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# Load Settlement Behavior Of Footings On Weak Soil Improved By Floating Granular Piles

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## ABSTARCT

*This paper introduces a rational method for predicting the load settlement behavior of rigid footing resting on weak soil reinforced by a group of floating granular piles. The floating granular piles are considered an economic alternative system to fully penetrated granular piles in case of deep weak soil layer or in case of lightly loaded structures. Based on the unit cell concept, an equation is developed for calculating the stress concentration ratio between floating granular pile and surrounding weak soil. The homogenization concept in conjunction with the stress concentration ratio is used to develop a method for predicting the load settlement curve of rigid footing resting on weak soil reinforced by a group of floating granular piles. A computer program called GPILES is developed for predicting the load settlement curve using the developed method. For the purpose of validation comparisons are made between the load settlement curves obtained by the developed procedure and the measured load settlement curves from two full scale field load tests. Good agreements are obtained between measured and predicted load settlement curves.*

**Key words**: load settlement curve, footing, weak soil, floating granular piles

## INTRODUCTION

Granular piles (i.e., sand compaction piles and stone columns) are extensively used to improve the bearing capacity and to reduce settlements of weak soils (i.e., soft clay or loose sand). In addition to their reinforcement effect, granular piles decrease the length of drainage path in clayey soil, thereby increasing the rate of consolidation. Based on the method of granular piles installation, the weak soil around the granular piles is compacted due to the lateral displacement of the soil during installation, and hence improved stiffness of the soil.

Granular piles may be fully penetrated and resting on strong soil layer (i.e., end bearing granular piles) or partially penetrated (i.e., floating granular piles). The floating granular piles are considered an economic alternative system to fully penetrated granular piles in case of deep weak soil layer or in case of lightly loaded structures. The effectiveness and behavior of floating granular piles is largely influenced by parameters such as granular pile length and diameter, strengths of granular pile material and surrounding soil, method of construction, flexibility of the footing and the number of granular piles beneath the footing.

Several literature pertaining to the analysis of fully penetrated granular piles are found (e.g., Priebe 1995; Poorooshasb and Meyerhof 1997; Shahu et al. 2000; Abdelkrim and Buhan 2007) but, a little number of literature concerning the analysis of floating granular piles are found (e.g., Sivakumar et al. 2004; Ishikura et al. 2007; Kirsch 2009; Zahmatkesh and Choobbasti 2010).

Three failure mechanisms of granular piles are described by several researchers (e.g., Wood et al. 2000; Sivakumar et al. 2004; Kempfert and Gebreselassie 2006). These are: (1) bulging failure, (2) local shear failure or punching failure, and (3) general shear failure within the reinforced zone below the footings.

This paper investigates the load settlement behavior of a rigid footing resting on weak soil reinforced by a group of floating granular piles (Fig. 1) considering punishing failure mechanisms, identified by Kempfert and Gebreselassie (2006). A rational method is developed based on the unit cell concept, the stress concentration ratio and the homogenization method. The developed method are validated against field measurements from two full scale field load tests and shown to be valid.

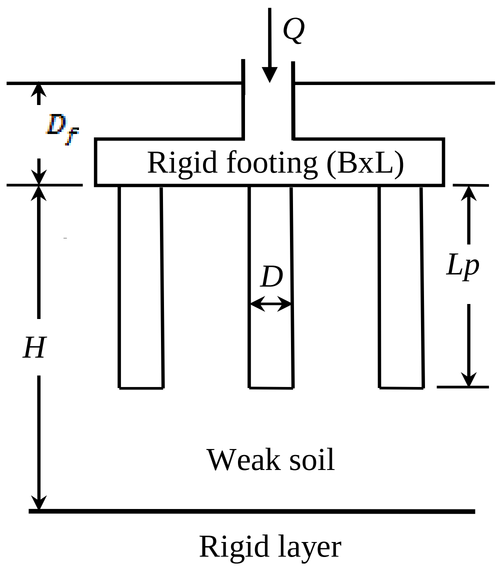


Fig. 1. The analyzed problem

## MATERIALS AND METHODS

### SETTLEMENT PREDICTION METHODS OF COMPOSITE GROUND

Most of the settlement prediction methods of composite ground, calculate the settlement of weak soil reinforced by granular piles in terms of settlement ratio, $\beta$ . The settlement ratio is defined as:

$$\beta = \frac{Settlement\ of\ treated\ soil}{Settlement\ of\ untreated\ soil} \tag{1}$$

The settlement ratio is dependent on the properties of granular piles material and surrounding soil and the geometry of the granular piles. Once the settlement ratio is known, the settlement of treated soil can be calculated as a function of the untreated soil settlement. The settlement of the untreated soil is usually calculated by the classical methods. The most common methods for settlement prediction of composite ground are presented and discussed in the following paragraphs.

The equilibrium method described by Aboshi, et al. (1979) and Barksdale (1983). This method used in Japan for estimating the settlement of weak soil reinforced by sand compaction piles. The equilibrium method is based on the unit cell idealization in combination with the stress concentration ratio. The settlement ratio was calculated from the following equation.

$$\beta = \frac{1}{1+(n-1)A_r} \tag{2}$$

$$A_r = \frac{N_p A_p}{BL} \tag{3}$$

Where $A_r$ is the area replacement ratio; $n$ is the stress concentration ratio; $A_p$ is the cross sectional area of granular pile; $N_p$ is the number of granular piles beneath the footing; $B$ is the footing width and $L$ is the footing length

Priebe's method (Priebe 1995) is considered the most common method used in the literature for calculating the settlement of soft soil reinforced by fully penetrated granular piles (i.e., end bearing granular piles). The method is based on the unit cell concept and takes into consideration the angle of internal friction of the granular piles material. Kempfert and Gebreselassie (2006) pointed out that Priebe's method is strictly applicable to an infinite array of granular piles and has some empiricism in its development; however, it is found to work very well for most applications.

Goughnour and Bayuk (1979) proposed a more elaborate prediction method. The unit cell is discretized vertically, and the stress state in the soil and the granular pile is initially assumed to be elastic. An iterative process is then used to calculate the strains and stresses within the soil and the granular pile, and modifications are made to ensure equilibrium and compatibility within each of the elements.

Alamgir et al. (1996) presented a rational analysis method where, nonuniform surface deformations are considered. Uniform deformation is assumed at the top of the granular piles (i.e., rigid footing), but the settlement in the surrounding soil varies from a maximum at the center point between the granular piles to a minimum adjacent to the pile.

Poorooshasb and Meyerhof (1997) presented a method based on the unit cell idealization. The vertical settlement is assumed to be uniform across the surface (i.e., rigid footing). The derivation of the equations and the model assumptions are quite similar to those of Priebe (1995), but the main difference lies in the nature of the granular pile deformation characteristics. The details of the method can be found in the original reference.

Shahu et al. (2000) proposed a simple theoretical approach to predict the settlement of uniformly loaded soft ground reinforced by granular piles with granular mat on top. The approach is based on the unit cell concept and incorporates the equal strain condition, the distribution of shear stresses and the load sharing between granular pile and soil.

Abdelkrim and Buhan (2007) proposed an elastoplastic homogenization method applied to weak soil reinforced by granular piles. According to this method, the composite reinforced soil is regarded, from a macroscopic point of view, as a homogeneous anisotropic continuous medium, the elastic and plastic properties of which obtained from the solution to an auxiliary problem attached to the reinforced soil representative cell.

Zahmatkesh and Choobbasti (2010) investigated the performance of granular piles in soft clay using the finite element program, PLAXIS. The 15-noded triangular elements were used. Interface elements were used at the interface between the granular pile and soft clay. The analyses employed elastic–perfectly plastic constitutive model following the Mohr–Coulomb failure criterion. The column installation was simulated for calculating the stresses due to compaction of soil.

## EQUATION FOR STRESS CONCENTRATION RATIO

Upon placing a footing on weak soil reinforced by a group of granular piles, a concentration of stress occurs in the granular piles and an accompanying reduction in stress occurs in the surrounding weak soil. Stress concentration occurs because the granular piles are considerably stiffer than the surrounding soil. Since the deflection in the two materials is approximately the same, from equilibrium considerations, the stresses in the stiffer granular piles must be greater than the stress in the surrounding soil. The stress concentration ratio, $n$, is defined as the ratio of the stress carried by granular pile to the stress carried by the surrounding soil. The value of $n$ is dependent on the applied load, the footing rigidity, the properties of granular pile material and weak soil and the geometrical dimensions. The stress concentration ratio can be obtained by measurement of stresses in full scale instrumentations, or estimated as the ratio of the constrained modulus of granular pile material divided by the constrained modulus of the surrounding weak soil. The later method generally gives high values of $n$. Reported values of the stress concentration ratio were found to vary between 2 and 6 (e.g. Bergado et al. 1996; Etezad et al. 2006).

Figure 2 shows the unit cell and stresses acting on floating granular pile and surrounding soil. For calculating the stress concentration ratio, it is assumed that the load transferred through the shear stresses along the soil-granular pile interface and end bearing at the granular pile tip (Suleiman and White 2006; Madhav et al. 2009).

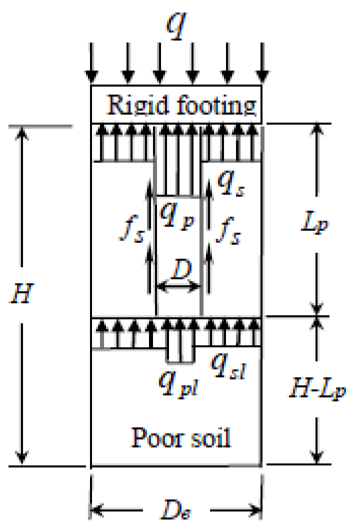


Fig. 2. Unit cell and stresses acting on the granular pile and surrounding soil

The bearing behavior of the system is characterized by sharing the load between granular pile and surrounding soil. In the case where the rigid footing undergoes a constant settlement the following equation may be derived.

$$q = q_p A_r + q_s(1 - A_r) \tag{4}$$

Where $q_s$ is the stress carried by weak soil; $q_p$ is the stress carried by the granular pile and $q$ is the average applied pressure.

The vertical stresses acting on the granular pile and surrounding soil at the bottom of the reinforced zone can be calculated from the following equations:

$$q_{pl} = q_p - \frac{T}{AA_r} \tag{5}$$

$$q_{sl} = q_s + \frac{T}{A(1 - A_r)} \tag{6}$$

$$\mathrm{T} = \pi \mathrm{DL_p f_s} = \pi \mathrm{DL_p}(\alpha \mathrm{c}) \tag{7}$$

Where $q_{pl}$ is the vertical stress at the granular pile tip; $q_{sl}$ is the vertical stress on the soil at the bottom of reinforced zone; $T$ is the sum of friction force on the surface area of granular pile; $L_p$ is the granular pile length; $D$ is the granular pile diameter; $f_s$ is the skin friction; c is the weak soil cohesion; $\alpha$ is the coefficient of friction reduction effect and $A$ is the cross sectional area of unit cell of equivalent diameter $D_e$ (where $D_e = \sqrt{4BL/\pi N_p}$ )

The vertical stresses acting on the granular pile and surrounding soil at the mid height of the reinforced zone can be calculated from the following equations:

$$q_{pl/2} = q_p - \frac{0.5T}{AA_r} \tag{8}$$

$$q_{sl/2} = q_s + \frac{0.5T}{A(1 - A_r)} \tag{9}$$

The average stress concentration ratio at the mid height of the reinforced zone can be calculated from the following equation:

$$n = \frac{q_{pl/2}}{q_{sl/2}} \tag{10}$$

Total settlement of the granular pile, $S_p$ , and the surrounding soil, $S_s$ , can be calculated from the following equations:

$$S_p = S_{p1} + S_{p2} \tag{11}$$

$$S_s = S_{s1} + S_{s2} \tag{12}$$

Where $S_{p1}$, $S_{s1}$ are settlements of granular pile and surrounding soil in reinforced zone and $S_{p2}$, $S_{s2}$ are settlements of granular pile and surrounding soil in the soil layer below the reinforced zone. $S_p$ and $S_s$ can be calculated from the following equations.

$$S_p = \frac{L_p}{E_p}\left[q_p - \frac{0.5T}{AA_r}\right] + \frac{I_{Bp}(H-L_p)}{E_s}\left[q_p - \frac{T}{AA_r}\right] \tag{13}$$

$$S_s = \frac{L_p}{fE_s}\left[q_s + \frac{0.5T}{A(1-A_r)}\right] + \frac{I_{Bs}(H-L_p)}{E_s}\left[q_s + \frac{T}{A(1-A_r)}\right] \tag{14}$$

Where $f$ is the improvement factor quantifying the increase in soft soil modulus of elasticity in the reinforced zone due to granular pile construction (Handy 2001, Kirsch 2006, and Richards et al. 2007); $E_p$ , $E_s$ are the modulus of elasticity of granular pile and surrounding soil, respectively; $I_{Bp}$ , $I_{Bs}$ are the influence values of Boussinesq equation for calculating the increase in the vertical stresses at the mid-height of soil layer below the reinforced zone due to $q_{pl}$, $q_{sl}$ and can be calculated from the following equations:

$$I_{Bp} = 1.0 - \frac{1.0}{\left[1.0 + \left(\frac{D}{H-L_p}\right)^2\right]^{3/2}} \tag{15}$$

$$I_{Bs} = 1.0 - \frac{1.0}{\left[1.0 + \left(\frac{D_e - D}{H-L_p}\right)^2\right]^{3/2}} \tag{16}$$

Satisfying the compatibility condition at the soil-granular pile interface below the footing (i.e., $S_p$ = $S_s$), the following equation in $q_p$ and $q_s$ is obtained:

$$\frac{L_p}{E_p}\left[q_p - \frac{0.5T}{AA_r}\right] + \frac{I_{Bp}(H-L_p)}{E_s}\left[q_p - \frac{T}{AA_r}\right] =$$

$$\frac{L_p}{fE_s}\left[q_s + \frac{0.5T}{A(1-A_r)}\right] + \frac{I_{Bs}(H-L_p)}{E_s}\left[q_s + \frac{T}{A(1-A_r)}\right] \tag{17}$$

By substituting Eq.(4) into Eq.(17), the following equation can be derived to calculate $q_s$:

$$\mathrm{q_s} = \frac{\mathrm{C_1 q - C_2}}{\mathrm{C_3}} \tag{18}$$

Where $C_1$, $C_2$ and $C_3$ are constants and can be calculated from the following equations:

$$\mathrm{C_1} = \frac{\mathrm{E_s}}{\mathrm{A_r E_p}} + \frac{\mathrm{I_{Bp}}(\lambda - 1)}{\mathrm{A_r}} \tag{19}$$

$$\mathrm{C_2} = \frac{\mathrm{0.5E_s}}{\mathrm{A_r A E_p}} + \frac{\mathrm{I_{Bp}}(\lambda - 1)}{\mathrm{A_r A}} + \frac{0.5}{(1-\mathrm{A_r})\mathrm{Af}} + \frac{\mathrm{I_{Bs}}(\lambda - 1)}{(1-\mathrm{A_r})\mathrm{A}} \tag{20}$$

$$C_3 = \frac{1}{f} + I_{Bs}(\lambda - 1) + \frac{(1-A_r)E_s}{A_r E_p} + \frac{I_{Bp}(\lambda-1)(1-A_r)}{A_r} \tag{21}$$

Where $\lambda = H/L_p$ is the depth ratio and the remaining parameters as previously defined.

From Eqs.(4), (8), (9), (10) and (18), the following equation can be derived for the average stress concentration ratio at the mid height of reinforced zone.

$$n = \left[\frac{(1-A_r)}{A_r}\right]\left[\frac{[AC_3-(1-A_r)AC_1]q+[(1-A_r)AC_2-0.5C_3]T}{[(1-A_r)AC_1]q+[0.5C_3-(1-A_r)AC_2]T}\right] \tag{22}$$

The parameters in Eq. (22) as previously defined.

## Deformation Parameters of Composite Ground

Based on the homogenization concept (Fig. 3), Omine et al. (1999) developed the following equation for calculating the equivalent modulus of elasticity and the equivalent coefficient of volume compressibility for composite ground (i.e., weak soil reinforced by vertical granular piles).

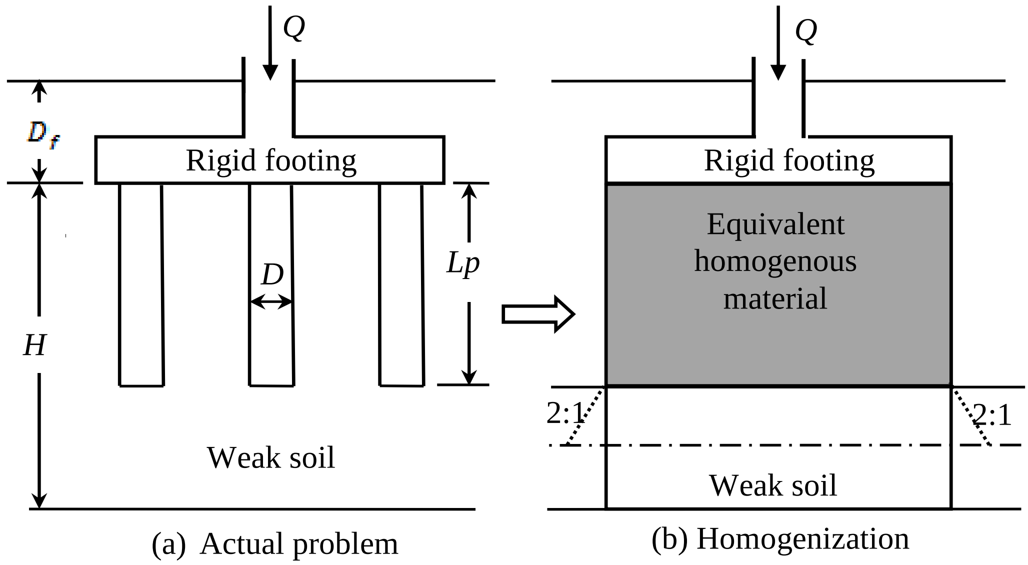


Fig. 3. The concept of homogenization

$$E_{eq} = \frac{1+(n-1)A_r}{\frac{nA_r}{E_p} + \frac{(1-A_r)}{E_s}} \tag{23}$$

$$m_{veq} = \frac{A_r n m_{vp} + (1-A_r)m_{vs}}{1+(n-1)A_r} \tag{24}$$

Where $E_{eq}$ is the equivalent modulus of elasticity of composite ground; $m_{vq}$ is the coefficient of volume compressibility of granular pile material; $m_{vs}$ is the coefficient of volume compressibility of soft clay and $m_{veq}$ is the equivalent coefficient of volume compressibility of composite ground. The remaining parameters of Eqs. (23) and (24) are as previously defined.

## METHOD FOR PREDICTING THE LOAD SETTLEMENT CURVE

To predict the load settlement curve, settlements of a rigid footing resting on weak soil reinforced by a group of floating granular piles are calculated at different values of footing pressure (i.e., $0.1q_{ult}$, $0.2q_{ult}$, $0.3q_{ult}$, $0.4q_{ult}$,...., $q_{ult}$ ). The ultimate bearing pressure, $q_{ult}$, can be estimated from full scale field load test and/or from published equations in the literature.

Settlement at each value of footing pressure is calculated based on the following steps.

1. Calculate the stress concentration ratio, n, from Eq. (22).
2. Calculate the equivalent modulus of elasticity of reinforced zone from Eq. (23) using the calculated *n* value, the area replacement ratio and the properties of granular pile material and surrounding soil.
3. The reinforced zone below the footing is subdivided into (*N-1*) layers of equal thickness and the weak soil below the reinforced zone is considered one layer (i.e., layer No. *N*).
4. The average vertical stress, $q_v$, at the centerline of each layer within the reinforced zone due to the footing pressure can be calculated by Boussinesq equation (Das, 1997).
5. The average vertical stress at centerline of weak soil layer below the reinforced zone is calculated by 2:1 slope method as shown in Fig. 3.
6. The settlement of treated soil due to the footing pressure can be calculated from the following equation.

$$S_t = \sum_{i=1}^{i=N} \frac{q_{vi}}{E_i} h_i \tag{25}$$

   Where $h_i$ is the thickness of the layer $i$; $E_i$ = $E_{eq}$ for the layers within the reinforced zone; $E_i$ = $E_s$ for the weak soil layer below the reinforced zone; and $N$ is the number of layers.

7. The settlement of untreated soil, $S$ , (i.e., before improvement) due to the footing pressure can be also calculated from Eq.(25). In this case, the average vertical stress at centerline of each soil layer is calculated by Boussinesq equation and the modulus of elasticity of all soil layers taken equal to $E_s$.
8. The settlement ratio, $\beta$, can be calculated from the following equation.

$$\beta = \frac{S_t}{S} \tag{26}$$

For a rigid footing resting on saturated soft clay reinforced by a group of floating granular piles, the previous steps can be used with the following modification to calculate the consolidation settlement:

1. Replacing the equivalent modulus of elasticity of reinforced zone, $E_{eq}$, by the equivalent constrained modulus, $1/m_{veq}$, (where $m_{veq}$ is the equivalent coefficient of volume compressibility of reinforced zone that can be calculated from Eq. 24).

2. Replacing the modulus of elasticity of weak soil layer, $E_s$ , by the constrained modulus, $1/m_{vs}$, (where $m_{vs}$ is the coefficient of volume compressibility of soft clay)

Hand calculation of load settlement curve using the above procedure takes time and may be subjected to errors due to the large number of parameters and complicated equations. The problem will be more difficult and time consuming in case of parametric study. Therefore, a FORTRAN computer program is developed and called GPILES to predict the load settlement curve and settlement ratio at different load level.

## VALIDATION OF THE DEVELOPED METHOD

For the purpose of validation, comparison between measured and predicted load settlement curves of a rigid footing resting on weak soil reinforced by a group of floating granular piles for two full scale field load tests are presented and discussed in the following sections.

### i)Field test by Kirsch (2009)

Kirsch (2009) reported the results of full scale field load test on a rigid footing resting on 11.0 m of soft clay reinforced by five floating granular piles (i.e., area replacement ratio of 28%). The footing dimensions were 3.0 m x 3.0 m x 0.5 m. The diameter and length of floating granular piles were 0.8 m and 8 m, respectively. The configuration and spacing between granular piles were as shown in Fig. 4. The soil profile consists of 1 m thick sand layer underlain by a soft clay layer of thickness 11 m resting on a firm soil layer. The engineering properties of soft clay layer were as presented in Table 1.

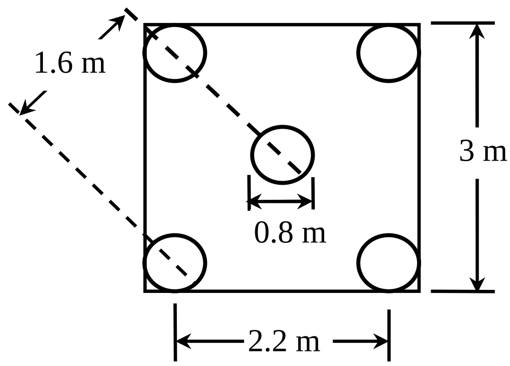


Fig. 4. The configuration and spacing between granular piles-not to scale (Kirsch 2009)

**Table 1. The engineering properties of soft clay layer (Kirsch 2009)**

| | |
|---|---|
| Natural water content | 0.636 |
| Plasticity index | 0.398 |
| Activity | 0.561 |
| Cohesion | 14 kPa |
| Compression index | 0.454 |
| Poisson’s ratio | 0.4 |
| Preconsolidation pressure | 55 kPa |

The load test was conducted as a maintained load test with loading stages held over a period of 10 days. The load settlement curve was measured under a contact pressure of 105 kPa and the stress concentration ratio was measured at different loading stages for center and edge granular piles. At the end of loading stage (i.e., 105 kPa) the stress concentration ratio for center and edge columns were measured to be 2.5 and 2.0, respectively. The results of the field load test were also showed that the instillation of granular piles raises the soft clay stiffness to a maximum of 2.5 times the initial stiffness (Kirsch 2009).
In the present analysis, the constrained modulus of soft clay is taken as 200 times its cohesion and the constrained modulus of granular pile material is taken 10 times the constrained modulus of soft clay according to Bowles (2001). The parameters used in the present analysis are as presented in Table 2.

**Table 2. The parameters used in the present analysis**

| | |
|---|---|
| The improvement factor, *f*. | 2.0 (Kirsch 2009) |
| Coefficient of friction reduction effect, □. | 0.135 (Ishikura et al. 2007) |
| The constrained modulus of soft clay. | 2800 kPa |
| The constrained modus of granular piles | 28000 kPa |

Figure 5 shows comparison between measured and predicted load settlement curves. As shown in Figure 5, good comparison exists between measured and predicted load settlement curve up to the load level of 55 kPa which is approximately half of the maximum load level, after that the predicted settlement is smaller than the measured settlement.

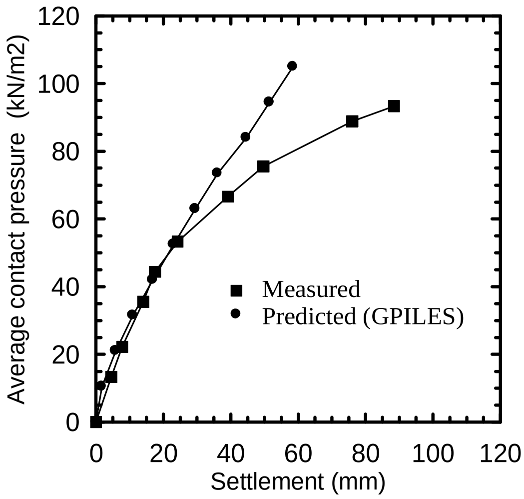


Fig. 5. Measured and predicted load settlement curves
For field test of Kirsch (2009)

The developed method is able, to some extent, to predict the nonlinearity of the load settlement curve as shown in Fig. 5. This is may be due to the change of the stress concentration ratio and the equivalent modulus of elasticity of composite ground as a function of the load level as shown in Figs. 6, 7.

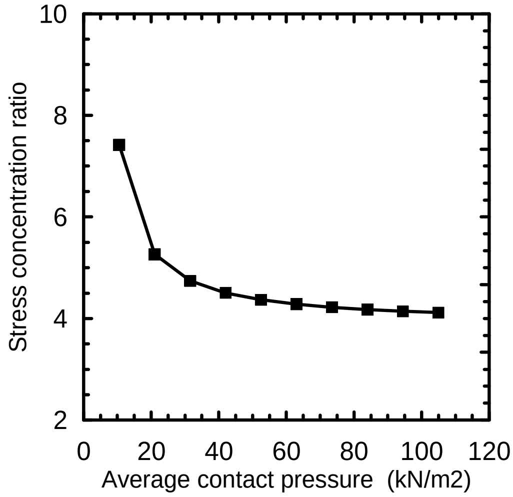


Fig. 6. Stress concentration ratio versus load level for field test of Kirsch (2009)

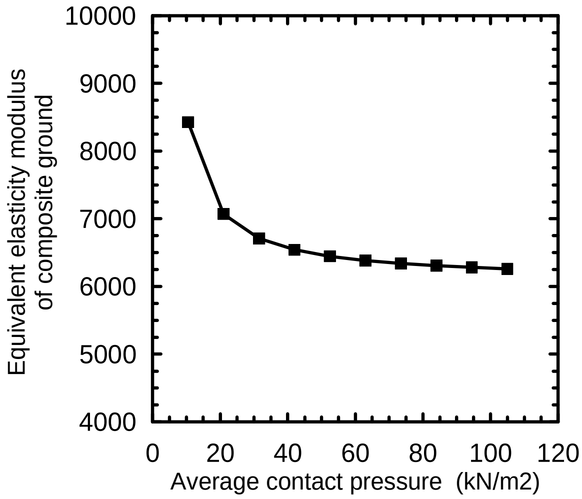


Fig. 7. Equivalent elasticity modulus of composite ground versus load level for field test of Kirsch (2009)

**ii) Hydrostatic Tests on Oil Storage Tanks**

Duzceer (2003) reported the results of hydrostatic tests of four oil storage tanks in Poti Oil Terminal, Georgia. The diameters of tanks 1, 2, 3 and 4 were 28.5 m, 28.5 m, 24.5 m, and 18.5 m, respectively. The analysis for tanks 1, 3 and 4 will be considered here. Raft foundations of oil storage tanks were rested on weak soil reinforced by a group of floating granular piles for settlement control and liquefaction mitigation. Granular piles were constructed in a square pattern with 2.2 m to 2.5 m spacing which corresponding to area replacement ratio of 12.5% to 16.5%. The diameter and length of granular piles were 1.0 m and 14.28 m, respectively. The subsoil consists of two layers of loose to medium dense silty sand underlain by medium stiff to stiff clay. Thicknesses and engineering properties of soil layers under each tank were as presented in Table 3.

**Table 3. Thicknesses and engineering properties of subsoil layers (Duzceer 2003)**

| Thickness of subsoil layers (m) | | | Classification | Natural unite weight ($kN/m^3$) | SPT (N) | Deformation Modulus (kPa) |
|---|---|---|---|---|---|---|
| Tank 1 (diameter = 28.5 m) | Tank 3 (diameter = 24.5 m) | Tank 4 (diameter = 18.5 m) | | | | |
| 10.5 | 7.5 | 6.0 | SP-SM | 17.5 | 8 | 10000 |
| 12.0 | 13.5 | 12.0 | SP-SM | 17.2 | 10 | 12000 |
| 17.5 | 14.0 | 17.0 | CL | 17.0 | 12 | 15000 |

The tanks were hydrostatically tested. Each tank was filled by sea water in 4 filling increments. Average settlements of the tanks were measured at different load level during the hydrostatic test up to the maximum load level of 180 kPa.

Figures 8, 9 and 10 show comparisons between average measured and predicted load settlement curves along with the predicted average settlements by Duzceer (2003) using PLAXIS finite element program and Priebe method at maximum load level of 180 kPa for tanks 1, 3 and 4, respectively. Referring to these figures it is observed that:

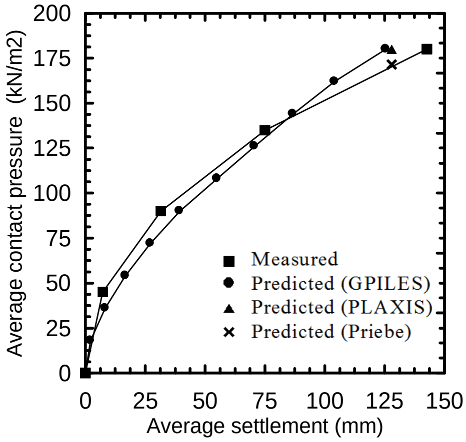


Fig. 8. Measured and predicted average load settlement curves for tank 1

1. For Tank 1, generally good comparison is obtained between measured and predicted load settlement curves as shown in Fig. 8. At the maximum load level of 180 kPa, the predicted settlement by the present method is approximately equal to the predicted settlement by PLAXIS program and slightly smaller than the predicted settlement by the Priebe method.
2. For Tank 3, the predicted settlements are slightly greater than the measured settlements as shown in Fig. 9. At the maximum load level of 180 kPa, the predicted settlement by the present method is slightly smaller than the predicted settlement by PLAXIS program and smaller than the predicted settlement by the Priebe method.
3. For Tank 4, the predicted settlement is slightly greater than the measured settlement as shown in Fig. 10. The predicted settlement by the present method at maximum load level of 180 kPa is approximately equal to the predicted settlement by PLAXIS program and smaller than the predicted settlement by the Priebe method.
4. For Tank 3 and Tank 4, the difference between predicted and measured settlements increases as the load level increases as show in Figs. 9, 10.

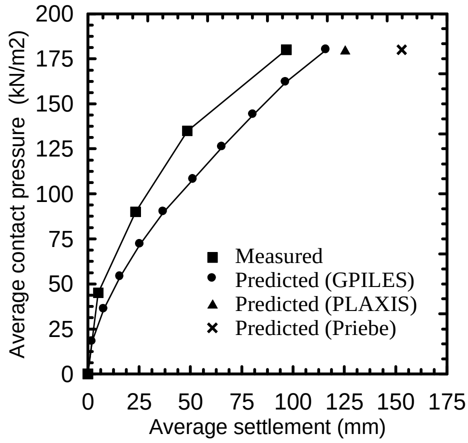


Fig. 9. Measured and predicted average load settlement curves for tank 3

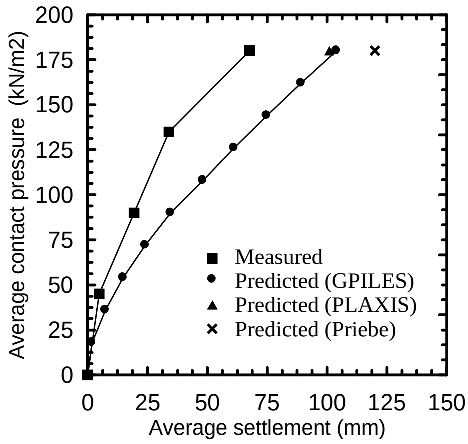


Fig. 10. Measured and predicted average load settlement curves for tank 4

5. At the maximum load level of 180 kPa, the three prediction methods (i.e., present method, PLAXIS program, and Priebe method) are considered conservative and predicted settlements greater than the measured settlements.

## CONCLUSION

A rational method based on the homogenization concept in conjunction with the stress concentration ratio for predicting the load settlement curve of rigid footing resting on weak soil reinforced by a group of floating granular piles is presented. The following conclusions can be drawn from this study.

1. The developed equation for the stress concentration ratio, $n$, is simple and realistic because the $n$ value varies with the depth and the load level or the deformation in the composite ground. As the load level increases or as the deformation in composite ground increases the stress transferred from the granular pile to the surrounding soil and consequently the stress concentration ratio decreases.

2. The developed program GPILES can be used to predict the load settlement curve of rigid footing resting on weak soil reinforced by a group of floating granular piles with satisfactory accuracy and to rapidly examine various design options.
3. The developed method is able to predict, to some extent, the nonlinearity of the load settlement curve due to the change of composite soil modulus with the change of load level.
4. More validation for the developed equation of the stress concentration ratio, *n*, and the developed method of settlement calculation is required.

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