5 MPa respectively. The maximum settlement (S_{max}) varies little when U is 10%. The settlement is 64 mm when G is 20 MPa, which is only 13 mm less than that when G = 10 MPa. As U increases, S_{max} becomes remarkably different. When U reaches 100%, the settlement is 386 mm when G is 20 MPa, which is 135 mm less than that when G is 10 MPa. This can be explained as follows:

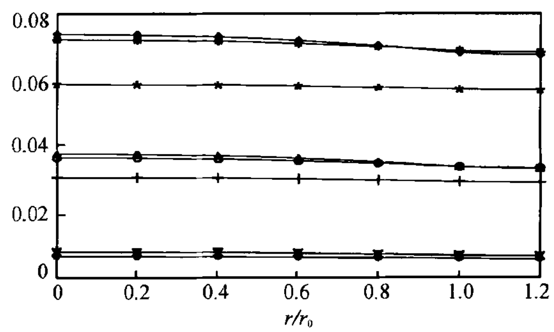


Fig. 4 Settlement-distance profiles for different shear modulus

● G^* = 7.357, U = 10%; + G^* = 4.159, U = 50%; ⋆G^* = 3.759, U = 100%; × G^* = 5.677, U = 10%; ○ G^* = 2.479, U = 50%; ¤ G^* = 2.08, U = 100%; ▼ G^* = 4.838, U = 10%; ▲ G^* = 1.63, U = 50%; ◆ G^* = 1.24, U = 100%

When the foundation soil begins to consolidate, $2t/U$ may be large enough not to be neglected and the value of G^* does not vary much, which make little difference in S_{max}. As U increases gradually to 100%, t can properly be omitted. At this time, the effect of variation of G on settlement is fully displayed. Because of great difference between $2t$ and GH_1, t can be neglected when calculating the final consolidation settlement and then G^* is $(GH_1)/(k_s r_0^2)$. However, it must be pointed out that when the shear modulus (G) is not very much larger than the elastic modulus (E_0), the binding effect of underlying soft soil ($2t$) cannot be omitted. Obviously, raising the value of G can significantly decrease the surface settlement.

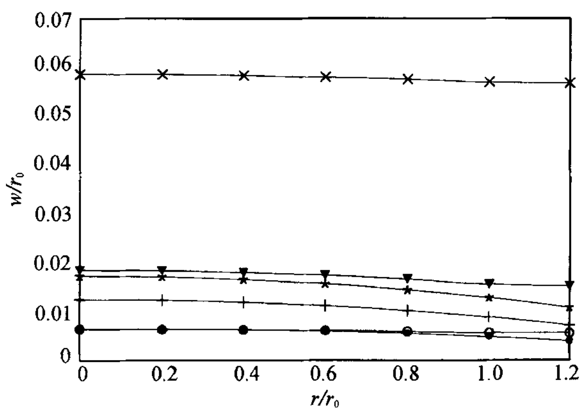


Fig. 5 Settlement-distance profiles for different radius

● R/r = 5, G^* = 7.357, U = 10%; + R/r = 5, G^* = 4.159, U = 50%; ⋆ R/r = 5, G^* = 3.759, U = 100%; ○ R/r = 1.2, G^* = 7.357, U = 10%; ▼ R/r = 1.2, G^* = 4.159, U = 50%; x R/r = 1.2, G^* = 3.759, U = 100%

Fig. 5 shows the settlement-distance profiles when ψ are 1.2 and 5 respectively for G = 20 MPa; and that ψ does not affect the both settlements much when the soft soil begins to consolidate (U = 10%). As U increases, the final consolidation settlement in the center reaches 173 mm for ψ = 5 and reaches 586 mm for ψ = 1.2. A conclusion can be drawn that the fill radius affects remarkably S_{max} when U is 100%, which cannot be obtained by the traditional stress dispersion method. " δ " represents the slope of the final consolidation settlement at the center and that at the edge. When ψ is 5, δ is 0.48% and when ψ is 1.2, δ is 0.16%. Therefore, another conclusion can be drawn that with the same shear modulus, the general settlement of the fill is more similar when ψ is small at different consolidation degree.

The data above can be used to calculate settlements. From Fig.6, a conclusion can be drawn that the settlement decreases as the value of H_1 increases. When U is 100%, S_{max} is 173 mm when H_1 is 3 m while it is 217 mm when H_1 is 2 m; and is 298 mm when H_1 is 1 m. It is obvious that the thickness (H_1) of the fill affects settlement remarkably. As H_1 increases evenly, S_{max} decreases more and more rapidly.

Table 2 The effects of H_1 on G^* (G = 20 MPa, ψ = 5)

G^*	U = 10%	U = 50%	U = 100%
H_1 = 3 m	7.357	4.159	3.759
H_1 = 2 m	6.237	3.04	2.64
H_1 = 1 m	5.12	1.92	1.52

With the traditional layer-wise summation method at $Z/R_0 = 3/10 = 0.3$, the stress dispersion angle is 20° and unit weight of soft soil is 17.8 kN/m^3. The designed additional stress at the bottom of the fill is

$$\sigma_z = \frac{\pi r_0^2(p - \sigma_c)}{\pi R_0^2} = \frac{\pi \times 10^2 \times 150}{\pi \times 12^2} = 104.2\ \text{kPa}$$

The thickness of each divided soft soil layer is 4 m. At last, the result can be obtained that the final consolidation settlement in the center is 599 mm while that in the edge is 305 mm.

Comparing the results using the layer-wise summation method with those using the method provided in this paper, we can observe that the corresponding results of S_{max} are almost the same for G = 20 MPa、H_1 = 3 m、ψ = 1.2. But for the differential settlement (S_{dif}), there is a large difference between the two results, one is 294 mm with the layer-wise summation method, and the other is 18 mm. With the method provided in this paper, the granular fill can adjust to S_{dif} because of its rigidity while the with layer-wise summation method, the ability of the granular fill to adjust the stress and strain in the underlying soft soil cannot be taken into account. Since the traditional method does not take the interaction between granular fill and soft soil into account reasonably, the result does not conform to engineering practice very well. The method given in this paper can predict and calculate the maximum settlement and the differential settlement more accurately.

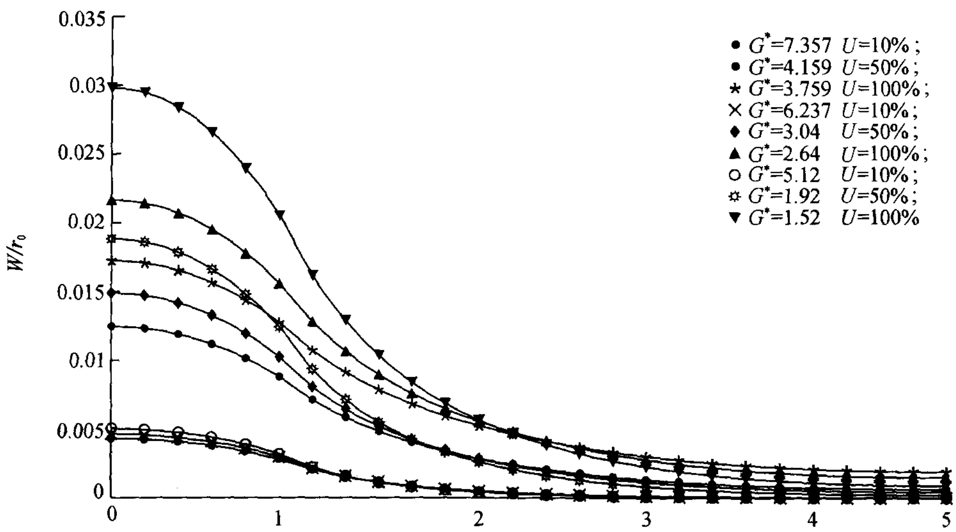


Fig.6 Settlement-distance profiles for different thickness of the granular fill

CONCLUSIONS

Generalized conclusions drawn from the calculation above are summarized as follows:

1. The foundation model provided above has a simple response function and can be well applied to the study of the settlement response of

granular fill on soft soil under oil tanks.

2. It is very useful with granular fill reinforcement of soft soils to reduce S_{max} and S_{dif}. The larger G is, the more remarkable is the effect of the granular fill in reducing settlement. t can be omitted when calculating the final consolidation settlement, which leads to $G^* = (GH_1)/(k_s r_0^2)$. However, when G is not much larger than E_0, the binding effect $(2t)$ of the underlying soft soil cannot be disregarded.

3. The radius of the granular fill can affect S_{max} and S_{dif}, too. The larger the radius is, the smaller S_{max} is. When ψ is relatively small (but still > 1), S_{dif} is smaller than that of larger ψ. For soft soil whose shear strength is only one tenth that of the granular fill, the granular fill becomes more rigid. when R_0/r_0 is smaller.

4. The effect of the thickness (H_1) on the settlement can be determined entirely through calculation and is especially remarkable for S_{max}.

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