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## A NON-LINEAR METHOD TO ESTIMATE ELASTIC SETTLEMENT OF SHALLOW FOUNDATIONS USING SMALL-STRAIN STIFFNESS

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### ABSTRACT

In this paper, we investigated how to utilize the small-strain stiffness order to estimate the settlement of shallow foundation on granular soils. For this purpose, a power law equation between normalized shear modulus and shear strain was presented. Based on elasticity theory and proposed equation, a new method in term of small-strain stiffness was suggested order to estimate the immediate settlement. In order to evaluate the proposed method, a series case history was studied, that included the loading tests and seismic geophysical tests. 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.

### 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), Schmertmann (1970), Schmertmann et al. (1978), Burland and Burbidge (1985). Meyerhof (1956) and Peck and Bazaraa (1969) methods are similar to the one proposed by Terzaghi and Peck (1948). 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, the mentioned correlations overpredict settlements. Seismic wave velocity measurements have been used to characterize in-situ soil and rock stiffness's 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 ( $E_{max}$ ).

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

Where  $\rho$  = mass density of the soil.

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

Where:  $\nu$  = 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,  $G_{max}$  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).

Elhakim(2005) A closed-form algorithm is proposed for generating non-linear load displacement curves for footings and mats within an equivalent elastic framework. Sheehan (2005) a study conducted to investigate how well measured soil stiffnesses determined by field and laboratory dynamic tests predict the settlement of a shallow footing on a granular soil. Moxhay(2008) a computerized method has been developed for predicting settlement from the small-strain stiffness data obtained in continuous surface wave surveys(CSWS). park et al(2010) developed a series of

empirical parameters, termed the *modulus reduction factor* (MRF), to obtain a secant modulus that defines the settlement at a particular footing pressure.

In this paper is provided a new method order to determine the settlement of shallow foundation with measuring the in situ shear wave velocity, determine the maximum shear modulus and based on elasticity theory. The suggested relationship in this paper will be modified maximum 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.

## 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  $E_s$  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  $G_{max}$  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. This was the approach of Atkinson & Salfors (1991). 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:

- Parameters at very small (ideally reference) strain levels (e.g.  $E_0$ ,  $\nu_0$  and  $G_0$ ).
- 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,  $G_{max}$  is too high for direct use in computing settlement of shallow foundation. Therefore, small-strain stiffness must be modified based on stress levels or strain levels.

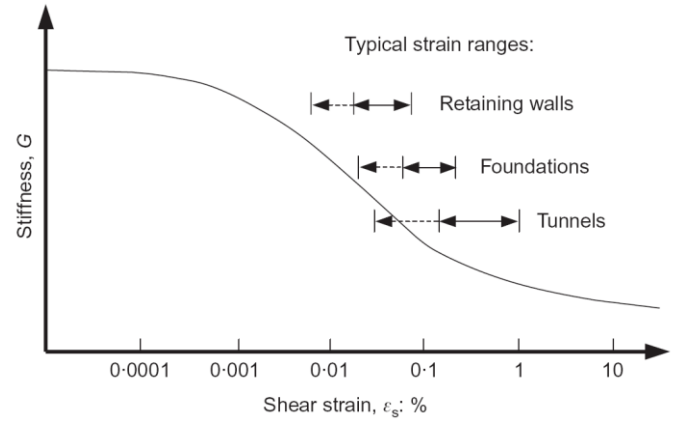


Fig. 1 Typical stiffness variation and strain ranges for different structures

## 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:

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

Where  $\gamma_{\%}$  = shear strain in percent.

Bands defining  $G/G_{max}$  versus shear strain for sands (Seed and Idriss 1970) 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  $G/G_{max}$  versus shear strain, generally falls near the center of the range of data for sands defined by Seed et al. (1970).

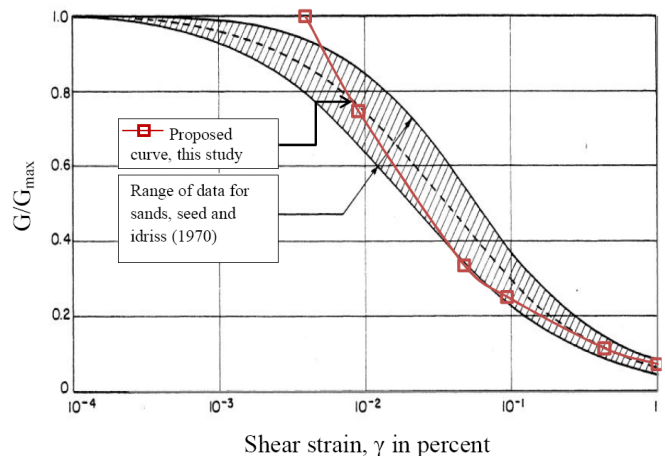


Fig. 2 Comparison of the proposed curve in this study and range of data for sands defined by Seed et al. (1970).

## 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,  $E_{\max}=2(1+\nu)G_{\max}$ .

**Step3:** The vertical strain at the centre of the layers,  $\varepsilon$ , are then calculated from the elasticity theory.

$$\varepsilon = \frac{\sigma_z}{E} - \nu \left( \frac{\sigma_x}{E} + \frac{\sigma_y}{E} \right) \quad (4)$$

Where  $\sigma_z$ ,  $\sigma_y$  and  $\sigma_x$  = vertical and horizontal stress,  $\nu$  = Poisson 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 (day,2006).

Hence from eq. (4):

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

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

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

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

$$\frac{G}{G_{\max}} = \frac{0.0725}{\sqrt{\varepsilon_{\%}(1+\nu)}} = \frac{0.0636}{\sqrt{\varepsilon_{\%}}} \quad (7)$$

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

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

**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,  $\sigma_z$ , is then calculated from the Boussinesq formula.

$$\sigma_z = q \left[ 1 - \frac{1}{\left( 1 + \left( \frac{B/2}{B} \right)^2 \right)^{\frac{3}{2}}} \right] = 0.285q \quad (9)$$

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

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

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

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

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

## 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,
- 2- Texas A&M University, USA and
- 3-Vattahammar, Sweden.

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

## SITE CONDITION AND FIELD TEST SITE

### 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  $18 \text{ kN/m}^3$ .

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 (CSWS) 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. 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.

Table 1. Case histories general specification

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



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.

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.

## 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).

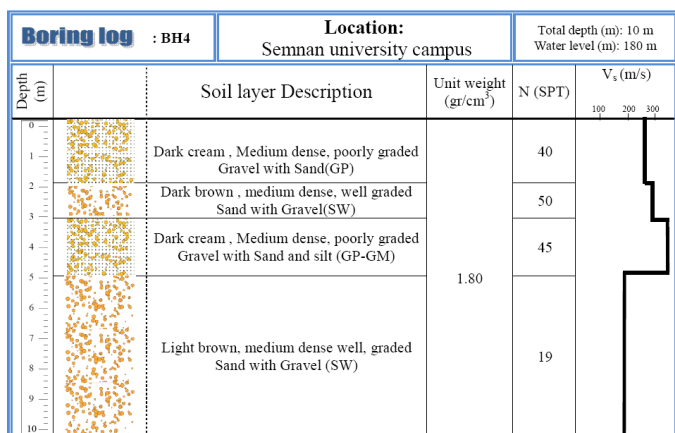


Fig. 3 Boring log for the semnan university campus, Iran.



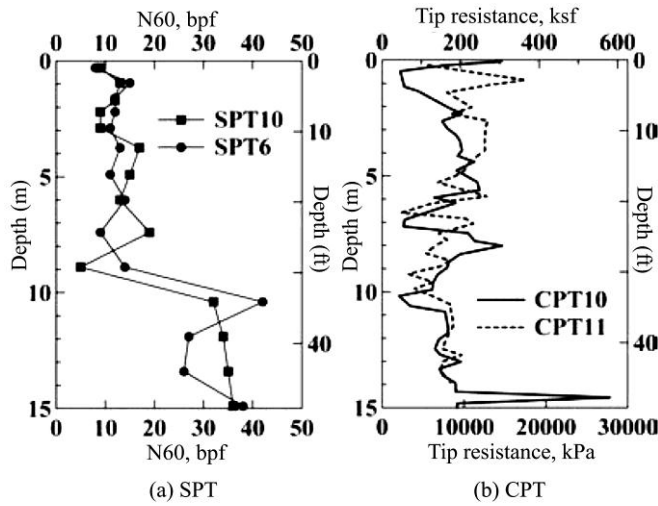


Fig. 6 SPT and CPT profiles at the Texas A&M University, USA

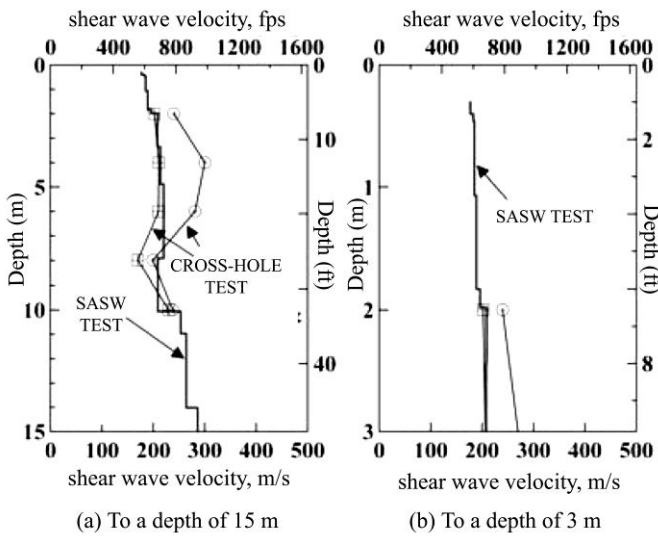


Fig. 7 Shear wave velocity profiles at the Texas A&M University, USA

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.

### 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. 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.

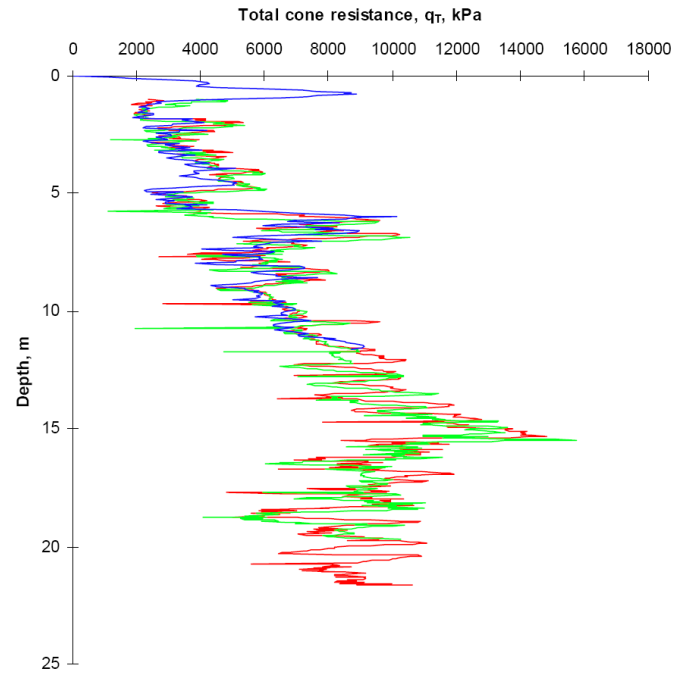


Fig. 8 Total cone resistance measured in three CPT tests in the test field at Vattahammar, Sweden.

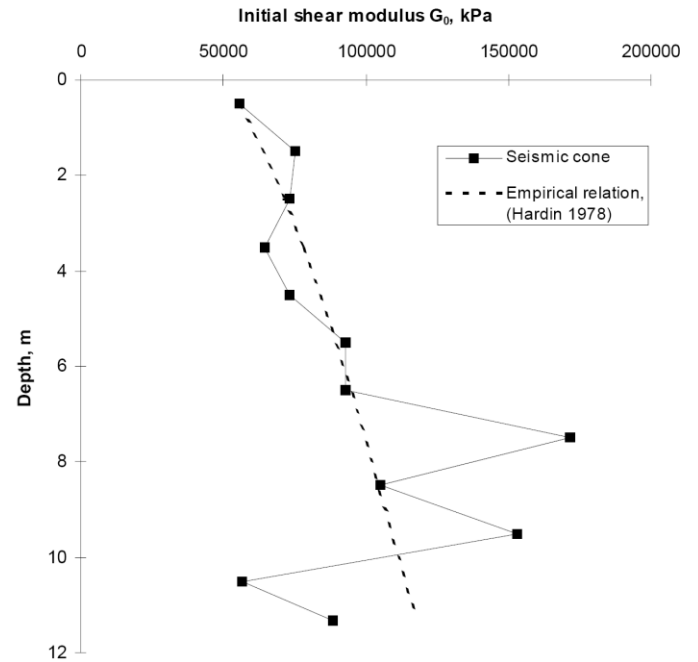


Fig. 9 Measured and estimated initial shear moduli in the test field at Vattahammar, Sweden.

## 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
Peck and Bazaraa (1969) (das et al 2009)	$S(mm) = C_w C_D \frac{0.53q(kPa)}{(N_1)_{60}} \left( \frac{2B}{B+0.3} \right)^2$	$C_D$ = embedment correction factor; $C_w$ = water table correction factor; $N$ = corrected SPT-N value;
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.
Mayne and Poulos (1999)	$S_{footing} = \frac{q_{net} B I_G I_F I_E (1-\nu^2)}{E_0}$	$\nu$ = Poisson's ratio; $q_{net}$ = applied bearing pressure; $E_s$ = modulus of elasticity of bearing soil; $I_G$ , $I_F$ and $I_E$ influence factor.

The proposed methods in Table 2 to predict the settlement of shallow foundations on cohesionless soils based on SPT N values and CPT point resistance,  $q_c$ .

#### INPUT PARAMETERS FOR EMPIRICAL METHODS

In this study, the correlation between  $E$  and  $N_{60}$  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  $N_{60}$  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  $q_c$  values for normally consolidated sands (Bowles 1996):

$$E(kPa) = 500(N+15) \quad (13)$$

$$E(kPa) = 2-4 q_c \quad (14)$$

## 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. Figures 10 through 22 give load-displacement curves for the, 13 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.

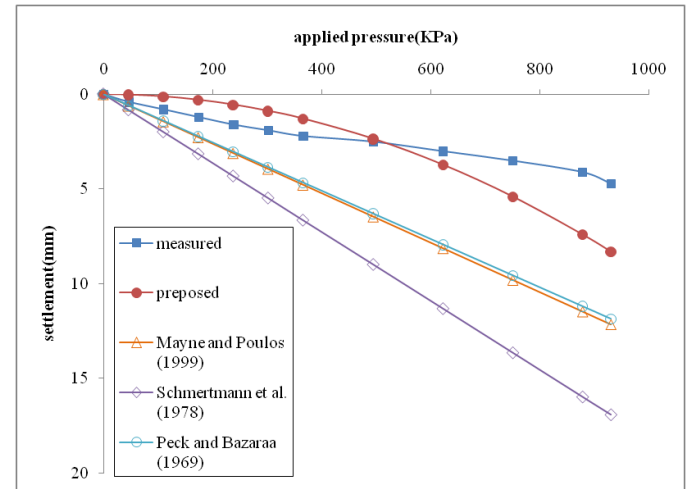


Fig. 10 Comparison of the predicted and measured settlements for the load test on the 0.45 m diameter plate in the semnan university campus, Iran.

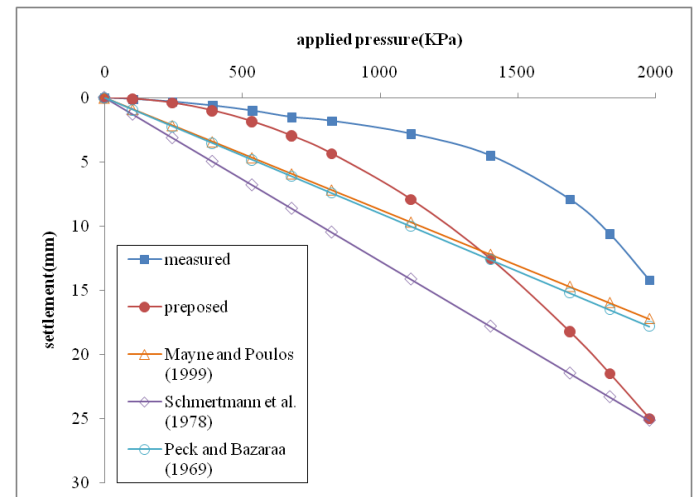


Fig. 11 Comparison of the predicted and measured settlements for the load test on the 0.3 m diameter plate in the semnan university campus, Iran.

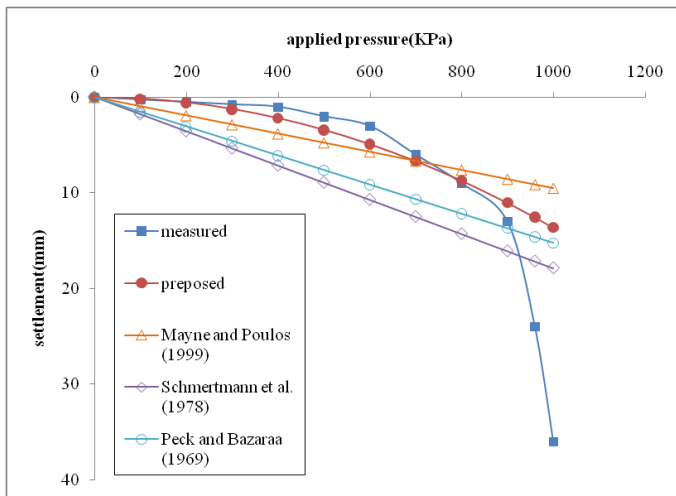


Fig. 12 Comparison of the predicted and measured settlements for the load test on the 0.25 m diameter plate in the test field at Texas A&M University, USA.

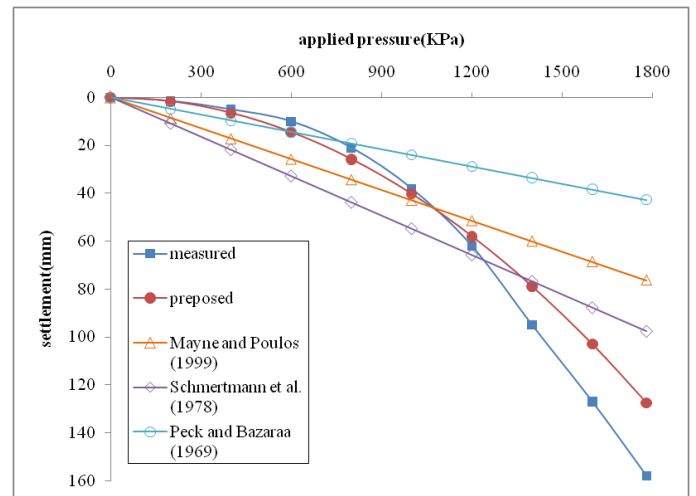


Fig. 15 Comparison of the predicted and measured settlements for the load test on the 1 x 1 m footing in the test field at Texas A&M University, USA.

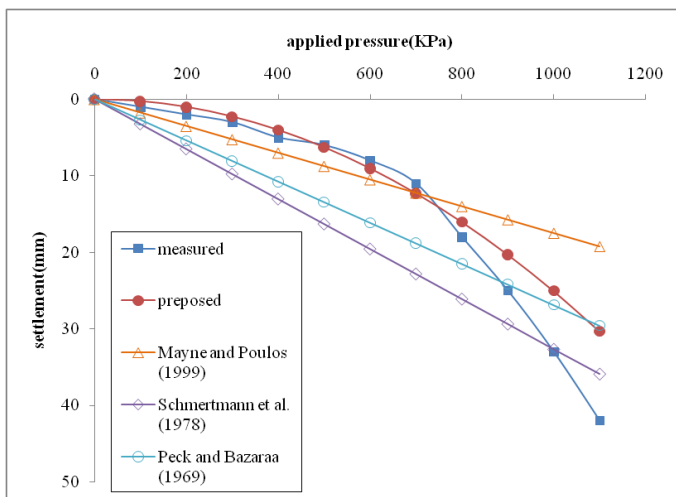


Fig. 13 Comparison of the predicted and measured settlements for the load test on the 0.46 m diameter footing in the test field at Texas A&M University, USA.

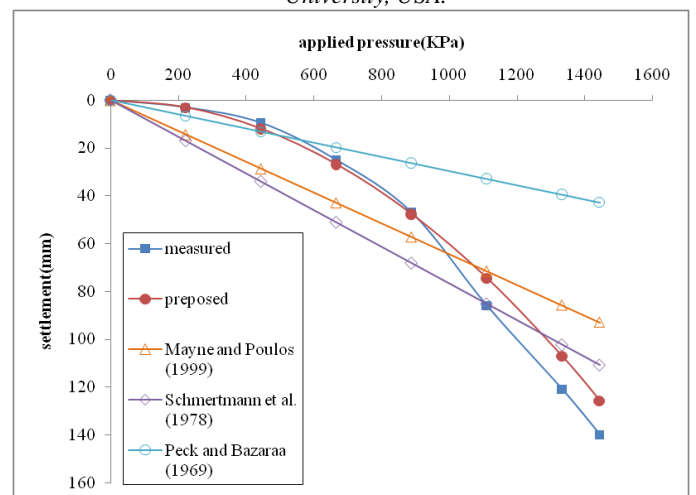


Fig. 16 Comparison of the predicted and measured settlements for the load test on the 1.5 x 1.5 m footing in the test field at Texas A&M University, USA.

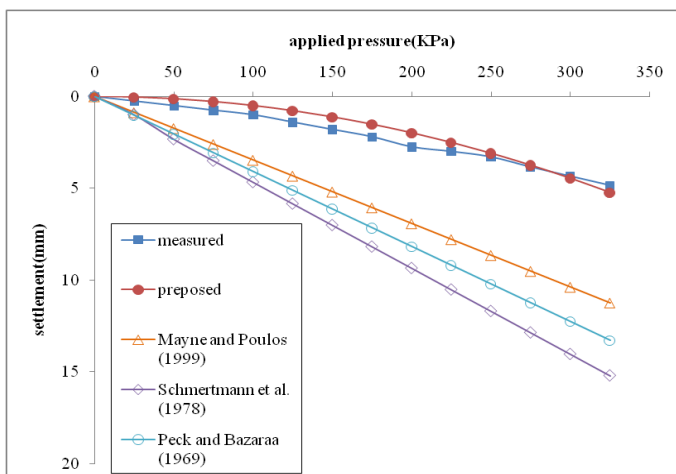


Fig. 14 Comparison of the predicted and measured settlements for the load test on the 0.91 m diameter footing in the test field at Texas A&M University, USA.

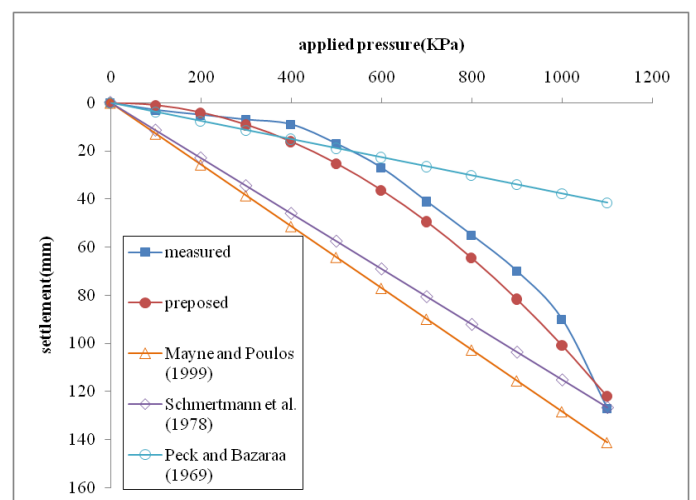


Fig. 17 Comparison of the predicted and measured settlements for the load test on the 2.5 x 2.5 m footing in the test field at Texas A&M University, USA.



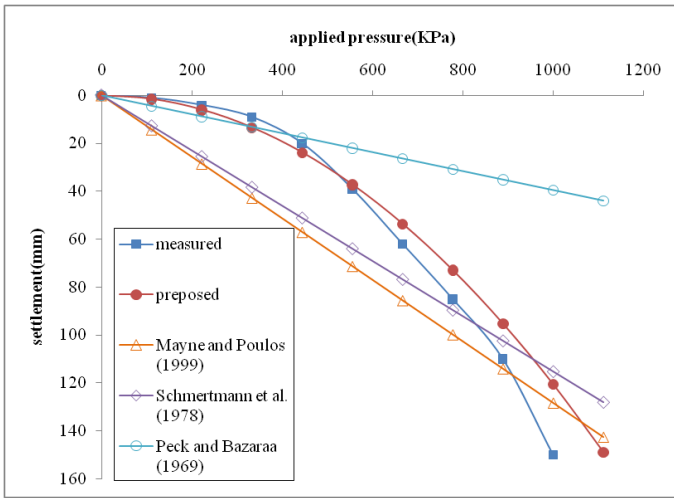


Fig. 18 Comparison of the predicted and measured settlements for the load test on the 3 x 3 m footing (south) in the test field at Texas A&M University, USA.

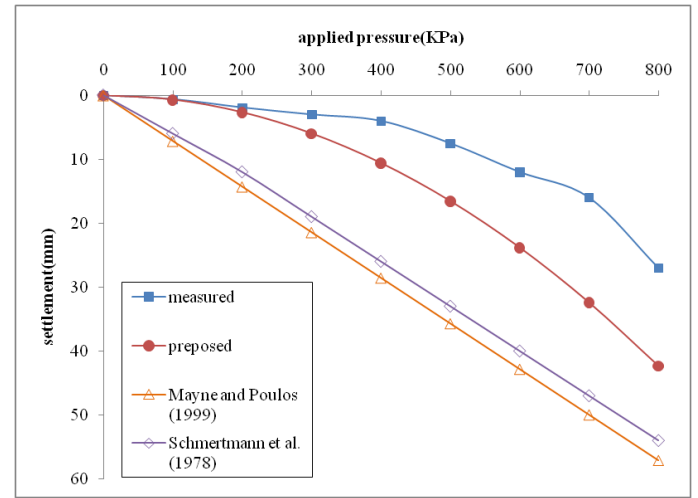


Fig. 21 Comparison of the predicted and measured settlements for the load test on the 1 x 1 m plate in the test field at Vatthammar, Sweden.

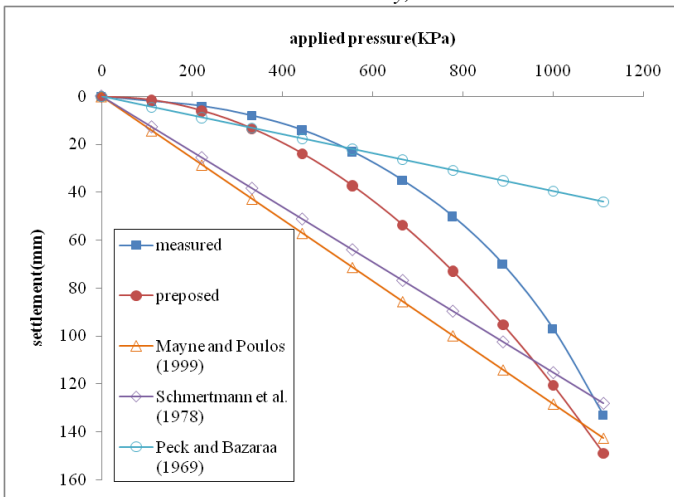


Fig. 19 Comparison of the predicted and measured settlements for the load test on the 3 x 3 m footing (north) in the test field at Texas A&M University, USA.

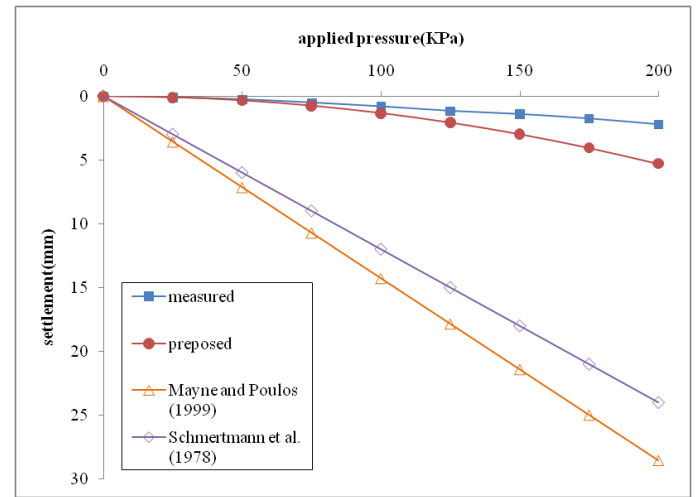


Fig. 22 Comparison of the predicted and measured settlements for the load test on the 2 x 2 m plate in the test field at Vatthammar, Sweden.

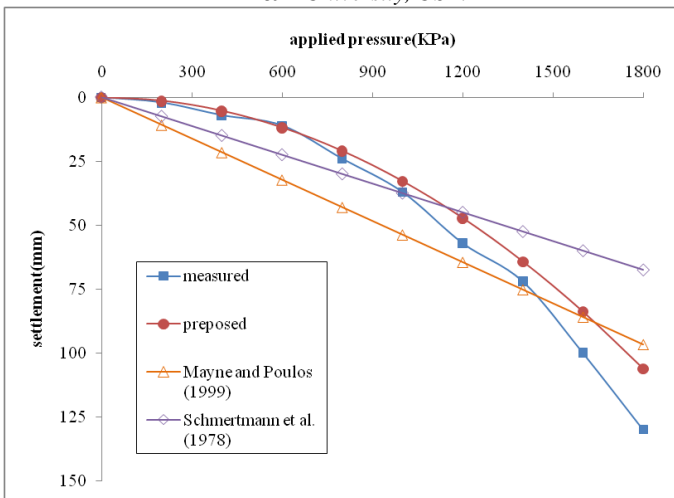


Fig. 20 Comparison of the predicted and measured settlements for the load test on the 0.5 x 0.5 m plate in the test field at Vatthammar, Sweden.

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 23 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. 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 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

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

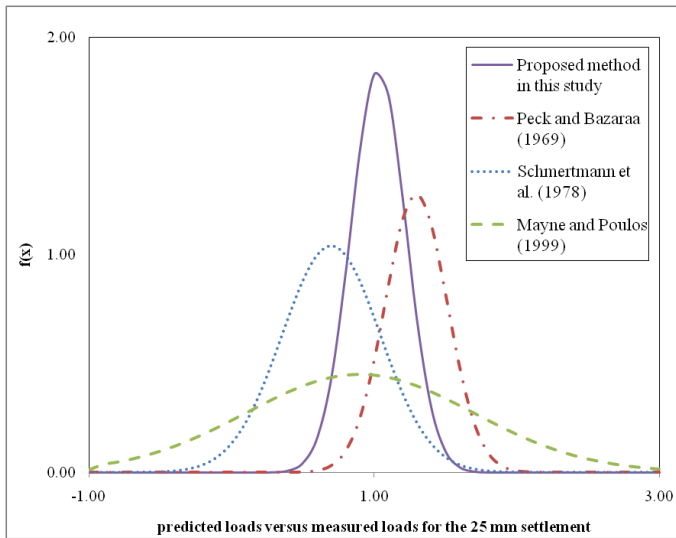


Fig. 23 Normal curves for the ratio of the predicted loads to measured loads @ 25 mm settlement, for the studies methods

## 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  $\log(\gamma)$ , 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,  $\gamma$ , curve for granular soils based on data from recent scientific paper and reports. This method modified the small-strain 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 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.

## 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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