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Scale Effect of Plate Load Tests in Unsaturated Soils

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ABSTRACT: The applied stress versus surface settlement (*SVS*) behavior from in-situ plate load tests (*PLT*s) is valuable information that can be used for the reliable design of shallow foundations. In-situ *PLT*s are commonly conducted on the soils that are typically in a state of unsaturated condition. However, in most cases, the influence of matric suction is not taken into account while interpreting the *SVS* behavior of in-situ *PLT*s. In addition, the sizes of plates used for load tests are generally smaller in comparison to real sizes of footings used in practice. Hence, in-situ *PLT* results should be interpreted taking account of not only matric suction but also the scale effects. In the present study, discussions associated with the uncertainties in interpreting the *SVS* behavior of *PLT*s taking account of matric suction and scale effect are detailed and discussed.

Keywords: Unsaturated Soil, Matric Suction, Plate Load Test, Shallow Foundation, Settlement

1. INTRODUCTION

Bearing capacity and settlement are two key parameters required in the design of foundations. There are several techniques available today to determine or estimate both the bearing capacity and settlement behavior of foundations based on experimental methods, in-situ tests and numerical models including the finite element analysis (hereafter referred as *FEA*). However, in spite of these advancements, various types of damages can be caused to the superstructures placed on shallow foundations due to the problems associated with the settlements leading to cracks, tilts, differential settlements or displacements. This is particularly true for coarse-grained soils such as sands in which foundation settlements occur quickly after construction. Due to this reason, the settlement behavior is regarded as a governing parameter in the design of shallow foundations in coarse-grained soils [1]-[5]. Several foundation design codes suggest restricting the settlement of shallow foundations placed in coarse-grained soils to 25 mm and also limit their differential settlements (e.g. [6]). Such design code guidelines suggest that the rational design of shallow foundations can be achieved by estimating the applied stress versus surface settlement (hereafter referred to as *SVS*) behavior instead of estimating the bearing capacity and settlement separately.

The most reliable testing method to estimate the *SVS* behaviors of shallow foundations is in-situ plate load tests (hereafter referred to as *PLTs)*. In-situ *PLT*s are commonly performed on soils that are typically in a state of unsaturated condition. This is particularly true in arid or semi-arid regions where the natural ground water table is deep. Hence, the stresses associated with the constructed infrastructures such as shallow foundations are distributed in the zone above the ground water table, where the pore-water pressures are negative with respect to the atmospheric pressure (i.e., matric suction). Several researchers showed that the applied stress versus surface settlement (hereafter referred to as *SVS*) behaviors from model footings [7]-[10] or in-situ *PLTs* [11], [12] are significantly influenced by matric suction. However, in most cases, the in-situ *PLT* results are interpreted without taking account of the negative pore-water pressure above ground water table. In other words, the influence of capillary stress or matric suction towards the *SVS* behavior is conventionally ignored in engineering practice. Moreover, the *PLT*s are generally conducted with small sizes of plates (either steel or concrete) in comparison to the real sizes of foundations. Due to this reason, the scale effect has been a controversial issue in implementing the *PLT* results with certainty into the design of shallow foundations. These details suggest that the reliability of the design of shallow foundations based on the *PLT* results can be improved by taking account of the influence of not only matric suction but also the scale effects associated with the plate size on the *SVS* behaviors.

In the present study, two sets of in-situ plate and footing load test results in unsaturated sandy and clayey soils available in the literature are revisited. Based on the results of these studies, an approach is presented such that the uncertainties associated with the scale effects are reduced or eliminated. In addition, discussions on how to interpret the in-situ *PLT* results taking account of matric suction are also presented and discussed. Moreover, a methodology to estimate the variation of *SVS* behavior with respect to matric suction using the *FEA* is introduced.

2. BACKGROUND

2.1 Plate Load Test

In-situ *PLT*s are generally conducted while designing shallow foundations [13] or pavement structures [14]-[16] to estimate the reliable design parameters (i.e., bearing capacity and displacement) or to confirm the design assumptions. Fig. 1 shows typical *SVS* behavior from a *PLT*. The peak stress is defined as ultimate bearing capacity, q_{ult} for general failure; however, in the case where well-defined failure is not observed (i.e., local or punching failure), the stress corresponding to the 10% of the width of a foundation [13] or the stress corresponding to the intersection of elastic and plastic lines of the *SVS* behavior is typically regarded as q_{ult} [7], [11], [17].

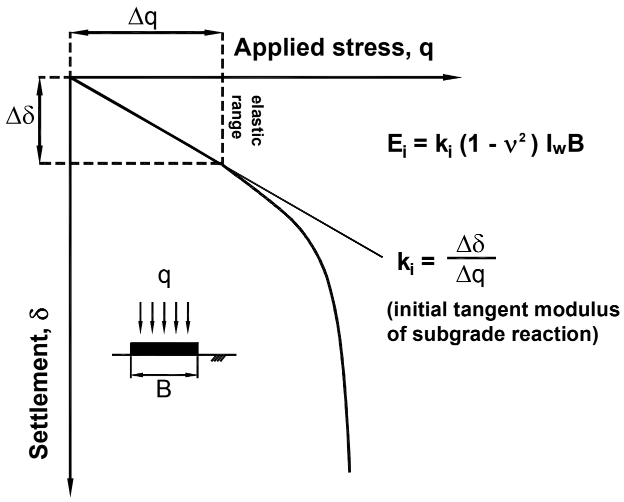


Fig. 1. Schematic stress versus settlement behavior from plate load test.

The elastic modulus can be estimated based on the modulus of subgrade reaction, k that is defined as the slope of *SVS* behavior (i.e., δ versus q) using the theory of elasticity as shown in Fig. 1. The maximum elastic modulus (i.e., initial tangent elastic modulus, E_i) can be computed using the modulus of subgrade reaction, k value in the elastic range of *SVS* behavior (i.e., k_i).

$$E = \frac{\left(1-\nu^2\right)}{\left(\delta / q\right)} B I_w = k\left(1-\nu^2\right) I_w B \tag{1}$$

where E = elastic modulus, ν = Poisson's ratio, δ, q = settlement and corresponding stress, B = width (or diameter) of bearing plate, I_w = factor involving the influence of shape and flexibility of loaded area, and k = modulus of subgrade reaction

Ultimate bearing capacity, q_{ult} and elastic modulus, E estimated based on the *SVS* behavior from a *PLT* are representative of soils within a depth zone which is approximately 1.5B - 2.0B [18]. The model footing test results in sands also showed that the settlement behavior of relatively dry sand is similar to that of sand with a ground water table at a depth of 1.5B below the model footing [19]. These results indirectly support that the increment of stress due to the load applied on the model footing is predominant in the range of 0 to 1.5B - 2B below model footings. Modeling studies results (e.g. [3]-[5]) are also consistent with these observations. This fact also indicates that the *SVS* behavior from *PLT* is influenced by plate (or footing) size since different sizes result in different sizes of stress bulbs and mean stresses in soils. This phenomenon which is conventionally defined as 'scale effect' needs to be investigated more rigorously to rationally design the shallow foundations.

2.2 Scale Effects of Plate Load Test in Cohesionless Soils

The Terzaghi bearing capacity factor, N_γ decreases with an increase in the width of footings [20]. Various attempts have been made by several researchers to understand the reasons associated with the scale effects. Three main explanations that are generally accepted are summarized below.

i) Reduction in the internal friction angle, ϕ' with increasing footing size (i.e., nonlinearity of the Mohr-Coulomb failure envelop) [20]-[22].
ii) Progressive failure (i.e., different ϕ' mobilized along the slip surfaces below a footing) [23],[24].
iii) Particle size effect (i.e., the ratio of soil particles to footing size) [23],[25].

The reduction in ϕ' with an increasing footing size is attributed to the fact that the larger footing size contributes to a higher mean stress in the soils. The higher mean stress in soils then leads to lower ϕ' due to the nonlinearity of the Mohr-Coulomb failure envelop when tested over a large stress range. According to Hettler and Gudehus [22], there is lack of consistent theory to explain the progressive failure mechanism in the soils below different sizes of footings. In addition, the particle size effect for the in-situ plate (or footing) load test (hereafter the term 'plate' is used to indicate both steel plate and concrete footing) can be neglected since the ratio of plate size, B to d_{50} (i.e., grain size corresponding to 50% finer from the grain size distribution curve) for in-situ *PLT*s are mostly greater than 50 -100 [26].

3. PLATE LOAD TEST RESULTS

In this study, two sets of in-situ *PLT* results in sandy and clayey soils available in the literature are revisited to discuss the scale effects.

Briaud and Gibbens [27] conducted several series of in-situ square footing (i.e., 1m, 2m, 2.5m, and 3m) load tests in unsaturated sandy soils. These studies were summarized in a symposium held at the Texas A&M University in 1994 [27] and the results are shown in Fig. 2.

Consoli et al. [28], in another study, conducted in-situ *PLT*s in a unsaturated low plasticity clayey soil (I_p = 20%) using three steel circular plates (i.e., 0.3m, 0.45m, and 0.6m; *PLT*) and three concrete square footings (i.e., 0.4m, 0.7m, and 1.0m; *FLT*) (Fig. 3).

As can be seen in Fig. 2 and Fig. 3, the bearing capacity increases with decreasing plate size and different settlements are observed under a certain stress. The *SVS* relationships clearly show that their behavior is dependent on the plate size (i.e., scale effect).

4. ELIMINATION OF SCALE EFFECT OF SHALLOW FOUNDATIONS

Briaud [29] suggested that scale effect in Fig. 2 can be eliminated by plotting the *SVS* behaviors as 'applied stress' versus 'settlement/width of footing' (i.e., δ/B) curves (i.e., normalized settlement) as shown in (2) and Fig. 4. Similar trends of results were reported in the literature [30],[31].

$$\frac{\delta}{B} = \frac{q\left(1-\nu^2\right)}{E} I_w \tag{2}$$

According to the report published by Briaud and Gibbens [32], this behavior can be explained using the analogy of triaxial tests (Fig. 5). Triaxial tests conducted on identical sand samples under the same confining pressure with different size top platens is analogous to conducting *PLT*s using different sizes of footings.

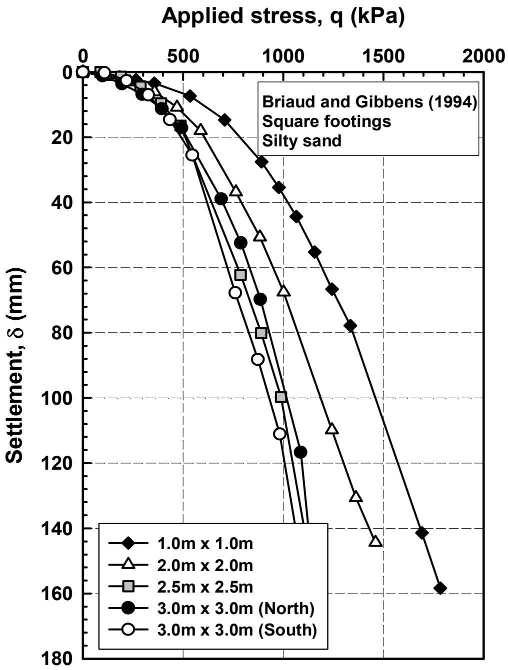


Fig. 2. Stress versus settlement behavior from in-situ footing load tests in silty sand (data from [27]).

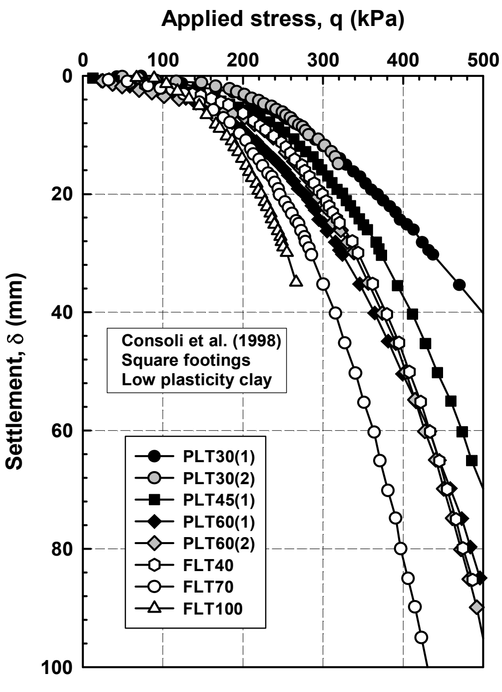


Fig. 3. Stress versus settlement behavior from in-situ plate and footing load tests in low plasticity clay (data from [29]).

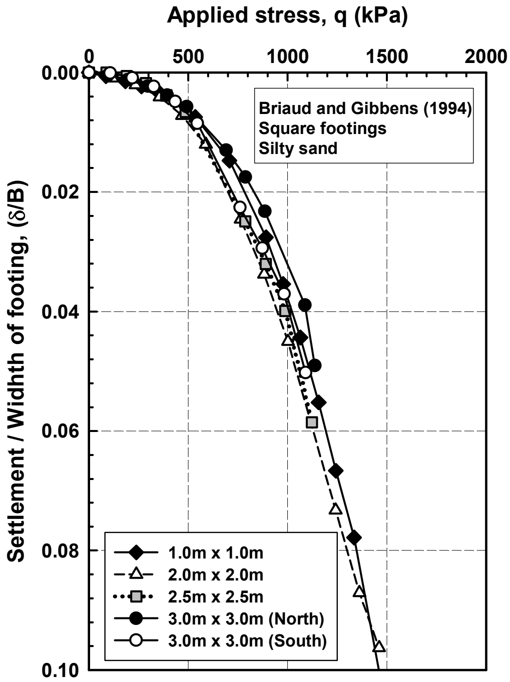


Fig. 4. Normalized in-situ footing load test results in silty sand (data from [32]).

The stresses versus strain behaviors for the triaxial samples are unique regardless of the diameter of the samples (i.e., the same stress for the same strain). This concept is similar to relationship between q versus δ/B from *PLT*s since the term, δ/B can be approximated as axial strain.

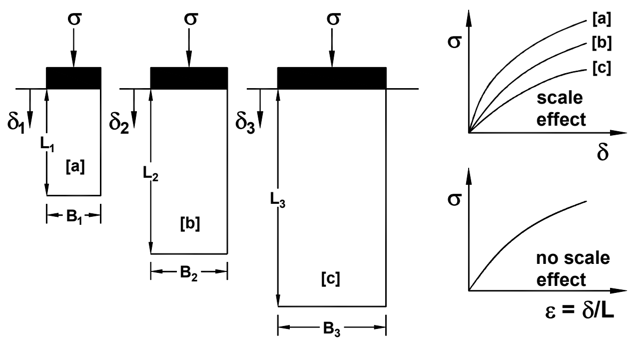


Fig. 5. Triaxial test/shallow foundation analogy (modified after [32]).

Consoli et al. [28] suggested that the scale effect of the *PLT*s in Fig. 3 can be eliminated when the applied stress and settlement are normalized with unconfined compressive strength, q_u and the footing width (or diameter), B respectively as shown in (3) and Fig. 6.

$$\left(\frac{q}{q_u}\right) = \left\{\frac{C_d}{q_u C_s}\right\}\left(\frac{\delta}{B}\right) \tag{3}$$

where q = applied stress, q_u = unconfined compressive strength at the depth of embedment, δ = surface settlement, B = width of footing, C_s = coefficient involving shape and stiffness of loaded area (I_w in (2)) and C_d = coefficient of deformation (= $E/(1-\nu^2)$).

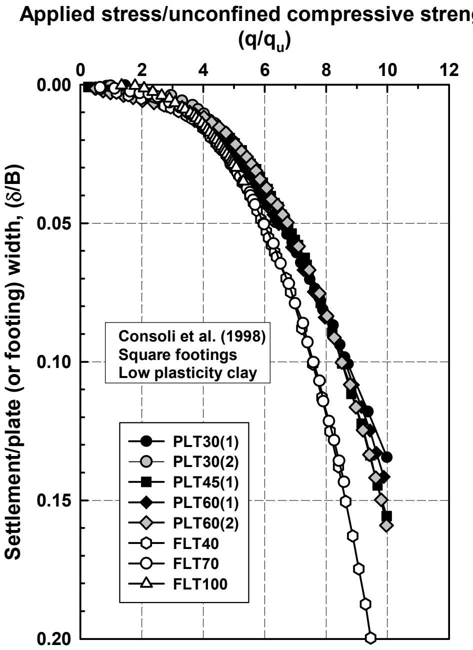


Fig. 6. Normalized in-situ plate and footing load test results in low plasticity clay (data from [28]).

They also analyzed *PLT*s results in sandy soils [33],[34] and showed that that the concept in (3) can be extended to the *PLT* results in sandy soils as well. As can be seen in Fig. 4 and Fig. 6, the curves, 'δ/B versus q' falls in a narrow range. Consoli et al. [28] suggested that such a behavior can be observed at sites where the soils are homogeneous and isotropic in nature.

5. SCALE EFFECTS ASSOCIATED WITH DIFFERENT PLATE SIZES IN UNSATURATED SOILS

5.1 Average Matric Suction Value

Matric suction distribution profile is mostly not uniform with depth in fields. In this case, the concept of '*average matric suction*' [3],[35] can be used as a representative matric suction value to interpret mechanical properties of a soil at a certain matric suction distribution profile. The average matric suction, Ψ is defined as a matric suction corresponding to the centroid of the suction distribution diagram from 0 to 1.5B - 2B depth.

As discussed earlier, the stress increment in a soil due to a load or a stress act on a shallow foundation is predominant in the range of 0 to 1.5B - 2B. Hence, when loads are applied on two different sizes of footings, the sizes of stress bulbs (in the depth zone of 0 to 1.5B - 2B) are different (Fig. 7). In other words, the stress bulb for the smaller footing (i.e., B_1) is shallower in comparison to that of the larger footing (i.e., B_2). These facts indicate that the *SVS* behaviors from *PLT*s are governed by E and ν values within the stress bulb. If the matric suction distribution profile is uniform with depth, the average matric suction value is the same regardless of footing size. However, if the matric suction distribution profile is non-uniform, the average matric suction value is dependent on the footing size. For example, the average matric suction value for the smaller plate, B_1 (i.e., Ψ_1) is greater than that of larger plate, B_2 (i.e., Ψ_2). In this case, the concept shown in (2) and (3) cannot be used to eliminate the scale effect of plate size since q_u [36], E_i [37], ν [5],[38] are not constant but vary with respect to matric suction. More discussions are summarized in later sections.

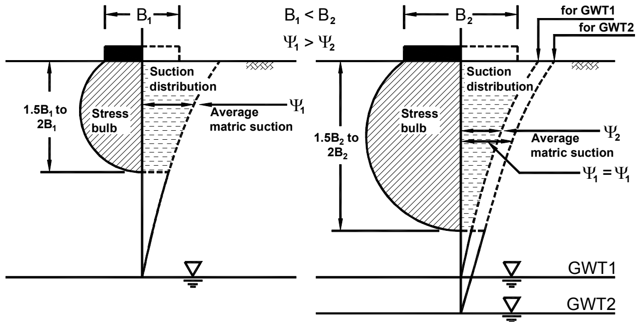


Fig. 7. Average matric suction values for different footing sizes under idealized non-constant matric suction distribution profile.

5.2 Variation of E_i with respect to Matric Suction for Coarse-grained Soils

Oh et al. [37] analyzed three sets of model footing test results in unsaturated sands [7], [9] and showed that the initial tangent elastic modulus, E_i is significantly influenced by matric suction. Based on the analyses, they proposed a semi-empirical model to estimate the variation of E_i with respect to matric suction using the Soil-Water Characteristic Curve (*SWCC*) and the E_i for saturated condition along with two fitting parameters, α and β.

$$E_{i(unsat)} = E_{i(sat)}\left[1+\alpha\frac{(u_a-u_w)}{(P_a/101.3)}\left(S^{\beta}\right)\right] \tag{4}$$

where $E_{i(sat)}$ and $E_{i(unsat)}$ = initial tangent elastic modulus for saturated and unsaturated conditions, respectively, P_a = atmospheric pressure (i.e., 101.3 kPa), and α, β = fitting parameters

They suggested that the fitting parameter, β = 1 is required for coarse-grained soils (i.e., plasticity index, I_p = 0%; *NP*). The fitting parameter, α is a function of footing size and the values between 1.5 and 2 can be recommended for large sizes of footings in field conditions to reliably estimate E_i (Fig. 8) and elastic settlement (Fig. 9). Vanapalli and Oh [39] analyzed model footing [37] and in-situ *PLT* [11], [12] results in unsaturated fine-grained soils and suggested that the fitting parameter, β = 2 is required for fine-grained soils. The analyses results also showed that the inverse of α (i.e., 1/α) nonlinearly increases with increasing plasticity index, I_p. The upper and the lower boundary relationship can be used for soils with low and high matric suction values, respectively at a certain I_p value (Fig. 10).

5.3 Variation of c_u with respect to Matric Suction for Fine-grained Soils

Oh and Vanapalli [36] analyzed six sets of unconfined compression test results [40]-[44] and showed that the c_u

value is a function of matric suction. Based on the analyses, they proposed a semi-empirical model to estimate the variation of undrained shear strength of unsaturated soils using the *SWCC* and undrained shear strength under

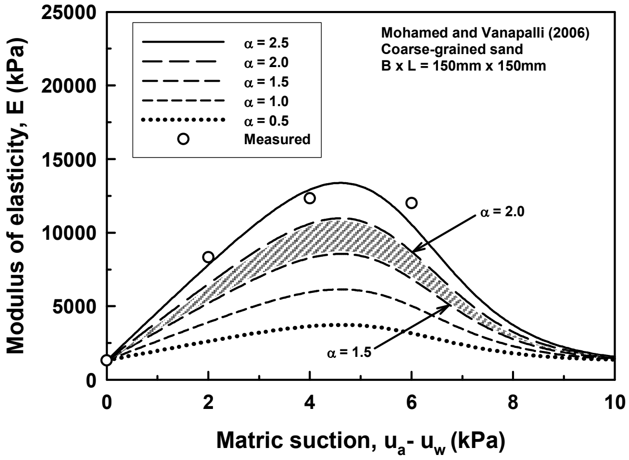


Fig. 8. Variation of initial tangent elastic modulus with the fitting parameter parameter, α for the 150mm ×150mm model footing (data from [9]).

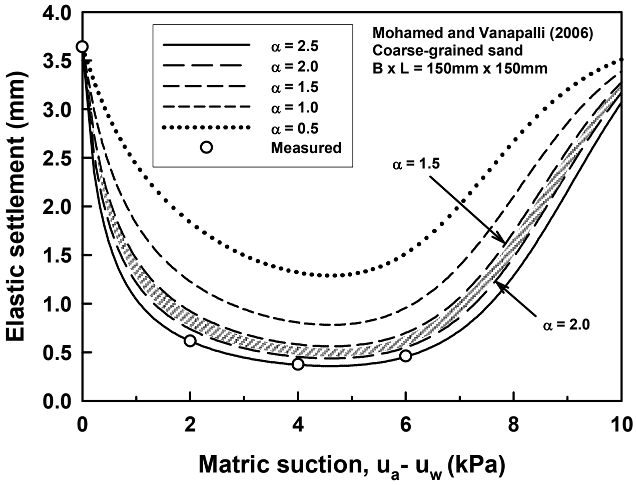


Fig. 9. Variation of elastic settlement with the fitting parameter parameter, α for the 150mm × 150mm model footing (data from [9]).

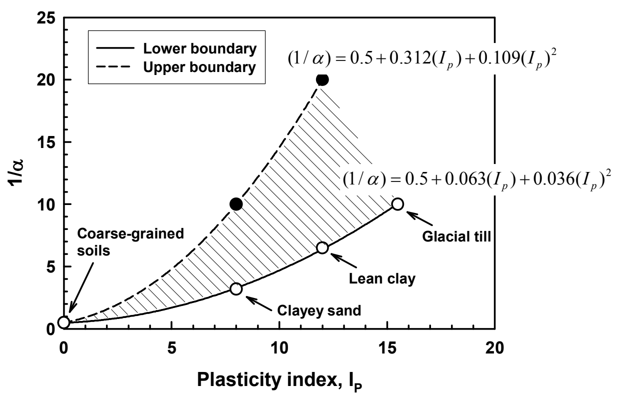


Fig. 10. Relationship between ($1/\alpha$) and plasticity index, I_p

saturated condition along with two fitting parameters, υ and μ as shown in (5). The form of (5) is same as (4).

$$c_{u(unsat)} = c_{u(sat)} \left[1 + \frac{(u_a - u_w)}{(P_a / 101.3)} \frac{(S^{\upsilon})}{\mu} \right] \tag{5}$$

where $c_{u(sat)}$, $c_{u(unsat)}$ = shear strength under saturated and unsaturated condition, respectively, P_a = atmospheric pressure (i.e., 101.3 kPa) and υ, μ = fitting parameters.

The fitting parameter, $\upsilon = 2$ is required for unsaturated fine-grained soils. Fig. 11 and Fig. 12 show the comparison between the measured shear strength values and those estimated using (5).

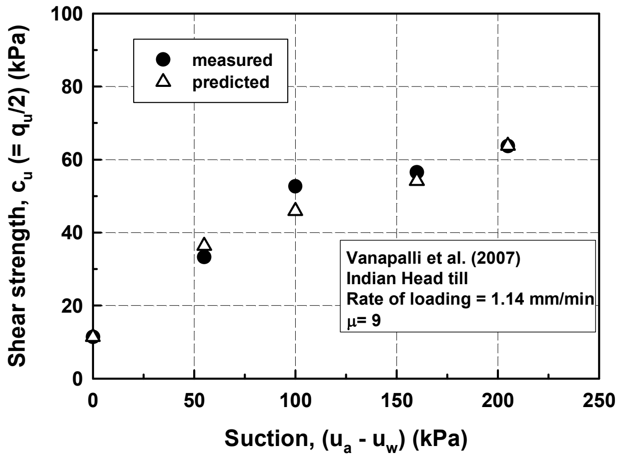


Fig. 11. Comparison between the measured and the estimated shear strength (data from [40]).

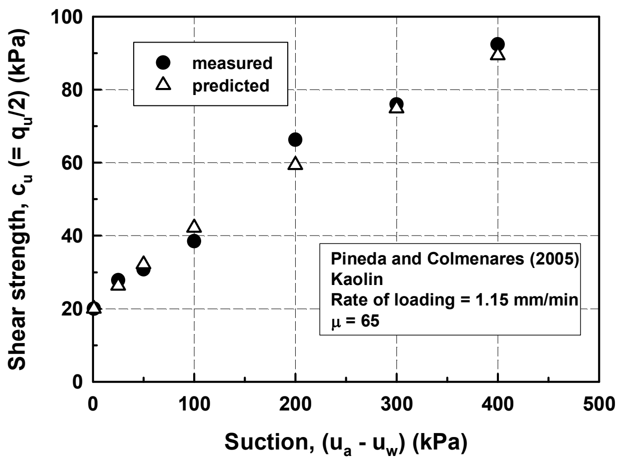


Fig. 12. Comparison between the measured and the estimated shear strength (data from [42]).

Fig. 13 shows the relationship between the fitting parameter, μ and plasticity index, I_p on semi-logarithmic scale for the soils used for the analysis. The fitting parameter, μ was found to increase linearly with two different slopes depending on the I_p values as shown in Fig. 13 and (6).

$$\begin{cases} \mu = 6.3055 \cdot e^{0.0298(I_p)} & for \;\; 8 \le I_p(\%) \le 15.5 \\ \mu = 2.305 \cdot e^{0.088(I_p)} & for \;\; 15.5 \le I_p(\%) \le 60 \end{cases} \tag{6}$$

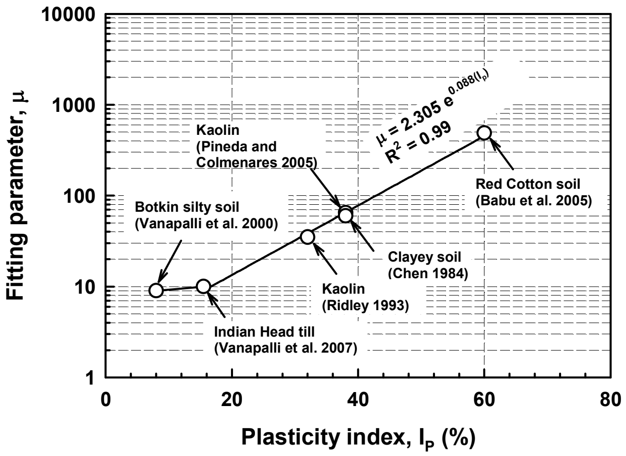


Fig. 13. Relationship between plasticity index, I_p and the fitting parameter, μ.

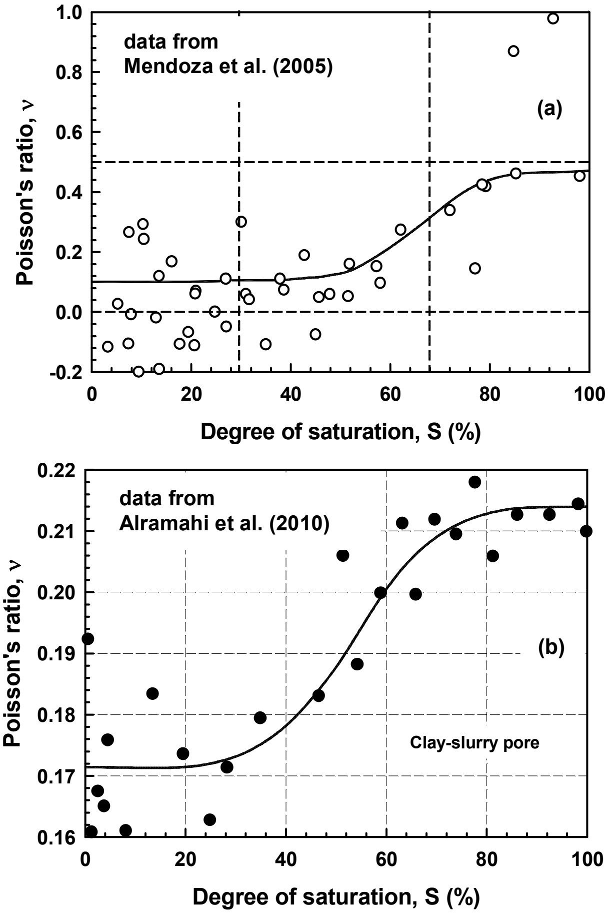


Fig. 14. Variation of Poisson's ratio, ν with respect to degree saturation [5].

5.4 Variation of Poisson's Ratio, ν with respect to Matric Suction

The Poisson's ratio, ν is typically considered to be a constant value in the elastic settlement analysis of soils. This section briefly highlights how ν varies with matric suction by revisiting published data from the literature.

Mendoza et al. [45] and Alramahi et al. [46] conducted bender element tests to investigate the variation of small-strain elastic and shear modulus with respect to degree of saturation for Kalolinite and mixture of glass beads and Kaolin clay, respectively. Oh and Vanapalli [5] reanalyzed the results and back-calculated the Poisson's ratio, ν with respect to degree of saturation. The analyses of the results showed that ν is not constant but varies with the degree of saturation as shown in Fig. 14(a) and (b).

6. RE-ANALYSES OF FOOTING LOAD TEST RESULTS FROM BRIAUD AND GIBBENS [29]

The site selected for the in-situ footing load tests was predominantly sand (mostly medium dense silty sand) from 0 to 11m overlain by hard clay layer (Fig. 15). The ground water table was observed at a depth of 4.9m and the soil above the ground water table was in a state of unsaturated condition. In this case, different footing sizes may result in different average matric suction values. In other words, scale effect cannot be eliminated with normalized settlement since the soils at the site are not "homogenous and isotropic". Despite this fact, as can be seen in Fig. 4, the *SVS* behaviors from different sizes of footing fall in a narrow range. This behavior can be explained by investigating the variation of matric suction with depth at the site as follows.

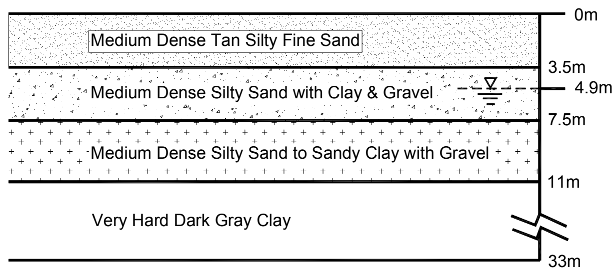


Fig. 15. The average soil profile at the test site [27].

Fig. 16 shows the grain size distribution curves for the soil samples collected from three different depths (i.e., 1.4m - 1.8m, 3.5m - 4.0m, and 4.6m - 5.0m). The grain size distribution curve for the Sollerod sand shown in Fig. 16 is similar to the sand sample collected at the depth of 1.4m to 1.8m. The reasons associated with showing the grain size distribution curve of Sollerod sand will be discussed later in the paper.

The properties of the soils collected from the depths, 0.6m and 3.0m are summarized in Table 1.

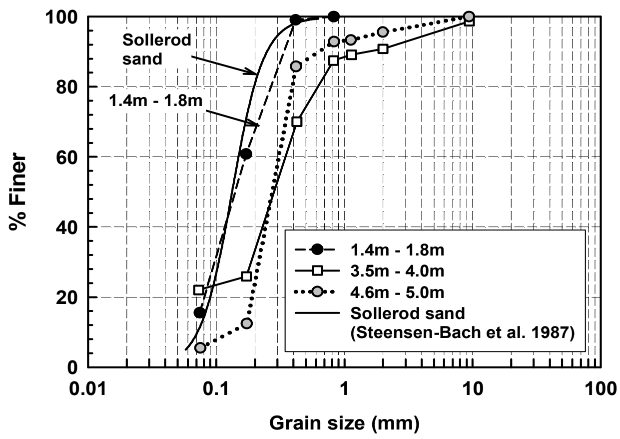


Fig. 16. Grain size distribution curves for the soil samples collected from three different depths [27] and Sollerod sand [7].

Table 1. Summary of the soil properties [27].

Property	Sand (0.6m)	Sand (3.0m)
Specific gravity, G_s	2.64	2.66
Water content, w (%)	5.0	5.0
Void ratio, e	0.78	0.75
Effective cohesion, c' (kPa)	0	0
Effective internal friction angle, ϕ' (°)	35.5	34.2

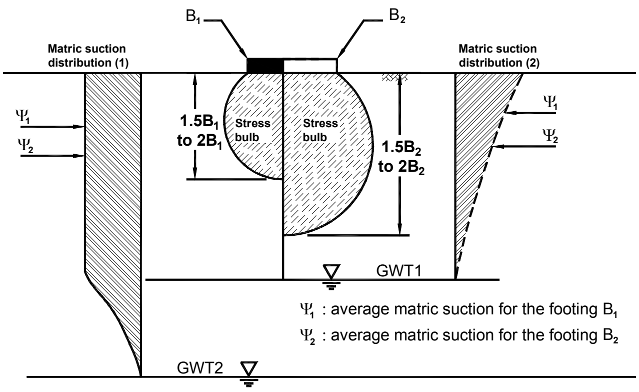


Fig. 17. Average matric suction for different sizes of footing under idealized uniform and non-uniform matric suction distribution.

As shown in Table 1, the water content at the depths of 0.6m and 3.0m is 5%. This implies that the matric suction value can be assumed to be constant up to the depth of approximately 3.0m. The field matric suction distribution profile is consistent with the typical matric suction distribution profile above ground water table for the coarse-grained soils. In other words, matric suction increases gradually (which is close to hydrostatic conditions) up to residual matric suction value and thereafter remains close to constant conditions (i.e., matric suction distribution (1) in Fig. 17). This matric suction distribution profile resulted in the same average matric suction value regardless of footing size (for this study). However, it also should be noted that the average matric suction value for each footing can be different if a non-uniform matric suction distribution profile is available below the footings (i.e., matric suction distribution (2) in Fig. 17).

7. VARIATION OF *SVS* BEHAVIORS WITH RESPECT TO MATRIC SUCTION

After construction of shallow foundations, the soils below them typically experience wetting – drying cycles due to the reasons mostly associated with the environmental conditions (i.e., rain infiltration or evaporation). Hence, it is also important to estimate the variation of the *SVS* behaviors with respect to matric suction.

Oh and Vanapalli [4],[5] conducted finite element analysis (*FEA*) using the commercial finite element software, SIGMA/W (Ver. 2007; [47]) to simulate *SVS* behavior of in-situ footing ($B \times L$ = 1m × 1m) load test results ([27]; Fig. 4) on unsaturated sandy soils. The *FEA* was performed using elastic-perfectly plastic model [48] extending the approach proposed by Oh and Vanapalli [3]. The square footing was modeled as a circular footing with an equivalent area (i.e., 1.13m in diameter; axisymmetric problem).

The Soil-Water Characteristic Curve (*SWCC*) can be used as a tool to estimate the variation of initial tangent elastic modulus, E_i [37] and total cohesion, c with respect to matric suction [49] using (4) and (7), respectively.

$$c = c' + (u_a - u_w)(S^{\kappa})\tan\phi' \tag{7}$$

where c = total cohesion, c' and ϕ' = effective cohesion and internal friction angle for saturated condition, respectively, $(u_a - u_w)$ = matric suction, S = degree of saturation, and κ = fitting parameter (κ = 1 for sandy soils with plasticity index, I_p = 0 %); [50]).

Information on the *SWCC* was not available in the literature for the site where the in-situ footing load test was carried out. Hence, the *SWCC* for the Sollerod sand shown in Fig. 18 was used for the analysis as an alternative based on the following justifications.

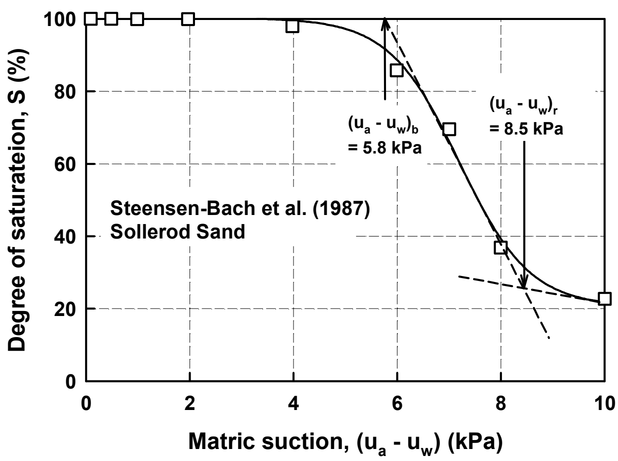
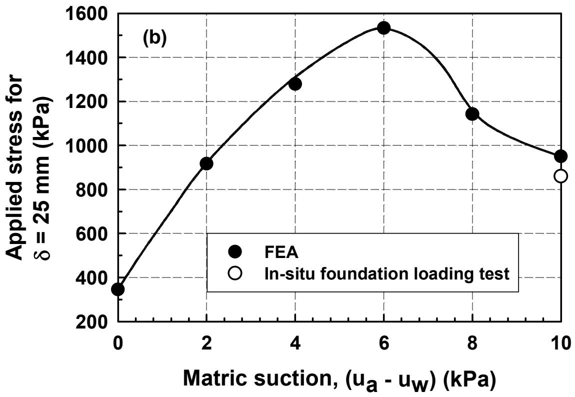


Fig. 18. *SWCC* used for the analysis (data from [7]).

Among the grain size distribution curves shown in Fig. 16, the grain size distribution curve for the range of depth, 1.4m - 1.8m can be chosen as a representative grain size distribution curve since the stress below the footing, 1m × 1m is predominant in the range of 0 to 1.5m - 2m (i.e., 1.5B - 2B) below the footing. This grain size distribution curve is similar to that of Sollerod sand (see Fig. 16) used by Steensen-Bach et al. [7] to conduct model footing tests in a sand to understand influence of matric suction on the load carrying capacity. In addition, the shear strength parameters for the Sollerod sand (c' = 0.8 kPa and ϕ' = 35.8°) are also similar to those of the sand where the in-situ footing load tests were conducted (see Table 1). The influence of wetting – drying cycles (i.e., hysteresis) and external stresses on the *SWCC* is not taken into account in the analysis due to the limited information.

The variation of *SVS* behavior with respect to matric suction from the *FEA* is shown in Fig. 19.



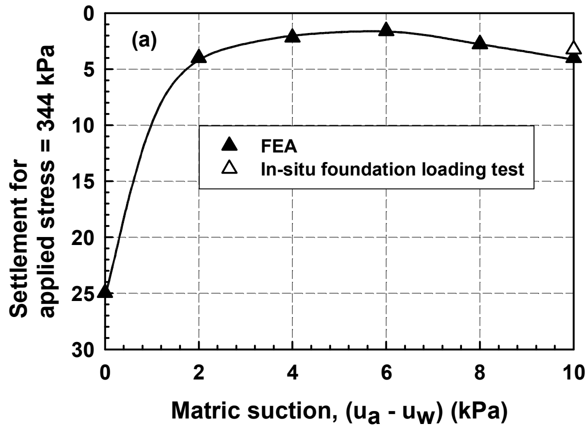


Fig. 20(a) and (b) shows the variation of settlement under the same stress of 344 kPa and the variation of stress that can cause 25mm settlement for different matric suction values, respectively. The stress 344 kPa is chosen since the settlement for saturated condition at this stress is 25mm. The settlement at the matric suction of 10 kPa (i.e., field condition) is approximately 4 mm and then increases up to 25mm (i.e., permissible settlement) as the soil approaches saturated conditions under the constant stress (i.e., 344 kPa). The permissible settlement, 25mm can be induced at 2.7 times less stress as the soil approaches saturated conditions (i.e., from 10 kPa to 0 kPa). The results imply that settlements can increase due to decrease in matric suction. It is also of interest to note that such a problem can be alleviated if the matric suction of the soil is maintained 2 kPa value.

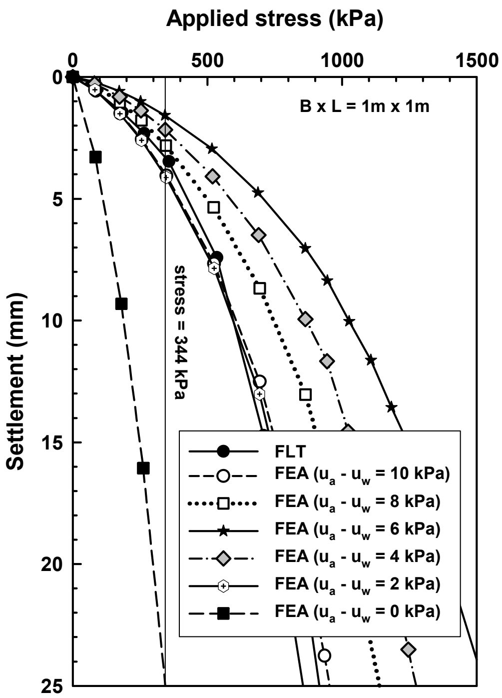


Fig. 19. Variation of *SVS* behavior with respect to matric suction.

8. SUMMARY AND CONCLUSIONS

The focus of the present study is to better understand the scale effects of plate load test (*PLT*) results and suggest guidelines that can be used in practice for their rational interpretation. *PLT* is regarded as the most reliable testing method to estimate the applied stress versus surface settlement (*SVS*) behavior of shallow foundations. However, there are uncertainties in interpreting the *PLT* results for soils that are in a state of unsaturated condition. This is mainly attributed to the fact that the *SVS* behavior from the *PLT*s is significantly influenced by both the footing size and the capillary stresses (i.e., matric suction). Previous studies showed that the scale effect can be eliminated by normalizing settlement with footing size. This methodology is applicable to the soils that are homogeneous and isotropic with depth in nature such as saturated or dry soils. In case of unsaturated soils, matric suction distribution profile with depth should be taken into account to judge whether or not this methodology is applicable. This is because if the matric suction distribution profile is non-uniform with depth different plate sizes lead to different average matric suction values. In other words, the soil below the plates cannot be regarded as homogeneous and isotropic since strength, initial tangent elastic modulus, and the Poisson's ratio, ν are function of matric suction. These facts indicate that the reliable design of shallow foundations based on the *PLT* results can be obtained only when the results are interpreted taking account of the matric suction distribution profile with depth and influence of average matric suction value on the *SVS* behavior.

In case of the shallow foundations rested on unsaturated sandy soils it is also important to estimate the variation of *SVS* behavior with respect to matric suction. This can be achieved by conducting the *FEA* using the methodology presented in this paper. Based on the *FEA* for the in-situ footing (1m × 1m) load test results discussed in the paper [28], unexpected problems associated with settlement are likely due to decrease in matric suction. Such a problem can be alleviated if the matric suction of the soil is maintained at a low of 2 kPa.

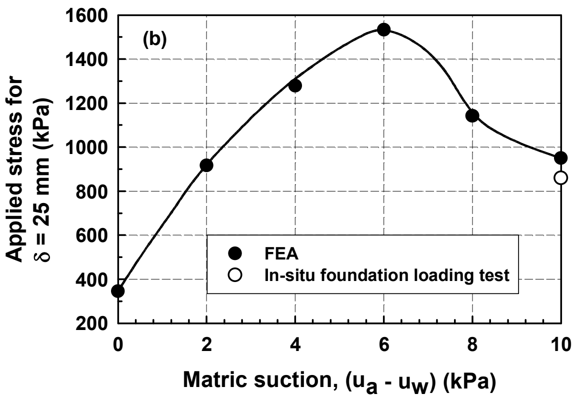


Fig. 20. Variation of (a) settlement under the applied stress of 344 kPa and (b) stress that can cause 25mm settlement with respect to matric suction.

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