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Modeling the stress versus settlement behavior of shallow foundations in unsaturated cohesive soils extending the modified total stress approach

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Abstract

The mechanical behavior of unsaturated soils can be interpreted using either modified total stress or a modified effective stress approach depending on the type of soils and various scenarios of drainage conditions of pore-water and pore-air. Recent studies suggest that the bearing capacity of unsaturated cohesive soils can be more reliably estimated using the modified total stress approach (MTSA) rather than the modified effective stress approach (MESA). In the present study, a modeling technique (extending Finite Element Analysis, FEA) is proposed to estimate the bearing capacity of shallow foundations in unsaturated cohesive soils by simulating the vertical stress versus surface settlement behaviors of shallow foundations extending the MTSA. The proposed technique is verified with the model footing test results in unsaturated cohesive soils. Commercial finite element software, SIGMA/W (GeoStudio 2012, Geo-Slope Int. Ltd.) is used for this study. Details of estimating the unsaturated soil parameters (i.e. total cohesion, modulus of elasticity and Poisson's ratio) required for the FEA are also presented taking account of the influence of matric suction. Good agreements were observed between the measured bearing capacity values and those from the FEA extending the MTSA.

Keywords: Unsaturated soil; Finite element analysis; Stress versus settlement; Shallow foundation; Modulus of elasticity; Poisson's ratio (**IGC**: E3/E13/E14)

1. Introduction

The bearing capacity of saturated soils can be estimated by extending either the effective stress approach (ESA; Terzaghi, 1943) or the total stress approach (TSA; Skempton, 1948). Criterion for determining appropriate approach between the ESA and the TSA are based on the soil type and drainage condition of pore-water during the loading stages. Shallow spread footings are commonly used as foundations of light structures. In many cases (especially in arid and semi-arid regions), the water table is relatively deep, and the pressure bulb typically lies within the vadose zone where soils are in unsaturated condition with negative pore-water pressure. Soil desaturation associated with lowering the natural ground water level or water evaporation from the soil surface results in an increase in the bearing capacity compared to the saturated soil condition. Various research related to the bearing capacity of unsaturated soils suggest that this increase can be attributed to the influence of soil suction (Broms, 1964; Steensen-Bach et al., 1987; Fredlund and Rahardjo, 1993; Oloo et al., 1997; Costa et al., 2003; Rojas et al., 2007; Balzano et al., 2012). Nevertheless, there are still uncertainties

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as to which approach (i.e. ESA or TSA) is more appropriate in the reliable estimation of the bearing capacity of unsaturated soils. Vanapalli and Mohamed (2007) and Schanz et al. (2011) showed that the bearing capacity of unsaturated sandy soils can be reliably estimated extending the ESA taking account of the influence of matric suction. Oh and Vanapalli (2013a) conducted model footing tests in unsaturated glacial till (i.e. Indian Head till) and concluded that the TSA based on the unconfined compressive strength of unsaturated soils can provide a more reasonable bearing capacity of unsaturated cohesive soils. These research studies related to the bearing capacity of unsaturated cohesionless and cohesive soils indicate that the bearing capacity of unsaturated soils should be estimated considering the type of soil and the drainage conditions of pore-water and pore-air. The ESA and the TSA for unsaturated soils are designated as the modified effective stress approach (MESA) and the modified total stress approach (MTSA), respectively.

In-situ plate load test (PLT) is one of the most reliable testing techniques for estimating the bearing capacity of shallow foundations. In-situ PLTs are typically carried out on soils that are in a state of unsaturated condition; hence, there are uncertainties in analyzing the in-situ PLT results in terms of scale effect and matric suction distribution profile with respect to depth (Oh and Vanapalli, 2013b). For this reason, numerical analysis is commonly used as an alternative to the in-situ PLTs to estimate the bearing capacity of unsaturated soils. The results from numerical analyses provide technically acceptable solutions for soil-structure stress and deformation characteristics below shallow foundations (Hanna, 1987; Consoli et al., 1998; Bose and Das, 1997; Lee and Salgado, 2002; Edwards et al., 2005; Hjiaj et al., 2005; Osman and Bolton, 2005). Limited studies were undertaken to predict the variation of bearing capacity of unsaturated soils with respect to matric suction by simulating the stress versus settlement (hereafter referred to as SVS) behaviors of shallow foundations in unsaturated soils using the Finite Element Analysis (FEA) (Abed and Vermeer, 2004; Oh and Vanapalli, 2011a; Le et al., 2013). Various constitutive models are available in the literature to simulate the deformation characteristics of unsaturated soils (Alonso et al., 1990; Kohgo et al., 1993; Wheeler and Sivakumar, 1995; Cui and Delage, 1996; Karube, 1997; Sun et al., 2000; Gallipoli et al., 2003). These models are comprehensive and can model many scenarios of different geotechnical problems extending the principles of unsaturated soil mechanics. The mathematical framework proposed by Borja (2004) to estimate deformation and strain localization in unsaturated soils considering both drained and undrained conditions is a notable contribution in this direction. However, these constitutive models require various soil parameters and determination of these parameters from experimental investigations is rather difficult, time consuming and cumbersome (D'Onza et al., 2015).

Oh and Vanapalli (2011a) proposed a simple numerical modeling technique for engineering practice applications to simulate the SVS behaviors of shallow foundations for unsaturated cohesionless soils based on the MESA. Two main parameters are required for extending this simple technique; namely, total cohesion and elastic modulus. In the present study, this numerical modeling technique is extended to simulate the SVS behaviors of a model footing in an unsaturated cohesive soil based on the MTSA. The commercial finite element software, SIGMA/W (GeoStudio 2012, Geo-Slope Int. Ltd.) was used for the FEA. Comparisons are made between the measured SVS behaviors and bearing capacity values and those estimated using the MESA, MTSA, and FEA.

2. Background

2.1. Estimation of bearing capacity of unsaturated soils using the MESA

In unsaturated cohesionless soils, (i) both the pore-air and pore-water in soils are in drained condition during the loading stages; and, (ii) general failure mode is expected for relatively high-density soils. These two conditions justify the use of MESA in estimating the bearing capacity of unsaturated cohesionless soils using the effective shear strength parameters (c', ϕ', and ϕ^b). Oloo (1994) proposed a method that can be used to design unpaved roads considering the influence of matric suction on the bearing capacity of pavement structures [Eq. (1)].

$$q_{ult(unsat)} = [c' + (u_a - u_w)\tan\phi^b]N_c\xi_c + q_0N_q\xi_q + 0.5B\gamma N_\gamma\xi_\gamma \quad (1)$$

where $q_{ult(unsat)}$ = ultimate bearing capacity of an unsaturated soil, c' = effective cohesion, ϕ' = effective internal friction angle, ϕ^b = internal friction angle due to the contribution of matric suction, $(u_a - u_w)$ = matric suction, γ = soil unit weight, q_0 = overburden pressure, B = width of footing, N_c, N_q, N_γ = bearing capacity factors, and ξ_c, ξ_q, ξ_γ= shape factors.

Vanapalli and Mohamed (2007) further improved Eq. (1) to estimate the ultimate bearing capacity of surface footings in unsaturated soils by taking account of nonlinear variation of shear strength with respect to suction (Vanapalli et al., 1996) [Eq. (2)].

$$q_{ult(unsat)} = \begin{bmatrix} c' + (u_a - u_w)_b(1 - S^{(\psi_{BC})}\tan\phi') \\ +(u_a - u_w)_{AVR}S^{(\psi_{BC})}\tan\phi' \end{bmatrix} N_c\xi_c + 0.5B\gamma N_\gamma\xi_\gamma \quad (2)$$

where $(u_a - u_w)_b$ = air-entry value, $(u_a - u_w)_{AVR}$ = average matric suction, S = degree of saturation, ψ_{BC} = fitting parameter for bearing capacity, N_c, N_q = bearing capacity factors from Terzaghi (1943), N_γ = bearing capacity factor from Kumbhokjar (1993), and ξ_c, ξ_q, ξ_γ = shape factors (Vésic, 1973).

Based on the research by Oloo [1994, Eq. (1)] and Vanapalli and Mohamed [2007, Eq. (2)], Vahedifard and Robinson (2016) proposed unified method to estimate the ultimate bearing capacity of unsaturated soils in variably saturated soils under steady flow [Eq. (3)].

$$q_{ult(unsat)} = \begin{bmatrix} c' + (u_a - u_w)_b(1 - S_{e,AVR}) \tan \phi' \\ +\{(u_a - u_w)S_e\}_{AVR} \tan \phi' \end{bmatrix} N_c \xi_c + q_0 N_q \xi_q + 0.5 B \gamma N_\gamma \xi_\gamma \quad (3)$$

where S_e = effective degree of saturation [= $(S - S_r)/(1 - S_r)$], S_r = residual saturation, $S_{e,AVR}$ = average effective degree of saturation.

2.2. Estimation of bearing capacity of unsaturated soils using MTSA

The Terzaghi (1943) bearing capacity equation is commonly recommended for soils that exhibit dilatancy, which leads to a well-defined failure surface (i.e. general shear failure conditions) (Yamamoto et al., 2008). However, this behavior is not typically observed for the in-situ PLTs in unsaturated cohesive soils (Larson, 1997; Schnaid et al., 1995; Consoli et al., 1998; Costa et al., 2003; Rojas et al., 2007). Consoli et al. (1998) conducted in-situ PLTs to determine the bearing capacity of an unsaturated cohesive soil (CL). The bearing capacity values were overestimated when they were calculated using the full-strength parameters; however, good agreement was observed when calculated using values 2/3 those of the full-strength parameters. This complies with Terzaghi's (1943) recommendations for punching shear failure. The model footing [Oh and Vanapalli, 2013a, B × L = 0.05 m × 0.05 m, Fig. 1 (a)] and plate load [Larsson, 1997, B × L = 0.5 m × 0.5 m, Fig. 1(b)] tests in unsaturated cohesive soils also showed no signs of heave or settlements outside the footing and plate. These results manifest that bearing capacity of in-situ PLT results in unsaturated cohesive soils are governed by punching shear failure mechanism (Larson, 2001).

Finnie and Randolph (1994) conducted physical model tests to investigate the bearing response of surface foundation on uncemented calcareous sand. The centrifuge tests that simulated five different diameters of circular foundations (i.e. D = 1, 3, 5, 7, and 10 m) showed that average SVS relationships can be linear and are insensitive to the footing diameter for highly compressible soils. Similar linear load versus settlement behavior from in-situ PLT (B × L = 2 m × 2 m) was reported by Larsson (1997). Yamamoto et al. (2008) suggested that, for punching shear failure, rigid-plastic-based method cannot be used to explain response of shallow foundations. For such a scenario, a deformation pattern within a bulb of compressed material below shallow foundations should be considered to reliably estimate SVS behaviors. From these observations, it can be postulated that the bearing capacity of shallow foundations in unsaturated cohesive soils is governed by the compressibility of the soil underneath.

For unsaturated cohesive soils, a reasonable assumption can be made that pore-air pressure is the atmospheric pressure (i.e. drained condition) and pore-water is under the undrained condition (i.e. constant water content) throughout the loading stage. This pore-air and water content condition during loading stages can be reliably simulated by conducting Constant Water content (CW) tests (Rahardjo et al., 2004; Infante Sedano et al., 2007). Tang et al. (2016) also showed that in-situ PLTs are typically carried out under the constant water content condition and χ $(u_a - u_w)$ (where χ is effective stress parameter, $(u_a - u_w)$ is matric suction, u_a = pore-air pressure, u_w = pore-water pressure) can be assumed to be constant in the interpretation of results without introducing significant error. The CW test is, however, time-consuming and requires elaborate testing equipment. For this reason, conventional unconfined compression tests for unsaturated cohesive soils are recommended instead of CW test based on the following assumptions for justification; (i) the drainage condition in unconfined compression test for unsaturated cohesive soils is the same as the CW test and (ii) the shear strength

(a) (b)

Fig. 1. (a) Indentation from a model footing test in an unsaturated cohesive soil (Oh and Vanapalli, 2013a, B × L = 0.05 m × 0.05 m) and (b) pattern of cracks in the ground around the plate with no observable heave (Larsson, 1997, B × L = 0.5 m × 0.5 m).

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obtained from the unconfined compression test provides conservative estimates since the contribution of confining pressure towards shear strength is neglected. Extending this approach, Oh and Vanapalli (2013a) suggested that the bearing capacity of a surface footing on an unsaturated cohesive soil can be estimated using the unconfined compressive strength of a soil block (i.e. A-A′-B-B′ in Fig. 2) below the footing [Eq. (4)].

$$q_{ult(unsat)} = [c_{u(unsat)}]\left[1 + 0.2\left(\frac{B}{L}\right)\right]N_{c(unsat)} = \left[\frac{q_{u(unsat)}}{2}\right]\left[1 + 0.2\left(\frac{B}{L}\right)\right]N_{c(unsat)} \tag{4}$$

where $q_{ult(unsat)}$ = the ultimate bearing capacity of unsaturated cohesive soil, $c_{u(unsat)}$ = the undrained shear strength of unsaturated cohesive soil, $q_{u(unsat)}$ = the unconfined compressive strength of unsaturated cohesive soil, $N_{c(unsat)}$ = the N_c factor for unsaturated soil, B, L = the width and length of a foundation, respectively, [1 + 0.2 (B/L)] = the shape factor proposed by Meyerhof (1963) and Vesić (1973) for the undrained condition (1.2 for a square foundation).

According to Oh and Vanapalli (2013a), the average of back-calculated $N_{c(unsat)}$ value is 4.3, which leads to 5.16 (=1.2 × 4.3) times $c_{u(unsat)}$ based on the unconfined compressive strength of an unsaturated cohesive soil under a square footing (i.e. B/L = 1). Eq. (4) takes the same form as the bearing capacity of saturated soils under the undrained condition extending the $\phi_u = 0$ approach (Skempton, 1948). However, from a theoretical point of view, the $\phi_u = 0$ approach cannot be applied to unsaturated soils since unsaturated soils experience volume change due to the compressibility of an air-water mixture during shearing even under undrained conditions. Nonetheless, reasonable agreements were observed between the measured bearing capacity values and those estimated using Eq. (4). This is mainly because the undrained shear strength of unsaturated soils based on unconfined compression test can be mathematically written using the extended Mohr-Coulomb failure envelope [Fig. 3, Eq. (5), Fredlund and Rahardjo, 1993). Hence, in the case of surface footing, total cohesion can be replaced with half of the unconfined compressive strength of an unsaturated soil sample.

$$c_{u(unsat)} \approx c' + (\sigma_f - u_a)\tan\phi' + (u_a - u_w)_f \tan\phi^b \tag{5}$$

where $c_{u(unsat)}$ = the undrained shear strength of unsaturated soil, $(\sigma_f - u_a)$ = the net normal stress at failure, and $(u_a - u_w)_f$ = the matric suction at failure.

Meyerhof (1974) investigated the bearing capacity of continuous foundations on a dense sand (stronger) layer above soft clay (weaker) layer (Fig. 4a) considering the failure as an inverted uplift problem under axial load (Meyerhof, 1973). For relatively small H/B ratios, punching shear failure takes place in the top layer, followed by general shear failure in the bottom layer. The ultimate bearing capacity of a foundation can be estimated using Eq. (6) by taking the sum of total cohesion and total passive force as equivalent to the forces on the punching failure surfaces.

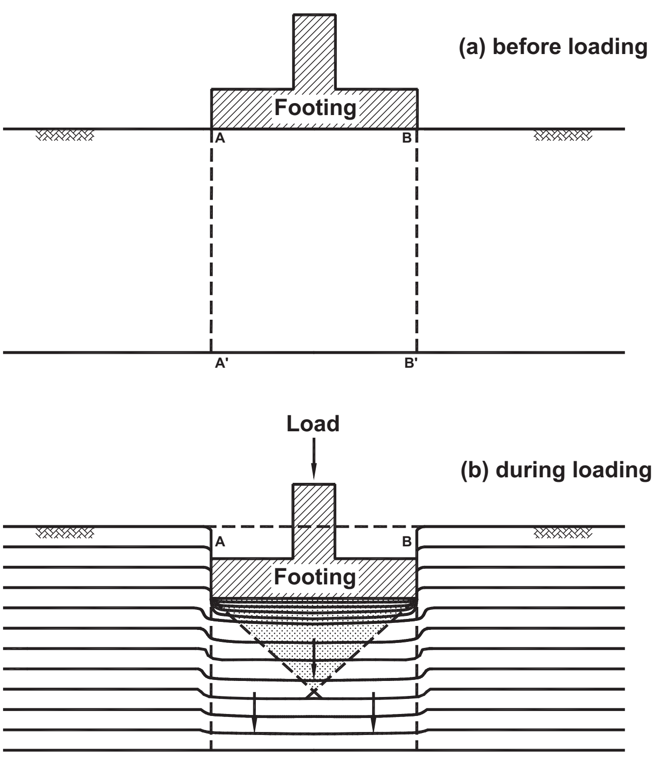

Fig. 2. Punching shear failure mechanism in unsaturated fine-grained soils below a footing used by Oh and Vanapalli (2013a).

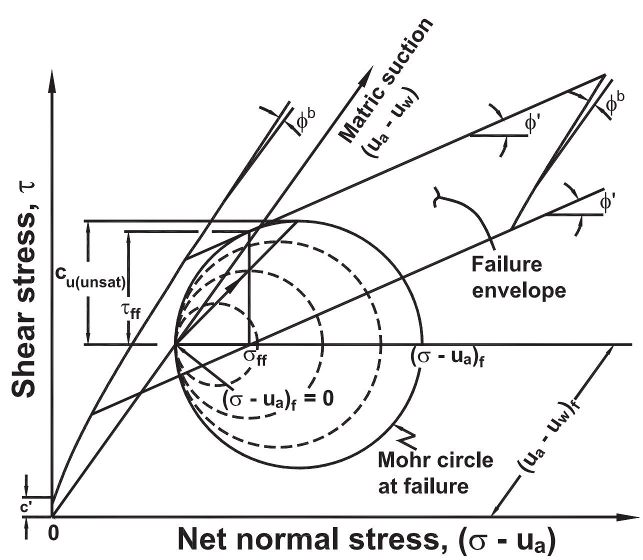

Fig. 3. Typical stress path followed during unconfined compression test on unsaturated soil with soil suction (modified after Fredlund and Rahardjo, 1993).

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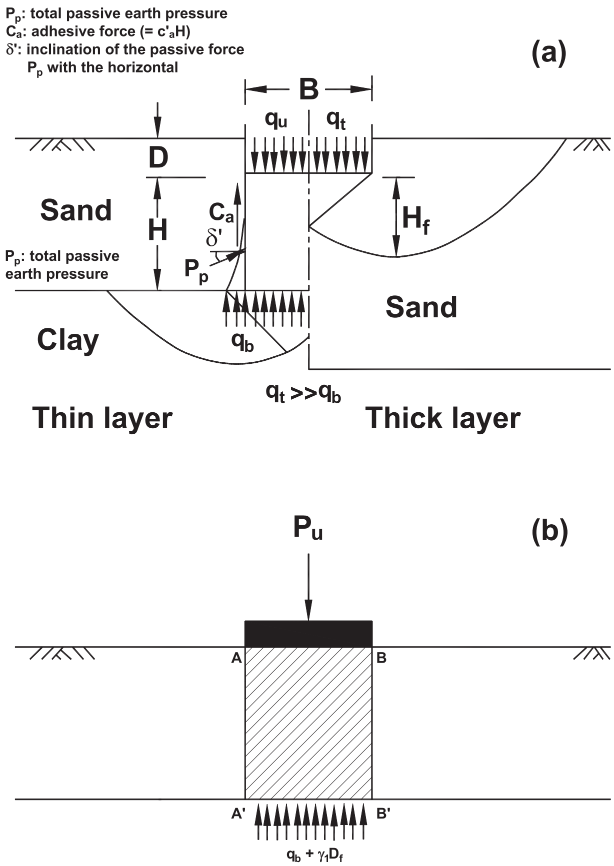

Fig. 4. Bearing capacity of a continuous foundation on layered soil (after Meyerhof, 1974).

$$q_{ult} = q_b + \frac{2(C_a + P_p \sin\delta')}{B} - \gamma_1 H = q_b + \frac{2c'_a H}{B} + \frac{P_p \sin\delta'}{B} - \gamma_1 H \tag{6}$$

Eq. (6) can be simplified as the form in Eq. (7) for a rectangular foundation.

$$q_{ult} = q_b + \left(\frac{2c'_a H}{B}\right)\left(1+\frac{B}{L}\right) + \gamma_1 H^2\left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s \tan\phi'_1}{B}\right) - \gamma_1 H \leqslant q_t \tag{7}$$

where, q_b = the bearing capacity of the bottom soil layer, q_t = the bearing capacity of the top soil layer, C_a = adhesive force (=c'_a H), c'_a = the adhesive cohesion, H = the distance from the base of the shallow foundation to the bottom of the lower layer, B = the width of the shallow foundation, L = the length of the shallow foundation, (1 + B/L) = the shape factor, D_f = the embedded depth, K_s = the punching shear coefficient, ϕ'_1 = the effective internal friction angle of the top layer, γ_1 = the unit weight of the top layer.

If the top layer is a stronger saturated clay and the bottom layer is a weaker saturated clay, Eq. (7) can be modified as Eq. (8) assuming that (i) the bearing pressure at the bottom clay surface is $q_b + \gamma_1 D_f$ and (ii) the bearing capacity of upper clay layer is governed only by the compressibility of the soil block A-A′-B-B′ (i.e. P_p in Eq. (6) or the third term including K_s in Eq. (7) = 0) (Fig. 4(b))

$$q_{ult} = q_b + \left(\frac{2c_a H}{B}\right)\left(1+\frac{B}{L}\right) + \gamma_1 D_f \tag{8}$$

where c_a = total cohesion.

Eq. (8) can be extended to the bearing capacity of unsaturated cohesive soils. Larson (1997) described from the in-situ PLTs (B × L = 2 m × 2 m) in the unsaturated cohesive soil that all significant settlements occur within a depth of 2B below the plate, with about 87% of settlements within a depth of 1B. Hence, if it is assumed that H in Fig. 4(a) is 1.5B (i.e. H/B = 1.5) and stress that is transferred into the lower layer is negligible (i.e. $q_b + \gamma_1 D_f = 0$) the bearing capacity of a surface square footing ($D_f = 0$) on an unsaturated cohesive soil can be calculated using Eq. (9).

$$q_{ult(unsat)} = \left(\frac{2c_a H}{B}\right)\left(1+\frac{B}{L}\right) = 2c_a\left(\frac{H}{B}\right)\left(1+\frac{B}{L}\right) = 6c_{u(unsat)} \tag{9}$$

It is interesting to note that the average of two factors {i.e. 5.58 = (5.16 [Eq. (4)] + 6 [Eq. (9)])/2} is close to 6.17 (= 5.14 × 1.2): this is commonly used for estimating the bearing capacity of a square foundation in saturated soil under undrained condition (10% less than the conventional value).

2.3. Estimation of stress versus settlement of shallow foundations in unsaturated cohesionless soils using finite element method

Abed and Vermeer (2004) studied the SVS behaviors of a footing using the Barcelona Basic Model (BBM) (for unsaturated condition) and Modified Cam Clay (MCC) model (for saturated condition). Good agreements were observed between the theoretically calculated bearing capacity values and those from the numerical analysis. The numerical analyses were carried out extending the MESA. Ghorbani et al. (2016) studied the load-displacement curves for statically loaded rigid footings in both fully saturated and unsaturated soils using an extended MCC. Tang et al. (2016) investigated bearing capacity of shallow foundations in unsaturated soils using elastic-perfectly plastic Mohr-Coulomb model. Hydraulic hysteresis (i.e. wetting, drying and scanning) and three different drainage conditions (i.e. constant suction, constant moisture content, and constant distribution of suction) were considered in the analyses (i.e. flow-deformation analysis). The results showed that the bearing capacity following the drying path in comparison to wetting path is approximately twice the value. They also concluded

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that the differences in bearing capacity values obtained from elastic-perfectly plastic Mohr-Coulomb model and complex models (Baker, 2004; Serrano et al., 2005) are negligible. Oh and Vanapalli (2011a) proposed a technique to simulate SVS behaviors of shallow foundations in unsaturated cohesionless soils using Finite Element Analysis (FEA). The analyses were conducted for model footing (100 mm × 100 mm) tests in an unsaturated sand (Mohamed and Vanapalli, 2006) using elastic – perfectly plastic Mohr-Coulomb model extending the MESA. In the present study, the approach proposed by Oh and Vanapalli (2011a) is extended to simulate SVS behaviors of model footing tests in an unsaturated cohesive soil extending the MTSA.

3. Numerical modeling of stress versus settlement behavior of a model footing

3.1. Model footing test results used in the study

In the present study, laboratory model footing (B × L × H = 50 mm × 50 mm × 50 mm) test results in an unsaturated cohesive soil were used for the FEA. The tests were conducted on the glacial till (Indian Head till; $W_L = 32.5\%$, $W_P = 17.0\%$, $I_p = 15.5\%$, $G_s = 2.72$) that was statically compacted in a specially designed soil tank (Dia. = 300 mm and Height = 300 mm) by applying the compaction stress of 350 kPa. Matric suction value of the initially compacted soil was 205 kPa at the water content of 13.2%. Five different matric suction distribution profiles with depth (Fig. 5) were achieved by following three different procedures.

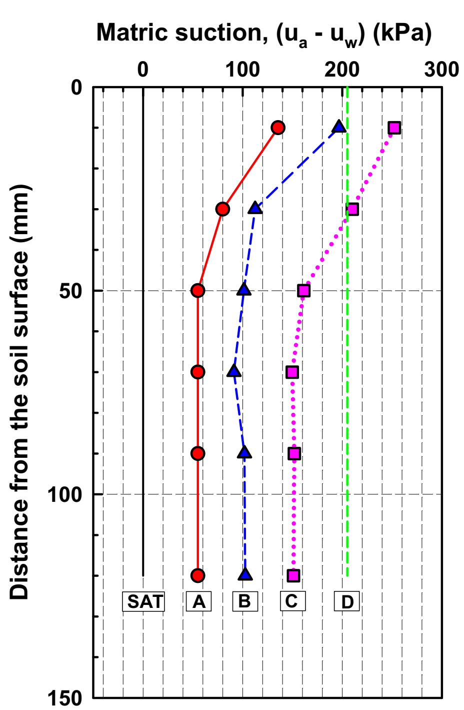

Fig. 5. Variation of matric suction with depth in compacted soils.

a. Compaction (D in Fig. 5; as compacted soil)
b. Compaction – saturation (SAT in Fig. 5)
c. Compacted – saturation – desaturation (air-drying in a moisture controlled box; A, B and C in Fig. 5)

When a targeted matric suction distribution profile with depth was achieved a metal model footing was place on the center of compacted soil and then load was applied on it at a constant rate [1.14 mm/min (0.045 in./min)]. After each model footing test, samples were collected from the compacted soils at the locations outside the stress bulb using stainless-steel thin-wall tubes (50-mm diameter × 120-mm length) for the unconfined compression tests. More details are available in Oh and Vanapalli (2013a).

Since matric suction distribution profiles with depth are not uniform for the cases, A, B, and C, average (or representative) matric suction values, $(u_a - u_w)_{AVG}$ were used to theoretically calculate the bearing capacity values. Average matric suction is defined as the matric suction value corresponding to the centroid of matric suction distribution profile within 1.5B (B is width of a footing) from the base level of the model footing (Fig. 6). This is the zone of depth where the stress increment due to loading is predominant (Poulos and Davids, 1974). This approach was also used by Vahedifard and Robinson (2016) to estimate the ultimate bearing capacity of shallow foundations in unsaturated soils under steady flow for different levels of ground water table. The average matric suction values for the matric suction distribution profiles, A, B, and C were estimated to be 55, 100 and 160 kPa, respectively. The SVS behaviors for five different matric suction distribution profiles are shown in Fig. 7. The intersection of the tangent to the initial and final portions of a SVS behavior within 5 mm settlement (i.e. 10% of the width of model footing) was defined as the ultimate bearing capacity value.

3.2. Estimation of soil properties

3.2.1. Soil-Water Characteristic Curve

The Soil-Water Characteristic Curve (SWCC) of the Indian Head till was measured using a pressure plate apparatus extending the axis-translation technique following the ASTM Standard D6836-12 (2003) (Fig. 8). Best-fit analysis was conducted using Fredlund and Xing (1994) equation and the fitting parameters are shown in the figure.

$$S_e = \left\{ \frac{1}{\ln[e + (\psi/a)^n]} \right\}^m \tag{10}$$

where S_e = effective degree of saturation, e = Euler's number (2.71828), ψ = suction, and a, n, m = fitting parameters.

3.2.2. Total cohesion

Total cohesion, c is defined as the sum of the effective cohesion and the apparent cohesion (i.e. cohesion attributed to the contribution of matric suction). Two models [i.e., Eqs. (11) and (12)] are commonly used to estimate

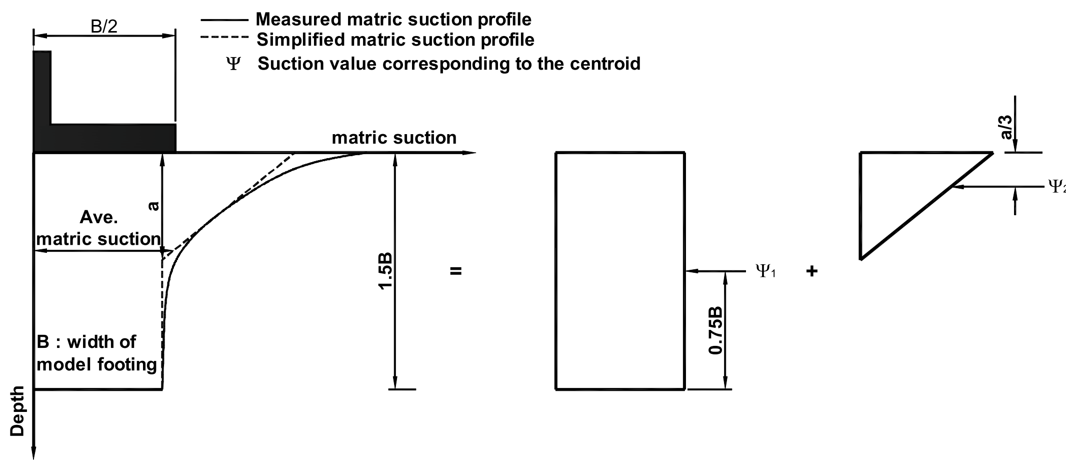

Fig. 6. Estimation of average matric suction value in unsaturated coarse-grained soils (modified after Oh and Vanapalli, 2011a).

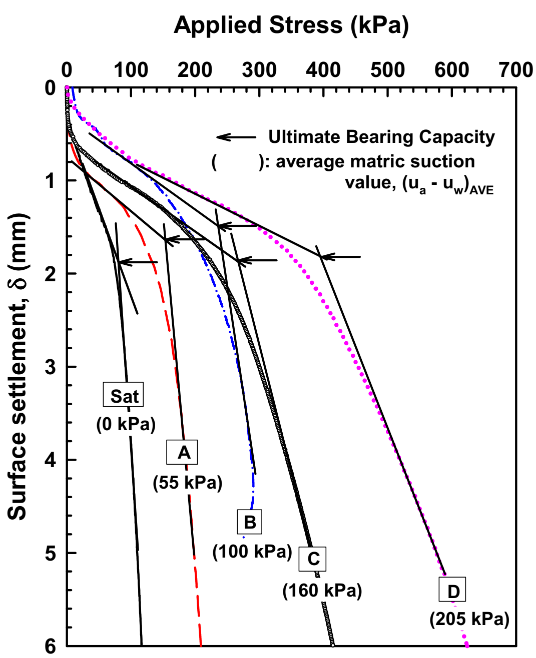

Fig. 7. Model footing test results.

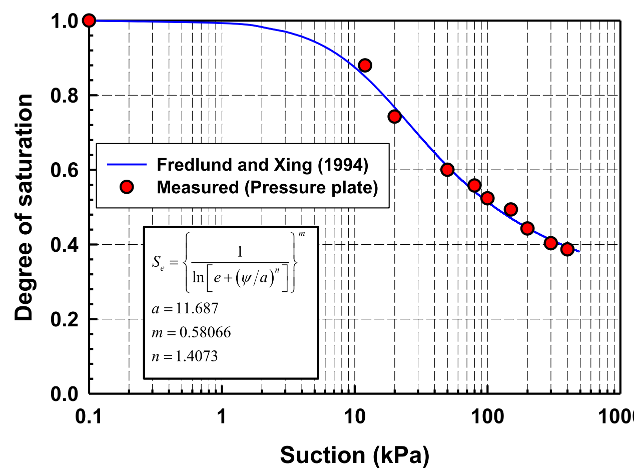

Fig. 8. Soil-water characteristic curve of Indian head till.

the variation of total cohesion with respect to matric suction (Vanapalli et al., 1996; Lu et al., 2009).

$$c = c' + (u_a - u_w)\Theta \tan\phi' = c' + (u_a - u_w)\left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right)\tan\phi' \tag{11}$$

$$c = c' + (u_a - u_w)S_e \tan\phi' = c' + (u_a - u_w)\left(\frac{S - S_r}{1 - S_r}\right)\tan\phi' \tag{12}$$

where Θ = normalized volumetric water content, θ_s, θ_r = volumetric water content for saturated and residual condition, respectively, S_e = effective degree of saturation, and S_r = degree of saturation for residual condition.

As explained using Eq. (4), in the present study, the bearing capacity of unsaturated cohesive soils was estimated extending the MTSA based on the unconfined compressive strength of a soil block below the model footing. For this reason, instead of using Eqs. (11) or (12), which are based on MESA, the variation of undrained shear strength with respect to matric suction was estimated using the model shown in Eq. (13) (Oh and Vanapalli, 2009; Vanapalli and Taylan, 2012). The unconfined compression test results for the specimens (Dia. = 50 mm, Height = 100 mm) collected from each matric suction distribution profile and the variation of undrained shear strength with respect to matric suction are shown in Fig. 9. Table 1 summarizes the measured undrained shear strength and those predicted using Eq. (13).

$$c_{u(unsat)} = c_{u(sat)}\left[1 + \frac{(u_a - u_w)}{(P_a/101.3)}(S^\eta)/\mu\right] \tag{13}$$

where, $c_{u(sat)}$, $c_{u(unsat)}$ = the undrained strength of saturated and unsaturated soil, respectively, $(u_a - u_w)$ = the matric suction, S = degree of saturation, P_a = the atmospheric pressure and η (=2), μ (=10) = the fitting parameters (Oh and Vanapalli, 2009).

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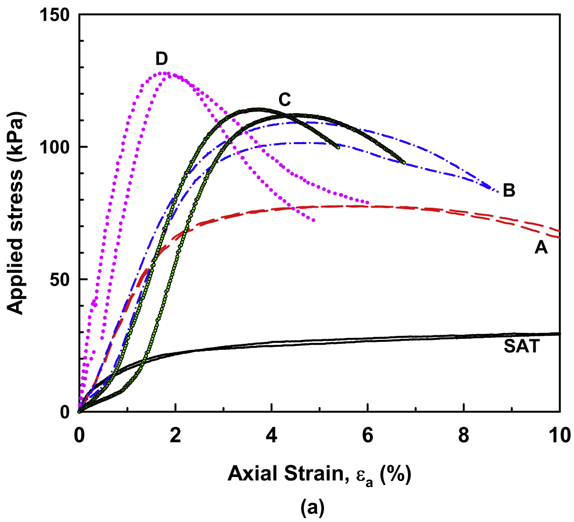

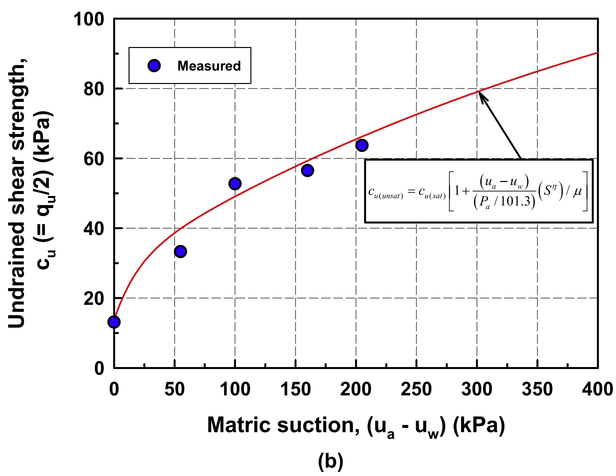

Fig. 9. (a) Applied stress versus axial strain behaviors and (b) variation of undrained shear strength with respect to matric suction from the unconfined compression test results.

3.2.3. Elastic modulus

The modulus of subgrade reaction from PLTs can be defined as Eq. (14). The subgrade reaction can be either be a tangent line slope or secant line slope. The initial tangent subgrade reaction estimated based on the model footing test results were used in the present study for the FEA.

$$k_{is} = \frac{\Delta q}{\Delta \delta} \tag{14}$$

where k_{is} = the initial tangent modulus of subgrade reaction, Δq, $\Delta \delta$ = the increment of contact pressure and corresponding change in settlement in the elastic range, respectively.

The elastic modulus required to analyze the settlement behaviors of a footing can be estimated using Eq. (15) (Timoshenko and Goodier, 1951).

$$E = \frac{(1-\nu^2)}{\Delta\delta/\Delta\sigma}BI = k_{is}(1-\nu^2)BI \tag{15}$$

where E = the elastic modulus, ν = Poisson's ratio, B = the width of a footing, and I = the influence factor (i.e. 0.79 for a circular footing and 0.88 for a square footing).

Oh and Vanapalli (2011a) suggested that SVS behaviors from the FEA can be more reliably estimated when modulus of elasticity is calculated using Eq. (16) instead of Eq. (15) as the influence of B, ν and I are already considered in the FEA as input parameters.

$$E = \frac{1.5B}{\Delta\delta/\Delta\sigma} = \frac{\Delta\sigma}{(\Delta\delta/1.5B)} \tag{16}$$

The variation of elastic modulus with respect to matric suction was estimated using the model shown in Eq. (17) (Fig. 10) (Oh et al., 2009; Wuttke et al., 2013; Adem and Vanapalli, 2013; Qi and Vanapalli, 2015; Han et al., 2016). It should be noted that Eq. (17) has not been validated for the wetting path of SWCC.

$$E_{unsat} = E_{sat}\left[1 + \alpha\frac{(u_a - u_w)}{(P_a/101.3)}S^{\beta}\right] \tag{17}$$

where, E_{sat}, E_{unsat} = the elastic modulus of saturated and unsaturated soil, respectively, and α (= 1/10), β (= 2) = the fitting parameters (Oh and Vanapalli, 2010a,b).

Eq. (17) requires the modulus of elasticity for saturated condition and the SWCC (i.e. matric suction versus degree of saturation) along with two fitting parameters, α and β. Vanapalli and Oh (2010) analyzed model footing tests results in unsaturated both cohesive and cohesionless soils and suggested that β = 1 and 2 can be used for cohesionless and cohesive soils, respectively. They also concluded that α

Table 1
Measured and predicted total cohesion and initial tangent elastic modulus.

$(u_a - u_w)_{AVG}$ (kPa)	S (%)	c_u[a] (kPa)	c_u[b] (kPa)	E_i[c] (kPa)	E_i[d] (kPa)
0	100	13.1	13.1	3516	3516
55	60	33.3	39.0	8857	9032
100	52	52.7	48.9	12,614	12,959
160	49	56.5	57.5	16,668	17,308
205	44	63.7	65.1	19,415	20,186

[a] Measured undrained shear strength [$=q_u/2$].
[b] Predicted undrained shear strength [Eq. (13)].
[c] Measured initial tangent elastic modulus.
[d] Predicted initial tangent elastic modulus [Eq. (17)].

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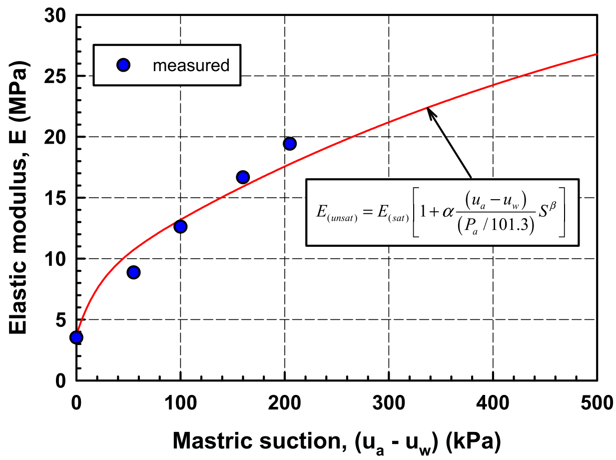

Fig. 10. Variation of initial tangent elastic modulus with respect to matric suction.

is a function of plasticity index, I_p. The measured and predicted initial tangent elastic modulus values are summarized in Table 1. Adem and Vanapalli (2013) proposed a simple approach to estimate heave/shrinkage movements of natural expansive soils with respect to time in terms of the matric suction variations and the corresponding values of the modulus of elasticity (referred to as modulus of elasticity based method, MEBM). Eq. (17) was successfully used to estimate the variation of modulus of elasticity with respect to matric suction in MEBM.

3.2.4. Coefficient of earth pressure at rest

The coefficient of earth pressure at rest for unsaturated soils can be estimated using Eq. (18) in terms of total stress (Fredlund and Rahardjo, 1993).

$$K_0 = \frac{(\sigma_h - u_a)}{(\sigma_v - u_a)} \tag{18}$$

where σ_v = total vertical stress, σ_h = total horizontal stress, u_a = pore-air pressure.

The stress versus strain relationship in the vertical and horizontal directions can be written as Eqs. (19) and (20), respectively assuming homogeneous, isotropic and linear elastic mass.

$$\varepsilon_v = \frac{\sigma_v - u_a}{E} - \frac{2v}{E}(\sigma_h - u_a) + \frac{u_a - u_w}{H_s} \tag{19}$$

$$\varepsilon_h = \frac{\sigma_h - u_a}{E} - \frac{v}{E}(\sigma_v + \sigma_h - 2u_a) + \frac{u_a - u_w}{H_s} \tag{20}$$

where ε_v = normal strain in the vertical direction, ε_h = normal strain in the horizontal direction, E = elastic modulus with respect to a change in $(\sigma - u_a)$, H_s = elastic modulus with respect to a change in $(u_a - u_w)$.

For the at rest (K_0) condition, the net horizontal stress can be written as Eq. (21) by setting $\varepsilon_h = 0$.

$$\sigma_h - u_a = \frac{v}{1-v}(\sigma_v - u_a) - (1-v)\frac{E}{H_s}(u_a - u_w) \tag{21}$$

The coefficient of earth pressure at rest then can be derived as Eq. (22) by normalizing Eq. (21) with $(\sigma_v - u_a)$.

$$K_0 = \frac{(\sigma_h - u_a)}{(\sigma_v - u_a)} = \frac{v}{1-v} - K_m\frac{(u_a - u_w)}{(\sigma_v - u_a)} \tag{22}$$

where $K_m = E/[(1 - v)H_s]$.

Coefficient of earth pressure at rest can also be defined in terms of effective vertical (σ'_v) and horizontal (σ'_h) stresses.

$$K'_0 = \frac{\sigma'_h}{\sigma'_v} \tag{23}$$

Lu and Likos (2006) suggested that Bishop (1959)'s effective stress equation (Eq. (24)) can be written as Eq. (25) using suction stress.

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \tag{24}$$

$$\sigma' = (\sigma - u_a) + \sigma^s \tag{25}$$

where χ = parameter and σ^s = suction stress.

Using Eqs. (23) and (25), a relationship between the total stress K_0 and the effective stress K'_0 can be established as Eq. (26).

$$K_0 = K'_0 - (1 - K'_0)\frac{(u_a - u_w)}{(\sigma_v - u_a)}\sigma^s \tag{26}$$

Eqs. (22) and (26) indicates that K_0 is a function of not only overburden stress, but also matric suction due to the parameter, H_s. Slatter et al. (2005) conducted one-dimensional consolidation tests on unsaturated cohesive soils to measure lateral pressures under controlled suction values. The results showed that lateral stress decreases with an increase in the mass of water lost from sample. For the matric suction and vertical stress values used in their study, K_0 was estimated to be a low value of 0.06. Oh et al. (2013) conducted nonfailure K_0 consolidation and the shear failure tests on residual soils in a suction-controlled triaxial setup. The results show that K_0 in terms of effective stress remains constant, but the K_0 defined by the total stress decreased by a maximum of about 30% as matric suction increases. Vahedifard et al. (2015) investigated the variation in the active earth pressure coefficient, K_a of clayey soil behind a wall for three different flow conditions; namely, no-flow, infiltration, and evaporation. The results showed that evaporation and infiltration cause a decrease and increase in K_a, respectively, when compared with the no-flow condition, which significantly affect the active earth pressure in clay. In the present study, the FEA was carried out for various K_0 values to investigate the influence of K_0 on the SVS behaviors of a model footing in an unsaturated cohesive soil.

3.2.5. Poisson's ratio

Poisson's ratio is commonly assumed to be a constant value in the numerical modeling studies of both saturated and unsaturated soils. This assumption is reasonable for cohesionless soils since the change in mean effective stress

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and associated volume change due to the wetting and drying cycles is negligible. A Poisson's ratio of $\nu = 0.3$ can be used as a typical value for cohesionless soils (Bowles, 2001). Drying and wetting cycles contribute to significant volume changes in cohesive soils, which results in different compressibility characteristics with respect to the degree of saturation. The mathematical relationship between E_{max} (maximum elastic modulus) and the G_{max} (maximum shear modulus) for a homogeneous, isotropic and linear elastic continuum can be expressed as Eq. (27).

$$E_{\max} = 2G_{\max}(1 + \nu) \tag{27}$$

If Eq. (27) is extended for unsaturated soils, it can be rewritten as Eq. (28).

$$E_{\max(unsat)} = 2G_{\max(unsat)}(1 + \nu) \tag{28}$$

Oh and Vanapalli (2010a,b, 2011b) revisited measured E_{max} and G_{max} values for both saturated and unsaturated conditions available in the literature to investigate the variation of Poisson's ratio with respect to degree of saturation. Fig. 11(a) shows the variation of back-calculated ν with respect to degree of saturation for the data provided by Mendoza et al. (2005). Negative values and those greater than 0.5 were ignored since those values are not realistic. A trend line is drawn assuming $\nu = 0.1$ representing the relatively dry condition, which was also supported by Lee and Santamarina (2005) study. Fig. 11(b) shows the variation of back-calculated ν values with respect to degree of saturation for clay-slurry pore provided by Alramahi et al. (2010). For both cases, Poisson's ratio is relatively high for saturated condition and then gradually decreases as soils desaturate. Similar behaviors were also observed for the sand studied by Kumar and Madhusudhan (2012). The results in Fig. 11 suggest that Poisson's ratio can be expressed as a function of degree of saturation [i.e. f(S); Eq. (29)]. It is also interesting to note that the variation of Poisson's ratio with respect to degree of saturation is similar to that of the SWCC behavior. Elastic modulus [Eq. (17)] and small-strain shear modulus models (Oh and Vanapalli, 2014; Dong et al., 2016; Alramahi, 2007) that incorporate the SWCC as a main tool support this statement.

$$\nu^* = \left[\frac{E_{\max(unsat)}}{2G_{\max(unsat)}}\right] - 1 = f(S) \tag{29}$$

where ν^* = Poisson's ratio of unsaturated soils, which is a function of degree of saturation.

Currently, no model is available in the literature to estimate the variation of Poisson's ratio with respect to matric suction. Hence, in the present study, the FEA was undertaken for various Poisson's ratio values (i.e. 0.1, 0.2, 0.3, 0.4, and 0.495).

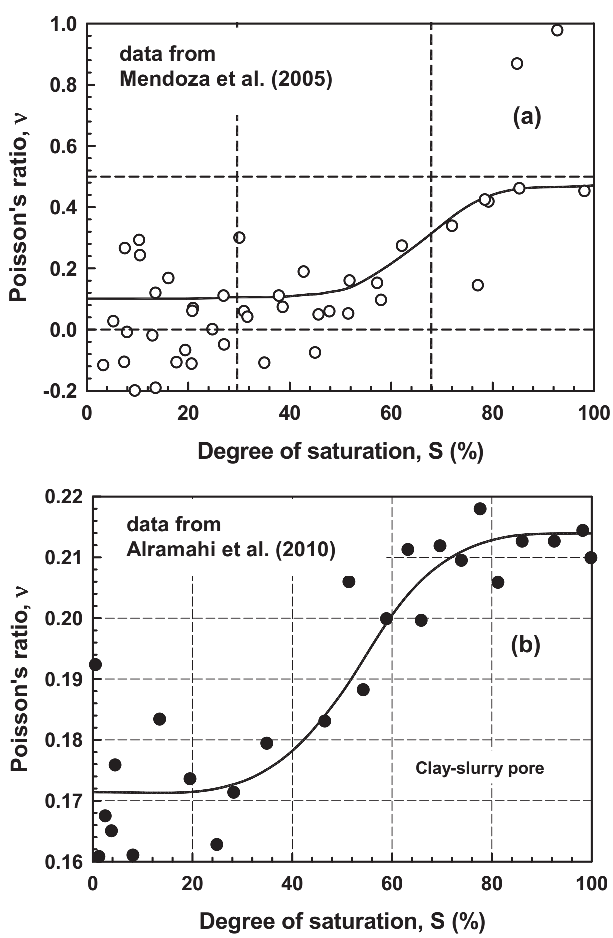

Fig. 11. Variation of Poisson's ratio with respect to degree saturation (modified after Oh and Vanapalli, 2011b).

4. Numerical analyses

Numerical analyses were performed using the commercial finite element software SIGMA/W (ver. 2012; Geo-Slope Int. Ltd.). Details are as follows.

4.1. Boundary conditions used in the FEA

Fig. 12 shows meshes along with boundary conditions used in the Finite Element Analysis (FEA). Total 714 elements were generated using Quads & Triangles mesh pattern for the soil. Element size is 0.005 m for the soil immediately below the model footing (from the surface of soil to the depth of 0.15 m (i.e. 3B)) and 0.01 m for the remainders. Boundary conditions were assumed to be restrained in horizontal direction along the center line and restrained in both horizontal and vertical directions at the bottom and the interface between soil and soil tank. Analyses were conducted as axisymmetric condition although the model footing test results used in the present study were obtained for a square footing. This can be justified based on the following experimental and numerical studies published in the literature. Cerato and Lutenegger (2006) conducted model footing tests on a sand using two different types of model footing; a square footing (B × L = 102 mm × 102 mm) and a circular footing (Diameter =

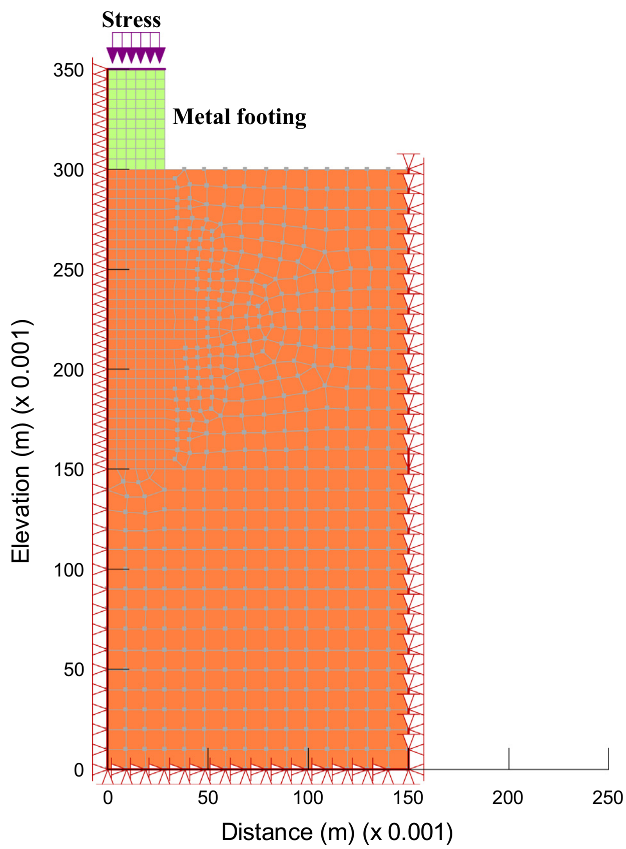

Fig. 12. Mesh and boundary conditions used in the FEA (axisymmetric condition).

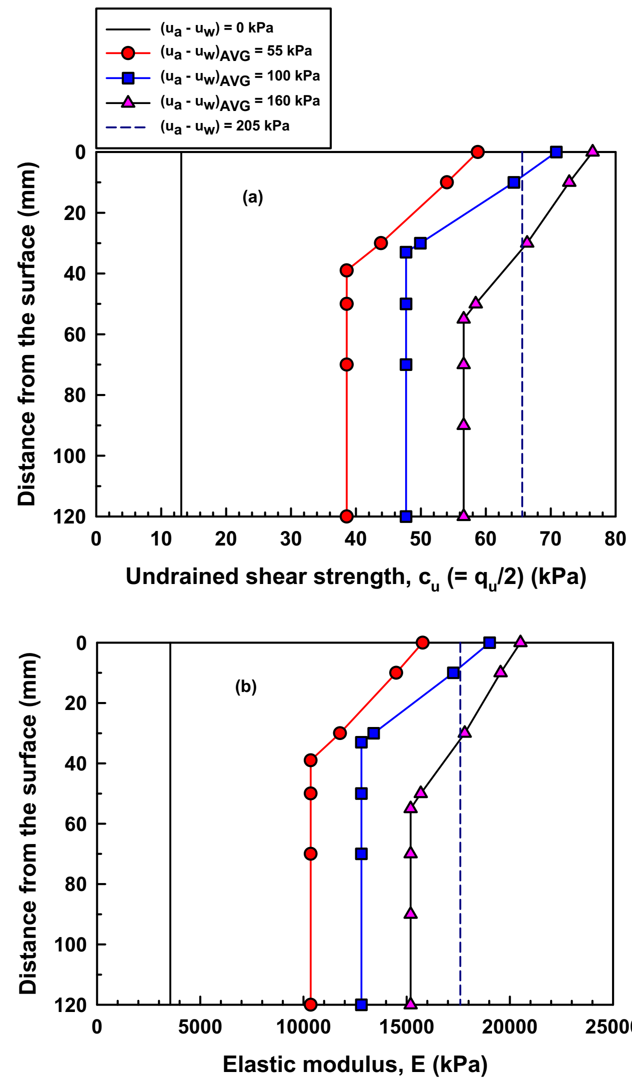

Fig. 13. Variation of (a) undrained shear strength and (b) elastic modulus with respect to depth.

102 mm). The results showed that the bearing capacity values using a square footing were approximately 1.25 times higher than those of a circular footing when an equivalent area is not considered. On the other hand, the three-dimensional numerical analysis by Gourvenec et al. (2006) showed that the difference in the bearing capacity values between a square footing and an equivalent circular footing was less than 3%. The (steel) model footing was simulated as a linear elastic material with significantly high modulus of elastic and zero weight. Total stress parameters were used to conduct FEA extending the MTSA. Analyses were carried out using elastic - perfectly plastic model with Mohr-Coulomb yield criterion (Chen and Zhang, 1991). Total cohesion and elastic modulus with respect to depth were calculated using Eqs. (13) and (17), respectively based on the matric suction distribution profiles (Fig. 5). One of main assumptions used in the present study is that the bearing capacity of unsaturated cohesive soils is governed by the compressibility of a soil block below the model footing. Hence, total internal friction and dilation angle of soils were assumed to be zero. These soil properties were then directly assigned to the meshes (Fig. 13). Stress was applied on the top of the model footing using stress boundary condition such that incremental stress can be gradually applied. The analyses were conducted with 0.1% displacement norm tolerance.

4.2. Influence of Poisson's ratio, ν on the SVS behaviors

Fig. 14 [(a)–(e)] shows the comparisons between the measured SVS behaviors and those predicted from the FEA for five different Poisson's ratio values (i.e. $\nu = 0.1$, 0.2, 0.3, 0.4, and 0.495). Based on the results in Fig. 14, the measured and estimated bearing capacity values are summarized in Table 2 and Fig. 15. The bearing capacity values increase with increasing Poisson's ratio values. This is because the higher Poisson's ratio contributes to higher lateral displacement, which leads to an increase in the resistance to the penetration of model footing into soil under confinement. Poisson's ratio close to 0.495 provided best agreement when compared with measured bearing capacity for saturated condition. For unsaturated conditions (except as compacted soil), better comparisons were achieved for the Poisson's ratios less than 0.3. For the 'as compacted sample', the estimated bearing capacity values were slightly lower than the measured values regardless

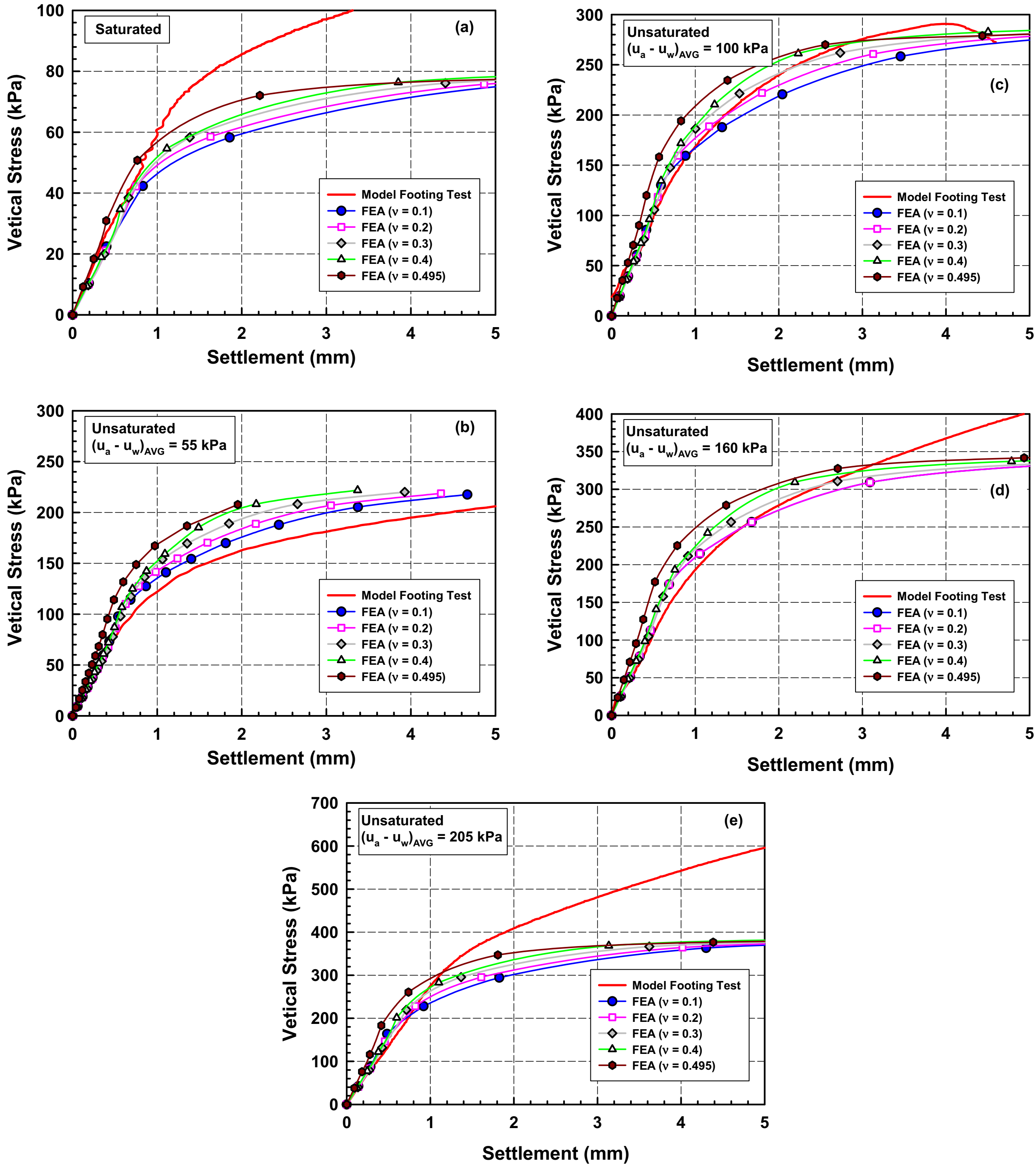

Fig. 14. Comparisons between measured SVS behaviors and those estimated using the FEA for different Poisson's ratios.

of Poisson's ratio values. Fig. 16 shows a comparison between the measured bearing capacity values and those estimated from different approaches such as the MESA (Eq. (2)), the MTSA (Eq. (4)) and the FEA along with MTSA (present study). Eq. (3) requires the residual degree of saturation that is not clearly defined in the measured SWCC (Fig. 8). Hence, the model proposed by Vanapalli and Mohammed (2007) was used to calculate bearing capacity values extending the MESA. The bearing capacity values estimated from the FEA using $\nu = 0.495$ and $\nu = 0.1$ were used as representative bearing capacity values for saturated and unsaturated conditions, respectively for the purpose of comparison. As can be seen (Fig. 16), the bearing capacity values estimated using the FEA with MTSA provided the best agreement when compared with the measured bearing capacity values.

Table 2
Comparisons between measured and estimated bearing capacity values.

$(u_a - u_w)_{AVG}$ (kPa)	$B.C$ (kPa)	$B.C_{0.1}$ (kPa)	$B.C_{0.2}$ (kPa)	$B.C_{0.3}$ (kPa)	$B.C_{0.4}$ (kPa)	$B.C_{0.495}$ (kPa)
0	80	63	64	68	73	73
55	153	180	188	195	197	205
100	233	235	249	262	269	269
160	257	296	296	300	312	323
205	384	333	350	353	361	360

B.C: measured bearing capacity from model footing test.
Subscript: Poisson's ratio used in the FEA.

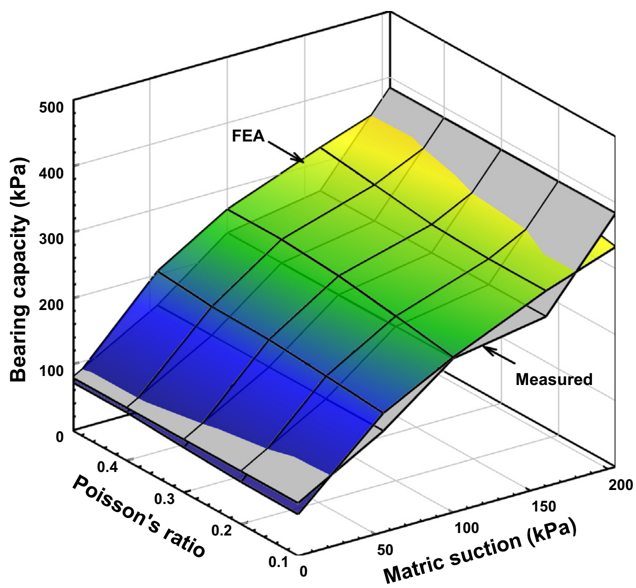

Fig. 15. Summary of the measured and estimated bearing capacity values for different Poisson's ratios.

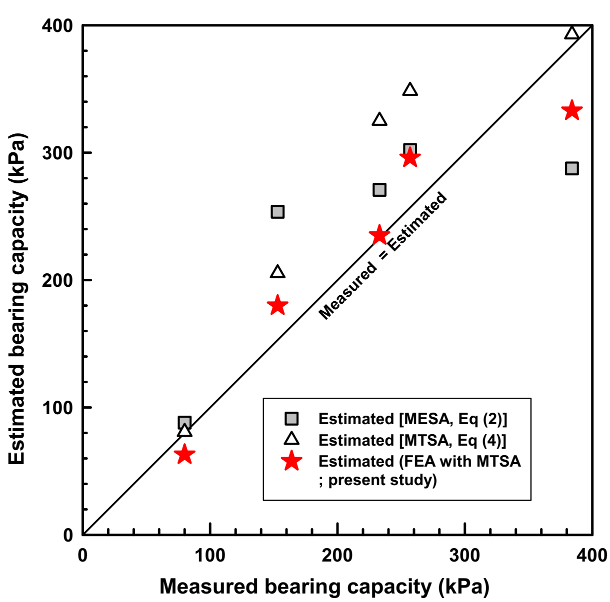

Fig. 16. Comparison between the measured bearing capacity values and those estimated using different approaches.

4.3. Influence earth pressure coefficient at rest, K_0 on the SVS behaviors

Another series of FEA were conducted to investigate the influence of K_0 on the SVS behaviors. This can be done by specifying K_0 value in the FEA, which is called the K_0 procedure in the SIGMA/W. This procedure overrides the initial stresses established in 'in-situ' conditions. The results showed that the SVS behaviors are not influenced by K_0 in the present study. This is because K_0 is a function internal friction angle and, as explained in Section 4.1, zero internal friction angle was used in the FEA to simulate SVS behaviors extending the conventional TSA. Georgiadis et al. (2003) carried out finite element analyses to investigate the influence of OCR on the behaviors of a single pile in an unsaturated soil in terms of total stress. A constitutive model developed for unsaturated soils was used for the FEA by implementing it into the Imperial College Finite Element Program (ICFEP). The OCR value was obtained from the OCR – K_0 – I_p relationships, as proposed by Brooker and Ireland (1965). The results showed that ultimate load decreases as OCR reduces. This indicates that the SVS behaviors of a shallow foundation can be affected by K_0 value, especially when a shallow foundation is located at a shallow depth where over consolidation is predominant due to desaturation. However, the influence of OCR on the SVS behaviors of shallow foundations cannot be considered in the proposed technique. Hence, the bearing capacity values obtained based on the estimated SVS behaviors can be considered to be conservative.

4.4. Justification of using the average matric suction

Comparisons of SVS behaviors estimated for varying matric suction and average matric suction values are shown in Fig. 17. For the FEA based on the average matric value, soil was assumed to be a single layer with a total cohesion and an initial tangent elastic modulus corresponding to the average matric suction value (Table 1). As can be seen, good agreement was observed between the measured and predicted SVS behaviors. This indicates that using an average matric suction value and its corresponding mechanical soil properties such as the shear strength, elastic modulus can provide reasonable SVS behavior without significant error.

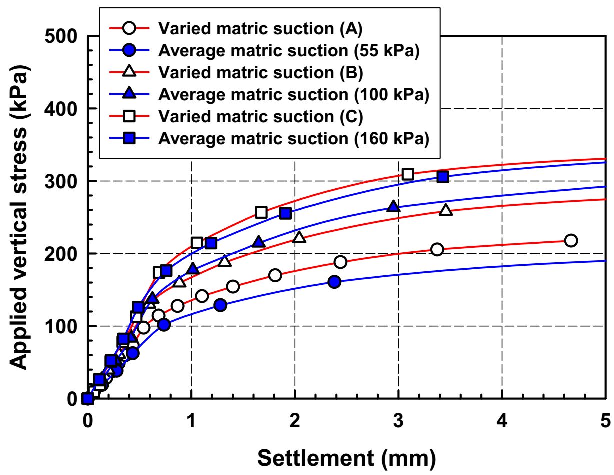

Fig. 17. Comparison of SVS behaviors estimated in the FEA for varied matric suction and average matric suction.

5. Summary and conclusions

In the present study, a simple Finite Element Analysis (FEA) approach extending Modified Total Stress Approach (MTSA) is proposed to estimate the bearing capacity of unsaturated cohesive soils. The conclusions obtained from the research can be summarized as below.

1. Poisson's ratio affects the stress versus settlement (SVS) behaviors of shallow foundations in unsaturated cohesive soils. Bearing capacity values estimated based on the SVS behaviors increase with increasing the Poisson's ratio. For saturated condition, the SVS behavior predicted using $\nu = 0.495$ provided better agreement with the measured SVS behavior. On the other hand, low values of Poisson's ratio (i.e. less than 0.3) are recommended to predict the SVS behaviors for unsaturated cohesive soils.
2. The SVS behaviors are not influenced by K_0 when the FEA is carried out extending the Modified Total Stress Approach (MTSA) due to the use of the zero internal friction angle.
3. Among the Modified Effective Stress Approach (MESA), Modified Total Stress Approach (MTSA), and FEA with MTSA (present study), the FEA with MTSA provided the most reasonable bearing capacity values when compared with measured bearing capacity values. These comparison studies suggest that the MTSA can be successfully extended in the simulation of the SVS behavior for shallow foundations in unsaturated cohesive soils.
4. The SVS behaviors in unsaturated soils can be reliably predicted using the concept of average matric suction without significant error.

The FEA method proposed in the present study is simple and useful for the geotechnical practitioners in the estimation of the bearing capacity of unsaturated cohesive soils using a commercial finite element software. However, more research is necessary to validate the proposed approach for the wetting condition.

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