Prediction of immediate settlement of shallow foundation over granular soils using small-strain stiffness

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Abstract: In this paper, we investigated how to utilize the small-strain stiffness in order to estimate the settlement of shallow foundations on granular soils. For this purpose, a power law equation between normalized shear modulus and shear strain was presented. Based on theory of elasticity and proposed equation, a new method in term of small-strain stiffness was suggested to estimate the immediate settlement. In order to evaluate the proposed method, a series of case history included plate and footing loading tests and seismic geophysical tests was studied. These field measurements are compared to the predicted values. The result indicated that the proposed method in this study can be effectively used to predict the settlement of footing on granular soils and that were more accurate than the SPT or CPT based predictions.

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1. Introduction

Shallow foundations are generally designed to satisfy bearing capacity and settlement criteria. In the design of shallow foundations, permissible settlement is often the controlling design criterion. Numerous methods have been developed over the years to estimate the settlement of shallow foundations. The most popular methods for settlement predictions, discussed commonly in textbooks, are the ones proposed by Terzaghi and Peck (1948), Peck and Bazaraa (1969), Schmertmann et al. (1978), Burland and Burbidge (1985). Two of the more recent methods are after Berardi and Lancellotta (1991) and Mayne and Poulos (1999). The conventional methods to estimate settlement of shallow foundations utilize correlations between measured settlements and some parameters from reasonably simple field tests, in particular standard penetration tests (SPT) and cone penetration test (CPT). In often, most current correlations overpredict settlements.

Seismic wave velocity measurements have been used to characterize in-situ soil and rock stiffnesses for use in the evaluation of the response of geotechnical sites to earthquake loading and machine vibrations. The velocity of propagation of a shear wave (V_S), which can then be converted to the shear modulus at small strains (G_{max}), and finally to Young's modulus at small strains (E_{max}).

(1) $$G_{max}= \rho . V_s^2$$

Where ρ = mass density of the soil.

(2) $$E_{max}=2(1+\upsilon)G_{max}$$

Where: ν = Poisson's ratio (0.15-0.35 for unsaturated cohesionless soils). In-situ direct estimation of maximum stiffness or small-strain stiffness (G_{max} or E_{max}), of soil is more effectively and reliably than those derived from resistance-based correlation or laboratory testing. However, Gmax is too high for direct use in computing foundation displacements using either simple elastic analytical methods or linear elastic-plastic constitutive models that are built-in to many commercial finite element programs. Therefore, a variety of models have been proposed to better represent the true soil stress-strain behavior (e.g. Jardine et al., 1986; Fahey and Carter, 1993; Rollins and et al., 1998).

The goal of the writers is how to utilize the small-strain stiffness in order to estimate the settlement of shallow foundations. We use the classical theories of elasticity for the analyses, measure the small-strain stiffness using seismic methods in the field; a new method was presented to estimate the immediate settlement. The suggested relationship in this paper will be modified small-strain stiffness of the soil layer in according to the level of foundation pressure. This research explores the use of the surface-wave seismic methods, specifically the SASW and CSW methods, to predict immediate settlement of shallow foundations on granular soils. Immediate settlement is obtained using the relationships' of elasticity theory based on the foundation width, stress field and small-strain stiffness. In order to validate the proposed method, the results of the survey of loading tests in three sites were evaluated and compared. Appropriate coincidence between the result of loading test and predicted settlement, shows the accuracy of proposed method.

2. Theoretical concepts

The non-linearity of stiffness with strain and stress level, coupled with different directions of loading and drainage conditions, makes it very difficult for a meaningful cross comparison of the various modules derived from the different tests, unless a consistent framework and reference stiffness are established. It is therefore a difficult issue to recommend a single test, or even a suite of tests, that directly obtains the relevant Es for all possible types of analyses in every soil type. This is because the modulus varies considerably with strain level or stress level.

The small-strain stiffness Gmax is a fundamental stiffness applicable to all types of geomaterials including clays, silts, sands, gravels, and rocks (Tatsuoka et al., 2001) for static and dynamic loading (Burland, 1989). Stiffness parameters may therefore, for practical purposes, be considered constant at very small strains, but can be expected to reduce as strains increase above this level. Because the strain levels around well-designed geotechnical structures such as retaining walls, foundations and tunnels are generally small (Fig. 1), measurements are required in order to determine two sets of parameters (clayton, 2011):

(a) Parameters at very small (ideally reference) strain levels (e.g. E_0, υ_0 and G_0).
(b) Stiffness parameters are altered by increasing strain and changing stress levels, during loading or unloading.

Jardine et al. (1986) and Mair (1993) have shown that the typical strain levels around geotechnical structures such as retaining walls, spread foundations, piles and tunnels fall in the range where soil stiffness changes most dramatically with strain and that for many structures they are in the range 0.01–0.1%. However, Gmax is too high for direct use in computing settlement of shallow foundation. Therefore, small-strain stiffness must been modified based on stress levels or strain levels.

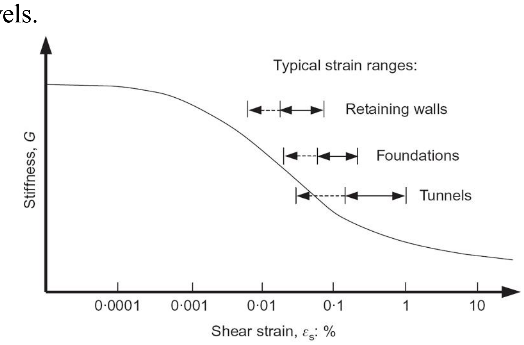


Fig. 1. Typical stiffness variation and strain ranges for different structures(clayton, 2011)

2.1 *Modification small-strain stiffness based on shear strain*

The shear modulus degradation with shear strain is commonly shown in normalized form, with current G divided by the maximum G_{max} (or G_0). The relationship between G/G_0 and logarithm of shear strain is well recognized for dynamic loading conditions (e.g., Vucetic and Dobry, 1991). In order to modify the small-strain stiffness, laboratory data for variations soil stiffness with various shear strains were collected from recent scientific papers and reports. The power law relationship was presented for modification small-strain stiffness by the shear strain:

(3) $$G/G_{MAX} = \frac{0.0725}{\sqrt{\gamma_{\%}}}$$

Where $\gamma_{\%}$= shear strain in percent. Bands defining G/Gmax versus shear strain for sands (Seed et al 1984) are shown in Fig. 2. In this figure, the proposed equation (3) by the authors is drawn. The proposed curve in this study for defining G/Gmax versus shear strain generally falls near the center of the range of data for sands defined by Seed et al (1984).

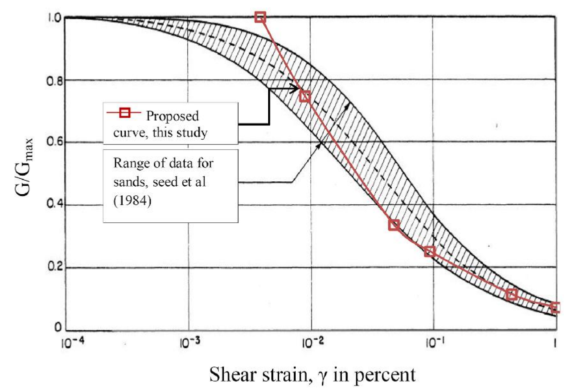


Fig. 2. Comparison of the proposed curve in this study and range of data for sands defined by Seed et al (1984).

3. Proposed method for prediction settlement of shallow foundation

Based on the measured small-strain stiffness and elasticity theory, a new method has been developed which uses these values to calculate Young's Modulus, E, at the practical strain levels experienced in actual foundation conditions and so enables ground settlements to be predicted. Suggested Steps to predict settlement in terms of small-strain stiffness, are as follow:

Step1: Average values of G_{max} in layers from the base of the foundation to twice the foundation width. The seismic methods of SASW and CSW are then conducted to measure the shear wave velocity and shear modulus (G_{max}) of the soil profile with depth.

Step2: Determine the maximum Stiffness (E_{max}) from small-strain stiffness, $E_{max}=2(1+\upsilon)G_{max}$.

Step3: The vertical strain at the centre of the layers, ε, are then calculated from the elasticity theory.

(4) $$\varepsilon = \frac{\sigma_z}{E} - \upsilon(\frac{\sigma_x}{E} + \frac{\sigma_y}{E})$$

Where σ_z, σ_y and σ_x = vertical and horizontal stress, υ= passion ratio this means Poisson's Ratio is assumed to be 0.3 and E= young modulus. With axial symmetric loading condition, $\sigma_x = \sigma_y$ and equal to $k_0.\sigma_z$ that k_0 is coefficient of lateral earth pressure at rest (dimensionless). For soil deposits that have not been significantly preloaded, a value of k_0 = 0.5 is often assumed in practice. Hence from eq.

(5) $$\varepsilon = 0.7\frac{\sigma_Z}{E}$$

Step4: Modify the small-strain stiffness. The relationship between shear strain and axial strain is as following:

(6) $$\gamma_{\%} = (1+\upsilon)\varepsilon_{\%}$$

Where $\varepsilon_{\%}$= axial strain in percent = 100. ε. Substituting eq. (6) into eq. (3) and υ=0.3 yields**:**

(7) $$G/G_{MAX} = \frac{0.0725}{\sqrt{\varepsilon_{\%}(1+\upsilon)}} = \frac{0.0636}{\sqrt{\varepsilon_{\%}}}$$

Step5: Calculate the axial strain. With regard to eq. (5) and eq. (7), enabling us to write:

(8) $$\varepsilon_{\%} = \left(\frac{1101.\sigma_z}{E_{MAX}}\right)^2$$

Step6: The settlement of foundation is obtained by multiplying the calculated strain in the soil layer thickness. The soil layer thickness considers from the bottom of the footing to a depth of 2B below the footing. Hence, the vertical stress at the centre of the layer at depth equal to B below the footing, σ_z, is then calculated from the Boussinesq formula.

(9)

$$\sigma_z = q\left[1 - \frac{1}{(1+\left(\frac{B/2}{B}\right)^2)^{\frac{3}{2}}}\right] = 0.285q$$

Where q= applied pressure at foundation level. Therefore, substituting eq. (9) into eq. (8), we obtain:

(10) $$\varepsilon_{\%} = \left(\frac{313.75q}{E_{MAX}}\right)^2$$

The settlement s of the soil layer, may be expressed from eq.(10) , as:

(11) $$S = \frac{\varepsilon_{\%}}{100}.2B = \left(\frac{313.75q}{E_{\max}}\right)^2.\frac{B}{50}$$

Where s= settlement and B= diameter of footing. This is the desired expression to determine the settlement of circular footing in granular soils.

4. Case histories

In order to evaluate the accuracy of the proposed method in this paper, eq. (11), for estimation settlement of shallow foundation, a database of 13 load tests on footings and large plates from three sites was compiled, as summarized in Table 1. The case histories are: 1-Semnan university, I.R.IRAN (amini,2012), 2- Texas A&M University (Briaud,1997 and park et al, 2010) and 3-Vattahammar, Sweden(Larsson, 1997).

For each case, in-depth geotechnical, loading test and geophysical site investigations have been conducted and soil parameters have been determined.

Table. 1. Case histories general specification

Site No.	Reference	Location	Soil Type	Footing shape	Footing sizeB (m)
①	Amini(2012)	IRAN	Sand with gravel	Circular	0.45
①	Amini(2012)	IRAN	Sand with gravel	Circular	0.30
②	Briaud (1997)	USA	Sand, silty sand	Square	1
②	Briaud (1997)	USA	Sand, silty sand	Square	1.5
②	Briaud (1997)	USA	Sand, silty sand	Square	2.5
②	Briaud (1997)	USA	Sand, silty sand	Square	3
②	Briaud (1997)	USA	Sand, silty sand	Square	3
②	Park et al (2010)	USA	Sand, silty sand	Circular	0.91
②	Park et al (2010)	USA	Sand, silty sand	Circular	0.46
②	Park et al (2010)	USA	Sand, silty sand	Circular	0.25
③	Larsson (1997)	SWEDEN	Silt	Square	0.5
③	Larsson (1997)	SWEDEN	Silt	Square	1
③	Larsson (1997)	SWEDEN	Silt	Square	2

4.1 Site condition and field test site

4.1.1 Semnan university, iran

Soil deposit at this site is granular. The top layer is poorly graded gravel with sand with 2 m thickness and the next layer is well-graded sand with gravel that extends to a depth 4 m. The ground water table is at a depth of about 180 m and the total unit weight was about $18kN/m^3$.

The results from SPT tests that were performed close to our footing locations is shown in Fig.3. As part of our investigation, seismic continuous surface wave system (CSWS) tests were performed to obtain the shear wave velocity profile with depth. The continuous surface-wave (CSW) method is a geophysical exploration technique to evaluate the subsurface stiffness structure using a vibrator and more than three receivers, as depicted in Fig 4. Surface wave method provide a non-invasive technique of obtaining soil shear wave velocity that overcome some of the limitations associated with the more commonly used invasive field methods. Two circular steel plates with diameters of 0.45m and 0.30m were loaded based on ASTM D1194. The plate loading test procedure involved application of load by jacking against a large truck and measuring settlements.

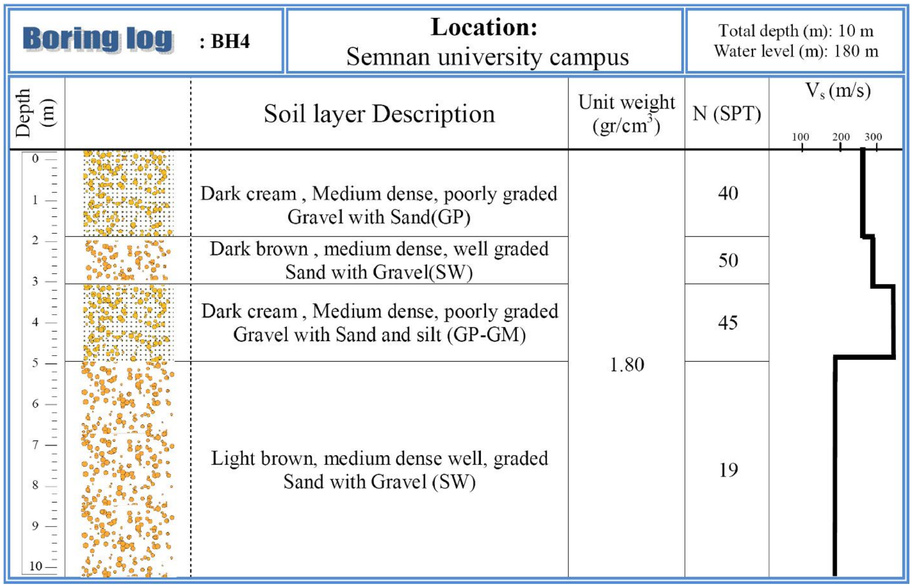


Fig. 3. Boring log for the semnan university campus, Iran. N= SPT value,V_s= shear wave velocity (m/s)

Fig. 4. Small-strain stiffness measurements at the semnan university campus (Iran) using continuous surface wave system

Fig. 5. Plate load testing at the semnan university campus, Iran.

Each stage consisted of building up the load during a period of 10–20 seconds, followed by a “resting period” of about four minutes where the loading process stopped. In Settlements were measured at two locations on the steel plate (Fig. 5). One reference frame was placed near the plate to support displacement potentiometers that were arranged in an equilateral triangle on each steel plate. Loads were applied in stages. The resting period, there was a slight reduction in load and continued settlements at a decreasing rate. Then the next loading stage began. The peak loads on the steel plates were limited by the weight of truck.

4.1.2 Texas A&M University, USA

Soil at the site is generally cohesionless. Four layers were indicated by Briaud and Gibbens (1994). The top layer is medium dense, tan silty fine sand with a thickness of 3.5 m. That layer extends to a depth of about two times the width of largest footing and thus the deeper layers of sandy soil that extend to a depth of 7 m, and deeper hard clay, had a negligible effect on settlements and are thus not considered further. Briaud and Gibbens (1994) presented results from SPT and CPT tests that were performed (Fig.6).

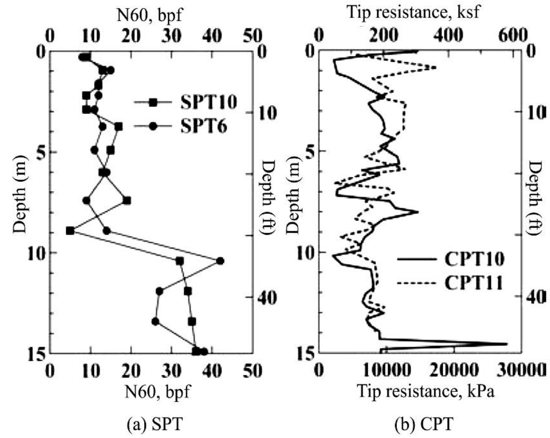


Fig. 6. SPT and CPT profiles at the Texas A&M University, USA (park et al 2010)

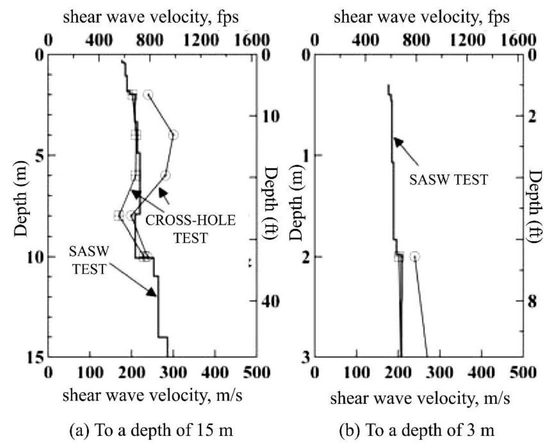


Fig. 7. Shear wave velocity profiles at the Texas A&M University, USA (park et al 2010)

They also presented crosshole tests results in this area, as well as park et al (2010) seismic spectral-analysis-of-surface-waves (SASW) tests were performed that showed tolerably uniform shear wave velocities (V_S), (Fig. 7). Briaud and Gibbens (1994) five, full-scale, reinforced concrete footings of different sizes were constructed. Each footing was loaded to failure and detailed load-settlement measurements were recorded. Also Park et al (2010) Two circular concrete footings with diameters of 0.91m (36 in.) and 0.46m (18 in.) and one, 0.25-m (10-in.) diameter steel plate were loaded.

4.1.3 Vattahammar, Sweden

According to the visual inspection of the soil samples and the sounding test results, the soil profile consisted of silt to great depths. Below 5 meters depth, the soil was classified as somewhat clayey. The free ground water level was located lower than 11 meters below the ground surface.

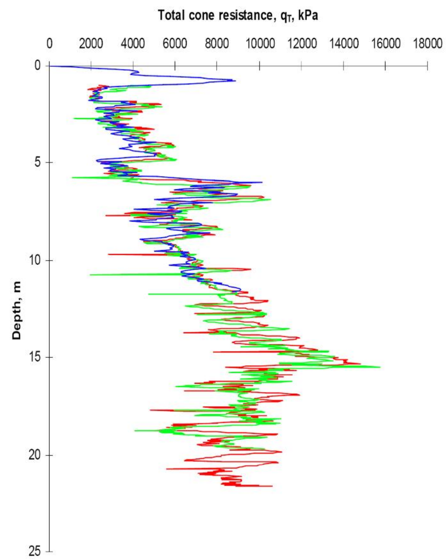


Fig. 8. Total cone resistance measured in three CPT tests in the test field at Vatthammar, Sweden (Larsson 1997)

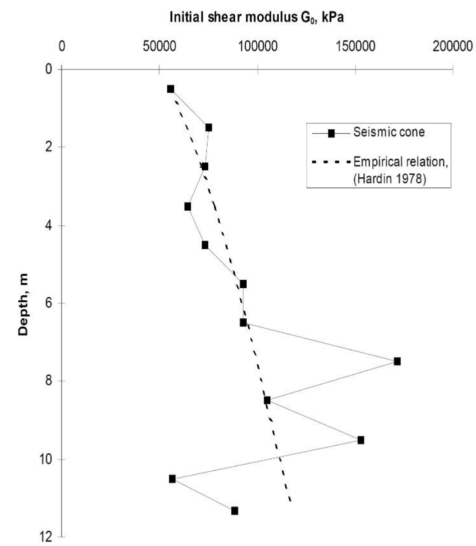


Fig. 9. Measured and estimated initial shear moduli in the test field at Vatthammar, Sweden (Larsson 1997).

The combined results of the three tests performed with the ordinary CPT equipment are shown in terms of total cone resistance, q_T, in Fig. 8. The initial shear modulus at small strains, Gmax, was then evaluated from the shear wave velocity and the bulk density (Fig. 9). Three, full-scale, reinforced concrete footings of different sizes were loaded.

5. Results and discussion

In order to validate the proposed method in term of small-strain stiffness, a comparison were conducted between predicted settlement and measured settlement in 15 case studies. As well as, to better demonstrate the accuracy of this method, the settlement for our case studies were estimated by three conventional methods among the available methods have been selected to be incorporated in settlement predictions, that are summarized in Table 2.

Table. 2. Summary of Settlement Prediction Methods

Method	Expression for settlement	Definitions	Explanations
Peck and Bazaraa (1969)	$S(mm) = C_w C_D \frac{0.53q(kPa)}{(N_1)_{60}}\left(\frac{2B}{B+0.3}\right)^2$	C_D = embedment correction factor; C_w = water table correction factor; N = corrected SPT-N value;	$C_D = 1.0 - 0.4\left(\frac{\gamma D_f}{q}\right)^{0.5}$ $C_w = \frac{\sigma_0}{\sigma'_0}$
Schmertmann et al. (1978)	$S_{footing} = C_1 C_2 q_{net} \sum_{z=0}^{z=2B} \frac{I_z dz}{E}$	S = settlement; C_1 = foundation depth correction factor; C_2 = soil creep factor; q = applied pressure; I_z = strain influence factor; and E_s = modulus of elasticity.	$C_1 = 1 - 0.5\frac{\sigma'_0}{q_{net}} \geq 0.5$ $C_2 = 1 + 0.2\log\left(\frac{t'}{0.1}\right)$
Mayne and Poulos (1999)	$S_{footing} = \frac{q_{net} B' I_G I_F I_E (1-\upsilon^2)}{E_0}$	υ = Poisson's ratio; q_{net} = applied bearing pressure; Es = modulus of elasticity of bearing soil; I_G, I_F and I_E influence factor.	$I_F = \frac{\pi}{4} + \frac{1}{4.6 + 10\left(\frac{E_f}{E_0 + \frac{B'}{2}k}\right)\left(\frac{2t}{B'}\right)^3}$ $I_E = 1 - \frac{1}{3.5\exp(1.22\upsilon - 0.4)\left[\left(\frac{B'}{D_f}\right) + 1.6\right]}$

The proposed methods in Table 1 to predict the settlement of shallow foundations on cohesionless soils based on SPT N values and CPT point resistance, qc.

5.1 *Input parameters for empirical methods*

In this study, the correlation between E and N60 from SPT data is used as suggested by Coduto (2001) for silty sand:

$$E = 50000\sqrt{OCR} + 12000N_{60} \quad (12)$$

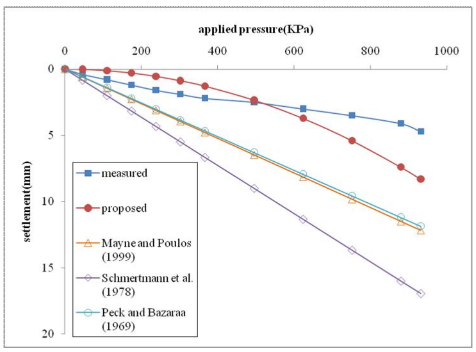
Where E is in psf, OCR is the overconsolidation ratio, and $N60$ is the standard penetration resistance in blows/30 cm corrected to a hammer efficiency of 60%. The following correlations were used to obtain the Modulus of Elasticity with SPT N values and CPT qc values for normally consolidated sands (Bowles 1996):

(13) E (kPa) =500(N+15)

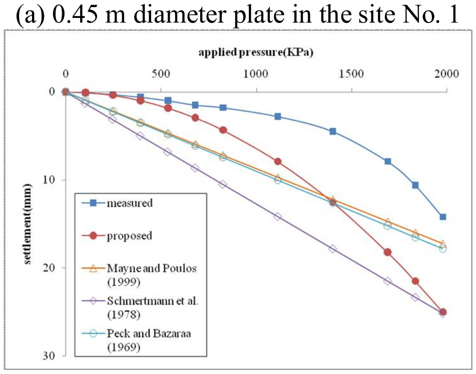
(14) E (kPa) =2-4 qc

5.2 Prediction results and comparison

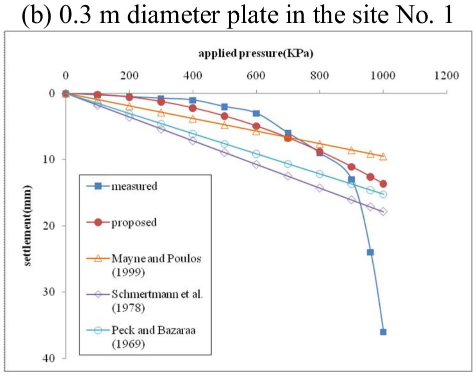
The settlement predicted by the proposed method in term of small-strain stiffness and three conventional methods were compared to the measured results by presenting a series load-displacement curves. Figure 10 give load-displacement curves for the13 footings. The results of the comparison indicate that the predicted settlements by the proposed method in this study are closer to measured settlements than the other methods. It means that the new method predicts the footing settlement with less overestimation or underestimation than the other methods.



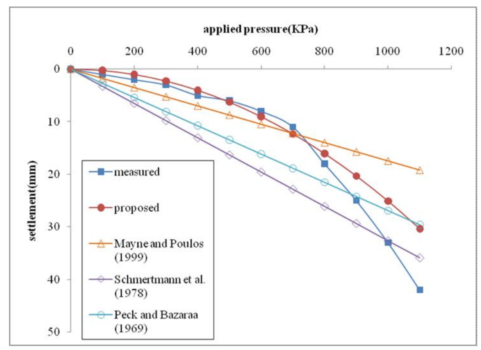
(a) 0.45 m diameter plate in the site No. 1



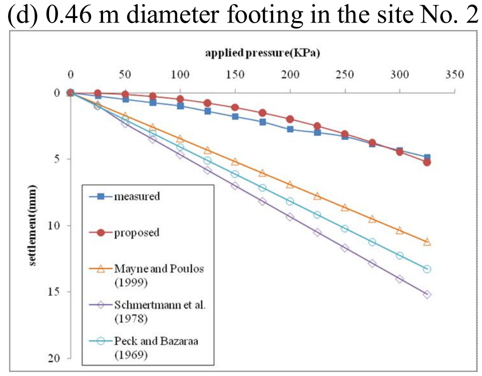
(b) 0.3 m diameter plate in the site No. 1



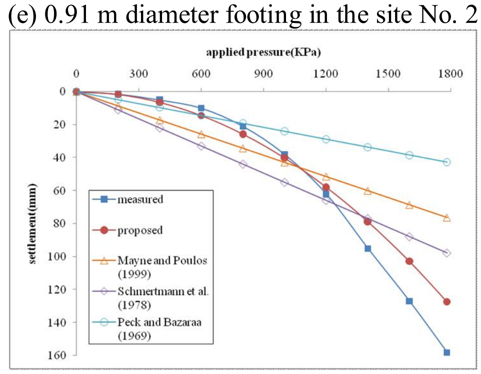
(c) 0.25 m diameter plate in the site No. 2



(d) 0.46 m diameter footing in the site No. 2

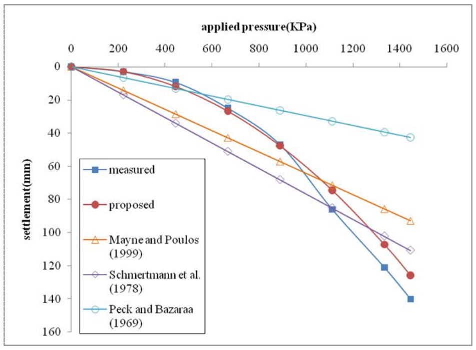


(e) 0.91 m diameter footing in the site No. 2

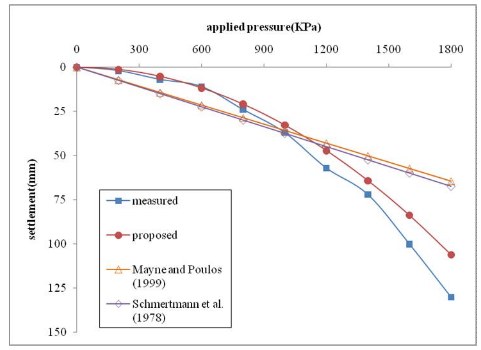


(f) 1 x 1 m footing in the site No. 2

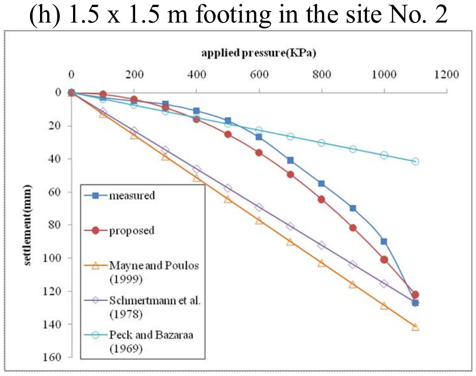
Fig. 10. Comparison of the predicted and measured settlements for the studies methods in three site



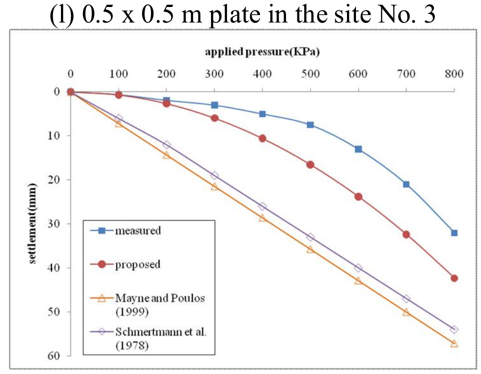
(h) 1.5 x 1.5 m footing in the site No. 2



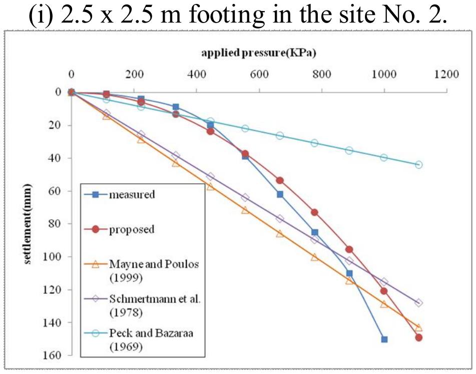
(l) 0.5 x 0.5 m plate in the site No. 3



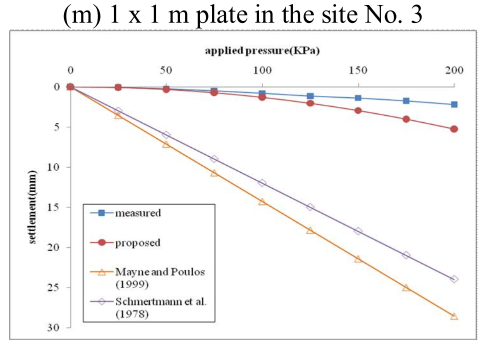
(i) 2.5 x 2.5 m footing in the site No. 2.



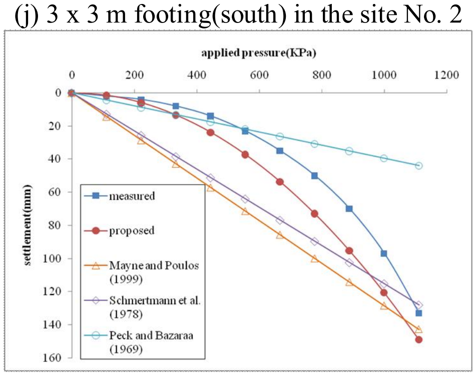
(m) 1 x 1 m plate in the site No. 3



(j) 3 x 3 m footing(south) in the site No. 2



(n) 2 x 2 m plate in the site No. 3



(k) 3 x 3 m footing(north) in the site No. 2

Fig. 10. (Continued)

Comparison of predicted versus measured load for 25 mm settlement from the proposed method in this paper and three conventional method is presented in Tables 3. In Fig 11 shows the curve of normal variations of the proposed methods. Carefully at the curves, can be seen that the normal curve of the proposed method is close to one. This can be confirmed the more accuracy of the proposed method than the other methods.

Table. 3. Comparison between measured and predicted loads for 25 mm settlement

Footing size, B (m)	Location	Predicted load for 25 mm settlement (KPa)				
		Peck and Bazaraa (1969)	Schmertmann et al. (1978)	Mayne and Poulos (1999)	Proposed method in this study	Measured
0.25	USA	1645	1397	2630	1355	***970***
0.46	USA	930	780	1430	997	***900***
1x1	USA	1042	455	585	790	***850***
1.5x1.5	USA	844	326	395	645	***667***
2.5x2.5	USA	663	236	195	490	***576***
3x3 (north)	USA	631	217	194	456	***578***
3x3 (south)	USA	631	217	194	456	***500***
0.5x0.5	SWEDEN	---------	667	465	875	***820***
1x1	SWEDEN	---------	381	353	616	***780***

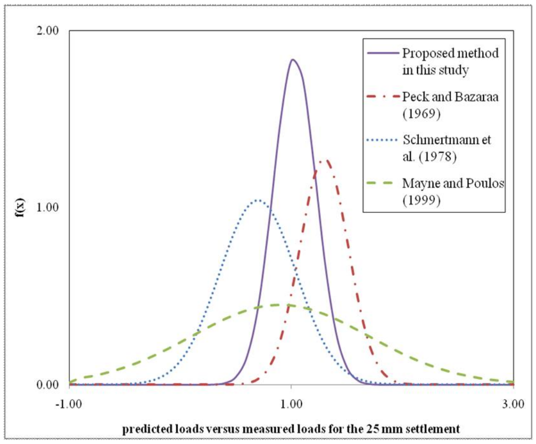


Fig. 11. Normal curves for the ratio of the predicted loads to measured loads @ 25 mm settlement, for the studies methods.

Specification of the normal curves is shown in Table. 4. Based on the normal curves of the proposed method in this study, the ratio of the predicted loads to measured loads is 0.98 at the nine case histories. It means that the new method predicts the foundation settlement with less overestimation or underestimation than the other methods. The results of the comparison indicate better accuracy and less scatter for the proposed method than other methods. Good agreement was obtained between measured and predicted soil deformation data.

Table. 4. Specification of the normal curves for the studied methods.

	Peck and Bazaraa (1969)	Schmertmann et al. (1978)	Mayne and Poulos (1999)	Proposed method in this study
mean	1.25	0.65	0.85	0.98
standard deviation	0.22	0.34	0.80	0.19

5.3 Discussion

Among major aspects for analysis and design of foundations, the bearing capacity and settlement aspects are interactive and commonly realized by geotechnical engineers. Most existing methods used to predict settlement of footings in granular soils are empirical and involve correlating measured settlements with parameters from tests that are convenient and widely used SPT and CPT, but which do not measure, directly, a relevant soil property.

The small-strain shear modulus G_{max} is a fundamental soil property that is applicable to both monotonic static and dynamic loading conditions. In-situ direct estimation of small-strain stiffness of soil is more effectively and reliably than those derived from in-situ tests such as SPT or CPT and laboratory testing. Yet, G_{max} is too stiff for direct use in computing foundation displacements. For dynamic tests, modulus reduction curves G/G_{max} versus log (γ), have been developed to calculate the shear modulus at a given strain level (e.g. Vucetic and Dobry 1991).

In this study, we proposed a new method in term of small-strain stiffness in order to estimate the settlement of footing in granular soils. For this purpose, a power law relationship was presented to define the mean normalized shear modulus, G/G_{max}, versus shear strain, γ, curve for granular soils based on data from recent scientific paper and reports. This method modified the small-stain stiffness according to stress levels or corresponding strain level. In order to evaluate the prediction method in term of maximum stiffness (G_{max}), a series of case histories were conducted.

The comparison between settlement predicted and measured, demonstrate the accuracy of the proposed method in this paper. The settlement predicted by the proposed method is closer than the settlement predicted by Schmertmann(1978), Peck and Bazaraa (1969) and Mayne and Poulos (1999) to measured settlement.

6. Conclusion

In the present study, estimation of the settlement of circular footings on granular soils was investigated based on shear wave velocity (V_s) and the shear modulus at small strains (G_{max}), and Young's modulus at small strains (E_{max}).

The results of this study are as follow:

1. The advantage of using a real soil property (such as E_{max}) in settlement predictions/analyses, field seismic measurements make it possible to provide information about a whole site much more accurately than can be obtained with point

measurements in soil borings or soundings. The seismic measurements have considerable advantage of being made in situ on undisturbed soil.
2. The surface wave method such as SASW and CSWS have several advantages over more conventional borehole methods like cross-hole including (1) the adverse effects of the presence of the borehole and poor receiver coupling are avoided; (2) depending on the source of ground vibration, frequencies used in surface wave testing can be much lower than borehole geophysical methods and thus closer to the frequencies encountered during dynamic loading of a site and (3) the noninvasive nature of surface wave measurements makes the test more versatile and economical.
3. The soil behavior is non-linearity and the stiffness of soil reduced with increasing the strain level. For this purpose, we a power law formula presented to predict the variations of stiffness according to strain level.
4. Based on theory elasticity and the proposed formula, a new method was developed in term of small-strain stiffness in order to estimate the immediate settlement of footing.
5. In order to validate the proposed method, the results of the survey of loading tests in four sites were evaluated and compared. Appropriate coincidence between the result of loading test and predicted settlement, shows the accuracy of proposed method in comparison to other methods.
6. Evaluation of the normal curves for the studies method shows that the average of the ratio of the calculated load to the measured load at 25 mm settlement are for the proposed method in this study, 0.98, Peck and Bazaraa method, 1.25, Schmertmann method, 0.65, and for Mayne and Poulos method, 0.85. This comparison shows that the proposed method is better than other methods with standard deviation equal to 0.19.
7. In general, predictions based on in situ parameters from seismic measurements are closer to measured settlement under service loads.

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