Numerical Simulation of the Mechanical Behavior of Single Piles in Expansive Soil

Yunlong Liu, *Ph.D candidate, University of Ottawa, Ottawa, Canada; email:* liuyunlongzzu@hotmail.com
Sai K. Vanapalli, *Professor, University of Ottawa, Ottawa, Canada; email:* vanapall@eng.uottawa.ca
Christ-Fabel Nyambere, *Undergraduate student, University of Ottawa, Ottawa, Canada; email:* cnyam098@uottawa.ca
Hamze Mohamoud, *Undergraduate student, University of Ottawa, Ottawa, Canada; email:* hmoha077@uottawa.ca

ABSTRACT: *A numerical method, PEI (Pile behavior in Expansive soil upon Infiltration) developed at the University of Ottawa can be used as a tool for the prediction and interpretation of the mechanical behavior of piles in expansive soils taking account of the influence of matric suction changes associated with water infiltration. This program is employed in back analyses of the in-situ single pile test results for a case study at the Colorado State University (CSU) field test site, which has expansive soil deposits. Four single piles were drilled into the expansive shale of the Pierre Formation west of Fort Collins, Colorado. The piles were 350 mm in diameter and were installed to a depth of 7.6 m. During installation, slope indicator-type VS embedment strain gauges were embedded on the piers at a depth of 1.8 m, 3.1 m, 4.3 m and 5.5 m below the ground surface. Water content variations of the surrounding soil along the pile depth were measured periodically from 1995 to 2004, in addition to other measurements, which include pile axial force distribution and pile head displacement. Extensive analysis regarding this case study with respect to the mechanical behavior of pile associated with changes in water content is available in the literature. In this paper, this case study is illustrated in an innovative way to understand the influence of matric suction on the mechanical behavior of the pile using the program PEI. In addition, in-situ measurements of both pile head displacements and pile axial force distribution are compared with the results of numerical simulations using the PEI. Analyses of the results show that measured in-situ and numerical results are in good agreement. The results of this study are of significant interest for the practicing engineers who routinely design pile foundations in expansive soils.*

KEYWORDS: Pile, Expansive soil, Infiltration, Suction, Numerical simulation.

SITE LOCATION: Geo-Database

INTRODUCTION

The most destructive geotechnical and geological hazards in the United States are associated with expansive soils (Benvenga 2005). The residual soils derived from Pierre Shale Formation in Colorado region underlie several major infrastructures and have properties similar to these of typical expansive soils, which swell and shrink with water content increases or decreases, respectively. Various studies were conducted for understanding the expansive soil properties at Colorado State University (CSU) test site during the past four decades, which are widely documented in the literature (Porter 1977; Goode 1982; Chapel 1998; Durkee 2000; Abshire 2002; Benvenga 2005 and Nelson et al. 2011). A commonly used foundation type in this region and in other expansive soils regions in North America is the pile and grade beam system. However, limited research studies were undertaken to interpret the mechanical behavior of expansive soils and propose rational design procedures for pile foundations.

One of the most well-known and documented field investigation case studies was conducted for a period close to 10 years from June 1995 to April 2004 at the CSU test site to understand the mechanical behavior of piles in expansive soils. As a part of this study, four reinforced concrete piles of 350 mm in diameter were installed to a depth of 7.6 m, and were subjected to natural water content variations due to the influence of evaporation and infiltration conditions. No load was applied on the

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pile head during the entire testing period. The pile axial strains, water content of soil around the pile, free ground heave and pile head displacement were measured for interpreting the mechanical behavior of these piles over the testing period of 10 years. The treasurable data of this investigation has been presented and analyzed in the literature extensively by various scholars in the literature (Reichler 1997; Chapel 1998; Durkee 2000; Abshire 2002; Benvenga 2005). For example, Benvenga (2005) provided a detailed discussion about the development of the uplift force during the wetting process and linked the uplift force development and ground heave to the swelling pressure. The pile uplift force is estimated with an empirical relationship by taking into account of the influence of the swelling pressure on the pile-soil interaction, with the aid of adhesion factor α. The data acquired from this case study results were also used by Nelson et al. (2011) for validation of a numerical program they developed for interpretation of the load transfer mechanism in pile foundations. In this case study, a key stress state variable, matric suction that significantly influences the mechanical behavior of piles in unsaturated soils was not measured. Due to this reason, most of the analysis related to this case study was interpreted considering only the variations of water content. However, in the present study, variation of the mechanical behaviors of single piles were illustrated in an innovative way using unsaturated soil mechanics with the numerical simulation program, Pile behavior in Expansive soil upon Infiltration (PEI), which was developed at the University of Ottawa, Canada (Liu and Vanapalli, 2019). One of the key advantages of PEI is that it considers the influence of matric suction as an independent stress state variable. The analyses of the case study results of pile behavior show that measured in-situ values and numerical results using the PEI are in good agreement. The step-by-step procedure for implementation of the PEI is succinctly summarized in the Appendix of this paper. The PEI is a relatively simple numerical method that can be used routinely in the design of pile foundations in expansive soils. For this reason, the results of this study are of significant interest to practicing engineers.

SUMMARY OF SITE INVESTIGATION STUDIES AT THE COLORADO STATE UNVERSITY (CSU) TEST SITE

Exploratory borings were drilled by several researchers at the Colorado State University (CSU) field test site in order to investigate the soil profile characteristics for various projects, in vicinity of the test piles. The borings that are most relevant to the current study were those presented by Durkee (2000). As shown in Figure 1, borings BH-1, BH-2 and BH-3 were drilled approximately 30 m southeast of the piles. The descriptions of soil samples collected from the borings that were comprehensively described by Durkee (2000) are summarized in this paper. In each of the three boreholes, approximately 51 to 152 mm of vegetation and silt were underlain by a yellowish-tan clay layer to a depth of approximately 0.9 to 1.5 m. A reddish-yellow clay layer extending to a depth of 1.5 to 2.1 m was found in boreholes BH-1 and BH-2 that was not observed in borehole BH-3. The clay is underlain by reddish-brown weathered clay shale that extends to approximately 6 m. All boreholes yielded several montmorillonite seams ranging from 102 to 152 mm in thickness. Pile F130 lied directly along the strike corresponding to boring BH-1. For this reason, the soil profile at pile F130 can be assumed similar to that of BH-1. The soil layers were extended beyond pile F130 in order to determine the approximate soil profiles for the remainder three piles. The soil profile for each of the research piles (as shown in Figure 2), determined from the borings, did not differ much to make a well-defined distinction between layers from one pile to the next. For this reason, the soil profile for BH-1 & F130 (see Figure 2) was used for all the four pile locations.

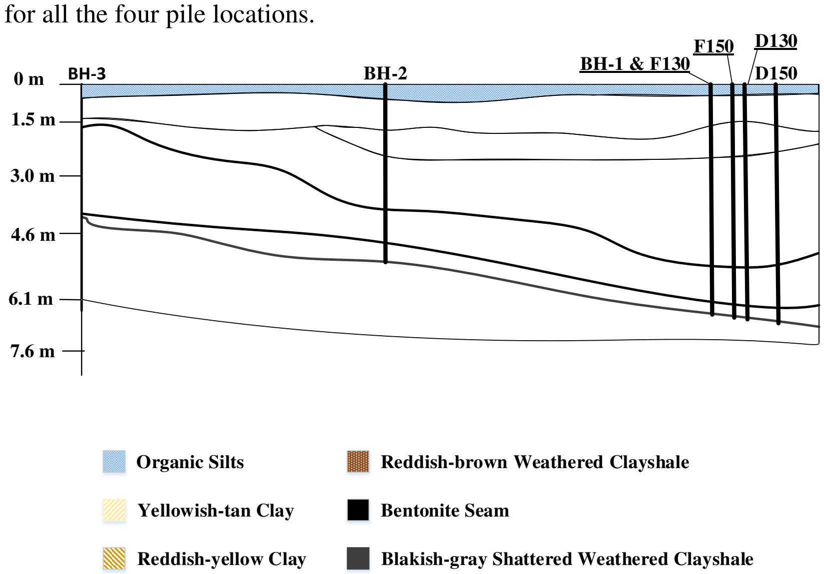

Figure 1. Positions of borings, piles and soil profile (Figure not in scale. Modified after Benvenga 2005).

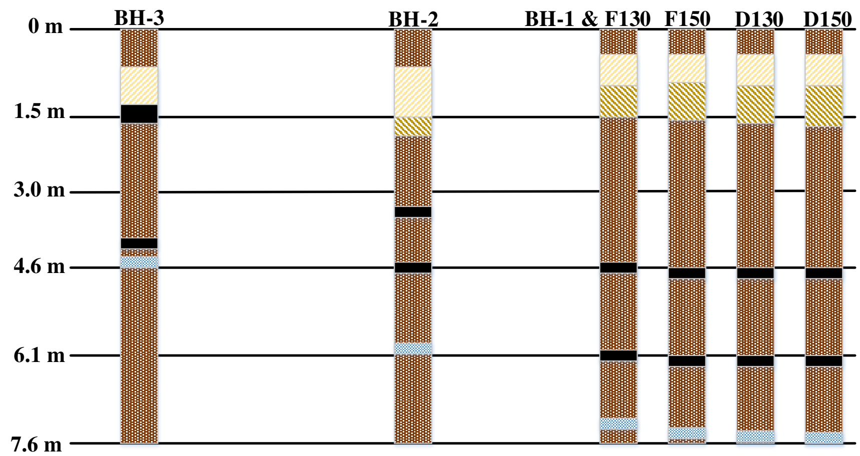

Figure 2. Soil profile at the positions of boreholes and piles (modified after Benvenga 2005).

DETAILS OF INSTRUMENTATION ON THE PILES IN THE CSU TEST SITE FOR COLLECTING DATA

Figure 3 shows the details of the four concrete single piles installed at the CSU test site. Strain gauges were mounted in the center of each of the concrete piles at depths of 1.8, 3, 4.3 and 5.5 m. The gauges were attached to the reinforcing cage prior to placement of the cage in the excavation. A strain gauge was also mounted in a 152 mm diameter concrete cylinder to verify the field strain readings and provide calibration and control data. Each gauge is 11 mm in diameter and the flanges at each end are 32 mm in diameter. The end flanges are intended to provide a good embedment in the concrete. A steel bar was fixed on the pile head for the measurement of pile head displacement. A Campbell Pacific Nuclear Corp. (CPN) 501 DR depth probe was used to collect water content and density data at the site. Measurements were taken at 0.3 m intervals to a depth of 6m. The CPN measures subsurface density using a gamma source and a Geiger Mueller detector. The water content data were used to determine the depth of wetting of the soil. The swell pressure acting on the piles was also calculated based on the water content of the soil. Water content readings were taken at different depths. Survey pins were installed at the ground surface for the ground displacement measurement.

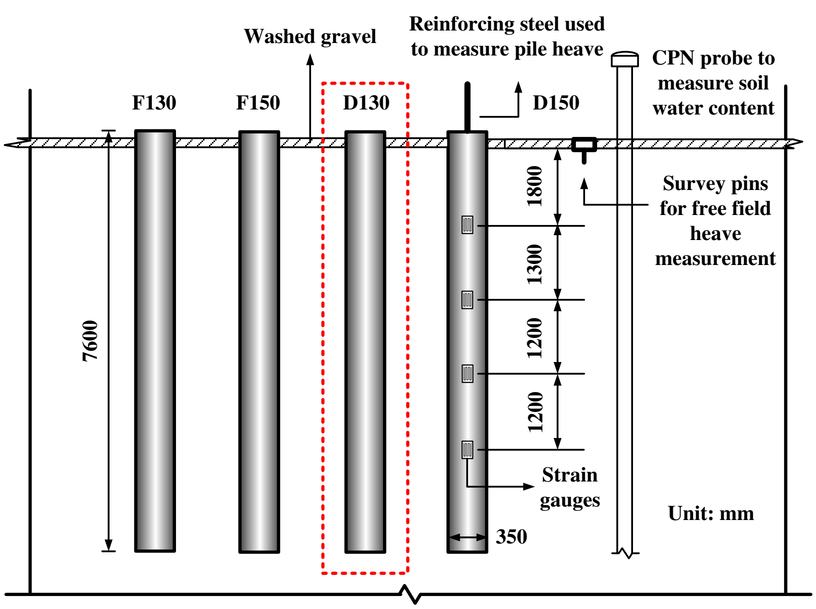

Figure 3. Diagram of the drilled reinforced concrete pile at CSU expansive soil test site (modified after Benvenga 2005).

PILE TEST RESULTS

All four single piles tested at CSU showed similar behavior; for this reason, only one pile, identified as D130 (see Figure 3), was selected as an example in the present study for illustrating the mechanical behavior of piles taking account influence of the environmental factors. Table 1 shows the variation of wetting depth, which varied from between 3 to 4 m at different time intervals. The volumetric water content variation at six different time points during the 10-year period are shown in Figure 4. The volumetric water content varied from 20% to around 33% at different depth levels. The measured ground and pile head displacements during the investigation period were presented by Benvenga (2005), which is summarized in Figure 5. From February 1997 to August 1997, the soil experienced ground heave of 64 mm. However, in October 2002, the soil experienced a settlement of around 2 mm below the zero point. The soil swelled approximately 6 mm (with reference to the zero point) from that time point. The pile head experienced upward displacements from 3 to 20 mm. In October 2002, as discussed earlier, in spite of ground settlement, the pile recorded an uplift displacement of around 15 mm. Such a ground settlement contributes a downward friction on the pile shaft and leads to further settlement. These results suggest there may be some error either in the measurement of ground displacement or in the pile head displacement. Figure 6 shows the pile axial force distribution at different times. According to Benvenga (2005), cracks were generated in the lower part of the pile after August 1997. The strain gauges at the depth of 4.3 m and 5.5 m recorded unreasonably high strains, which contributed in the estimation of unreasonably high pile axial forces. For this reason, to alleviate some of the limitations with the collected data, Benvenga (2005) proposed an empirical relationship relating the total uplift forces with swelling pressure multiplied by a single coefficient, which is referred to as adhesion factor α. In addition, Benvenga (2005) suggested regression equations for the estimation of the swelling pressure from volumetric water content. In other words, an average swelling pressure for different depths was used in the calculation of the total pile uplift force. The total uplift force that was computed using the average swelling pressure times the empirical coefficient, α.

Table 1. Wetting depth at different time points.

Time points	Feb-97	Oct-97	Oct-02	Jun-03	Sep-03	Apr-04
Wetting depth (m)	3.6576	3.9624	3.048	3.3528	3.9624	3.3528

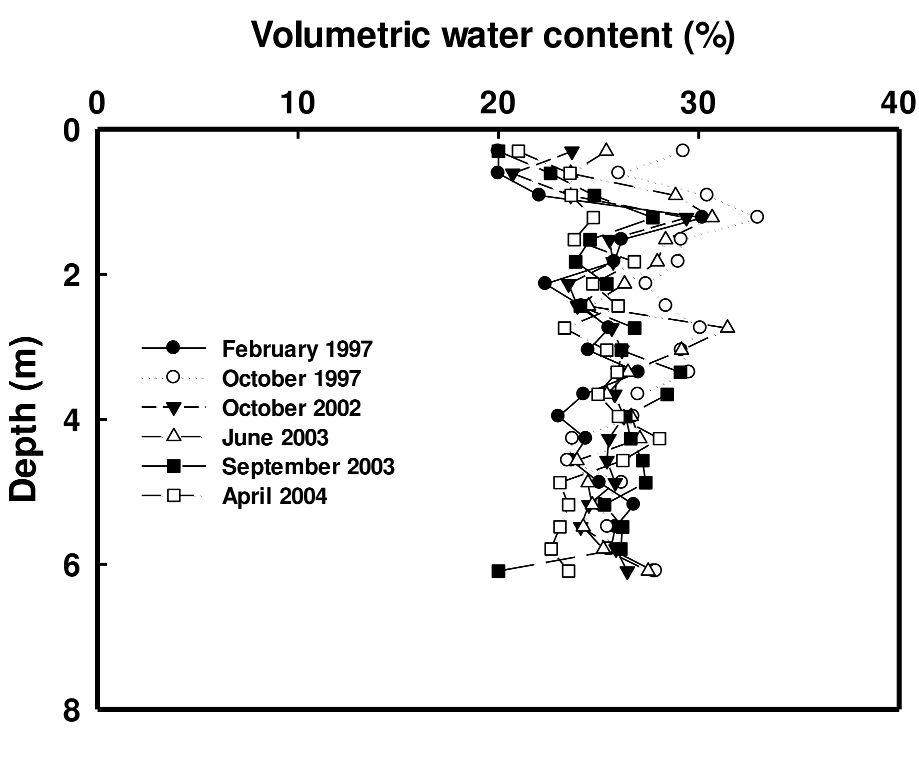

Figure 4. Measured volumetric water content variations.

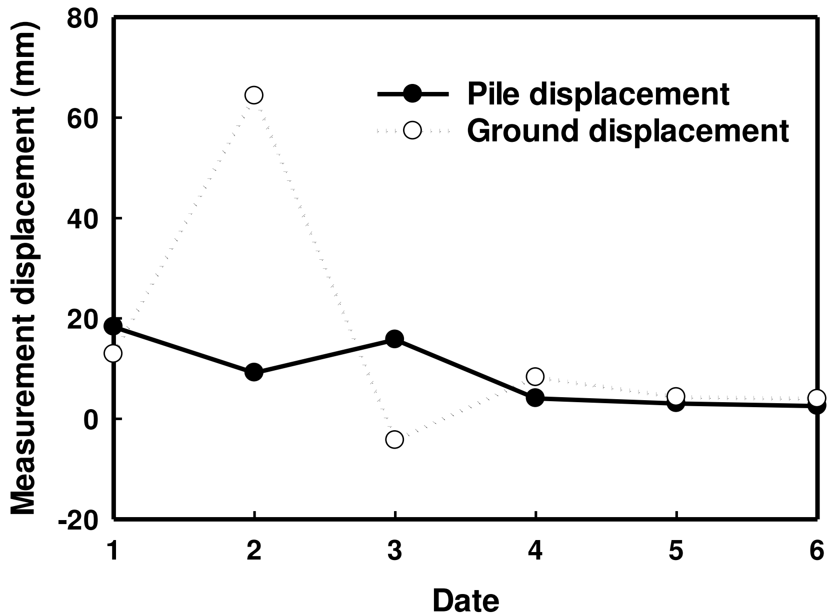

Figure 5. Measured pile and ground displacement [1-February 1997; 2- August 1997; 3-October 2002; 4- June 2003; 5-September 2003; 6-April 2004].

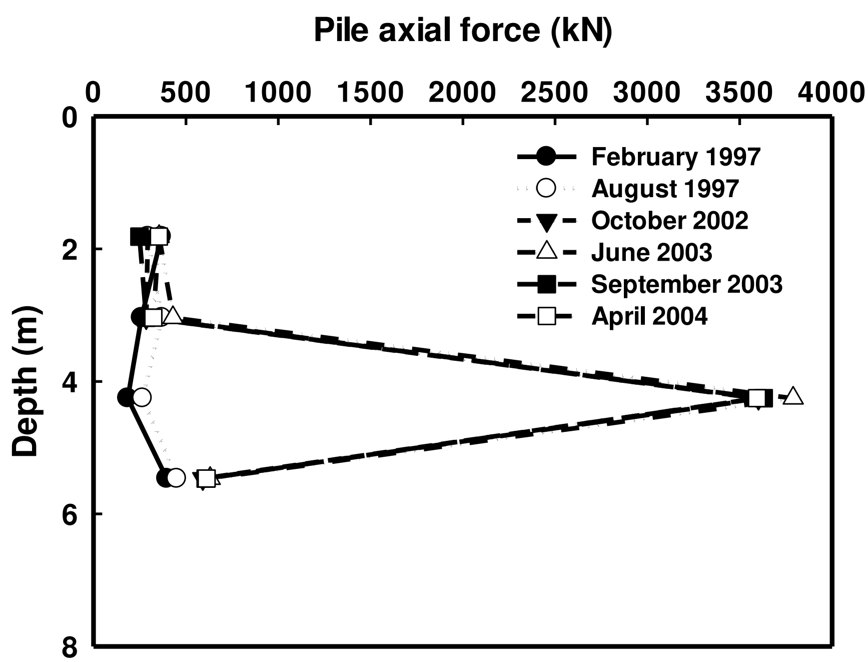

Figure 6. Measured pile axial force distributions.

NUMERICAL SIMULATION PROGRAM, PEI

The load transfer method proposed by Coyle and Reese (1966) is widely utilized to predict the load settlement behavior of a single pile subjected to axial load. This method is based on a relatively simple analytical procedure that can be applied to any complex composition of soil layers with a nonlinear stress-strain relationship for a nonhomogeneous medium or any other variation in the section along a pile (Coyle and Reese 1966). The traditional load transfer method is modified considering the influence of volume expansion of soil around the pile on the pile-soil relative displacement in the numerical simulation program, PEI (Liu and Vanapalli 2019). The step-by-step procedure of the computer program PEI is summarized in the Appendix. Employing several assumptions, experimental results of the CSU pile tests are illustrated using this numerical simulation program to highlight to influence of matric suction and/or matric suction variations associated with wetting on the mechanical behavior of piles in expansive soils.

PREDICTION OF THE PILE MECHANICAL BEHAVIOR USING THE PEI

The mechanical behavior of pile D130 was predicted using the program PEI taking into account the influence of environmental factors and was compared with the measured data. In order to simplify the calculations, average soil properties of different soil layers were used in the present study. In addition, an empirical relationship between constant vertical swelling pressure and volumetric water content of the soil proposed by Benvenga (2005) [Eq. (1)] was considered. Figure 7 summarizes the variation of constant volume vertical swelling pressure of expansive soil with respect to depth using the water content measurements from Figure 4.

$$y = 12.949\ln(x) - 69.45 \tag{1}$$

where x is constant volume vertical swelling pressure with units in psf; y is the volumetric water content.

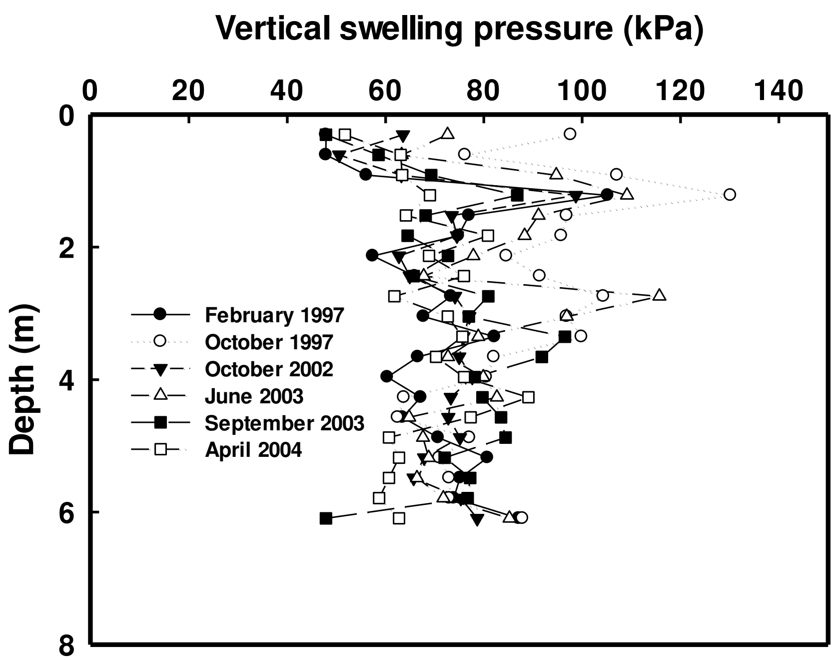

Figure 7. Constant volume vertical swelling pressure at different time points (Modified after Benvenga 2005).

Matric suction values were not measured at the CSU test site; this may be due to the limitations in instrumentation when this case study was initiated, which was about 25 years ago. For this reason, in the present study matric suction profiles were back calculated using a semi-empirical equation proposed by Tu and Vanapalli (2016) (see Eq. (2)).

$$P_{SC} = P_{S0} + \beta_c \psi_i \left(\frac{S}{100}\right)^2 \tag{2}$$

where P_{SC} is the vertical swelling pressure (kPa) for compacted soils; S is the degree of saturation; P_{S0} is the intercept on the P_S axis at zero suction value (P_{S0} varies from 0 to 120 kPa, an average 55 kPa is suggested compacted expansive soils); $\beta_c = (0.011e^{0.107I_P} - 7.872\,\rho_{d,max} + 13.706)/2$; I_P is the index of plasticity; $\rho_{d,max}$ = maximum dry density; ψ_i is the soil suction.

This semi-empirical equation is capable of predicting the constant volume vertical swelling pressure tested in the laboratory based on soil suction, degree of saturation and some other basic soil properties. Using the information summarized in Figure 7 from Benvenga (2005) and Eq. (2), the soil water characteristic curve (SWCC) and variations of suction were back-calculated and summarized as Figure 8 and Figure 9, respectively. It is more reliable to use the wetting SWCC; however, in the present analysis the back-calculated SWCC, is the drying curve. A drying SWCC is used due to lack of data for estimating the wetting curve from the drying curve (for example, Pham et al. 2005). In addition, the hysteresis phenomenon is not considered in this study due to lack of information.

In order to use Eq. (2), plasticity index, I_p and maximum dry density ($\rho_{d,max}$) are necessary. Benvenga (2005) used the average soil properties for calculation of the vertical swelling pressure. An average value of maximum dry density of 17.7 kN/m^3 was used. The plasticity index, I_p, for different soil layers along the depth of the pile was not available in Benvenga (2005). Nelson et al. (2015) summarized that the expansive soils in the Front Range area of Colorado typically have a liquid limit ranging from 35 to 75 percent and a plasticity index ranging from 15 to 50 percent. For this reason, an average plasticity index, I_p of 30 was used for different soil layers. This value falls in the range of medium swelling potential (25 to 35) according to the expansive soil classification system proposed by O'Neill and Poormoayed (1980).

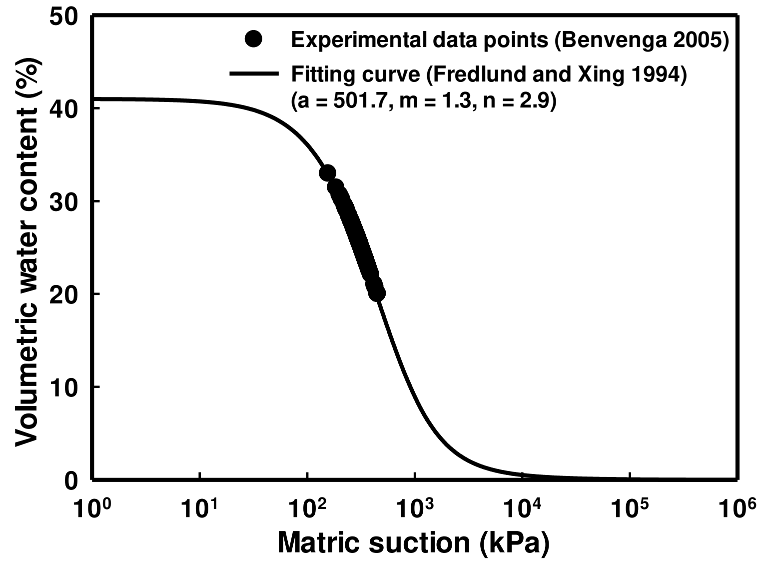

Figure 8. Estimated SWCC using the Fredlund and Xing (1994) model.

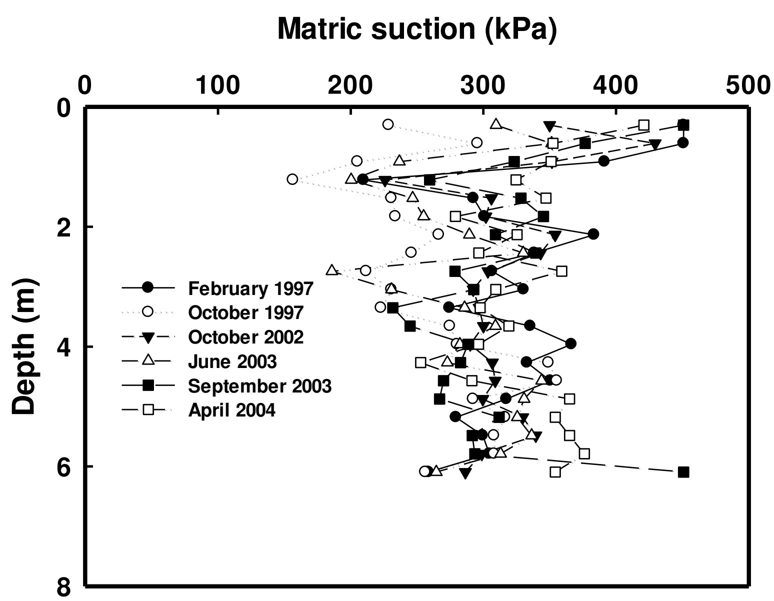

Figure 9. Variation of matric suction with respect to depth at during the test period from 1997 to 2004.

Benvenga (2005) estimated the average ultimate shaft friction, f_s between the pile and soil using Eq. (3) and suggested the range of α varies from 0.6 to 1.0 in October 1997. Using the SWCC and matric suction profiles shown in Figure 8 and Figure 9, it is possible to use Eq. A5 (see Appendix) to back calculate the average shear strength properties of the pile-soil interface.

$$f_s = \alpha \sigma'_{cv} \tag{3}$$

where f_s is the average ultimate shaft friction; α is the empirical adhesion coefficient; σ'_{cv} is corresponding constant volume vertical swelling pressure.

Employing Eq. (A5), the effective pile-soil interface cohesion, c_a and the effective pile-soil interface friction angle δ' are estimated as 15 kPa and 25°, respectively by trial and error method. Comparisons among back calculated results and the estimations made by Benvenga (2005) using different α values regarding shaft friction distribution on October 1997 are shown in Figure 10. In addition, for the application of PEI, the residual interface shear strength and the pile-soil relative displacement corresponding to the peak interface shear strength, is necessary (see Appendix, for more details). Since there was no data available from Benvenga (2005), empirical relationships are employed which set the ratio of residual interface shear strength to the peak interface shear strength, β_s and pile soil relative displacement corresponding to the peak interface shear strength S_u as 0.85 and 0.01 m, respectively. Both of these parameters are in the range suggested by Zhang and Zhang 2012).

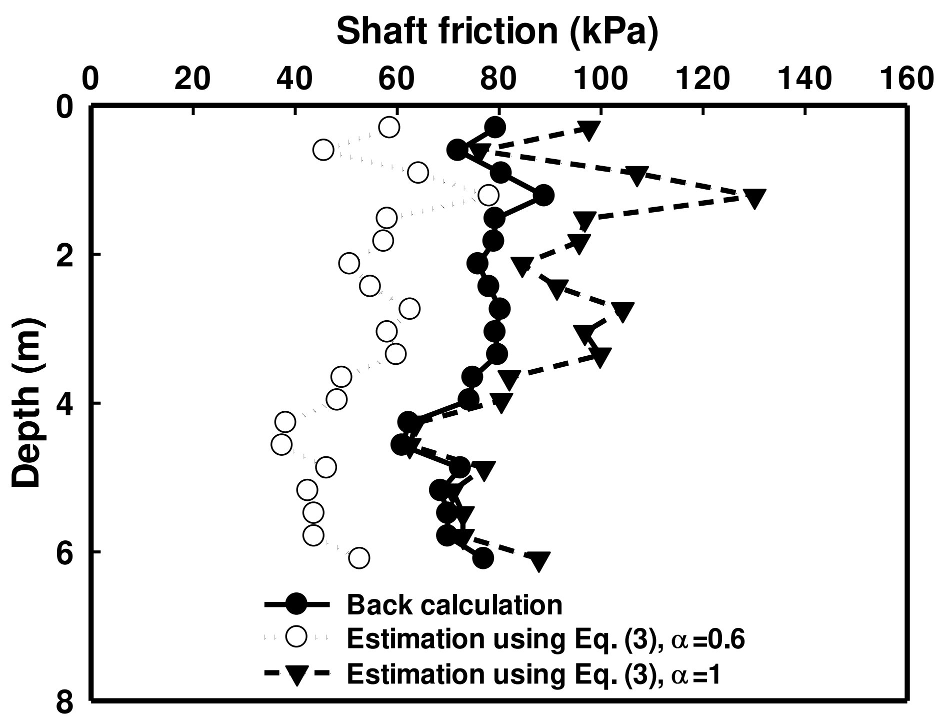

Figure 10. Comparison of pile shaft friction distribution from the case study by Benvenga (2005).

Using the matric suction profile at the beginning of the test (February 1997) and at specific time points (i.e., October 1997, June 2003, September 2003 and April 2004), the mechanical behavior of pile is analyzed employing the program PEI. The pile mechanical behavior in October 2002 is not discussed due to associated errors within the measurement of the pile head and ground displacement. The likely errors with respect to measurements in October 2002 were discussed in an earlier section. Figure 11 summarizes the measured pile head displacements and the predicted values using the proposed program PEI and Benvenga (2005). There is a reasonably good comparison between the measured and predicted values using the PEI.

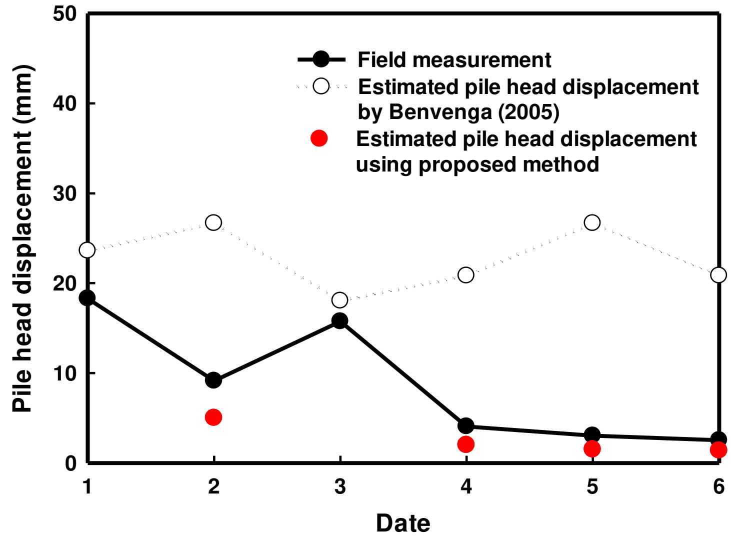

Figure 11. Comparison of pile head displacement [1-February 1997; 2- August 1997; 3-October 2002;4- June 2003; 5-September 2003; 6-April 2004].

Figure 12 provides comparisons between the measured pile axial force distributions and predicted values using the PEI at different time points. The PEI method that uses the modified load transfer method provides reasonable estimations of the mechanical behavior of the piles taking account of the influence of environmental factors (i.e., wetting and drying).

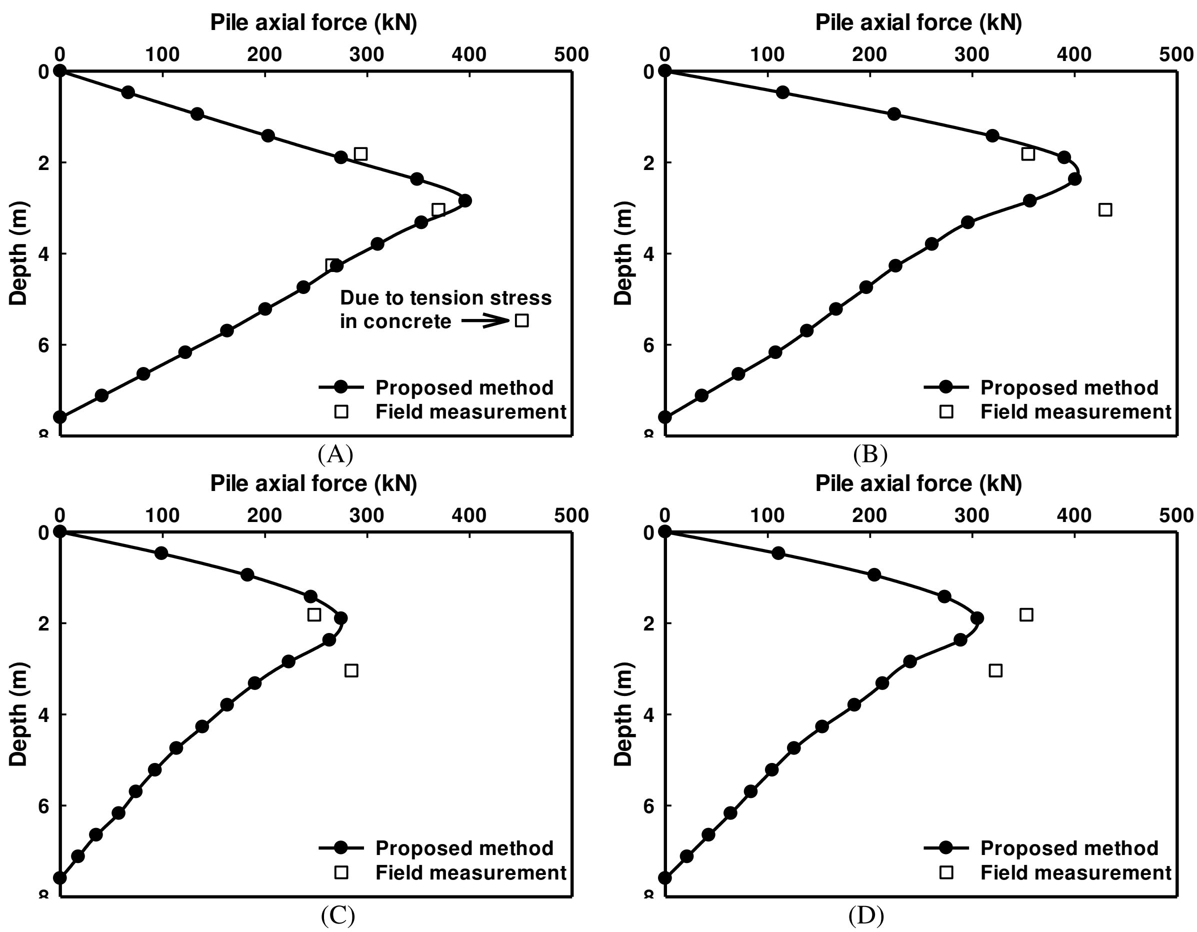

Figure 12. Comparison of pile axial force distribution (A) October 1997; (B) June 2003; (C) September 2003; (D) April 2004.

DISCUSSION

The load carrying capacity of a single pile is conventionally calculated from the contributions arising from the shaft friction and base resistance in non-expansive soils. However, both the pile shaft friction and pile base resistance are influenced by matric suction and/or matric suction variations in unsaturated expansive soils. Eq. A5 highlights that the interface shear strength (i.e., peak shaft friction) is influenced by both matric suction variations and lateral earth pressure changes associated with variations in matric suction due to wetting and drying conditions (Liu and Vanapalli, 2019). The post peak interface shear strength (residual pile shaft friction, τ_{sr}) as illustrated in Eq. A6 can be determined by taking account of lateral earth pressure acting on the pile shaft and assuming the disruption of air-water menisci along the failure surface in the post peak stage (Hamid and Miller 2009). It is however important to take account of pile-soil relative displacement changes due to the ground heave during the wetting process or ground settlement in the drying process for reliable estimation of load carrying capacity of piles in expansive soils.

The numerical simulation program PEI summarized in this paper is comprehensive and takes account of various factors, which include matric suction and volume change behavior of an expansive soil to simulate the mechanical behavior of piles in unsaturated expansive soils. A comprehensive discussion is available in Liu and Vanapalli (2019). The information required for using PEI include matric suction profile and soil properties, which include the saturated elastic modulus, saturated interface shear strength properties, Poisson's ratio, plasticity index and maximum dry density of the expansive soil. The mechanical behavior of a pile (D130) tested at Colorado State University (CSU) were simulated and analyzed using the PEI. Good comparisons of pile head displacement (Figure 11) and pile axial force distribution (Figure 12) between the numerical simulation and field measurements highlight the need for taking account of the influence of matric suction for reliable estimation of the load transfer mechanism in pile foundations. This study highlights the importance of extending the mechanics of unsaturated soils in the rational design of pile foundations for unsaturated expansive soils.

CONCLUSIONS

Conventionally used foundations in expansive soils, which are typically in a state of unsaturated condition are piles; these soils experience significant volume changes due to water content changes associated with wetting and drying conditions. The soil structure interaction has a significant influence on the mechanical behavior of piles in unsaturated expansive soils. One of the stress state variables, matric suction, is sensitive to changes in water content conditions. In this paper, the in-situ pile tests results conducted at Colorado State University (CSU) for a period of nearly 10 years are summarized. The data collected from this case study were discussed, interpreted and illustrated in an innovative way using unsaturated soil mechanics through numerical simulation program PEI, which was developed at the University of Ottawa, Canada. The step-by-step computation procedure of PEI provided in Appendix demonstrates the influence of matric suction and matric suction-induced volume change of an expansive soil on the mobilization of pile shaft friction and pile base resistance. More information about the PEI is available in Liu and Vanapalli (2019). The mechanical behavior of piles, which include the pile head displacement and the pile axial force distribution were predicted using the numerical simulation program PEI in the present study to provide good comparisons with the measured field data. The proposed approach in this paper is relatively simple and promising and can be used as a tool in the interpretation of the mechanical behavior of piles in expansive soils and assist the practicing engineers in the rational design of pile foundations.

APPENDIX

The steps that are involved in the computer program PEI are summarized below.

1. The pile is divided into a number of segments [as shown in Figure I (A)].

2. Assume a small base movement [as shown in Figure I (B)], ρ_t and calculate corresponding pile base resistance according to Eq. (A1).

$$\tau_b = \begin{cases} 0 & \rho_t < 0 \\ k_b \rho_t & \rho_t > 0 \end{cases} \tag{A1}$$

Randolph and Wroth (1978) proposed a model for the determination of k_b shown as Eq. (A2).

$$k_b = \frac{4G_{sb}}{\pi r_0 (1-\upsilon_b)} \tag{A2}$$

where G_{sb} and υ_b are the shear modulus and Poisson's ratio of the soil below the pile base, respectively.

3. A pile-soil relative displacement, ρ_3, may be assumed at mid-height of the bottom segment (for the first trial, it is suggested to use a value of $\rho_3 = \rho_t$) [as shown in Figure I (B)].

4. The matric suction profile keeps changing during the infiltration process. The different transfer curve models at different depths for a certain matric suction profile can be estimated using Eq. (A3) [as shown in Figure I (C) and (D)].

$$\tau_s(z) = \frac{S_s(z)[a + cS_s(z)]}{[a + bS_s(z)]^2} \tag{A3}$$

$$\begin{cases} a = \dfrac{\beta_s - 1 + \sqrt{1-\beta_s}}{2\beta_s} \dfrac{S_{su}}{\tau_{su}} \\ b = \dfrac{1 - \sqrt{1-\beta_s}}{2\beta_s} \dfrac{1}{\tau_{su}} \\ c = \dfrac{2 - \beta_s - 2\sqrt{1-\beta_s}}{4\beta_s} \dfrac{1}{\tau_{su}} \end{cases}$$

$$\beta_s = \frac{\tau_{sr}}{\tau_{su}} \tag{A4}$$

where $\tau_s(z)$ is the interface shear stress (shaft friction) at a given depth, z; $S_s(z)$ is the pile-soil relative displacement at a given depth, z; S_{su} is pile-soil relative displacement corresponding to peak interface shear strength; τ_{su} is the peak interface shear strength; τ_{sr} is the residual interface shear strength; from a series of field tests on bored piles under compression loading, Zhang et al. (2010. 2011a and 2011b) demonstrated that the value of β_s is within the range of 0.83-0.97. The peak interface shear strength can be estimated using Eq. (A5) and the residual interface shear strength can be estimated using Eq. (A6) (Liu and Vanapalli 2019). Hamid and Miller (2009) studies suggest the residual interface shear strength (τ_{sr}) is influenced only by net normal stress because there is a disruption of air-water menisci along the failure surface due to continued shearing beyond the peak interface shear strength, τ_{su}. In other words, the matric suction contribution to the shear strength significantly reduces due to the reduction in the area of the air-water menisci beyond τ_{su}.

$$\tau_{su} = c'_a + \tan\delta'\{\frac{(1-\upsilon-2\upsilon^2)P_{s(\psi mi-0)}}{1-\upsilon^2 - \frac{P_{s(\psi mi-0)}}{E_{a(\psi mi-0)}}(1+\upsilon)(1-\upsilon-2\upsilon^2)} - \frac{(1-\upsilon-2\upsilon^2)P_{s(\psi mw-0)}}{1-\upsilon^2 - \frac{P_{s(\psi mw-0)}}{E_{a(\psi mw-0)}}(1+\upsilon)(1-\upsilon-2\upsilon^2)} + \frac{\upsilon}{1-\upsilon}\sigma_{vs} + (u_{af} - u_{wf})\frac{\theta-\theta_r}{\theta_s-\theta_r}\} \tag{A5}$$

where ψ_{mi} and ψ_{mw} are initial matric suction and matric suction upon wetting, ψ_{mi} has a value higher than ψ_{mw}); $P_{S(\psi mi-0)}$ is the vertical swelling pressure (constant volume condition) generated from ψ_{mi} to 0; $P_{S(\psi mw-0)}$ is the vertical swelling pressure (constant volume condition) generated from ψ_{mw} to 0; $E_{a(\psi mi-0)}$ is the elastic modulus corresponding to a matric suction variation from ψ_{mi} to 0; $E_{a(\psi mw-0)}$ is the elastic modulus corresponding to a matric suction variation from ψ_{mw} to 0; υ is Poisson's ratio; σ_{vs} is the total vertical pressure due to soil unit weight and surcharge; c_a' is the effective interface cohesion; $(u_{af} - u_{wf})$ is the matric suction at failure; δ' is the interface soil internal friction angle; θ is the current volumetric water content; θ_r is the residual volumetric water content; θ_s is the volumetric water content at a saturation of 100%.

$$\tau_{sr} = c'_{ar} + (\sigma_{nf} - u_{af})\tan\delta'_r \tag{A6}$$

where (σ_{nf} - u_{af}) is the net normal stress at failure; c'_{ar} is the residual effective cohesion; δ'_r is the residual interface friction angle. In Eq. (6), both the residual effective cohesion (c'_{ar}) and the residual interface friction angle (δ'_r) should be determined from experimental studies.

If there is no experimental data available on the residual interface shear strength, the empirical relationship between the peak and residual interface shear strength can be extended for estimating the pile-unsaturated soil interface. A value in the range of 0.83 to 0.97, which is typically designated as β_s is suggested based on a series of field tests on bored piles under compression loading according to the experimental studies from Zhang et al. (2010, 2011a and 2011b). In other words, the value of residual interface shear strength can be assumed as 83% to 97% of peak interface shear strength. From a series of field tests for bored piles (diameter from 0.7-1.1 m) in different kinds of soils (e.g., mud, clay, sandy silt, silty clay), Zhang and Zhang (2012) summarized that the value of S_{su} varies within a range from 5 to 25 mm.

5. From the estimated transfer curve model, the interface shear strength corresponding to pile-soil relative displacement ρ_3 can be obtained as τ_3. It should be noted that for the pile-soil relative displacement in the active zone, the ground heave should be taken into account. The ground heave within a certain soil layer can be estimated using Eq. (A7) proposed by Adem and Vanapalli (2016).

$$\Delta h = h\left[\frac{(1+\upsilon)(1-2\upsilon)}{E_a(1-\upsilon)}\right]\Delta(u_a - u_w) \tag{A7}$$

where Δh is the heave of soil; h is the thickness of the calculated soil layer; $\Delta(u_a - u_w)$ is the matric suction reduction.

6. The load Q_3 on the top of segment 3 [as shown in Figure I (B)] can then be calculated as

$$Q_3 = P_t + \tau_3 L_3 P_3 \tag{A8}$$

where L_3 is the length of segment 3; P_3 is the average perimeter of segment 3.

7. The elastic deformation at the mid-point of the pile segment (assuming a linear variation of load in the segment) is calculated as

$$\Delta'\rho_3 = \left(\frac{Q_m + P_t}{2}\right)\left(\frac{L_3}{2A_3E_p}\right) \tag{A9}$$

where $Q_m = \frac{Q_3 + P_t}{2}$; A_3 is the area of segment 3; E_p is the elastic modulus of the pile.

8. The new pile-soil relative displacement at the middle point of the segment 3 is then given as Eq. (A 10).

$$\rho'_3 = \rho_t + \Delta\rho'_3 \tag{A10}$$

9. The calculated ρ'_3 is compared with the estimated value of ρ_3 from step (3).

10. If the computed movement ρ'_3 does not agree with ρ_3 within a specified tolerance (in this study, 10^{-11}m is used), step (2) to step (10) are repeated and a new midpoint pile-soil relative displacement is calculated.

11. When convergence is achieved, the next segment up is considered. This iteration technique is continued until the value of pile head load (Q_1) and pile head displacement (ρ_0) are obtained.

12. Following the procedure detailed from step (1) to step (11), the pile axial force is calculated from the bottom segment to the top segment, for a certain matric suction profile, by assuming a series of pile base movement (ρ_t), the pile head load displacement response can be obtained.

From step (1) to step (12), a series of curves reflecting the relationship between the pile head load and pile head displacement for different matric suction profile can be obtained. Keep the pile head (Q_1) as a certain value, through these curves, the pile head displacement (ρ_0) variations in the infiltration process can be estimated. Since the proposed program is based on the traditional load transfer method, the application of PEI is also limited by the fundamental limitations of traditional load transfer method. Initially no proper account is taken of the continuity of the soil mass in PEI.

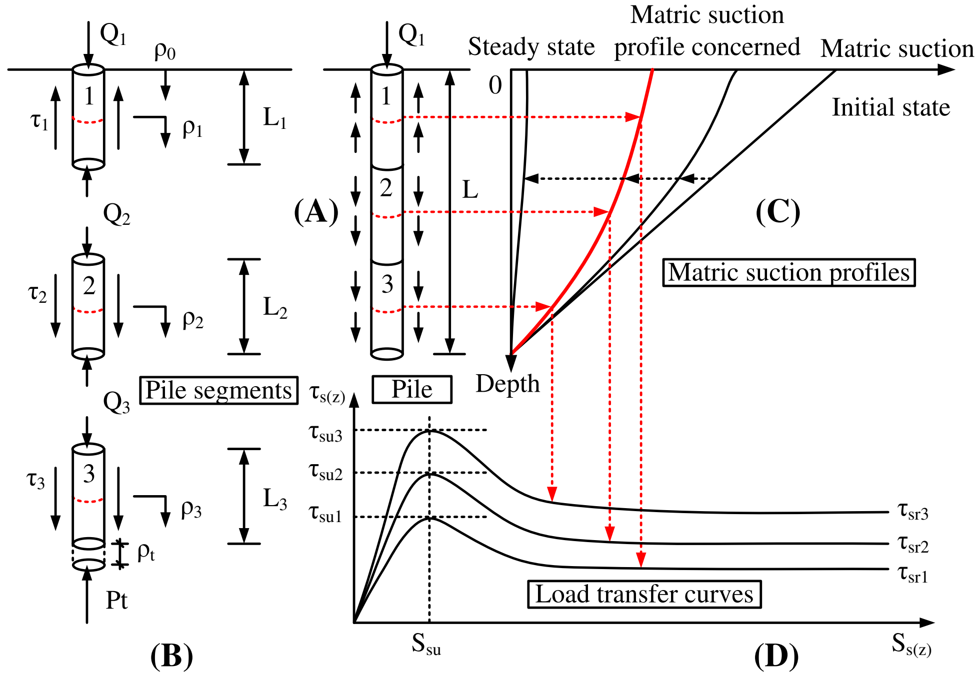

Figure I. Schematic to provide key details of the numerical simulation program PEI for obtaining the load transfer curves.

REFERENCES

Abshire, M. S. (2002). *Differential moisture migration and heave beneath slabs over dipping expansive shale*, M.S. thesis, Colorado State Univ., Fort Collins, CO.

Adem, H.H., and Vanapalli, S.K. (2016). “Heave prediction in a natural unsaturated expansive soil deposit under a lightly loaded structure.” *Geotech. Geolo. Eng.*, 34(4), 1181-1192.

Benvenga, M. M. (2005). *Pier-soil adhesion factor for drilled shaft piers in expansive soil*, M.S. thesis, Colorado State Univ., Fort Collins, CO.

Chapel, T. A. (1998). *Field investigation of helical and concrete piers in expansive soils*, M.S. thesis, Colorado State Univ., Fort Collins, CO.

Coyle, H.M., and Reese, L.C. (1966). “Load transfer for axially loaded piles in clay.” *J. Soil. Mech. Found. Div.*, 92(SM2), 1-26.

Durkee, D. B. (2000). *Active zone depth and edge moisture variation distance in expansive soils*, Ph.D. dissertation, Colorado State Univ., Fort Collins, CO.

Goode, J. C. (1982). *Heave prediction and moisture migration beneath slabs on expansive soils*, M.S. thesis, Colorado State Univ., Fort Collins, CO.

Hamid, T.B., and Miller, G.A. (2009). “Shear strength of unsaturated soil interfaces.” *Can. Geotech. J.*, 46(5), 595-606

Liu, Y., and Vanapalli, S.K. (2019). “Load displacement analysis of a single pile in an unsaturated expansive soil.” *Compu. Geotech.*, 106, 83-98.

Nelson, J.D., Chao, K.C., Overton, D.D., and Nelson, E.J. (2015). *Foundation Engineering for Expansive Soils*, John Wiley & Sons, Hoboken, New Jersey.

Nelson, J. D., Thompson, E. G., Schaut, R. W., Chao, K. C., Overton, D. D., and Dunham-Friel, J. S. (2011). “Design procedure and considerations for piers in expansive soils.” *J. Geotech. Geoenviron. Eng.*, 138(8), 945-956.

O’neill, M. W., and Poormoayed, N. (1980). “Methodology for foundations on expansive clays.” *J. Geotech. Eng. Div.*, 106 (GT12), 1345-1367.

Pham, H. Q., Fredlund, D. G., and Barbour, S. L. (2005). “A study of hysteresis models for soil-water characteristic curves.” *Can. Geotech. J.*, 42(6), 1548-1568.

Porter, A. A. (1977). *The mechanics of swelling in expansive clays*, M.S. thesis, Colorado State Univ., Fort Collins, CO.

Randolph, M.F., and Wroth, C.P. (1978). “Analysis of deformation of vertically loaded piles.” *J. Geotech. Eng. Div.*, 104(12), 1465-1488.

Reichler, D. K. (1997). *Investigation of variation in swelling pressure values for an expansive soil*, M.S. thesis, Colorado State Univ., Fort Collins, CO.

Tu, H.Y., and Vanapalli, S.K. (2016). “Prediction of the variation of swelling pressure and 1-D heave of expansive soils with respect to suction using the soil water retention curve as a tool.” *Can. Geotech. J.*, 53(8), 1213-1234.

Zhang, Q.Q., Zhang, Z.M., Yu, F., and Liu, J.W. (2010). “Field performance of long bored piles within piled rafts.” *Proce. Institution of Civil Engineers: Geotechnical Engineering*, 163(6), 293-305.

Zhang, Z.M., Zhang, Q.Q., and Yu, F. (2011a). “A destructive field study on the behavior of pile under tension and compression.” *J. Zhejiang University-Science A*, 12(4), 291-300.

Zhang, Z.M., Zhang, Q.Q., Zhang, G.X., and Shi, M.F. (2011b). “Large tonnage tests on super-long piles in soft soil area.” *Chinese J. Geotech. Eng.*, 33(4), 535-543.

Zhang, Q.Q., and Zhang, Z.M. (2012). “A simplified nonlinear approach for single pile settlement analysis.” *Can. Geotech. J.*, 49(11), 1256-1266.