Coupled Hydrological-Geotechnical Model for Determinine Bearing Capacity and Elastic Settlement of Foundations

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COUPLED HYDROLOGICAL-GEOTECHNICAL MODEL FOR DETERMINING BEARING CAPACITY AND ELASTIC SETTLEMENT OF FOUNDATIONS

A Dissertation
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Doctor of Philosophy
Civil Engineering

by
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ABSTRACT

This dissertation presents a coupled hydrological-geotechnical framework to investigate the performance of shallow and deep foundations under hydrological events such as heavy rainfall and drought. The variation in performance of foundation, interface between the structure and ground surface, is caused by the uncertainties associated with not only the geotechnical parameters but also the hydrological parameters that include intensity and duration of hydrological events and water table depth. The impact of such hydrological events significantly alters the performance of foundations by changing the soil strength and stiffness parameters of subsurface soil which may lead to foundation failures. Such failures can cause damage to the supporting structure. Therefore, to better understand the performance of geotechnical systems under different hydrological events and also to build sustainable and resilient infrastructure systems, the design of geotechnical systems should be carried out in a coupled hydrological-geotechnical manner considering the site-specific geotechnical and hydrological parameters. To this end, a numerical framework is developed based on the partially saturated soil mechanics principles and applied to a number of sites in the United States to show the impacts of hydrological events in the performance of shallow and deep foundations. In this framework, the one-dimensional Richards’ equation is numerically solved to compute the spatial and temporal variation of the degree of saturation and matric suction in subsurface soil due to the site-specific rainfall, evapotranspiration, and water table depth as model boundary conditions. Then, the critical settlement and bearing capacity of foundations (as critical design values) are calculated using the average degree of saturation and matric
suction within the foundation influence zone. It is worth mentioning that two different
design methodologies based on the probabilistic analysis and single extreme hydrological
cycle are considered in the proposed framework to have a better assessment of foundation
performance. The results show that the hydrological parameters have a significant impact
on the performance of shallow and deep foundations, and in general, they improve the
predicted foundation design values obtained from conventional methods in terms of the
settlement and bearing capacity. The proposed method can be used as a decision-making
tool for selecting the suitable design values of foundations in engineering practice.
DEDICATION

To my beloved parents and my wife.
ACKNOWLEDGMENT

I would like to express my sincere gratitude and appreciation to my advisor, Dr. Nadarajah Ravichadran, for his guidance, support, insights, and constant encouragement throughout my Ph.D. studies. He gave me this valuable opportunity to explore interesting topics and develop my research with freedom and enthusiasm along with his thoughtful guidance and support. I would also like to thank and recognize the other members of my doctoral committee, Dr. Ronald D. Andrus, Dr. Ashok Mishra, and Dr. Weichiang Pang, for providing me with additional guidance, valuable ideas, and critique.

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CHAPTER 1
INTRODUCTION

1.1 OVERVIEW AND MOTIVATION

According to NOAA (National Oceanic and Atmospheric Administration), the average surface temperature across the contiguous 48 states has risen at an average rate of 0.14°F per decade and predicts that the air temperature raises about 2°F across the United States by the end of the century (NOAA 2016) (Fig. 1.1a). This increase in air temperature, as the main climatic parameter, has altered the climate pattern which leads to increase the frequency of severe hydrological events such as heavy rainfall, flood, and drought across the United States. The recent research by U.S. Global Research Center (updated from Karl et al. 2009) indicates that not only the frequency of heavy rainfall has been increased from 1958 to 2012, but also the intensity and duration of this event have been increased across the United States (Fig. 1.1b). Increasing the frequency of heavy rainfall subsequently increases the chance of occurrence for a flood event. The 2014 New York flood, 2015 Missouri flood, 2016 Oklahoma flood, 2016 Louisiana flood, 2017 California flood and 2017 Houston flood are some of the examples of severe floods occurred recently in the United States.

The impact of such hydrological events significantly affects the performance of many earthen structures in particular shallow and deep foundations through changing strength and stiffness of subsurface soil which may lead to foundation failures under different hydrological events. Such failures can cause damage to the superstructure and subsequently cause human lives and financial losses (Orense 2004, Varden 2015).
Since the conventional geotechnical design methods ignore the impact of hydrological events, several large-scale studies have been conducted to assess various aspects of climate change on the performance of geotechnical systems. However, there is still a clear gap in the state of knowledge in term of evaluating the resiliency of foundations against different hydrological events. Thus, to understand the performance of foundation under hydrological events and to build sustainable and resilient infrastructure systems, the design of foundations should be carried out systematically in a coupled hydrological-geotechnical manner to incorporate site-specific hydrological and geotechnical parameters for more realistic and accurate design. A better design procedure will require a better understanding of partially saturated soil mechanic principles and utilization of site-specific hydrological parameters including rainfall, evapotranspiration and water table data.
1.2 OBJECTIVES

The objectives of this study are: (1) to develop a coupled hydrological-geotechnical framework for various geotechnical systems subjected to heavy rainfall, evapotranspiration, and water table depth (all these will be referred to as hydrological loads in this document), (2) to propose different design methodologies based on the probabilistic analysis and single extreme hydrological cycle, and (3) to demonstrate the application of the proposed methods for shallow foundation, drilled shaft, and driven pile at a number of sites in the United States with significantly different climatic conditions.

1.3 DISSERTATION ORGANIZATION

The dissertation consists of six chapters. The introduction is presented in the current chapter, Chapter 1, to introduce and organize the entire dissertation. Chapters 2, 3 and 4 present the coupled hydrological-geotechnical design framework for the shallow foundation, drilled shaft, and driven pile based on the probabilistic approach. In Chapter 5, the coupled hydrological-geotechnical framework of a shallow foundation is expanded to investigate the performance of foundation considering site-specific single extreme hydrological cycle. The conclusion of the dissertation and future work are presented in Chapter 6.

In Chapter 2, the coupled hydrological-geotechnical framework is introduced and applied for the design of shallow foundation subjected to historical heavy rainfall and water table depth. To apply the proposed framework, first, a mathematical model is developed based on the Richards equation for computing the spatial and temporal
variation of the degree of saturation and matric suction in subsurface area, then the mathematical model is solved using the historical heavy rainfall and water table as the top and bottom boundary conditions, respectively. Afterward, the Monte Carlo simulation is employed to randomly generate rainfall intensity and water table depth from their respective probability distributions. In the next step, the average matric suction and degree of soil saturation within the influence zone of the foundation is computed from the results of the solution of Richards’ equation. Then, the ultimate bearing capacity and settlement are calculated using the equations that consider the effects of matric suction and degree of saturation through changing the soil strength and stiffness parameters. Finally, the design values of the foundation are determined and selected based on the mean of the best-fitted probability distribution to the average degree of saturation (or matric suction) calculated from the previous step.

In Chapter 3, the newly developed model is applied for the design of drilled shaft subjected to historical resultant infiltration (including heavy rainfall and evapotranspiration) and water table depth. To apply the proposed framework to the design of drilled shaft, first, the numerical solution of the Richards equation is considered to capture the variation of the degree of saturation and matric suction along the shaft length and below the tip considering the resultant infiltration and water table depth as upper and lower boundary conditions, respectively. Then, Monte Carlo simulation is used to randomly generate the input variables associated with the hydrological parameters from its probability distribution to compute the axial capacity and elastic settlement of various drilled shafts in partially saturated soil for the study areas. Finally, the design
axial capacity and settlement of the drilled shafts are selected based on the mean of the best-fitted probability distribution to the average matric suction beneath the subsurface.

In Chapter 4, the newly developed model is applied for the design of driven pile subjected to historical resultant infiltration (including heavy rainfall, evapotranspiration, and surface runoff) and water table depth. To apply the proposed framework, first, the numerical solution of the Richards equation is considered to capture the variation of the degree of saturation and matric suction along the pile length considering the resultant infiltration and water table depth as upper and lower boundary conditions, respectively. Then, Monte Carlo simulation is used to randomly generate the input variables associated with the hydrological data from its probability distribution to compute the axial bearing capacity and elastic settlement of various driven piles in partially saturated soil for the study areas. Finally, the design axial capacity and settlement of the drilled shafts are selected based on the mean of the best-fitted probability distribution to the average matric suction (or degree of saturation) beneath the ground surface.

In Chapter 5, the newly developed coupled hydrological-geotechnical framework is updated to investigate the performance of shallow foundation subjected to a single extreme hydrological cycle and corresponding hydrological loads. To apply the proposed framework, first, a hydrological-geotechnical model is developed for the shallow foundation. Then, the site-specific extreme hydrological cycle and the corresponding hydrological loads are determined based on the Standardized Precipitation Evapotranspiration Index (SPEI) (Vicente-Serrano 2010). In the next step, the Richards equation is used to compute the spatial and temporal variation of water content and
pressure head of the underlying soil due to site-specific hydrological loads as the updated model boundary conditions. Afterward, the average matric suction and soil degree of saturation are computed within the foundation influence zone during the extreme hydrological cycle. In the next step, the computed average degree of saturation and matric suction are used to calculate the ultimate bearing capacity and elastic settlement using the developed hydrological-geotechnical model. Finally, the critical values of the settlement and ultimate axial capacity are determined as the design values of the shallow foundation.

Finally, the overall summary of the conclusions and the recommendations for future research studies are provided in Chapter 6 of this dissertation.

1.4 CONTRIBUTIONS OF THIS DISSERTATION

The major contributions of this dissertation are:

- A fully coupled hydrological-geotechnical framework is developed based on the partially saturated soil mechanics principles to incorporate the site-specific hydrological loads into the conventional design procedure of foundations.
- The proposed framework considers the impact of hydrological events including heavy rainfall and drought on foundation performance.
- The proposed framework presents two different design methodologies based on the probabilistic analysis and single extreme hydrological cycle to assess the performance of geotechnical systems accurately.
The proposed framework may significantly improve the sustainability and resiliency of infrastructure when applied in foundation design.

1.5 REFERENCES


CHAPTER 2

DESIGN OF SHALLOW FOUNDATION CONSIDERING SITE-SPECIFIC RAINFALL AND WATER TABLE DATA–THEORETICAL FRAMEWORK AND APPLICATION

2.1 INTRODUCTION

According to the National Climatic Data Center (NCDC), nearly 30 percent of the contiguous U.S. experienced moderate to severe hydrological events such as heavy rainfall, flood, and drought which ultimately influence the spatial and temporal variation of the degree of saturation of the subsurface soil. The effect of degree of saturation of the soil on its mechanical and flow behaviors are well documented in recent years. These findings indicate that the design of any geotechnical systems must be performed considering the hydrological parameters to accurately quantify their performance.

Shallow foundation, a common type of foundation used to support small to medium size of structures and transfer its loads to the near-surface soil, is commonly designed for the worst case geotechnical conditions. That is, the soil is fully saturated with the water table at the ground surface even if the historical water table is well below the influence depth of the shallow foundation and expected to be the same during the lifetime of the structure. This way of foundation design is too conservative since the underlying soil is mostly under partially saturated condition and water table fluctuates with time. Recent case studies indicate that the variation of the soil degree of saturation and the matric suction significantly affect the soil shear strength (Lu and Likos 2004,
Fredlund et al. 2012, Briaud 2013). To account for the effect of the degree of saturation and matric suction, numerous shear strength equations have been developed for predicting or estimating the shear strength of partially saturated soil. Many of these equations use the Soil Water Characteristic Curve (SWCC) as the controlling parameter together with the saturated shear strength parameters to predict the shear strength of soil under partially saturated condition (Fredlund et al. 1978, Vanapalli et al. 1996, Oberg and Sallfors 1997, Garven and Vanapalli 2006, Guan et al. 2010, Sheng 2011, Borana et al. 2015). Furthermore, various research studies have been conducted to investigate the impact of the partially saturated soil on the behavior and shear strength of soil interface with other construction materials (Zhan and Ng 2006, Khoury et al. 2010, Borana et al. 2016). The change of soil shear strength under partially saturated condition subsequently affects the bearing capacity and settlement of different types of foundation. The contribution of partially saturated soil shear strength towards the bearing capacity of shallow foundation has been the subject of fairly extensive study for coarse-grained soils (Steensen-Bach et al. 1987, Costa et al. 2003, Mohamed and Vanapalli 2006, Vanapalli and Mohamed 2007) and fine-grained soils (Schnaid et al. 1995, Oh and Vanapalli 2009, Oh and Vanapalli 2013). In terms of foundation settlement, numerous research studies have been undertaken to study the effect of matric suction on soil modulus of elasticity using model footing and plate load tests which lead to develop various semi-empirical models for investigating the variation of soil stiffness in partially saturated soil (Agarwal and Rana 1987, Costa et al. 2003, Vanapalli and Oh 2010, Vanapalli and Oh 2010, Vanapalli and Adem 2013). Thus, it can be concluded that the deterministic design
approach is either conservative or non-conservative, depending on the site condition where the near-surface soil is initially partially saturated during the design life of the structure, and it is highly affected by hydrological events such as rainfall. A better design procedure will require a thorough understanding of the behavior of partially saturated soil and utilization of site-specific hydrological and geotechnical conditions. Illustrated in Fig. 2.1 is a shallow foundation with a hydrological cycle that changes the degree of saturation of the soil within the influence zone (1.5*foundation width (B)).

![Figure 2.1. Hydrological cycle and its influence on the foundation](image)

In recent years, a number of efforts have been undertaken to assess the effect of hydrological events on the mechanical behavior of partially saturated soils. These research mostly focused on changing the soil shear strength and soil inter-particle force under hysteresis wetting and drying front of underlying soil due to hydrological events (Han et al. 1995, Nishimura and Fredlund 2002, Rahardjo et al. 2004, Thu et al. 2006, Melinda et al. 2004). In addition to the studies mentioned above, some research has addressed the impact of the hydrological events on the behavior of various geotechnical
systems. Most of these studies investigated the slope stability problem under different flux conditions and soil types through either numerical method or experimental tests (Rahardjo et al. 1995, Kim et al. 2004, Lu and Likos 2006, Vahedifard et al. 2015). However, limited studies have been performed for the shallow and deep foundations on this subject (Vahedifard and Robinson 2016, Kim et al. 2017, Ravichandran et al. 2017, Mahmoudabadi and Ravichandran 2017).

The objectives of this study are to develop a procedure for coupling site-specific hydrological parameters with geotechnical parameters to find a more realistic design approach and also to understand the impacts of hydrological parameters on the bearing capacity and settlement of shallow foundation in a probabilistic manner. The procedure requires several steps. First, the flow of water into the soil is modeled using Richards’ equation and solved considering the infiltration and water table as the top and the bottom boundary conditions, respectively. Next, Monte Carlo simulation is used to randomly generate the input variables from the probability distribution of water table and rainfall intensity which lead to computing the average degree of saturation and matric suction within the influence zone of the foundation. The degree of saturation and matric suction are then used to calculate the bearing capacity and elastic settlement of a shallow foundation. Finally, the design values of the foundation are selected based on the mean of the best-fitted probability distribution to the average degree of saturation (or matric suction) within the foundation influence zone. Two sites in the United States are selected to demonstrate the procedure.
2.2 A FRAMEWORK FOR COUPLING GEOTECHNICAL AND HYDROLOGICAL DATA IN DESIGN PROCESS

The incorporation of the hydrological parameters into a conventional shallow foundation design entails: (1) developing a mathematical model for computing the spatial and temporal variation in the degree of saturation and matric suction, (2) implementing and solving the mathematical model using the resulting infiltration and water table as the top and bottom boundary conditions, respectively, (3) employing Monte Carlo simulations to randomly generate rainfall intensity and water table depth from their probability distribution, (4) computing the average matric suction and degree of soil saturation within the influence zone of the foundation considering generated input data, (5) computing the ultimate bearing capacity and settlement using the equations that consider the effects of matric suction and degree of saturation, and (6) determining the design values of the foundation that were selected based on the mean of the best-fitted probability distribution to the average degree of saturation (or matric suction) calculated from the previous step within the foundation influence zone. The details of these six steps are presented below.

2.2.1 Mathematical Model for Flow of Water in Soil and its Solution

Procedure

The one-dimensional vertical movement of water through the partially saturated soil was represented by Richards’ equation (Richards 1931) which is shown in Eq. 2.1.
\[
\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z} \left[ K \left( 1 + \frac{\partial \psi}{\partial z} \right) \right]
\]  

(2.1)

where \( t \) is the time, \( z \) is the depth from the ground surface, \( \theta \) is the volumetric water content, \( K \) is the hydraulic conductivity of partially saturated soil, \( \psi \) is the pressure head, and \( \frac{\partial \psi}{\partial z} \) is the hydraulic gradient. Although the problem considered in this study is three-dimensional in nature, this one-dimensional model shown above is reasonably accurate for predicting the vertical movement of the water (Celia et al. 1990, Van Dam et al. 2000). Since the pressure head, \( \psi \), is considered as the primary variable to be determined using the Richards equation here, the \( \theta \) and \( K \) must be expressed as functions of \( \psi \). The \( K \) of the partially saturated soil is expressed as the product of saturated hydraulic conductivity (\( K_{\text{sat}} \)) and relative hydraulic conductivity (\( K_r \)), i.e., \( K = K_{\text{sat}} K_r \). A Soil Water Characteristic Curve (SWCC) is then used to express \( K_r \) in terms of \( \psi \). The other parameter in the Richards equation, \( \theta \), is also expressed in terms of \( \psi \) using the same SWCC. Among the many SWCCs and corresponding relative hydraulic conductivity functions available in the literature, the equations proposed by van Genuchten (1980) were used in this study. The equations of SWCC and corresponding \( K_r \) functions are given in Eqs. 2.2 and 2.3, respectively,

\[
\theta(\psi) = \theta_r + \frac{(\theta_s - \theta_r)}{\left[ 1 + (\alpha \psi)^n \right]^{(1-1/n)}}
\]  

(2.2)
where $\alpha$ and $n$ are SWCC fitting parameters, $\theta_s$ is saturated water content, and $\theta_r$ is residual water content.

There are numerous procedures available in the literature for solving the Richards equation for a given set of initial and boundary conditions (van Genuchten 1982, Feddes et al. 1988, Celia et al. 1990, Warrick 1991, Zaidel and Russo 1992, Baker 1995, Pan et al. 1996, Romano et al. 1998, Van Dam et al. 2000). In this study, the Finite Volume Method (FVM) was used to solve that equation with the initial and boundary conditions. The spatial and temporal discretization and the summary of the solution procedure are given in Eqs. 2.4 to 2.10 and the spatial and temporal discretization is graphically shown in Fig. 2.2.

$$K_r(\psi) = \frac{1 - (\alpha \psi)^{n-1} \left[ 1 + (\alpha \psi)^{n} \right]^{-(1-1/n)}}{\left[ 1 + (\alpha \psi)^{n} \right]^{-(1-1/n)/2}}$$ (2.3)
In order to solve the Richards equation, the equation must first be written in terms of the pressure head for a one-dimensional vertical infiltration. Thus, $\frac{\partial \theta}{\partial t}$ is expressed as

$$C \frac{\partial \psi}{\partial t} = \frac{\partial \psi}{\partial t}$$

and substituted in Eq. 2.1,

$$C \frac{\partial \psi}{\partial t} - \frac{\partial}{\partial z} \left( K \frac{\partial \psi}{\partial z} + K \right) = 0$$

(2.4)

where $C$ is the specific moisture capacity ($\frac{\partial \theta}{\partial \psi}$). Next Eq. 2-4 is integrated with respect to the time ($t$) and depth ($z$) as follows,

$$\int_{i-1/2}^{i+1/2} \int_{t-\Delta t}^{t+\Delta t} C \frac{\partial \psi}{\partial t} dt dz - \int_{i-1/2}^{i} \int_{t-\Delta t}^{t+\Delta t} \frac{\partial}{\partial z} \left( K \frac{\partial \psi}{\partial z} + K \right) dz dt = 0$$

(2.5)
where \( i \) is the node number (start from ground surface) and \( \Delta t \) is the time interval. The left-hand side of the integration is rewritten in a discretized form, which is expressed in Eq. 2.6.

\[
\int_{i-1/2}^{i+1/2} \int_{t}^{t+\Delta t} C \frac{\partial \psi}{\partial t} \, dt \, dz = \int_{i-1/2}^{i+1/2} \left[ C \left( \psi_i^{t+\Delta t} - \psi_i^t \right) \right] \, dz = C_{i}^{t+\Delta t} \left( \psi_{i}^{t+\Delta t} - \psi_{i}^{t} \right) \Delta z
\]  

(2.6)

The right-hand side is first solved for spatial variation, as shown in Eq. 2.7, and then, the integration is discretized into the temporal form, as shown in Eq. 2.8.

\[
\int_{i-1/2}^{i+1/2} \int_{t}^{t+\Delta t} \left( K \frac{\partial \psi}{\partial z} + K \right) \, dz \, dt = \int_{i-1/2}^{i+1/2} \left[ \left( K \frac{\partial \psi}{\partial z} + K \right)_{i+1/2}^{t+\Delta t} + \left( K \frac{\partial \psi}{\partial z} + K \right)_{i-1/2}^{t} \right] \, dt
\]  

(2.7)

\[
C_{i}^{t+\Delta t} \left( \psi_{i}^{t+\Delta t} - \psi_{i}^{t} \right) \Delta z = \frac{\Delta z}{K_{i+1/2}^{t+\Delta t} - \psi_{i+1/2}^{t+\Delta t}} - \left( K_{i-1/2}^{t+\Delta t} - \psi_{i-1/2}^{t+\Delta t} \right) \Delta t
\]  

(2.8)

Considering \( m \) as the iteration level and pressure head at iteration \( m+1 \) as the unknown value and \( \Delta z \) as the depth interval, the complete spatial and temporal form of Richards’ equation is expressed in Eq. 2.9.

\[
C_{i}^{t+\Delta t,m} \left( \psi_{i}^{t+\Delta t,m+1} - \psi_{i}^{t} \right) \Delta z = \frac{\Delta z}{K_{i+1/2}^{t+\Delta t} - \psi_{i+1/2}^{t+\Delta t,m+1}} - \left( K_{i-1/2}^{t+\Delta t} - \psi_{i-1/2}^{t+\Delta t,m+1} \right) \Delta t
\]  

(2.9)

Finally dividing the Eq. 2.9 by \( \Delta z \) and rearranging the formulation, the final form of the Richards equation is written as follow (Eq. 2.10).
\[ C_{i}^{t+\Delta t,m} \left( \psi_{i}^{t+\Delta t,m+1} - \psi_{i}^{t} \right) = \left[ \left( K_{i+1/2}^{t+\Delta t} \frac{\psi_{i}^{t+\Delta t,m+1} - \psi_{i-1}^{t+\Delta t,m+1}}{(\Delta z)^{2}} - K_{i-1/2}^{t+\Delta t} \frac{\psi_{i}^{t+\Delta t,m+1} - \psi_{i+1}^{t+\Delta t,m+1}}{(\Delta z)^{2}} \right) + \left( K_{i+1/2}^{t+\Delta t} - K_{i-1/2}^{t+\Delta t} \right) \right] \Delta t \] (2.10)

Since the obtained numerical solution of the Richards equation is a time-consuming process, a MATLAB code was developed to solve the Eq. 2.10. The code was then installed on the Clemson University supercomputer, the Palmetto Cluster, to perform the simulation (considering a large number of input scenarios systematically using Monte Carlo method) in a reasonable series of runtimes. The variation in both hydraulic head and water content, as explained above, can then be solved in terms of the ultimate bearing capacity and elastic settlement for the partially saturated soil.

### 2.2.1.1 Boundary and initial conditions

The one-dimensional water infiltration into the soil profile with a specific water table is shown in Fig. 2.3 for purposes of illustrating the problem and the boundary conditions. The top and bottom boundary conditions are displayed and located on the ground surface and the water table level, respectively.

![Figure 2.3. Partially saturated zone of a soil profile with vertical infiltration](image)

Figure 2.3. Partially saturated zone of a soil profile with vertical infiltration
In this study, both the pressure head and flux boundary conditions were applied at the top boundary depending upon the magnitude of the rainfall and the specific moisture capacity. In the case of ponding, when the infiltration rate is greater than the saturated hydraulic conductivity, the pressure head type boundary condition ($\psi$) is applied. When all water infiltrates into the soil, the flux type boundary condition, resultant infiltration intensity ($q$), is applied, which is computed using the actual rainfall data. In addition, the soil was assumed to be at the residual condition (residual stage in SWCC) at the beginning of each simulation. Since the water table location and resultant infiltration vary with climatic conditions for each specific location, appropriate values must be determined in a probabilistic manner considering historical rainfall and the water table data.

### 2.2.2 Model Verification

The accuracy of the presented framework should be tested through a comparison of the results of numerical Richards’ equation, which is the main algorithm of the proposed procedure, with other validated solution methods. In order to accomplish that comparison, a generalized solution developed by Celia et al. (1990) for pressure head boundary condition was used to verify the water infiltration process of the proposed approach under a given pressure head boundary condition (Fig. 2.4). All constants, which are used to verify the proposed model, are listed in Table 2.1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>$\alpha$</th>
<th>$\theta_s$</th>
<th>$\theta_r$</th>
<th>$n$</th>
<th>$m$</th>
<th>$K_{sat}$ (cm/s)</th>
<th>$\psi_{top}$ (cm)</th>
<th>$\psi_{bottom}$ (cm)</th>
<th>$t$ (day)</th>
<th>$\Delta t$ (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Values</td>
<td>0.0335</td>
<td>0.368</td>
<td>0.102</td>
<td>2</td>
<td>0.5</td>
<td>0.00922</td>
<td>-75</td>
<td>-1000</td>
<td>1</td>
<td>144</td>
</tr>
</tbody>
</table>
2.2.3 Bearing Capacity Criteria

The contribution of the matric suction and the degree of saturation towards the bearing capacity of partially saturated soils has been the subject of fairly extensive study (Steensen-Bach et al. 1987, Costa et al. 2003, Mohamed and Vanapalli 2006, Vanapalli and Mohamed 2007, Oh and Vanapalli 2009, Oh and Vanapalli 2013). Among the many available equations, the ultimate bearing capacity equation proposed by Vanapalli and Mohamed (2007) was used in this study to predict the nonlinear variation of ultimate bearing capacity in partially saturated soils \( q_{u(unsat)} \) with respect to the matric suction for the shallow foundation (Eq. 2.11),

\[
q_{u(unsat)} = \left[ c' + (u_a - u_w) \right] b \left( \tan \phi' - S' \tan \phi' \right) + (u_a - u_w) \tan \phi' \right] N_c F_{cs} F_{cd} + \gamma D N_q F_{qs} F_{qs} + 0.5 \gamma B N_{qs} F_{qs} F_{qs}
\]

(2.11)
where \( c' \) is the effective cohesion intercept, \( \gamma \) is the moist unit weight of soil, \( D \) is the foundation depth, \( B \) is the foundation width, \( N_c, N_q \) and \( N_r \) are the non-dimensional bearing capacity factors that are functions of the soil effective friction angle \( \phi' \), \( F_s \) and \( F_d \) are the shape and depth factors, respectively, \( (u_a-u_w)_b \) is the air entry value which is computed from the SWCC, \( (u_a-u_w)_{avg} \) is the average matric suction within the foundation influence zone, \( S \) is the degree of saturation, and \( \psi_a \) is the shear strength fitting parameter which is expressed in Eq. 2.12 (Vanapalli and Mohamed 2007).

\[
\psi_a = 1.0 + 0.34(I_p) - 0.0031(I_p)^2
\]  

(2.12)

where \( I_p \) is the plasticity index of the soil. The average matric suction in the above bearing capacity equation is calculated using the Eq. 2.13,

\[
(u_a-u_w)_{avg} = \frac{\sum_{i=1}^{p}[(u_a-u_w)_i]}{p} = \frac{\sum_{i=1}^{p}[-\psi_a \gamma_w]}{p} = -\psi_{avg} \gamma_w
\]  

(2.13)

where \( (u_a-u_w)_i \) is the matric suction at \( i^{th} \) node, \( p \) is the last soil node within the foundation influence zone, \( \psi_i \) is the pressure head at \( i^{th} \) node, \( \psi_{avg} \) is the average pressure head within the foundation influence zone, and \( \gamma_w \) is the unit weight of water. The average matric suction and degree of saturation are key variables, which affect the ultimate bearing capacity of the soil within the influence zone of foundation and were calculated using the procedure described above.
2.2.4 Settlement Criteria

The elastic settlement of the foundation was calculated using the simplified equation shown in Eq. 2.14 (Bowles 1987),

\[ S_e = q_0 (\alpha_s B') \frac{1 - \mu_s^2}{E_s} I_x I_f \]  

(2.14)

where \( S_e \) is the elastic settlement of the foundation, \( q_0 \) is the net pressure at the bottom of the foundation due to applied structural load, \( \alpha_s \) is a non-dimensional parameter that depends on the point at which settlement is calculated for a flexible foundation, \( B' \) is the effective dimension of the foundation, \( \mu_s \) is the Poisson’s ratio, \( I_x \) and \( I_f \) are factors associated with the shape and depth of the foundation, respectively, and \( E_s \) is the average modulus of elasticity of the soil within the influence zone. Of all these parameters, \( E_s \) is the only parameter that is affected by the degree of saturation and matric suction of the soil within the influence zone. Since the degree of saturation and the matric suction are computed following the procedure described before, the elastic settlement can be computed if \( E_s \) is expressed as a function of the degree of saturation and matric suction.

Various empirical equations have been proposed to predict the elastic modulus of soil as a function of matric suction and degree of saturation (Steensen-Bach et al. 1987, Schnaid et al. 1995, Costa et al. 2003, Rojas et al. 2007, Oh et al. 2009, Vanapalli and Oh 2010, Vanapalli and Adem 2013). In this study, the equation proposed by Oh et al. (2009), shown in Eq. 2.15, was used to estimate the modulus of elasticity in partially saturated coarse-grained soils (\( E_{s(unsat)} \)).
\[ E_{s(sat)} = E_{s(sat)} \left[ 1 + \alpha_e \frac{u_o - u_w}{P_{atm}/101.3} S^\beta_e \right] \] (2.15)

where \( E_{s(sat)} \) is the modulus of elasticity under the saturated condition at strain level of 1\%, \((u_o-u_w)\) is the matric suction, \(\alpha_e\) and \(\beta_e\) are fitting parameters, and \(P_{atm}\) is atmospheric pressure. For coarse- and fine-grained soils, the recommended fitting parameter, \(\beta_e\) is equal to 1 and 2, respectively. Also, the fitting parameter \(\alpha_e\) depending upon the plasticity index \((I_p)\) can be computed using the following empirical equation (Eq. 2.16), developed by Oh et al. (2009).

\[ \frac{1}{\alpha_e} = 0.5 + 0.312(I_p) + 0.109(I_p)^2 \quad (0 \leq I_p (\%) \leq 12) \] (2.16)

It should be noted that the consolidation settlement will also affect the total settlement of the shallow foundation at a given time after the occurrence of hydrological events. However, the consolidation settlement was not considered in this study.

2.3 APPLICATION OF PROPOSED METHOD TO STUDY AREAS

The application of the proposed procedure requires the computation of the average matric suction and the degree of saturation within the influence zone of shallow foundation using the site-specific historical rainfall and water table records. These two random variables are considered as the boundary conditions that change with return periods as will be discussed in more detail in the rainfall and water table distribution section. Since these variables have time-independent uncertainty for any specific location, the design process of shallow foundation should be carried out in a probabilistic
manner to account the site-specific uncertainty of historical rainfall and water table depth. In another word, if the highest historical rainfall intensity and higher water table depth are considered as a worst case of foundation design, the probability of concurrence of these events simultaneously is significantly low during the lifetime of structure which will lead to an overdesign approach. Thus, to a better assessment of shallow foundation performance, the design procedure should consider all the event possibilities through the Probability Density Function (PDF) of site-specific hydrologic parameters.

In order to perform the probabilistic analysis, first, the distributions of the historical rainfall and water table were used to generate random input variables to serve as the boundary conditions. The Monte Carlo simulation technique was then used to generate 10,000 random input variables to compute the bearing capacity and elastic settlement of a shallow foundation in the partially saturated soil. Finally, the mean of the best-fitted probability distribution to the average degree of saturation or matric suction was selected to find the design values of shallow foundation for study sites. Although it is possible to incorporate into the inherent randomness of shear strength parameters of the soil in the analysis, here the shear strength variables were kept as constants to allow for comparisons between the saturated condition and partially saturated condition with site-specific hydrological loads. However, the unit weight of the soil changed with varying degrees of saturation computed within the influence zone after each simulation. The flowchart of the procedure employed in this study is presented in Fig. 2.5.
2.3.1 Study Locations

Two sites were selected to demonstrate the proposed procedure and to show the variation of the ultimate bearing capacity and elastic settlement with a degree of saturation and matric suction. The first case study site was located in Victorville, California, which was selected for its semi-arid climate condition (with Aridity index (AI) = 0.43) and the availability of van Genuchten SWCC parameters for the Adelanto Loam soil type found in this region. The SWCC parameters of the Adelanto Loam (SM) were taken from the report by Zhang (2010), and the soil strength parameters of the site were obtained from a geotechnical report by Kleinfelder (Chowdhury 2006). The second case study site was located in Levelland, Texas with arid climatic condition (AI = 0.05). The soil strength parameters of this site were obtained from a geotechnical report provided by Amarillo Testing and Engineering, Inc. (Gonzalez 2009).
Figure 2.5. Simulation flowchart to incorporate the hydrological data into shallow foundation design
Despite the availability of soil parameters for this region, the van Genuchten parameters did not exist. Therefore, the van Genuchten parameters for Levelland were obtained from the class average value of hydraulic parameters for the twelve USDA (the U.S. Department of Agriculture) textural classes from the program Rosetta (Schaap 2000). The soil classification criteria in the geotechnical report were then used to determine the class best suited for the Levelland soil and was considered to be in the sandy-clay (SC-SM) textural class. The specified van Genuchten SWCC model for these two locations are presented in Fig. 2.6. In addition, the selected geotechnical parameters of both locations are listed in Table 2.2.

Table 2.2. Geotechnical and SWCC fitting parameters for the Victorville, CA and Levelland, TX sites

<table>
<thead>
<tr>
<th>Geotechnical and SWCC fitting parameters</th>
<th>Victorville, CA</th>
<th>Levelland, TX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil type</td>
<td>SM</td>
<td>SC-SM</td>
</tr>
<tr>
<td>Dry unit weight, $\gamma_d$ (kN/m$^3$)</td>
<td>16.20</td>
<td>18.56</td>
</tr>
<tr>
<td>Void ratio, $e$</td>
<td>0.605</td>
<td>0.401</td>
</tr>
<tr>
<td>Effective friction angle, $\phi'$ (deg.)</td>
<td>33</td>
<td>31</td>
</tr>
<tr>
<td>Effective cohesion intercept, $c'$ (kPa)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Plasticity index ($I_p$)</td>
<td>5</td>
<td>8</td>
</tr>
</tbody>
</table>
2.3.2 Historical Rainfall and Water Table Data and their Probability Distributions

The rainfall data for Victorville, CA and Levelland, TX were obtained from the National Climatic Data Center (NCDC) which records daily rainfall values. In this study, the annual maximum series were used and constructed by extracting the highest precipitation for a particular return period in each successive year. The maximum annual rainfall has been tabulated in millimeter for a return period of 76 years for both study sites. The depth of the water table is another factor which affects the matric suction and the degree of saturation of a partially saturated soil within the foundation influence zone. The required data were taken from the U.S. Geological Survey (USGS 2016) and for the same return period which was assumed for rainfall data. To determine the best fitting distribution for the annual maximum rainfall and water table data, the probability paper plotting technique was used. Type I Extreme Largest (Gumbel distribution), the Type II
Extreme Largest (Frechet distribution), and the Type III Extreme Largest (Weibull distribution) were checked for the best fit, and the Gumbel distribution was deemed the best regression based on R-squared test (R²-value). The Gumbel probability paper distribution parameters, mode (\(\mu_n\)) and standard deviation (\(\beta_n\)), can be determined using Eq. 2.17,

\[
x_j = y_j \beta_n + \mu_n = -\ln \left( -\frac{j}{r+1} \right) \beta_n + \mu_n
\]

where \(j\) is the data index (arranged in increasing order), \(x_j\) is the annual maximum historical rainfall or water table data, \(y_j\) is the linearized form of the cumulative density function of Gumbel distribution, and \(r\) is the number of data points. The probability plots of the rainfall and water table data based on the Gumbel distribution is shown in Figs. 2.7 and 2.8, respectively for both study sites.

Figure 2.7. Type I Extreme Largest (Gumbel distribution) for water depth and rainfall data for Victorville
2.3.3 Hydrological Loads

Two hydrological loads were considered in the proposed procedure, rainfall intensity, and water table depth. These loads are applied on the top and bottom boundary conditions, respectively. For each simulation, the rainfall and water table randomly selected from its distribution. The selected rainfall value is daily precipitation. To apply an in-flux rate on the top boundary condition (at the first node), the hourly rainfall intensity is calculated. While for the bottom boundary condition, the depth of water table is applied as pressure head. Finally, the average degree of saturation and matric suction within the influence zone for each foundation size were computed by applying the rainfall intensity and water table depth predicted by Eq. 2.17 for 10,000 random cases using Monte Carlo method. The duration of the rainfall was assumed to be three days to simulate the heavy rainfall condition.
2.3.3.1 Resultant infiltration-top boundary condition

The resultant infiltration can be computed by subtracting the surface runoff and evapotranspiration from the rainfall intensity data for each site. By applying the surface runoff and evapotranspiration to the model, the intensity of resultant infiltration decreases and this leads to a lower infiltration rate on the top boundary condition and subsequently lower degree of saturation for the entire site soil profile. In this study, in order to consider the worst case design scenario for the shallow foundation, the effect of surface runoff and the evapotranspiration were ignored, and it is assumed that all the rainfall infiltrates into the ground. However, it should be noted that the ponding effect was already incorporated into the framework and can be applied depending on the problem’s condition. Also, the runoff can be considered in the proposed procedure by quantifying and subtracting the value from the total rainfall intensity. Similarly, the evapotranspiration can be easily computed based on the Hamon (1961) method in terms of potential evapotranspiration (PET) and subtracted from the rainfall intensity.

2.3.4 Structural Load and Foundation Size

Strip foundations with width, \( B = 1.0, 1.5, \) and \( 2.0 \) m located at depths, \( D = 0.5, 0.75, \) and \( 1.0 \) m were used in this study to investigate the infiltration effect for different foundation influence zones. A uniform load of 200 kN/m was applied to all the cases studied.
2.4 RESULTS AND DISCUSSION

In order to compute the design values of each foundation, 10,000 simulations were analyzed considering random input variables (rainfall intensity and water table depth) to model various boundary conditions. These random values were generated using the Monte Carlo simulation based on the probability distribution of historical rainfall and water table data for each region. The Richards equation was then used to determine the spatial and temporal variations of degree of saturation within the foundation influence zone for each simulation. The analysis was performed considering the calculated resultant infiltration and the water table level of both the Victorville and Levelland site locations. A sample spatial variation of the degree of saturation within the subsurface is shown in Fig. 2.9, after considering the water table depth as the bottom boundary condition after 3 days of continuous rainfall. Note that the inherent soil characteristics and SWCC of each site location have a direct effect on the water infiltration process and subsequently depth to which the water penetrates. It can be seen that the water penetrates utmost 1.6 m into the subsurface area for Victorville, while this depth is almost 0.4 m for Levelland. The location of the water table is characterized by a tendency of remaining unchanged during the analysis for all the simulations because the final depth of the infiltrated water is above the water table level. As indicated by the findings in the figure, the various resultant infiltration and water table depth change the depth of infiltrated water and subsequently varies the matric suction and soil stiffness of subsurface, specifically that area close to the surface. This change, in turn, affects the ultimate bearing capacity and the elastic settlement of the shallow foundation.
After completion of each simulation for 3-day rainfall, the average value of the matric suction was calculated for the entire depth of the foundation influence zone (Eq. 2.13). Similarly, the average value of the degree of saturation is also calculated for the same depth. Then, the average matric suction and its corresponding degree of saturation are considered as the matric suction and the degree of saturation value of the site for calculating the ultimate bearing capacity and settlement. This process was performed for each foundation size over 10,000 simulations, and the results of the average matric suction and its corresponding degree of saturation for all different cases are plotted in Fig. 2.10 for both site locations. Note that the infiltration process is much more rapid in the Victorville than the Levelland region. This is because of the inherent differences in
the soil properties for these two locations, as previously discussed in the section entitled “Study Locations”. After three days of continuous rainfall, the degree of saturation in Victorville was between 55% and 84%, and between 25% and 35% for Levelland. This huge difference is caused by the existence of the fine-grained soil in the Levelland region which decreases the soil porosity and decreases the soil permeability. Since the water does not infiltrate deeply in the Levelland subsurface area, the saturation profile remains near constant within the influence zone of the footings, and a constant matric suction profile is derived for this region.

A change in the matric suction and degree of saturation affect the soil shear modulus, which in turn directly influences the elastic settlement of the shallow foundation. In general, an increase in the matric suction (or a decrease in the degree of saturation) has a considerable effect on reducing the foundation settlement.

![Figure 2.10. Average variation of matric suction with different degrees of saturation considering all various studied cases](image-url)
The variation of the elastic settlement for various degrees of saturation and matric suction for both of the study sites are shown in Figs. 2.11 and 2.12. Each line in the figures is produced by 10,000 analysis considering the random resultant infiltration intensity and water table depth as the top and bottom boundary condition, respectively which is already discussed. For each analysis, the spatial variation of the degree of saturation and matric suction within the foundation influence zone, similar to Fig. 2.9, was determined. Then, the average degree of saturation and matric suction (Eq. 2.13) were calculated within the influence zone of foundation and were used to compute the modulus of elasticity (Eq. 2.15) and subsequently the elastic settlement (Eq. 2.14). As shown in figures, the computed elastic settlement of various shallow foundations depicts a discrepancy for the Victorville and Levelland areas in terms of degree of saturation and matric suction. Based on the results, the elastic settlement of the foundation in Levelland is greater than in Victorville. For both locations, the width ($B$) of the foundation has a greater impact on the settlement in comparison to the depth ($D$). As shown in Figs 2.11 and 2.12, the foundation size with higher width value has a lower settlement in comparison to the other foundation sizes. This finding is reasonable since a wide foundation distributes the applied load in a larger surface area and leads to generate less pressure on the ground surface. In terms of the foundation depth, for foundations with the same width size, higher depth leads higher settlement which is also reasonable since it directly increases the applied load through the foundation weight. In the Victorville region, each of the studied cases has a minimum settlement which occurs within a range of 68% to 75% degree of saturation. As the results show, an increase in the matric suction
leads to a narrow range of settlement for the foundations with the same width except for the width equal to 1.0 m in which the influence zone is smaller than others, and the entire zone is influenced by the infiltrated water. For the Levelland region, a decrease in the elastic settlement for all different foundation sizes was observed with a slight increase in the degree of saturation so that the higher settlement values occur within the lower values of the degree of saturation.

Figure 2.11. Elastic settlement of Victorville site after 3-day continuous rainfall

Figure 2.12. Elastic settlement of Levelland site after 3-day continuous rainfall
In terms of the ultimate bearing capacity, the calculated values show a similar pattern for each foundation case due to the degree of saturation and matric suction for the Victorville region (Fig. 2.13) in which the depth factor governs the design parameter. In Levelland, the ultimate bearing capacity increases consistently with reducing the degree of saturation for all different cases (Fig. 2.14). The ultimate bearing capacity in Victorville also exhibits a maximum set value occurring within a range of 70% to 80% degree of saturation and with a 70 to 90 kPa matric suction.

Figure 2.13. Ultimate bearing capacity of Victorville, CA site location after 3-day continuous rainfall
It can be seen from Figs 2.11 and 2.13 when the soil matric suction reaches to the value of about 70 kPa for the Victorville, the soil shear strength decreases which also lead a reduction for the soil stiffness. Therefore, at this range of matric suction, the ultimate bearing capacity decreases due to the reduction of the soil shear strength and similarly the elastic settlement of the foundation increases in that range due to the reduction of soil stiffness. This behavior happens when the rate of shear strength changes in different stages of partially saturated condition. This finding is in a good agreement with the experimental tests conducted by Vanapalli et al. (1996) and Oh and Vanapalli (2013). The experimental results of these research show that the shear strength (or stiffness) of soil decreases (or increase) when the soil reaches to the residual state. According to those research, the residual state range for gravels, sand and silts and their mixture is generally between 0 and 200 kPa. These trends were not captured for the Levelland site because the
range of residual stage is higher due to the existence of fine-grained soil in this site. The range of residual stage for clays with low plasticity is generally between 500 to 1500 kPa.

2.4.1 Foundation Design Values Determination and Comparison with Deterministic Approach

The predictions from the proposed method and the conventional deterministic method were compared to quantify the influence of the hydrological events through a change in the degree of saturation (or matric suction) in the foundation design for the selected sites. First, the design values based on the proposed method must be determined separately over the wide range of degree of saturation (or matric suction) for each study site. Deriving these design values entails a determining the mean of the best-fitted probability distribution to the average degree of saturation (or corresponding matric suction) within the foundation influence zone by considering all possible rainfall and water table scenarios. All possible scenarios were considered as the previously illustrated simulations were undertaken with a consideration of the probability distributions of historical rainfall and water table data for each location. Therefore, the necessary results can be readily extracted from their antecedents. The mean value of the best-fitted probability distribution to the average degree of saturation or matric suction is deemed the best-selected input values for computing the foundation design values, the ultimate bearing capacity and the elastic settlement. Similar to deriving the best-fitted distributions of both rainfall and water table, the Type I Extreme Largest (Gumbel distribution), the Type II Extreme Largest (Frechet distribution), and the Type III Extreme Largest
(Weibull distribution) were checked for the best fit. The Frechet and Weibull distribution was deemed the best regression to the average degree of saturation based on Kolmogorov-Smirnov test (p-value) for the Victorville and Levelland sites, respectively. The best-fit probability distribution to the calculated average degree of saturation for each site is shown in Fig. 2.15 with the mean value of the degree of saturation for the Victorville and the Levelland sites 63.1% and 32.8%, respectively. By using the mean values of the degree of saturation, the corresponding matric suction can also be found for each site using the scheme in Fig. 2.10. After finding these two values, then the ultimate bearing capacity and elastic settlement of each foundation size can be easily determined from the design values detailed in Figs. 2.10 to 2.14.

![Figure 2.15. Best-fitted distribution of average degree of saturation within the foundation influence zone for Left: Victorville and Right: Levelland](image)

As shown in Tables 2.3 and 2.4, the ultimate bearing capacity of the foundation increases by as much as 230% to that of the conventional method used for the Victorville site, with an increase of approximately 10% for the Levelland site. An 87% decrease in
the foundation settlement was observed for the Victorville site, and a 49% decrease was observed for the Levelland site, respectively. These observations indicate that the effect of degree of saturation and matric suction improve the predicted performance of shallow foundation obtained from conventional methods.

Table 2.3. Comparison of soil resistances and settlement computed based on proposed and conventional methods in Victorville

<table>
<thead>
<tr>
<th>Width (B) (m)</th>
<th>Depth (D) (m)</th>
<th>Fully Saturated Condition</th>
<th>Difference (%), ([(\text{Unsat-Sat})/\text{Sat})]*100%</th>
<th>Ultimate Bearing Capacity (kPa)</th>
<th>Elastic Settlement (mm)</th>
<th>Ultimate Bearing Capacity (kPa)</th>
<th>Elastic Settlement (mm)</th>
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</thead>
<tbody>
<tr>
<td>2.00</td>
<td>1.00</td>
<td>1501.90</td>
<td>+173.72</td>
<td>19.45</td>
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Table 2.4. Comparison of soil resistances and settlement computed based on proposed and conventional methods in Levelland

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<thead>
<tr>
<th>Width (B) (m)</th>
<th>Depth (D) (m)</th>
<th>Fully Saturated Condition</th>
<th>Difference (%), ([(\text{Unsat-Sat})/\text{Sat})]*100%</th>
<th>Ultimate Bearing Capacity (kPa)</th>
<th>Elastic Settlement (mm)</th>
<th>Ultimate Bearing Capacity (kPa)</th>
<th>Elastic Settlement (mm)</th>
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<tr>
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2.5 SUMMARY AND CONCLUSION

The coupled geotechnical-hydrological model defined in this study was used to incorporate the historical rainfall and water table data with the conventional method used in shallow foundation design. This novel method evaluates ultimate bearing capacity and elastic settlement due to the matric suction and the degree of saturation of the soil within the foundation influence zone. The infiltration of rainfall through initially partially saturated subsurface soil was modeled using the one-dimensional Richards’ equation considering both rainfall intensity and water table depth as the top and bottom boundary conditions, respectively. To calculate the bearing capacity and settlement of various foundation sizes, the average degree of saturation and matric suction within the influence zone were computed by applying 10,000 random input values corresponding to the rainfall and water table distributions using Monte Carlo simulation. Finally, the mean of the best-fitted probability distribution to the average degree of saturation or matric suction is selected to find the design values of shallow foundation for each site.

Two sample sites were selected in this study to show the variation of ultimate bearing capacity and elastic settlement with matric suction (or degree of saturation), Victorville, CA and Levelland, TX. After three days of continuous rainfall and ignoring the effect of surface runoff and evapotranspiration, the degree of saturation in Victorville was between 55% and 84%, and between 25% and 35% in Levelland. The significant difference in the ranges was caused by the existence of fine-grained soil in the Levelland region which decreases the soil porosity and the permeability. The matric suction in a shallow foundation design was also found to increase the ultimate bearing capacity of a
foundation by almost 230% of the bearing capacity compared to the fully saturated condition. However, the effect of the matric suction can be changed depending upon the depth of water infiltration into the soil. In terms of settlement criteria, the elastic settlement of various footing sizes has been decreased to approximately 87% and 40% of the settlement considering the fully saturated condition in Victorville and Levelland, respectively.

A comparison of the results determined that the common foundation design procedure overestimates the foundation design parameters in comparison with the actual condition of the site locations even for heavy rainfall events. Based on the current results, the matric suction, which is a significant parameter of partially saturated conditions, had a significant effect on the ultimate bearing capacity and elastic settlement of a shallow foundation located in a permeable soil medium.

2.6 REFERENCES


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CHAPTER 3

COUPLED GEOTECHNICAL-CLIMATIC DESIGN FRAMEWORK FOR DRILLED SHAFT SUBJECTED TO AXIAL LOAD

3.1 INTRODUCTION

Climate events such as heavy rainfall, flood, and drought have become frequent in recent years (Trenberth 2011). These events cause significant economic losses every year in the United States and around the world. The 2014 New York flood, 2015 Missouri flood, 2016 Oklahoma flood, 2016 Louisiana flood, 2017 California flood and 2017 Houston flood are some of the examples of severe floods in the United States. Also, the United States has been struggling with the severe droughts over a long period. The drought happened in North America during the 19th century, in the Southwestern United States (New Mexico and Texas) in 1950, and in the Midwest and Rocky Mountain regions in 2002 are the examples of the severe drought in the United States. According to the National Climatic Data Centre (NCDC) currently, 30% of the United States suffers from moderate to severe drought. These climatic events have notable damages to the above-ground structures through the foundations (Steenbergen et al. 2009). Foundation as an interface between the structure and ground surface is highly affected by the characteristics and properties of underlying soil. The soil properties are greatly influenced by the climatic events through changing the saturation level of the soil. However, the climatic events’ impact on the soil properties and subsequently the foundations which support various structures such as bridge, buildings, earth dams, and levees are ignored in the current design codes. In addition to that, the current loss estimation schemes simply
ignore the damage caused by foundation failures (bearing capacity and settlement) due to the climate events. Therefore, a coupled geotechnical-climatic model must be developed to predict the behavior of foundations under climatic events accurately. In this study, rainfall and evapotranspiration as two primary parameters of the climatic events are employed as input climatic data along with groundwater level to assess the performance of a drilled shaft.

The drilled shaft is a common type of deep foundation used to support superstructures and transfer its loads to the deep surface soil. The drilled shaft is commonly designed for the worst case geotechnical conditions in which the soil is fully saturated with the water table at the ground surface even if the historical water table is well below the shaft tip and expected to be the same during the lifetime of the structure (Kulhawy 1990, Das 2010, Briaud 2013). This way of design is too conservative since the underlying soil is mostly under partially saturated condition and water table fluctuates with time. Recent case studies indicate that variation of the soil degree of saturation and matric suction significantly affect the soil shear strength of the subsurface area (Fredlund et al. 1978 and 2012, Lu and Likos 2004). Over past decades, many research studies have been conducted to propose a method based on the Soil Water Characteristics Curve (SWCC) to account the variation of degree of saturation and matric suction in partially saturated soil (Vanapalli et al. 1996, Oberg and Sallfors 1997, Lee et al. 2005, Garven and Vanapalli 2006, Guan et al. 2010, Sheng 2011). Following that, several researchers investigated the influence of matric suction on the partially saturated soil behavior and interface shear strength (Khoury et al. 2010, Borana et al. 2015, Borana et al. 2016). The
change of soil shear strength subsequently affects the settlement and bearing capacity of different types of foundation. In recent years, numerous research studies have investigated the influence of matric suction and degree of saturation on the load carrying capacity of deep and shallow foundation using plate-load tests (Douthitt et al. 1998, Georgiadis et al. 2003, Costa et al. 2003) and model footing tests (Vanapalli et al. 2010, Vanapalli and Oh 2010b, Vanapalli and Taylan 2011a, 2011b and 2012). Also, in term of the soil stiffness, a number of research studies have been addressed the impact of matric suction on soil modulus elasticity for coarse-grained soils (Agarwal and Rana 1987, Steensen-Bach et al. 1987, Schnaid et al. 1995, Oh et al. 2009, Vanapalli and Oh 2010a) and fine-grained soils (Costa et al. 2003, Rojas et al. 2007, Vanapalli and Adem 2013). Thus, it can be concluded that the deterministic design approach can be either conservative or unconservative, depending on the site condition where the near-surface soil is initially partially saturated during the design life of the structure, and it is highly affected by the climatic events.

The conventional drilled shaft design methods must be revised to incorporate the site-specific climatic parameters for a better assessment. A better design procedure will require a thorough understanding of the behavior of partially saturated soil and utilization of site-specific climate parameters including historical rainfall, evapotranspiration and water table data along with geotechnical parameters. In recent years, a limited number of efforts have been undertaken to assess the influence of climatic events on the behavior of partially saturated soils for various geotechnical systems. Vahedifard et al. (2015) were developed a new framework to evaluate the influence of steady vertical flow on effective
stress-based limit equilibrium analysis of partially saturated slopes. Moreover, Vahedifard and Robinson (2016) proposed a unified method based on model footing and plate load tests to estimate the ultimate bearing capacity of shallow foundation in partially saturated soil considering different surface flux boundary conditions and fluctuation of water table depth. Kim et al. (2017) studied the effect of rainfall on shallow foundation settlement using numerical analysis and compared its result with in-situ load tests for low-range matric suction. Mahmoudabadi and Ravichandran (2017) presented a framework to take the historical rainfall and water table into account for computing the bearing capacity of a shallow foundation. Following that Ravichandran et al. (2017) applied a probabilistic method to the design process of shallow foundation in partially saturated soils.

Since none of the abovementioned methods addressed a design approach for a drilled shaft, this study aims to develop a procedure for coupling site-specific climatic parameters and water table depth with geotechnical parameters to compute the shaft axial capacity and settlement. To this end, first, the numerical solution of the Richards equation was considered to capture the variation of the degree of saturation and matric suction along the shaft length considering the resultant infiltration (including historical rainfall and evapotranspiration) and water table data as upper and lower boundary conditions, respectively. Then, Monte Carlo simulations were used to randomly generate the input variables associated with the climatic data from its probability distribution to compute the axial capacity and elastic settlement of various drilled shafts in partially saturated soil for the study areas. Finally, the design axial capacity and settlement of the drilled shafts were
selected based on the mean of the best-fitted probability distribution to the average matric suction (or degree of saturation) beneath the subsurface. In addition, different time durations are selected to demonstrate the impact of climatic event duration on water infiltration process into different site conditions and subsequently its effect on the design parameters of a drilled shaft. The proposed procedure was illustrated through two sample applications in the United States.

3.2 COUPLED GEOTECHNICAL-CLIMATIC DESIGN OF DRILLED SHAFT

3.2.1 Axial Capacity - Safety Check

The ultimate axial compression capacity of the drilled shaft, $Q_{Ult}$, was calculated using the simplified equation shown in Eq. 3.1 (Kulhawy 1990),

$$Q_{Ult} = Q_{Skin} + Q_{Tip} - W$$

(3.1)

where $Q_{Skin}$ is the drilled shaft skin resistance, $Q_{Tip}$ is the drilled shaft tip resistance, and $W$ is the weight of the drilled shaft. To calculate the ultimate axial compression capacity of a drilled shaft, first, the skin and tip resistance need to be computed due to the soil matric suction and degree of saturation. The contribution of matric suction and degree of saturation towards the axial capacity of a drilled shaft in partially saturated soils has been the subject of numerous studies which were discussed in the literature in more detail.

3.2.1.1 Skin resistance

Among the many available equations, the ultimate axial capacity equation proposed by Vanapalli and Taylan (2012) was used in this study to predict the nonlinear
variation of skin resistance in partially saturated soils with respect to the matric suction for a circular drilled shaft (Eq. 3.2). The proposed equation considers the contribution of matric suction toward the skin resistance, \( Q^{Skin(u_a-u_w)} \), as an additive term to the conventional method considering the soil is fully saturated, \( Q^{Skin(sat)} \).

\[
Q_{Skin} = \sum_{j=1}^{p} (Q_{Skin(sat)} + Q_{j}^{Skin(u_a-u_w)}) = A_s \sum_{j=1}^{p} (f_{j}^{sat} + f_{j}^{(u_a-u_w)})
\]

\[
= \sum_{j=1}^{p} \left[ c_a + \beta_j \sigma_{z(j)} + (u_a - u_w)_{avg(j)} S_j \tan \delta^j \right] \pi BD
\]

where \( j \) is the shaft segment number, \( f_{j}^{(u_a-u_w)} \) is the contribution of matric suction towards the unit skin resistance for \( j \)th segment, \( f_{j}^{sat} \) is unit skin resistance at fully saturated condition for \( j \)th segment, \( A_s \) is shaft perimeter, \( p \) is the total number of shaft segment, \( c'_a \) is the soil adhesion under saturated condition, \( \beta_j \) is the Burland-Bjerrum coefficient for \( j \)th segment which is equal to \( K_0 \tan \delta^j \) (\( K_0 \) is mean lateral earth coefficient at rest), \( \delta^j \) is effective angle of interface between soil and shaft skin (Tariq and Miller 2009), \( \sigma_{z(j)} \) is vertical effective stress for \( j \)th segment, \( (u_a-u_w)_{avg(j)} \) is the average matric suction of \( j \)th segment, \( S_j \) is degree of saturation for \( j \)th segment, \( B \) is shaft diameter, \( D \) is shaft length, and \( \kappa \) is the fitting parameter which is described in details in Vanapalli and Fredlund (2000). The average matric suction in the proposed skin resistance equation is expressed in Eq. 3.3,

\[
(u_a - u_w)_{ave(j)} = \frac{\sum_{i=j}^{n_j} [(u_a - u_w)_i]}{n_j - i_j} = -\psi_{ave(j)} \gamma_w
\]
where \((u_{a_i}-u_{w})_i\) is the matric suction at \(i^{th}\) node of soil profile, \(i_j\) is the first node of \(j^{th}\) segment, \(n_j\) is the last node of \(j^{th}\) segment, \(\psi_{avg(j)}\) is the average pressure head of \(j^{th}\) segment, and \(\gamma_w\) is the unit weight of water. To have a precise calculation of skin resistance, at first step the drilled shaft is discretized to a numerous segments, and then the soil suction-related parameters such as degree of saturation, matric suction and unit weight are computed for each segment separately and finally the total skin resistance is equal to the summation of skin resistance for each segment. Fig. 3.1 displays the process of calculating the skin resistance for the drilled shaft in the partially saturated soil.

![Diagram](image.png)

Figure 3.1. The design process of drilled shaft in partially saturated soils

### 3.2.1.2 Tip resistance

In term of the tip resistance, the method presented by Kulhawy (1990) was employed in this study (Eq. 3.4) in which the impact of the partially saturated condition is
considered through the change of unit weight of the soil(\(\gamma\)) and also the effective stress at the tip point, \(\sigma'_{z(Tip)}\).

\[
Q_{Tip} = q_{Tip}A_{Tip} = \left(0.5\overline{\gamma}BN_q\xi_{qs}\xi_{qs} + \sigma'_{z(Tip)}N_q\xi_{qs}\xi_{qs}\xi_{qs}\right)0.25\pi B^2
\]  

(3.4)

where \(q_{Tip}\) is the tip bearing capacity, \(A_{Tip}\) is shaft tip area, \(\overline{\gamma}\) is the average unit weight from depth \(D\) to \(D+B\), \(N_q\) and \(N_q\) are the non-dimensional bearing capacity factors that are functions of the effective soil friction angle, and \(\xi_{qs}\), \(\xi_{qs}\), and \(\xi_{qs}\) are the shape, depth, and soil rigidity factors, respectively. The details of these factors are provided by Kulhawy (1990) (Table 3.1).

<table>
<thead>
<tr>
<th>Factor</th>
<th>Symbol</th>
<th>Value</th>
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<tr>
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</tr>
<tr>
<td></td>
<td>(\xi_{qs})</td>
<td>1 + tan(\phi')</td>
</tr>
<tr>
<td>Depth</td>
<td>(\xi_{qs})</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>(\xi_{qs})</td>
<td>1 + 2 tan(\phi'(1\sin\phi')) ((\pi/180)(\tan^{-1}(D/B)))</td>
</tr>
</tbody>
</table>
| Rigidity| \(\xi_{qs}\) | \(\xi_{qs}\) exp\{[-3.8tan\(\phi') + [(3.07sin\(\phi')(log_{10}(2I_{rr}))/((1+sin\(\phi')))]\}

\(\phi'\): Effective friction angle
\(I_{rr}\): Reduced rigidity index

**3.2.2 Elastic Settlement - Serviceability Check**

The total elastic settlement of a drilled shaft, \(S_e\) was calculated using the equation shown in Eqs. 3.5 to 3.8 (Das 2010),

\[
S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}
\]

(3.5)
\[ S_{e(1)} = \frac{Q_{\text{wtip}} + \zeta Q_{\text{wskin}}}{A_p E_p} \]  

(3.6)

\[ S_{e(2)} = \frac{q_{\text{wtip}} B}{E_{\text{soil}}}(1 - \mu_s^2)I_{wp} \]  

(3.7)

\[ S_{e(3)} = \left( \frac{Q_{\text{wskin}}}{A_s D} \right) \frac{B}{E_{\text{soil}}}(1 - \mu_s^2)I_{ws} \]  

(3.8)

where \( S_{e(1)} \) is the elastic settlement of drilled shaft, \( S_{e(2)} \) is the settlement of drilled shaft caused by the load at the shaft tip, \( S_{e(3)} \) is the settlement of drilled shaft caused by the load transmitted along the shaft skin, \( Q_{\text{wtip}} \) is the load carried at the shaft tip under working load condition, \( Q_{\text{wskin}} \) is the load carried by skin resistance under working load condition, \( \zeta \) is a coefficient which depends on the distribution of the unit skin resistance along the shaft skin and it is assumed 0.67, \( A_p \) is the cross-section area of shaft, \( E_p \) is the modulus of elasticity of the drilled shaft material (concrete), \( q_{\text{wtip}} \) is the shaft tip load per unit area under working load condition, \( E_{\text{soil}} \) is the soil modulus of elasticity at strain level of 1\%, \( \mu_s \) is the soil Poisson’s ratio, \( I_{wp} \) and \( I_{ws} \) are the tip’s and skin’s influence factor, respectively. In addition to the shear strength parameters, the soil modulus of elasticity \( (E_{\text{soil}}) \) is another parameter which is affected by the degree of saturation and matric suction of the soil profile along the shaft skin. Since the degree of saturation and the matric suction are computed following a numerical procedure which will be discussed in the next section, the elastic settlement can be computed if \( E_{\text{soil}} \) is also expressed as a function of the degree of saturation and matric suction.
As discussed in the literature review, various empirical equations have been proposed to predict the soil modulus of elasticity as a function of matric suction and degree of saturation. In this study, the equation proposed by Oh et al. (2009), shown in Eq. 3.9, was used to estimate the modulus of elasticity in partially saturated coarse-grained soils ($E_{\text{soil(unsat)}}$),

$$E_{\text{soil(unsat)}} = E_{\text{soil(sat)}} \left[ 1 + \alpha_e (u_a - u_w)_{\text{ave}} S^\beta_e \right]$$  \hspace{1cm} (3.9)

where $E_{\text{soil(sat)}}$ is the soil modulus of elasticity under the saturated condition at strain level of 1%, and $\alpha_e$ and $\beta_e$ are fitting parameters. For coarse- and fine-grained soil, the recommended fitting parameter, $\beta_e$, is equal to 1 and 2, respectively. Also, the fitting parameter $\alpha_e$ depending upon the plasticity index ($I_p$) is computed using the following empirical equation (Eq. 3.10), developed by Oh et al. (2009).

$$\frac{1}{\alpha} = 0.5 + 0.312(I_p) + 0.109(I_p)^2 \quad (0 \leq I_p(\%) \leq 12)$$  \hspace{1cm} (3.10)

It should be noted that to calculate the total settlement of a drilled shaft the consolidation settlement is also required. The consolidation settlement is omitted in this study for two reasons. The first reason is that the consolidation settlement is a long-term process which usually takes years to show significant settlement especially when the foundation is supported by fine-grained soil. However, in reality, the degree of saturation of the soil within the influence zone fluctuates with the rainfall intensity and duration and other factors. In this study, few days rainfall is considered which is a short duration compared to the time it takes to show significant consolidation. In such a situation, accurately computing the change in consolidation settlement due to the change in the
degree of saturation is a difficult task. That is why the immediate settlement (elastic settlement) is considered in this study. One could compute the consolidation settlement considering traditional consolidation parameters and add it with the elastic settlement for the sake of completeness. The second reason is that the lack of well–established correlations for computing the consolidation parameters such as compression index, recompression index and preconsolidation pressure as functions of the degree of saturation and/or matric suction. When such correlations are available, one could calculate the additional consolidation settlement due to the variation in the degree of saturation and add it with that of primary consolidation settlement based on saturated parameters.

As is expressed in this section, the ultimate axial capacity and settlement of drilled shaft directly relate to the degree of saturation and matric suction of underlying soil. Thus, these parameters need to be accurately calculated for any site condition. The procedure of calculating the site-specific degree of saturation and matric suction is described in the next section.

3.3 WATER FLOW MODELING IN PARTIALLY SATURATED SOIL AND VERIFICATION

The one-dimensional vertical movement of water through the partially saturated soil was represented by Richards’ equation (Richards 1931) which is shown in Eq. 3.11. This nonlinear partial differential equation derived from Darcy’s law, predicts a decrease of the water infiltration for the different flux rates in the subsurface area.
\[
\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z} \left[ K \left( 1 + \frac{\partial \psi}{\partial z} \right) \right]
\] (3.11)

where \( t \) is the time, \( z \) is the depth from the ground surface, \( \theta \) is the volumetric water content, \( K \) is the hydraulic conductivity of partially saturated soil, \( \psi \) is the pressure head, and \( \frac{\partial \psi}{\partial z} \) is the hydraulic gradient. Although the problem considered in this study is three-dimensional in nature, it is found that the one-dimensional model is reasonably accurate to predict the vertical movement of the water (Celia et al. 1990, Van Dam et al. 2000). Since the pressure head is considered as the primary variable to be determined in this study, the two other variables in Richards’ equation, \( \theta \) and \( K \), are required to be expressed as functions of \( \psi \). The hydraulic conductivity of partially saturated soil is expressed as \( K = K_{sat} K_r \), where \( K_{sat} \) is the hydraulic conductivity of the soil under the fully saturated condition, and \( K_r \) is the relative hydraulic conductivity of the soil under partially saturated condition. Both \( \theta \) and \( K_r \) are then expressed as functions of \( \psi \) using a Soil Water Characteristic Curve (SWCC). Among the many SWCCs and corresponding relative hydraulic conductivity functions, the equations proposed by van Genuchten (1980) were used in this study. The equations of SWCC and corresponding \( K_r \) functions are given in Eqs. 3.12 and 3.13, respectively,

\[
\theta(\psi) = \theta_r + \frac{\left( \theta_{sat} - \theta_r \right)}{\left[ 1 + (\alpha \psi)^n \right]^{1-1/n}}
\] (3.12)
where $\alpha$ and $n$ are SWCC fitting parameters, $\theta_s$ is saturated water content, and $\theta_r$ is residual water content. Many researchers have solved Richards’ equation using various numerical solution approaches (van Genuchten 1982, Feddes et al. 1988, Celia et al. 1990, Warrick 1991, Zaidel and Russo 1992, Baker 1995, Pan et al. 1996, Romano et al. 1998, Van Dam et al. 2000). In this study, the Finite Volume Method (FVM) was used to solve the Richards equation for various types of boundary conditions. The spatial and temporal discretization and the summary of the solution procedure are given in the following sections.

### 3.3.1 Mathematical Solution Procedure

To solve the Richards equation, first, the equation needs to be written in term of the pressure head for one-dimensional vertical infiltration. Thus, $\frac{\partial \theta}{\partial t}$ is expressed as

$$
\frac{\partial \theta}{\partial t} \cdot \frac{\partial \psi}{\partial t} = C \frac{\partial \psi}{\partial t}
$$

and substituted in Eq. 3.14,

$$
C \frac{\partial \psi}{\partial t} - \frac{\partial}{\partial z} \left( K \frac{\partial \psi}{\partial z} + K \right) = 0
$$

(3.14)

where $C$ is the specific moisture capacity ($= \frac{\partial \theta}{\partial \psi}$). Then, the Eq. 3.14 is integrated with respect to the time ($t$) and depth ($z$) as follow.
\[ \int_{i-1/2}^{i+1/2} \int_{t}^{t+\Delta t} C \frac{\partial \psi}{\partial t} dt dz - \int_{i-1/2}^{i+1/2} \int_{t}^{t+\Delta t} \frac{\partial}{\partial z} \left( K \frac{\partial \psi}{\partial z} + K \right) dz dt = 0 \]  

(3.15)

where \( i \) is the node number (start from ground surface) and \( \Delta t \) is the time interval. The left-hand side of the integration is rewritten in a discretized form, which is expressed in Eq. 3.16.

\[ \int_{i-1/2}^{i+1/2} \int_{t}^{t+\Delta t} C \frac{\partial \psi}{\partial t} dt dz = \int_{i-1/2}^{i+1/2} C \left( \psi^{i+\Delta t} - \psi^i \right) dz = C_i^{i+\Delta t} \left( \psi_i^{i+\Delta t} - \psi_i^i \right) \Delta z \]  

(3.16)

The right-hand side is first solved for spatial variation, as shown in Eq. 3.17, and then, the integration is discretized into the temporal form, as shown in Eq. 3.18.

\[ \int_{t}^{t+\Delta t} \int_{i-1/2}^{i+1/2} C \frac{\partial \psi}{\partial t} dt dz = \int_{i-1/2}^{i+1/2} \int_{t}^{t+\Delta t} \left[ \left( K \frac{\partial \psi}{\partial z} + K \right)_{i+1/2} - \left( K \frac{\partial \psi}{\partial z} + K \right)_{i-1/2} \right] dt \]  

(3.17)

\[ C_i^{i+\Delta t} \left( \psi_i^{i+\Delta t} - \psi_i^i \right) \Delta z = \left[ \left( K^{i+\Delta t}_{i+1/2} \psi_i^{i+\Delta t} - \psi_i^i \right) \Delta z + K^{i+\Delta t}_{i+1/2} \right] - \left[ \left( K^{i+\Delta t}_{i-1/2} \psi_i^{i+\Delta t} - \psi_i^i \right) \Delta z + K^{i+\Delta t}_{i-1/2} \right] \]  

(3.18)

Considering \( m \) as the iteration level and pressure head at iteration \( m+1 \) as the unknown value and \( \Delta z \) as the depth interval, the complete spatial and temporal form of Richards’ equation is expressed in Eq. 3.19.

\[ C_i^{i+\Delta t,m} \left( \psi_i^{i+\Delta t,m+1} - \psi_i^i \right) \Delta z = \left[ \left( K^{i+\Delta t}_{i+1/2} \psi_i^{i+\Delta t,m+1} - \psi_i^{i+\Delta t,m} \right) \Delta z + K^{i+\Delta t}_{i+1/2} \right] - \left[ \left( K^{i+\Delta t}_{i-1/2} \psi_i^{i+\Delta t,m+1} - \psi_i^{i+\Delta t,m} \right) \Delta z + K^{i+\Delta t}_{i-1/2} \right] \]  

(3.19)
Finally dividing the Eq. 3.19 by \( \Delta z \) and rearranging the formulation, the final form of the Richards equation is written as follow (Eq. 20).

\[
C_j^{i+\Delta t,m} \left( \psi_{i+\Delta z,m+1} - \psi_i \right) = \left[ \left( \frac{K_i^{i+\Delta t} \psi_{i+\Delta z,m+1} - K_i^{i} \psi_{i} }{ (\Delta z)^2 } \right) - K_{i-1/2}^{i+\Delta t} \psi_{i} - K_{i+1/2}^{i+\Delta t} \psi_{i+\Delta z,m+1} \right] \Delta t + \left( \frac{K_{i+1/2}^{i+\Delta t} - K_{i-1/2}^{i}}{\Delta z} \right) \Delta t 
\]

(3.20)

### 3.3.2 Initial and Boundary Conditions

The one-dimensional numerical scheme of water infiltration into the soil profile is shown in Fig. 3.2 to illustrate the problem and the boundary conditions. The soil was assumed to be at the residual condition (residual stage in SWCC) as an initial condition at the beginning of each simulation. The upper and lower boundary conditions are located on the ground surface and the water table level, respectively. In this study, both pressure head and flux boundary conditions were applied at the upper boundary depending upon the intensity of the resultant infiltration and the surface moisture capacity. In the case of ponding, when the infiltration rate is greater than the saturated hydraulic conductivity, the pressure head boundary condition is applied. On the contrary, when all water infiltrates into the soil, the flux boundary condition is applied. Since the water table location and resultant infiltration vary with climatic conditions for each specific location, appropriate values must be determined in a probabilistic manner considering historical rainfall, evapotranspiration, and water table data. The process of constructing the boundary conditions’ probability distribution will be discussed in the design application section.
3.3.2.1 Resultant infiltration – upper boundary condition

The resultant infiltration, \( F_{\text{Resultnat Infiltration}} \), as the upper boundary condition, is computed from subtraction of the in-flux from out-flux climatic parameters for each specific site which is expressed in Eq. 3.21.

\[
F_{\text{Resultnat Infiltration}} = \text{Influx} - \text{Outflux} = (F_{\text{Rainfall}}) - (F_{\text{Evapotranspiration}} + F_{\text{Runoff}})
\]

(3.21)

where \( F_{\text{Rainfall}} \) is the historical rainfall intensity, \( F_{\text{Evapotranspiration}} \) is the evapotranspiration intensity and \( F_{\text{Runoff}} \) is the surface runoff which is assumed to be zero in this study. One may consider topology and other site-specific parameters for calculating the resultant infiltration more accurately.

3.3.2.1.1 Rainfall

The historical rainfall data were obtained from the National Climatic Data Center (NCDC) which records daily rainfall values. In this study, the annual maximum series were used and constructed by extracting the highest precipitation in each successive year.
over a given return period. Then, the maximum annual rainfall has been tabulated for the same return period to determine the site-specific probability distribution of resultant infiltration.

### 3.3.2.1.2 Evapotranspiration

Land surface evaporation plus plant transpiration, evapotranspiration, is another climatic parameter which has a direct influence on the resultant infiltration, subsequently the degree of saturation and matric suction of the subsurface area. This parameter is dependent on the other environmental factors such as temperature, daylight time and saturated vapor density and can be simply computed based on the Hamon (1961) method in terms of Potential Evapotranspiration ($PET$) (Eq. 3.22),

$$PET = 0.1651 \times L_d \times RHOSAT \times KPEC$$  \hspace{1cm} (3.22)

where $L_d$ is daytime length, $T$ is the air average temperature, $KPEC$ is a calibration coefficient equal to 1, and $RHOSAT$ is saturated vapor density at a mean temperature calculated using Eq. 3.23,

$$RHOSAT = 216.7 \frac{ESAT}{T + 273.2}$$  \hspace{1cm} (3.23)

where $ESAT$ is the saturated vapor pressure and is calculated using the Eq. 3.24,

$$ESAT = 6.108e^{\left(\frac{17.27T}{T + 237.3}\right)}$$  \hspace{1cm} (3.24)
Using Hamon method, the daily potential evapotranspiration for the same return period, which was used for the rainfall data, is calculated based on the temperature values which was obtained from the National Oceanic and Atmospheric Administration (NOAA) of NCDC.

In brief, the key steps for computing the upper boundary condition, resultant infiltration, are: (1) extract the site-specific historical rainfall data and temperature from NCDC, (2) calculate the site-specific resultant infiltration considering the historical rainfall, evapotranspiration, and runoff, (3) find the best-fitted probability distribution function for the site-specific resultant infiltration data, and (4) generate random number based on the distribution function to apply as the upper boundary condition through Monte Carlo simulation for each analysis. The detail of this process is described in the sample application section for each study site.

3.3.2.2 Water table - lower boundary condition

The water table depth, the lower boundary condition for solving the Richards equation, is another climatic parameter that affects the matric suction and degree of saturation of the soil along the shaft skin and tip. The required data was taken from the U.S. Geological Survey (USGS 2016) for the seam return period selected for the rainfall data. It should be noted that the lower boundary condition is applied as the pressure head. Similar to the resultant infiltration, the key steps for computing the lower boundary condition are: (1) extract the site-specific historical water table data from USGS, (2) find the best-fitted probability distribution function for the site-specific historical water table
data, and (3) generate random number based on the distribution function to apply as the lower boundary condition through Monte Carlo simulation for each analysis. The details of this process are described in the sample application section for each study site.

### 3.3.3 Model Verification for Given Boundary Condition

The presented framework includes two primary algorithms, the Richards equation, and SWCC, which together can sort out the coupled geotechnical-climatic problem. Hence, the validity of the proposed method should be tested through a comparison of the results of the numerical Richards’ equation and the SWCC with either experimental data or other verified model. To accomplish that, a generalized solution developed by Celia et al. (1990), which is laid down on a set of experimental data, was used to verify the implemented numerical solution of water flow in the partially saturated soil in this study (Fig. 3.3).

![Figure 3.3. Numerical solution scheme verification for Richards’ Equation](image)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>0.0335</td>
</tr>
<tr>
<td>$\theta_s$</td>
<td>0.368</td>
</tr>
<tr>
<td>$\theta_r$</td>
<td>0.102</td>
</tr>
<tr>
<td>$n$</td>
<td>2.0</td>
</tr>
<tr>
<td>$K_{sat}$ (cm/s)</td>
<td>0.00922</td>
</tr>
<tr>
<td>$\psi_{top}$ (cm)</td>
<td>-75</td>
</tr>
<tr>
<td>$\psi_{bottom}$ (cm)</td>
<td>-1000</td>
</tr>
<tr>
<td>$t$ (day)</td>
<td>1</td>
</tr>
<tr>
<td>$\Delta t$ (sec)</td>
<td>144</td>
</tr>
</tbody>
</table>

Figure 3.3. Numerical solution scheme verification for Richards’ Equation
3.4 SITE-SPECIFIC DESIGN APPLICATION

The application of the proposed procedure requires the computation of temporal and spatial variation of matric suction and degree of saturation within the zone of influence. Because the skin resistance varies along the length of the shaft, to accurately calculate the effect of climatic parameters, the shaft needs to be divided into a number of segments along its length. The random variables associated with the climatic data are considered as the boundary conditions that change with return period and is discussed in details in the resultant infiltration and water table distribution section. Since these variables have time-independent uncertainty for each specific site, the probability analysis is required to adjust the design process. In another word, if the highest historical resultant infiltration rate and lowest water table depth are considered as the worst case scenario of boundary conditions, the probability of occurrence of these events simultaneously is significantly low during the lifetime of the structure. Considering this condition as one of the design cases can lead to an overdesign result. Thus, the design procedure should carry out through a probabilistic manner to consider all the joint occurrence possibilities of climate events. This way of analysis will lead to a more realistic design approach based on the Probability Density Function (PDF) of the climate-related geotechnical parameters for the drilled shaft.

To perform the probabilistic analysis, first, the probability distributions of historical resultant infiltration rate and water table depth were used by Monte Carlo simulation method to generate a set of random input variables. These input variables were considered as the boundary conditions in this study. Then, Richards’ Equation solution
was used to compute the temporal and spatial variation of soil degree of saturation and matric suction along the shaft skin. Afterward, the ultimate axial capacity and elastic settlement of each drilled shaft were calculated using the average degree of saturation and matric suction. This process was repeated for all the generated input variables. Finally, the mean of the best-fitted probability distribution to the average matric suction is selected to find the design axial capacity and settlement of drilled shaft for study sites. It should be noted that the inherent randomness of shear strength parameters of the soil can also be incorporated into the analysis, but to allow for comparisons between the saturated and partially saturated conditions, the shear strength variables were kept as constants throughout the analysis except the soil unit weight. The soil unit weight changes with varying degrees of saturation computed along the shaft skin for each simulation. The flowchart of the procedure employed in this study is presented in Fig. 3.4.
Figure 3.4. Simulation flowchart to incorporate the climatic data into drilled shaft design
3.4.1 Study Sites

Two sites in the United States were selected in this study to show the effects of climatic parameters on the ultimate axial capacity and elastic settlement of drill shafts. The first site is located in Salt Lake City, UT. The Salt Lake City site was selected due to its semi-arid climate and availability of van Genuchten SWCC parameters, in addition to the conventional geotechnical engineering design parameters, for the silty-clayey sandy (SC) soil type found in this region. The SWCC parameters were obtained from the report by Zhang (2010). The soil strength parameters of the site were obtained from a geotechnical report by GSH Geotechnical Inc. (2013), and Web Soil Survey developed by the United States Department of Agriculture (USDA 2016). A location in Riverside, CA, was selected as the second site which mostly contains the silty sand (SM) in this paper. For this site, the soil strength parameters were obtained from a geotechnical report provided by Converse Consultants (2016), and Web Soil Survey developed by the United States Department of Agriculture (USDA 2016). The SWCC parameters for the Riverside location were obtained from the report by Zhang (2010). The specified van Genuchten SWCC parameters model for these two locations are presented in Fig. 3.5. In addition, the basic strength and other geotechnical parameters for both locations are listed in Table 3.2.
### Table 3.2. Geotechnical parameters for the Salt Lake City, UT, and Riverside, CA sites

<table>
<thead>
<tr>
<th>Geotechnical Parameters</th>
<th>Salt Lake City, UT</th>
<th>Riverside, CA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight, $\gamma_d$ (kN/m³)</td>
<td>16.40</td>
<td>18.10</td>
</tr>
<tr>
<td>Void ratio, $e$</td>
<td>0.585</td>
<td>0.436</td>
</tr>
<tr>
<td>Effective friction angle, $\phi'$ (deg)</td>
<td>32</td>
<td>30</td>
</tr>
<tr>
<td>Effective adhesion, $c'_a$ (kPa)</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>Plasticity Index, $I_p$</td>
<td>11</td>
<td>7</td>
</tr>
</tbody>
</table>

![Figure 3.5. The specified van Genuchten SWCC model for Salt Lake City and Riverside sites](image)

#### 3.4.2 Probability Distribution of Boundary Conditions

As is discussed in previous sections, the boundary conditions, resultant infiltration as an upper boundary condition and water table depth as a lower boundary condition, are highly relied on the climatic parameters which change with various return periods. Thus, these boundary conditions need to be represented separately as a probability distribution instead of a deterministic value. The return period selected for this study was 117 years for both study locations. The process of producing the boundary conditions’ distribution is explained in details below.
### 3.4.2.1 Constructing probability distribution

To determine the best fitting distribution for the resultant infiltration and water table data, the probability paper plotting technique was used. Type I Extreme Largest (Gumbel distribution), the Type II Extreme Largest (Frechet distribution), and the Type III Extreme Largest (Weibull distribution) were checked for the best fit, and the Gumbel distribution was deemed the best regression based on the R-squared test ($R^2$-value). The Gumbel probability paper distribution parameters, mode ($\mu_n$) and standard deviation ($\beta_n$), can be determined using Eq. 3.25,

\[
x_v = y_v \beta_n + \mu_n = -\ln \left(-\ln \left(\frac{v}{r+1}\right)\right) \beta_n + \mu_n
\]  

(3.25)

where $v$ is the data index (arranged in increasing order), $x_v$ is the annual maximum historical rainfall or water table data, $y_v$ is the linearized form of the cumulative density function of Gumbel distribution, and $r$ is the number of data points. The probability plots of the resultant infiltration and water table data based on the Gumbel distribution are shown in Figs. 3.6 and 3.7, respectively for both study sites.
3.4.3 Climatic Loads

The climatic load is applied in the proposed framework through upper boundary (historical rainfall and evapotranspiration) and lower boundary (groundwater level) conditions as well as the time duration of climate event. As described before, the boundary condition is predicted through the probability distribution function for each
location using Eq. 3.25. In addition, the time duration of the climate events was assumed to be 1, 3 and 5 days in this study.

**3.4.4 Initial Design and Design Parameters**

Circular drilled shaft with width, $B = 0.9, 1.2$ and $1.5$ m and the shaft length, $D = 12, 15$ and $18$ m were used in this study to investigate the influence of partially saturated soil condition caused by various climate parameters on different drilled shaft sizes. The applied working load for each drilled shaft was calculated based on the fully saturated soil condition and a factor of safety equals to 3.0 for both skin and tip resistance.

**3.4.5 Computational Platform**

For considering the historical rainfall and water table depth in a probabilistic manner, the Richards equation must be solved around 10,000 times and the skin resistance, tip resistance and settlement must be calculated for each simulation. To simplify such repeated calculations, a MATLAB code was developed, parallelized and installed on the Clemson University’s High Performance Computing (HPC) System called Palmetto Cluster, a parallel computing facility. A simulation that took almost a month on a single processor PC was completed within a week with just four nodes on the Palmetto Cluster.
3.5 RESULTS AND DISCUSSION

3.5.1 3-Day Rainfall Analyses

3.5.1.1 Average matric suction and degree of saturation

To show the impact of rainfall and water table depth on the bearing capacity and settlement of drill shaft, a 3-day continuous rainfall was considered. The intensity of the rainfall and the water table depth vary with season and therefore considered in a probabilistic manner to compute average pressure head and the corresponding degree of saturation within the influence zone of the drilled shaft. For that, 10,000 scenarios of rainfall intensity and water table depth were randomly selected using the Monte Carlo simulation technique based on the probability distribution of the resultant infiltration and water table data for each site. The variation of average matric suction and the corresponding degree of saturation were computed based on the 10,000 simulations and plotted as shown in Fig. 3.8 for both site locations. The degree of saturation in Riverside ranges between 54% and 96% while this range is between 81% and 97% for Salt Lake City. This difference may be attributed to the difference in the soil type, intensity of rainfall and water table depth.
Figure 3.8. Variation of average matric suction with different degrees of saturation considering various cases. Left: Riverside, Right: Salt Lake City

3.5.1.2 Elastic settlement

Changing in matric suction and degree of saturation, due to the site-specific climatic parameters, affect the soil stiffness which directly influences the elastic settlement of the drilled shaft. As is presented in Fig. 3.9, increasing the matric suction has a considerable impact on reducing the drilled shaft elastic settlement for both sites. Each line in the figures represents the 10,000 analysis considering the random resultant infiltration intensity and water table depth as the upper and lower boundary condition, respectively which is already discussed. For each analysis, the spatial variation of the degree of saturation and matric suction beneath the ground surface was determined. Then, the average degree of saturation and matric suction were calculated along the shaft skin and were used to compute the soil stiffness and subsequently the elastic settlement. Based on the results, the elastic settlement of the drilled shaft in Salt Lake City shows the greater amount for the same size in comparison with the Riverside. It was found that each case in Riverside site experiences a significant decrease within a range of 78 to 102 kPa.
matric suction which is caused by the existence of water table at the tip level for different simulations. Afterward, the settlement reduces gradually. This significant decrease in the total elastic settlement is mainly caused by the shaft tip settlement in which the groundwater level reduction is increased the soil stiffness. It should be noted that this effect is not captured in the Salt Lake City site because of shallow groundwater depth in this area where the water level is placed well-above the shaft tip level over the selected period.

3.5.1.3 Ultimate axial capacity

Regarding the shaft skin and tip resistance, the results of the proposed method depict an increasing trend for each shaft size due to an increase of matric suction in both locations (Figs. 3.10 and 3.11).
However, as is observed from elastic settlement results, the effect of water table also has a considerable effect on increasing the tip resistance in the same suction range which is already discussed and highlighted in the figure. Also, it can be seen that the shaft tip shows greater resistance in comparison with the skin for each shaft size.
The ultimate axial capacity of different drilled shafts based on the proposed method is presented in Fig. 3.12 for both sites. As shown in the figure, the trend of increasing the axial capacity is similar to the trend of shaft tip resistance in which the shaft depth controls the ultimate design values. Based on the results, it can be concluded that the soil matric suction, which highly depends on the degree of saturation of the soil profile and site-specific climatic loads, plays an important role in the ultimate axial capacity of the drilled shaft and subsequently its design procedure.

![Figure 3.12. Ultimate axial capacity of various drilled shaft after 3-day continuous rainfall Left: Riverside, Right: Salt Lake City](image)

**3.5.1.4 Drilled shaft size**

According to the results, the size of the shafts has a significant impact on the elastic settlement and ultimate axial capacity. In case of the elastic settlement, it can be seen that the width of the shaft has a greater impact compared to the depth factor. On the other hand, for the ultimate axial capacity, it is found that the shaft depth factor has more
impact on the skin resistance in comparison with the width factor, while this is vice versa for the tip resistance.

3.5.1.5 Foundation design values determination and comparison with deterministic approach

To assess the impact of the climatic parameters in the design of drilled shaft, a comparison between the proposed method and the deterministic approach, in which the soil is assumed fully saturated, is required. To this end, first, the design settlement and axial capacity should be determined based on the proposed method considering all the possible scenarios of climatic parameters for each study site, separately. The climatic parameters, as is discussed before, alter the settlement and axial capacity of the drilled shaft through changing the matric suction and the degree of saturation of the underlying soil. Thus, a simple way to determine the realistic design values is to find the mean of the best-fitted probability distribution to the average matric suction (or degree of saturation) of the site considering all the scenarios. Since these scenarios are selected based on the probability distribution of boundary conditions (resultant infiltration and water table depth) for each location, it can be concluded that the calculated average matric suction covers all the possible scenarios of climatic parameters for designing the drilled shaft. Therefore, the mean value of the best-fitted probability distribution to the matric suction is deemed to the best-selected input value for computing the design settlement and axial capacity. The same distributions, which were used for finding the best-fitted distribution of boundary conditions, are again considered here. Weibull distribution was deemed the best regression to the average matric suction based on Kolmogorov-Smirnov test (p-
value) for both locations. As shown in Fig. 3.13, the mean value of matric suction for Salt Lake City and Riverside are 23.4 kPa and 49.8 kPa, respectively. Using the mean values, the ultimate axial capacity and elastic settlement of each shaft can be easily found from Figs. 3.9 and 3.12 as the design values. It should be noted that the average degree of saturation can also be found for each site using the Fig. 3.8.

In order to compare the proposed and deterministic design approach in drilled shaft design criteria, the elastic settlement and ultimate axial capacity including skin and tip resistance of each case study were computed using Das’ and Kulhawý’s (Kulhawy 1990, Das 2010) general equations, respectively for both sites. The design values of each case study using the presented method are determined based on the mean of the best-fitted probability distribution to the average matric suction within foundation influence zone for each study location. As are shown in Table 3.3 and 3.4, the ultimate axial capacity of each drilled shaft increases by as much as 40% of the conventional method in
Salt Lake City, while this is utmost 56% for Riverside. In case of the settlement criteria, the total elastic settlement of each drilled shaft decreases by utmost 34% and 30% at Salt Lake City and Riverside, respectively. It can be concluded from this comparison that the effect of matric suction in design parameters of the drilled shaft depends on the SWCC and also inherent soil characteristics of a site location, which is highly relied on the climatic parameters and also water table level.

Table 3.3. Comparison of pile resistances and settlement computed based on proposed and conventional methods in Riverside

<table>
<thead>
<tr>
<th>Width (B) (m)</th>
<th>Depth (D) (m)</th>
<th>Fully Saturated Condition</th>
<th>Difference (%), [(Unsat-Sat)/Sat]*100%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$Q_{Skin}$ (kPa)</td>
<td>$Q_{tip}$ (kPa)</td>
</tr>
<tr>
<td>1.50</td>
<td>18.0</td>
<td>2659.78</td>
<td>15076.75</td>
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<tr>
<td>1.50</td>
<td>15.0</td>
<td>1905.97</td>
<td>12556.05</td>
</tr>
<tr>
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<td>15.0</td>
<td>1524.78</td>
<td>8041.96</td>
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<td>12.0</td>
<td>1021.10</td>
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<td>18.0</td>
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<td>15.0</td>
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<tr>
<td>0.90</td>
<td>12.0</td>
<td>765.82</td>
<td>3619.57</td>
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Table 3.4. Comparison of pile resistances and settlement computed based on proposed and conventional methods in Salt Lake City

<table>
<thead>
<tr>
<th>Width (B) (m)</th>
<th>Depth (D) (m)</th>
<th>Fully Saturated Condition</th>
<th>Difference (%), [(Unsat-Sat)/Sat]*100%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$Q_{Skin}$ (kPa)</td>
<td>$Q_{Tip}$ (kPa)</td>
</tr>
<tr>
<td>1.50</td>
<td>18.0</td>
<td>2654.79</td>
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<td>1.50</td>
<td>15.0</td>
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<tr>
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<tr>
<td>0.90</td>
<td>12.0</td>
<td>787.11</td>
<td>4201.88</td>
</tr>
</tbody>
</table>

3.5.2 Parametric Study-Impact of Rainfall Duration

In this section, different time durations (1, 3 and 5 days) are selected to assess the design parameters of a drilled shaft with the width of 0.9 m and depth of 12 m. The analysis is performed considering the same boundary conditions which were used for the previous set of analysis. Fig. 3.14 shows the spatial variation of the degree of saturation with different time duration of resultant infiltration. As is discussed before, the inherent soil characteristics and SWCC of each site has a direct effect on the water infiltration process and subsequently the water penetration depth. It can be seen from the figure that when the time duration increase from 1 to 5 days, water penetrates utmost 1.0 m into the subsurface area for Riverside, while this depth is almost 3.2 m for Salt Lake City. Although, the location of the water table presents a tendency to remain unchanged during
the analysis for all simulations. The reason is that the final depth of infiltrated water is always placed above the water table level throughout all simulations.

Based on the finding from the results, various time durations change the depth of infiltrated water and subsequently vary the matric suction and soil stiffness of subsurface specifically the area close to the upper shaft segments. This change affects the shaft skin resistance and ultimately the axial capacity and elastic settlement of drilled shaft.

![Figure 3.14. Degree of saturation profile at the end of each rainfall duration](image)

As is presented in Fig. 3.15, the total elastic settlement of the drilled shaft is obtained from the proposed method considering different time durations of resultant
infiltration. It is clear that since water does not penetrate too deep into the soil for the Riverside site, the average matric suction and degree of saturation of shaft segments does not experience too many changes over different time periods which leads to small changes of elastic settlement for the drilled shaft. However, the elastic settlement of Salt Lake City shows noticeable changes due to different durations for lower matric suction values.

The skin resistance of the drilled shaft with various time durations is shown in Fig. 3.16 for both locations.
Similar to the results of elastic settlement for Riverside, the variation of skin resistance does not present any discrepancy, while there is a small change in shaft skin resistance in the Salt Lake City region for different time durations, although at higher matric suction the results become closer to each other.

Finally, the ultimate axial capacity of the drilled shaft is calculated based on the proposed method considering different time periods and presented in Fig. 3.17. Like previous results for the Riverside region, the time parameter slightly affects the ultimate axial capacity which is caused by the fine-grained soil existing in that region. However, the influence of water infiltration period is noticeably observed in the Salt Lake City in which the differences are higher at the lower matric suction, while they get close to each other at greater matric suction. Also, it is cleared that for longer time periods, the results of the ultimate axial capacity decrease because of increasing the degree of saturation of soil profile which leads the subsurface to become close to the fully saturated condition.
3.6 SUMMARY AND CONCLUSION

The coupled geotechnical-climatic scheme defined in this study was used to incorporate the climatic and subsurface data with the deterministic methods used in drilled shaft design. This novel method evaluates ultimate axial capacity and elastic settlement due to matric suction and degree of saturation of the soil along the shaft skin. The resultant infiltration of rainwater and evapotranspiration through initially partially saturated soil was modeled using the one-dimensional Richards’ equation considering both resultant infiltration rate and water table location as the upper and lower boundary conditions, respectively. To calculate the axial capacity and settlement of various drilled shafts, the average degree of saturation and matric suction along the shaft skin for each pile segment were computed by applying 10,000 random input values corresponding to the resultant infiltration and water table distributions using Monte Carlo simulation.
Two sample sites were selected in this study to show the variation of ultimate axial capacity and elastic settlement with matric suction; Riverside, CA and Salt Lake City, UT. After a three days continuous water infiltration and ignoring the effect of surface runoff, the degree of saturation in Riverside was between 54% and 96%, and between 97% and 80% in Salt Lake City. The significant difference in the ranges is caused by the existence of the fine-grained soil in the Riverside region which decreases the soil porosity and make it less permeable. It is also found that considering the matric suction in a drilled shaft design increases the ultimate axial capacity of a shaft by as much as 40% of the conventional method using fully saturated condition in Salt Lake City, while this is utmost 56% for Riverside. In case of the settlement criteria, the total elastic settlement of drilled shaft decreases by utmost 34% and 30% at Salt Lake City and Riverside, respectively. Also, the results show that the water table level had a noticeable impact on the design parameters of drilled shafts specifically in Riverside in which each case experience a significant decrease in the total settlement and increase in the ultimate axial capacity within a range of 78 to 102 kPa matric suction. This result is caused by the existence of the water table at the shaft tip level for different simulations. This significant change is mainly caused by the shaft tip in which the groundwater level reduction is increased the soil stiffness

However, the effect of the matric suction can be changed depending upon the depth of water infiltration into the soil. Thus, different time durations (1, 3 and 5 days) are selected to assess the design parameters of a drilled shaft with the width of 0.9 m and depth of 12 m. As the results presented, the inherent soil characteristics of each site
location have a direct effect on the water infiltration process and subsequently the depth at which water penetrates. When the time increase from 1 to 5 days, water penetrates utmost 1.0 m into the subsurface area for Riverside, while it goes deeper for Salt Lake City which was almost 3.2 m. Therefore, various time durations change the depth of infiltrated water and subsequently vary the matric suction and soil stiffness of subsurface specifically the area close to the upper shaft skin. This change affects the shaft skin resistance and ultimately the axial capacity and elastic settlement of drilled shaft. Because of greater water penetration in Salt Lake City, increasing the time duration of resultant infiltration lead to a decrease of ultimate axial capacity and raise of settlement in that region, while the design parameters of Riverside remain mostly unchanged due smaller depth on water penetration.

3.6 REFERENCES


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CHAPTER 4

A PROCEDURE FOR INCORPORATING CLIMATIC AND WATER TABLE DATA IN THE GEOTECHNICAL DESIGN OF DRIVEN PILE SUBJECTED TO AXIAL LOAD

4.1 INTRODUCTION

The climatic events such as heavy rainfall, flood, and drought have become frequent in recent years. These events cause significant economic losses every year in the United States such as the 2015 Missouri flood and 2016 Oklahoma flood. Also, the United States has been struggling with the drought over a long period such as the severe drought in the Midwest regions in 2002. According to the National Climatic Data Centre (NCDC) currently, 30% of the United States suffers from moderate to severe drought. These climatic events have notable damages to the above-ground structures through the foundations (Steenbergen et al. 2009). Foundation as an interface between the structure and soil is highly affected by the properties of the underlying soil. The soil properties are greatly influenced by the climatic events through changing the saturation level of the soil. However, the climate impact on the soil properties and subsequently the foundation's performance. Therefore, to better understand the performance of foundations when subjected to a climatic event, the site-specific climatic data must be incorporated into the geotechnical design in addition to the site-specific geotechnical data.

The driven pile is one of the common types of deep foundation which is used to support superstructures and transfer its loads to the deep surface soil. The driven pile is commonly designed for the worst case geotechnical condition in which the soil is fully
saturated with the water table at the ground surface even if the historical water table is well below the pile tip and expected to be the same during the lifetime of the structure (Briaud 2013). This way of design is too conservative since the underlying soil is mostly under partially saturated condition and water table fluctuates with time. Recent case studies indicate that variation of the soil degree of saturation and matric suction significantly affect the shear strength and deformation parameters (Fredlund et al. 1978, Vanapalli et al. 1996, Guan et al. 2010). The change of soil shear strength subsequently affects the settlement and bearing capacity of the foundation. Numerous research studies have been investigated the influence of matric suction and degree of saturation on the load carrying capacity of the shallow and deep foundation through plate load tests and model footing tests (Georgiadis et al. 2003, Vanapalli and Taylan 2012). Regarding soil stiffness, many researchers have studied the impact of matric suction on soil modulus elasticity and proposed semi-empirical model (Agarwal and Rana 1987, Oh et al. 2009). Thus, it can be concluded that the deterministic design approach considering that the soil is fully saturated can be either conservative or unconservative, depending on the site condition where the near-surface soil is initially partially saturated during the design life of the structure, and it is highly affected by the climatic events.

In addition to the efforts mentioned above, various research has been conducted in recent years to address the impact of climatic events on the performance of shallow foundation (Vahedifard and Robinson 2016, Kim et al. 2017, Ravichandran et al. 2017, Mahmoudabadi and Ravichandran 2017 and 2018). Since none of the abovementioned studies have addressed the performance of deep foundation under climatic events, thus in
this study a new framework for coupling the site-specific climatic parameters and water table depth with geotechnical parameters is developed based on partially saturated soil mechanics principles to accurately compute the axial capacity and settlement of driven pile.

4.2 A FRAMEWORK FOR INCORPORATING THE CLIMATIC DATA IN GEOTECHNICAL DESIGN PROCESS OF DRIVEN PILE

In order to incorporate the climatic data in the design process of a driven pile, first, the one-dimensional Richards’ equation was numerically solved to compute the temporal and spatial variation of the degree of saturation and matric suction along the pile skin and below the influence zone of the pile tip. The resultant infiltration and the water table depth were considered as the top and bottom boundaries, respectively for solving the Richards equations. Because the intensity and duration of rainfall and the depth of water table vary with time (seasonal and historical variation), the Monte Carlo simulation technique was used to generate the resultant infiltration and water table depth based on the site-specific historical rainfall, evapotranspiration, and water table depth in a probabilistic manner. In the next step, the computed degree of saturation and corresponding matric suction were then used to update the strength and settlement properties of the surrounding soil and the soil-pile interface. Since the degree of saturation varies with depth, the pile was divided into a number of segments along its length for accurately computing the skin resistance. Finally, the ultimate axial capacity and elastic settlement were calculated using the updated soil and interface properties. The details of these key steps are presented below.
4.2.1 Mathematical Model for Flow of Water in Soil and Solution Procedure

The one-dimensional Richards’ equation (Richards 1931), a nonlinear partial differential equation shown in Eq. 4.1, is used to model the flow of water through the partially saturated soil in this study. Although the problem considered in this study is three-dimensional in nature, it is assumed that the one-dimensional model is reasonably accurate to predict the vertical movement of the water (Van Dam et al. 2000).

\[
\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z} \left[ K \left( 1 + \frac{\partial \psi}{\partial z} \right) \right]
\]  

(4.1)

where \( \theta \) is the moisture content, \( t \) is the time, \( K \) is the hydraulic conductivity of partially saturated soil, \( \psi \) is the pressure head, \( z \) is the depth from the ground surface, and \( \frac{\partial \psi}{\partial z} \) is the hydraulic gradient. Since the pressure head is considered as the primary variable to be solved in this study, the two other variables in Richards’ equation, \( \theta \) and \( K \), are required to be expressed as functions of \( \psi \). The hydraulic conductivity of partially saturated soil is expressed as \( K = K_{sat}K_r \), where \( K_{sat} \) is the hydraulic conductivity of the soil at fully saturated condition and \( K_r \) is the relative hydraulic conductivity of the soil at partially saturated condition. In this study, both \( \theta \) and \( K_r \) are expressed as functions of \( \psi \) using a Soil Water Characteristic Curve (SWCC). Among the many SWCCs and corresponding relative hydraulic conductivity functions, the equations proposed by van Genuchten (1980) were used in this study. Finally by using the SWCC equations and relative hydraulic conductivity functions, the Eq. 4.1 is spatially and temporarily
discretized and solved using Finite Volume Method (FVM) by applying appropriate initial and boundary conditions which is shown in Eq. 4.2.

\[
C_{m,m}^{i+\Delta, i} \left( \psi_{i, m+1}^{i+\Delta, i} - \psi_{i, i}^{i+\Delta, i} \right) 
= \left[ \left( K_{i+1/2}^{i+\Delta, i} \psi_{i+1/2}^{i+\Delta, i} - K_{i-1/2}^{i+\Delta, i} \psi_{i-1/2}^{i+\Delta, i} \right) \right] \Delta t
\]

(4.2)

where \( C \) is the specific moisture capacity \( (\partial \theta / \partial \psi) \), \( m \) is the iteration number, \( i \) is the soil compartment node and \( \Delta z \) is the depth interval. A MATLAB code was developed and installed on the Clemson University’s supercomputer for solving this equation and to perform other calculations for a number of upper and lower boundary conditions. In this study, both pressure head and flux boundary conditions were applied at the upper boundary depending upon the intensity of the resultant infiltration and the specific moisture capacity. The water table depth was considered as the lower boundary. In addition, the soil was assumed to be at the residual condition (residual stage in SWCC) at the beginning of each simulation. Since the water table depth and resultant infiltration vary with climatic conditions for a location, appropriate values must be determined in a probabilistic manner considering historical rainfall, evapotranspiration, and water table data.

4.2.1.1 Resultant infiltration-upper boundary condition

The resultant infiltration, \( F_{\text{Resultant Infiltration}} \) as the upper boundary condition, is computed from subtraction of the in-flux from out-flux climatic parameters for each specific site which is expressed in Eq. 4.3,
\[
F_{\text{Resultant Infiltration}} = \text{Influx} - \text{Outflux} = (F_{\text{Rainfall}}) - (F_{\text{Evapotranspiration}} + F_{\text{Runoff}}) \quad (4.3)
\]

where \(F_{\text{Rainfall}}\) is the historical rainfall intensity, \(F_{\text{Evapotranspiration}}\) is the evapotranspiration intensity, and \(F_{\text{Runoff}}\) is the surface runoff. The site-specific rainfall data was obtained from the NCDC which records daily rainfall values. In this study, the annual maximum series were used and constructed by extracting the highest precipitation in each successive year over 76 years. The evapotranspiration is dependent on the other environmental factors such as temperature, daylight time, and saturated vapor density, which can be easily computed based on the Hamon method (1961) in terms of potential evapotranspiration (PET) that is expressed in Eq. 4.4,

\[
F_{\text{Evapotranspiration}} = PET = 0.1651 \times L_d \times RHOSAT \times KPEC \quad (4.4)
\]

where \(L_d\) is daytime length, \(RHOSAT\) is saturated vapor density at mean temperature, and \(KPEC\) is a calibration coefficient equal to 1. Using this method, the daily potential evapotranspiration of the same time period was calculated based on the temperature values which were obtained from the National Oceanic and Atmospheric Administration (NOAA). In order to consider the surface runoff, the runoff value was calculated using Eq. 4.5 based on the method developed by USDA-TR55 (1986). The residential district is considered as the site hydrological condition.

\[
F_{\text{Runoff}} = \frac{(P - 0.2 S)^2}{P + 0.8 S} \quad (4.5)
\]

where \(P\) is rainfall and \(S\) is potential maximum retention after runoff begins.
4.2.1.2 Water table-lower boundary condition

The water table depth is another factor which affects the matric suction and degree of saturation of a partially saturated soil within the influence zone of the pile. The required data was taken from the U.S. Geological Survey (USGS 2016) and the National Water Information System for the same time period which was assumed for the rainfall data.

4.2.2 Coupled Geotechnical-Climatic Design of Driven Pile

4.2.2.1 Axial capacity

The ultimate axial capacity of a driven pile, $Q_{ult}$, was calculated using the simplified equation shown in Eq. 4.6 (Kulhawy et al. 1983),

$$Q_{ult} = Q_{Skin} + Q_{Tip}$$

(4.6)

where $Q_{Skin}$ is the skin resistance and $Q_{Tip}$ is the tip resistance. To calculate the ultimate axial capacity of a driven pile, first, the skin and tip resistances of the driven pile need to be computed considering the soil matric suction and degree of saturation.

4.2.2.1.1 Skin resistance

Among the many available equations, the skin resistance equation proposed by Vanapalli and Taylan (2012) was used in this study to predict the nonlinear variation of skin resistance, $Q_{Skin}$, in partially saturated soils for a circular pile (Eq. 4.7). The proposed equation considers the contribution of matric suction toward the skin resistance,
\(Q^{\text{Skin}(u_a-u_w)}\), as an additive term to the conventional method considering the soil is fully saturated, \(Q^{\text{Skin(sat)}}\).

\[
Q_{\text{Skin}} = \sum_{j=1}^{p} (Q_{j}^{\text{Skin(sat)}} + Q_{j}^{\text{Skin}(u_a-u_w)}) = A_s \sum_{j=1}^{p} (f_{j}^{\text{sat}} + f_{j}^{(u_a-u_w)}) \\
= \sum_{j=1}^{p} \left[ c'_{a} + \beta_{j} \sigma'_{z(j)} + (u_a - u_w)_{avg(j)} (S_{j}^{*}) \tan \delta^{*} \right] \pi BD
\]

where \(j\) is the pile segment number, \(f_{j}^{(u_a-u_w)}\) is the contribution of matric suction towards the unit skin resistance for \(j\)th segment, \(f_{j}^{\text{sat}}\) is unit skin resistance at fully saturated condition for \(j\)th segment, \(A_s\) is the pile perimeter, \(p\) is the total number of pile segment, \(c'_{a}\) is the soil adhesion for saturated condition, \(\beta_{j}\) is the Burland-Bjerrum coefficient for \(j\)th segment which is equal to \(K_0 \tan \delta^{*}\) (\(K_0\) is mean lateral earth coefficient at rest), \(\delta^{*}\) is effective angle of interface between the soil and pile skin (Tariq and Miller 2009), \(\sigma'_{z(j)}\) is vertical effective stress for \(j\)th segment, \((u_a-u_w)_{avg(j)}\) is the average matric suction of \(j\)th segment, \(S_{j}\) is degree of saturation for \(j\)th segment, \(B\) is the pile diameter, \(D\) is the pile length, and \(\kappa\) is the fitting parameter.

4.2.2.1.2 Tip resistance

In term of the tip resistance of a driven pile, \(Q_{\text{Tip}}\), the method presented by Kulhawy et al. (1983) was employed in this study (Eq. 4.8) in which the impact of the partially saturated condition is considered through the change of unit weight of the soil and also the effective stress at the tip point, \(\sigma'_{z(Tip)}\).

\[
Q_{\text{Tip}} = q_{\text{Tip}} A_{\text{Tip}} = (\bar{N} B N^{*} + \sigma'_{z(Tip)} N^{*}) 0.25 \pi B^{2}
\]
where $q_{\text{Tip}}$ is the tip bearing capacity, $A_{\text{Tip}}$ is the pile tip cross-section area, $\gamma$ is the average unit weight from depth $D$ to $D+B$, $N^*_{\gamma}$ and $N^*_{q}$ are the non-dimensional bearing capacity factors that are functions of the effective soil friction angle. The detail of these factors are provided by Kulhawy et al. (1983). Fig. 4.1 displays the discretization procedure, boundary conditions and the process of calculating the skin and tip resistance for the driven pile in the partially saturated soil.

![Figure 4.1. Procedure for estimating soil resistances in partially saturated soils](image)

### 4.2.2.2 Elastic Settlement

The total elastic settlement of a driven pile, $S_e$ was calculated using Eq. 4.9 (Das 2010),

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)} = \left(\frac{Q_{\text{Tip}}}{A_p E_p} + \frac{Q_{\text{Skin}}}{A_p D E_{\text{soil}}} \right) + \frac{q_{\text{Tip}} B}{E_{\text{soil}}} (1 - \mu^2_s) I_{\text{tip}} + \left(\frac{Q_{\text{Skin}}}{A_p D E_{\text{soil}}} \right) \frac{B}{E_{\text{soil}}} (1 - \mu^2_s) I_{\text{tip}} \quad (4.9)$$

where $S_{e(1)}$ is the elastic settlement of driven pile, $S_{e(2)}$ is the settlement of driven pile caused by the load at the pile tip, $S_{e(3)}$ is the settlement of driven pile caused by the load
transmitted along the pile skin, $Q_{wtip}$ is the load carried at the pile tip under working load condition, $Q_{wskin}$ is the load carried by skin resistance under working load condition, $\zeta$ is a coefficient which depends on the distribution of the unit skin resistance along the pile skin and it is assumed 0.67, $A_p$ is the cross-section area of pile, $E_p$ is the modulus of elasticity of the pile material (concrete), $q_{wtip}$ is the pile tip load per unit area under working load condition, $E_{soil}$ is the soil modulus of elasticity at strain level of 1%, $\mu_s$ is the Poisson’s ratio of soil, $I_{wp}$ and $I_{ws}$ are the pile tip and skin influence factor, respectively. In addition to the shear strength parameters, the soil modulus of elasticity ($E_{soil}$) is another parameter which is affected by the degree of saturation and matric suction of the soil profile along the pile skin. Since the degree of saturation and the matric suction are computed following the procedure described before, the elastic settlement can be computed if $E_{soil}$ is also expressed as a function of the degree of saturation and matric suction. In this study, the equation proposed by Oh et al. (2009) shown in Eq. 4.10 was used to estimate the modulus of elasticity in a partially saturated condition ($E_{soil(unsat)}$).

$$E_{soil(unsat)} = E_{soil(sat)} \left[1 + \alpha_e (u_a - u_w)S^\beta_e\right]$$  \hspace{1cm} (4.10)

where $E_{soil(sat)}$ is the modulus of elasticity in a saturated condition at strain level of 1%, $\alpha_e$ and $\beta_e$ are fitting parameters.

**4.3 APPLICATION AND SAMPLE SIMULATION PROCESS**

The application of the proposed procedure requires the computation of temporal and spatial variation of matric suction and degree of saturation considering the site-
specific resultant infiltration and water table records. These two random variables are considered as the boundary conditions which change with different return periods. Therefore, the primary variables can be best estimated using probabilistic methods.

4.3.1 Studied Site Location

Victorville, CA was considered as the sample site in this study due to its semi-arid climate. The site soil type is Adelanto Loam (SM). The SWCC parameters of the Adelanto Loam soil were taken from the report by Zhang (2010) and the soil strength parameters of the site were obtained from a geotechnical report by GSH Geotechnical Inc. and Web Soil Survey developed by U.S. Department of Agriculture (USDA). The key input parameters are tabulated in Table 4.1.

<table>
<thead>
<tr>
<th>SWCC fitting parameters</th>
<th>Value</th>
<th>Geotechnical parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturated volumetric water content, $\theta_s$</td>
<td>0.423</td>
<td>Dry unit weight, $\gamma_d$ (kN/m$^3$)</td>
<td>16.20</td>
</tr>
<tr>
<td>Residual volumetric water content, $\theta_r$</td>
<td>0.158</td>
<td>Void ratio, $e$</td>
<td>0.605</td>
</tr>
<tr>
<td>Model parameter, $\alpha$ (m$^{-1}$)</td>
<td>0.321</td>
<td>Effective friction angle, $\phi'$ (deg)</td>
<td>33</td>
</tr>
<tr>
<td>Model parameter, $n$</td>
<td>2.11</td>
<td>Effective adhesion, $c'_a$ (kPa)</td>
<td>0</td>
</tr>
<tr>
<td>Hydraulic conductivity, $K_{sat}$ (cm/hr)</td>
<td>0.21</td>
<td>Plasticity index, $(I_p)$</td>
<td>5</td>
</tr>
</tbody>
</table>

4.3.2 Site Specific Resultant Infiltration and Water Table Distributions

To determine the best fitting distributions for the resultant infiltration and water table data, the probability paper plotting technique was used. Various probability distributions including Gumbel Frechet and Weibull distribution were checked for the best fit, and the Gumbel distribution was deemed to be the best regression for both input
variables. The Gumbel distribution parameters, $\mu_n$ and $\beta_n$, can be determined using Eq. 4.11,

$$x_v = y_v \beta_n + \mu_n = -\ln \left( -\ln \left( \frac{v}{r+1} \right) \right) \beta_n + \mu_n$$  \hspace{1cm} (4.11)$$

where $v$ is the data index (arranged in increasing order), $x_v$ is the annual maximum historical rainfall or water table data, $y_v$ is the linearized form of the cumulative density function of Gumbel distribution, and $r$ is the number of data points. The probability plots of the resultant infiltration and water table depth based on the Gumbel distribution is shown in Fig. 4.2.

![Gumbel distributions for water table depth and resultant infiltration for Victorville, CA site](image)

**Figure 4.2.** Gumbel distributions for water table depth and resultant infiltration for Victorville, CA site

### 4.3.3 Climatic Loads

The climatic load is applied in the proposed framework through upper and lower boundary conditions as well as the time duration of the climate event. As described before, the resultant infiltration and water table depth are applied through the site-specific probability distribution function (Eq. 4.11) for 10,000 random cases using Monte Carlo
method. In addition, the time duration of the climatic event was assumed to be 4 days in this study.

### 4.3.4 Structural Load and Driven Pile Size

Circular pile with diameter, $B = 0.3, 0.5, \text{ and } 0.7\, \text{m}$ and length, $D = 10, 13, \text{ and } 16\, \text{m}$ were used in this study to investigate the infiltration effect for different pile sizes. The applied working load for each driven pile was calculated based on the fully saturated soil condition, and a factor of safety equals to 3.0 for both skin and tip resistance.

### 4.4 RESULTS AND DISCUSSION

After analyzing all the simulations for a 4-day continuous rainfall, the computed average matric suction along the pile skin ranges between 64 and 96 kPa. Fig. 4.3 shows the variation of skin and tip resistance of various pile sizes with matric suction. In general, an increase in the matric suction (or decrease in the degree of saturation) slightly increase the pile skin and tip resistance. However, this increase is significant for the piles with 16 m depth that it is because of existing the water table at the pile tip level which also reduces the average matric suction.
Fig. 4.4 shows the variation of ultimate axial capacity and elastic settlement of various driven piles with matric suction. In general, an increase in the matric suction reduces the elastic settlement and increases the ultimate axial capacity. As shown in the figure, the increasing pattern of the axial capacity is similar to the skin resistance since it has more contribution to the ultimate capacity. Based on the results, it should be noted that the width of the pile has a greater effect on the ultimate axial capacity and settlement rather than the depth factor.
In order to investigate the impact of the matric suction in the pile design, a comparison between the presented method and the conventional design procedure, in which the soil is assumed to be fully saturated, is required. A simple way to determine the ultimate axial capacity and settlement from the proposed method is to find the site-specific design matric suction considering all the simulations. Since these simulations are performed based on the probability distribution of the boundary conditions, it can be concluded that the calculated average matric suction cover all the possible input values for designing the driven pile. Therefore, the mean value of the best-fitted probability distribution to the matric suction is deemed to the best-selected input values for computing the realistic design values of the driven pile. The same distributions, which were used for finding the best-fitted distribution of boundary conditions, are again considered here (Fig. 4.5).
Weibull distribution was deemed the best regression to the average matric suction based on the Kolmogorov-Smirnov test (p-value) for the study location. As shown in Fig. 4.5, the mean value of matric suction for the studied site is 87.8 kPa. The corresponding value of pile skin resistance, tip resistance, ultimate axial capacity, and settlement are then compared with the ones computed using Das’s (2010) and Kulhawy’s general equations considering fully saturated conditions (Table 4.2). As shown in Table 4.2, the skin resistance, tip resistance and ultimate axial capacity obtained using the proposed coupled geotechnical-climatic method are almost 160%, 80% and 130%, respectively greater than those computed using the conventional method. Furthermore, the elastic settlement computed from the proposed method is approximately 40% smaller than that of the conventional method.

Figure 4.5. Best-fitted distribution of predicted average matric suction within the length of the pile
Table 4.2. Comparison of soil resistances and settlement computed based on proposed and conventional methods

<table>
<thead>
<tr>
<th>Width (B) (m)</th>
<th>Depth (D) (m)</th>
<th>Fully Saturated Condition</th>
<th>Difference (%), (((\text{Unsat-Sat/} \text{Sat}) \times 100%)</th>
<th>$Q_{\text{Skin}}$ (kPa)</th>
<th>$Q_{\text{TIP}}$ (kPa)</th>
<th>$Q_{\text{ULT}}$ (kPa)</th>
<th>$S_e$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.30</td>
<td>10.0</td>
<td>285.90 98.25 384.15 5.15</td>
<td>186.00 80.59 159.04 -40.19</td>
<td>285.90</td>
<td>98.25</td>
<td>384.15</td>
<td>5.15</td>
</tr>
<tr>
<td>0.30</td>
<td>13.0</td>
<td>483.17 127.42 610.59 5.71</td>
<td>165.47 80.42 147.72 -34.68</td>
<td>483.17</td>
<td>127.42</td>
<td>610.59</td>
<td>5.71</td>
</tr>
<tr>
<td>0.30</td>
<td>16.0</td>
<td>731.91 156.58 888.49 6.52</td>
<td>145.09 73.23 132.43 -30.67</td>
<td>731.91</td>
<td>156.58</td>
<td>888.49</td>
<td>6.52</td>
</tr>
<tr>
<td>0.50</td>
<td>10.0</td>
<td>476.50 274.82 751.33 7.99</td>
<td>186.00 80.59 147.44 -42.55</td>
<td>476.50</td>
<td>274.82</td>
<td>751.33</td>
<td>7.99</td>
</tr>
<tr>
<td>0.50</td>
<td>13.0</td>
<td>805.29 355.84 1161.13 8.41</td>
<td>163.85 80.18 138.21 -40.67</td>
<td>805.29</td>
<td>355.84</td>
<td>1161.13</td>
<td>8.41</td>
</tr>
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<td>0.50</td>
<td>16.0</td>
<td>1219.84 436.86 1656.7 8.98</td>
<td>145.09 72.91 126.06 -36.30</td>
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<td>436.86</td>
<td>1656.7</td>
<td>8.98</td>
</tr>
<tr>
<td>0.70</td>
<td>10.0</td>
<td>667.10 542.40 1209.5 10.88</td>
<td>186.00 80.59 138.73 -43.29</td>
<td>667.10</td>
<td>542.40</td>
<td>1209.5</td>
<td>10.88</td>
</tr>
<tr>
<td>0.70</td>
<td>13.0</td>
<td>1127.4 701.19 1828.59 11.25</td>
<td>163.85 80.54 131.91 -42.04</td>
<td>1127.4</td>
<td>701.19</td>
<td>1828.59</td>
<td>11.25</td>
</tr>
<tr>
<td>0.70</td>
<td>16.0</td>
<td>1707.78 859.98 2567.76 11.73</td>
<td>145.09 72.59 120.81 -38.36</td>
<td>1707.78</td>
<td>859.98</td>
<td>2567.76</td>
<td>11.73</td>
</tr>
</tbody>
</table>

4.5 CONCLUSION

In general, the proposed method considering site-specific climatic data predicts higher axial capacity and lower settlement compared to the ones based on the conventional soil mechanics principles. Also, it is observed that the water table has a significant impact on the axial capacity and elastic settlement when the water table depth rises above the pile tip. Based on the observations of this study, it can be concluded that the proposed framework has improved the design procedure of driven pile through a new physics that relies on the more accurate and realistic input data. It is worth mentioning that the proposed framework can also be applied to various other earthen structures to improve their design for economical advantages, sustainability, and resiliency.

4.6 REFERENCES


GSH Geotechnical Inc. (2013). “Report Supplemental Geotechnical Study Lots 1901 To 1907, Eaglepoint Estates Approximately 950 South Parkway Drive North Salt Lake.” Salt Lake, UT.


CHAPTER 5
SHALLOW FOUNDATION DESIGN DURING EXTREME HYDROLOGICAL CYCLE AND WATER TABLE FLUCTUATION–THEORETICAL FRAMEWORK AND APPLICATION

5.1 INTRODUCTION

Recent records of climate data indicate an increase in the frequency of hydrological events, such as heavy rainfall, drought, and flood, followed by the variation of hydrological parameters including the precipitation and air temperature (IPCC, 2013). Current climatic models predict that the air temperature increases about 2°F across the United States by the end of the century with a prolonged period of drought (Melillo et al. 2014, Cheng et al., 2015). This significant increase of temperature is also observed by the historical records of temperature recorded across the United States (Mazdiyasni and AghaKouchak 2015, Shukla et al. 2015, Damberg et al. 2014). These changes in air temperature directly and/or indirectly affect the climate pattern which subsequently leads to an extreme hydrological event such as heavy rainfall, drought, and flood (Moftakhari et al. 2017, Hao et al. 2013, AghaKouchak et al. 2014). The impacts of such hydrological events are felt not only by the human beings but also by the built infrastructure systems, especially the earthen structure which transfer the structural loads to the subsurface soil. Thus, the current geotechnical design method of earthen structures should be evolved so that the impacts of emerging and projected hydrological events are incorporated into the design procedure for improving the resiliency, sustainability, and performance of these types of structures.
Shallow foundations are widely used to support small to medium size of structures and transfer its loads to the near-surface soil. This foundation is generally designed for the worst case geotechnical conditions that the soil is fully saturated with the water table at the ground surface even if the historical water table is well below the influence zone of the shallow foundation and expected to remain unchanged during the lifetime of the structure (Kulhawy 1990, Das 2010, Briaud 2013). Such a design procedure is conservative since the underlying soil is mostly under partially saturated condition and water table fluctuates with time. Recent case studies indicate that the variation of the soil degree of saturation and the matric suction due to hydrological events significantly affect the soil shear strength and stiffness (Fredlund et al. 1978 and 2012, Lu and Likos 2004). In recent years, a number of efforts have been undertaken to assess the effect of hydrological events on the mechanical behavior of variably saturated soils. These studies mostly have focused on changing the soil shear strength and soil inter-particle force under hysteresis wetting and drying front of underlying soil due to the hydrological events (Han et al. 1995, Nishimura and Fredlund 2002, Rahardjo et al. 2004, Thu et al. 2006, Melinda et al. 2004). In addition to the studies mentioned above, some research has been addressed the impact of the hydrological events on the behavior of various geotechnical (Vardon 2015, Turnbull 2016). In recent years, Numerous studies have been investigated the impact of the intensity and duration of precipitation on the slope stability, levee stability and earth walls problem through either numerical method or experimental tests (Rahardjo et al. 1995, Kim et al. 2004, Lu and Likos 2006, Vahedifard et al. 2015, Robinson et al. 2017, Jasim et al. 2017, Vahedifard et al. 2017a.
and 2017b). Following these research, limited number of studies have been addressed the impact of rainfall intensity and water table fluctuation on the performance of shallow foundation (Vahedifard and Robinson 2016, Kim et al. 2017, Ravichandran et al. 2017, Mahmoudabadi and Ravichandran 2017, Mahmoudabadi and Ravichandran 2018a) and deep foundations (Mahmoudabadi and Ravichandran 2018b).

While several large-scale studies have been conducted to assess various aspects of climate change, there is still a clear gap in the state of knowledge in terms of assessing the sustainability and resiliency of the earthen structures against hydrological events. This study aims to develop a procedure for incorporating the site-specific extreme hydrological cycle and its corresponding hydrological loads to improve the performance of shallow foundation. The design process of shallow foundation subjected to extreme hydrological cycle entails: (1) developing a geotechnical-hydrological model (2) developing a mathematical model for computing the spatial and temporal variation of water content and pressure head of underlying soil due to hydrological loads, (3) implementing and solving the mathematical model using the precipitation (or evapotranspiration) and water table depth as the upper and lower boundary conditions, respectively, (3) determine the site-specific extreme hydrological cycle based on the Standardized Precipitation Evapotranspiration Index (SPEI) (Vicente-Serrano 2010) and the corresponding hydrological loads during the cycle (4) computing the average matric suction and soil degree of saturation within the influence zone of the foundation during the cycle, (5) computing the ultimate bearing capacity and settlement using the developed geotechnical-hydrological model, and (6) determining the critical values of the settlement.
and ultimate axial capacity as the design values of the shallow foundation. In this study, two sites in the United States are selected to demonstrate the proposed procedure. The details of each of these steps are presented below.

5.2 COUPLED GEOTECHNICAL-CLIMATIC DESIGN OF SHALLOW FOUNDATION

5.2.1 Estimation of Elastic Settlement in Variably Saturated Soil

The elastic settlement of the foundation is calculated using the simplified equation proposed by Bowles (1987) (Eq. 5.1),

\[ S_e = q_0 \left( \alpha_s B' \right) \frac{1 - \mu_s}{E_{soil}} I_s I_f \]

where \( S_e \) is the elastic settlement of the foundation, \( q_0 \) is the net pressure at the bottom of the foundation due to applied structural load, \( \alpha_s \) is a non-dimensional parameter that depends on the point at which settlement is calculated for a flexible foundation, \( B' \) is the effective dimension of the foundation, \( \mu_s \) is the Poisson’s ratio, \( I_s \) and \( I_f \) are factors associated with the shape and depth of the foundation, respectively, and \( E_{soil} \) is the average modulus of elasticity of the soil within the influence zone. Of all these parameters, \( E_{soil} \) is the only parameter that is affected by the degree of saturation and matric suction of the soil within the influence zone. Since the degree of saturation and the matric suction are computed following a numerical procedure which will be discussed in the next section, the elastic settlement can be computed if \( E_{soil} \) is expressed as a function of the degree of saturation and matric suction.
5.2.1.1 Modulus of Elasticity

The effect of the hydrological loads in predicting the soil modulus of elasticity through changing the soil matric suction and degree of saturation has been the subject of fairly extensive studies in recent years (Agarwal and Rana 1987, Schnaid et al. 1995, Costa et al. 2003, Rojas et al. 2007, Oh et al. 2009, Vanapalli and Adem 2013). In this study, the equation proposed by Oh et al. (2009), shown in Eq. 5.2, is used to estimate the modulus of elasticity in variably saturated soils ($E_{soil(unsat)}$).

$$E_{soil(unsat)} = E_{soil(sat)} \left[1 + \alpha_e (u_a - u_w) S^\beta_e \right]$$

(5.2)

where $E_{soil(sat)}$ is the modulus of elasticity under fully saturated condition at a strain level of 1%, $(u_a - u_w)$ is the matric suction, $\alpha_e$ and $\beta_e$ are fitting parameters. For coarse- and fine-grained soils, the recommended fitting parameter, $\beta_e$ is equal to 1 and 2, respectively. Also, the fitting parameter $\alpha_e$ depending upon the plasticity index ($I_p$) can be computed using the following empirical equation (Eq. 5.3), developed by Oh et al. (2009).

$$\frac{1}{\alpha_e} = 0.5 + 0.063(I_p) + 0.036(I_p)^2 \quad (0 \leq I_p (%) \leq 16)$$

(5.3)

5.2.2 Estimation of Ultimate Bearing Capacity in Variably Saturated Soil

Various empirical equations have been proposed to address the contribution of the hydrological loads as a function of matric suction and degree of saturation in predicting the ultimate bearing capacity of a shallow foundation in variably saturated soil (Steensen-Bach et al. 1987, Mohamed and Vanapalli 2006, Vanapalli and Mohamed 2007, Oh and Vanapalli 2009, Oh and Vanapalli 2013). Of the many available equations, the ultimate
bearing capacity equation proposed by Vanapalli and Mohamed (2007) was used in this study to estimate the nonlinear variation of ultimate bearing capacity in variably saturated soils \( (q_{u\text{(unsat)}}) \) with respect to the matric suction and degree of saturation (Eq. 5.4),

\[
q_{u\text{(unsat)}} = \left[ c' + \left( u_a - u_w \right)_b \left( \tan \phi' - S'^{\psi_a} \tan \phi' \right) + \left( u_a - u_w \right)_{\text{avg}} S'^{\psi_a} \tan \phi' \right] N_c \zeta_c \zeta_d
+ \gamma D N_q \zeta_{q_q} \zeta_{q_d} + 0.5 \gamma B N_{\gamma} \zeta_{\gamma_q} \zeta_{\gamma_d}
\] (5.4)

where \( c' \) is the effective cohesion intercept, \( D \) is the foundation depth, \( B \) is the foundation width, \( \gamma \) is the average unit weight of soil within the foundation influence zone (from depth \( D \) to \( D+1.5B \)), \( N_c, N_q \) and \( N_{\gamma} \) are the non-dimensional bearing capacity factors that are functions of the soil effective friction angle \( \phi' \), \( \xi_{a} \) and \( \xi_{d} \) are the shape and depth factors, respectively (Table 5.1), \( (u_a-u_w)_b \) is the air entry value which is computed from the SWCC, \( (u_a-u_w)_{\text{avg}} \) is the average matric suction within the foundation influence zone, \( S \) is the degree of saturation, and \( \psi_a \) is the shear strength fitting parameter which is expressed in Eq. 5.5 (Vanapalli and Mohamed 2007).

\[
\psi_a = 1.0 + 0.34 \left( I_p \right) - 0.0031 \left( I_p \right)^2
\] (5.5)

where \( I_p \) is the soil plasticity index. The average matric suction in the above bearing capacity equation is calculated using Eq. 5.6,

\[
(u_a - u_w)_{\text{avg}} = \frac{\sum_{i=1}^{p} (u_a - u_w)_i}{p} = \frac{\sum_{i=1}^{p} [-\psi_a \gamma_w]}{p} = \psi_{\text{avg}} \gamma_w
\] (5.6)
where \((u_r-u_w)\) is the matric suction at \(i^{th}\) node, \(p\) is the last soil node within the foundation influence zone, \(\psi_i\) is the pressure head at \(i^{th}\) node, \(\psi_{avg}\) is the average pressure head within the foundation influence zone, and \(\gamma_w\) is the unit weight of water. The average matric suction and degree of saturation are key variables, which affect the ultimate bearing capacity of the soil within the influence zone of foundation and were calculated using the procedure described above. The equations of the bearing capacity factors for the shallow foundation are provided in Table 5.1.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Reference</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>(N_q)</td>
<td>Terzaghi (1943)</td>
<td>(e^{2(\pi/4-\phi'/2)\tan\phi/(2\cos^2(\pi/4+\phi'/2))})</td>
</tr>
<tr>
<td>(N_c)</td>
<td>Terzaghi (1943)</td>
<td>(\cot(\phi(N_q-1)))</td>
</tr>
<tr>
<td>(N_r)</td>
<td>Kumbhojkar (1993)</td>
<td>(0.5\tan((K_p\gamma/\cos^2(\phi'))-1))</td>
</tr>
<tr>
<td>(\xi_{cs})</td>
<td>Vesic (1973)</td>
<td>(1+(B/L)(N_q/N_c))</td>
</tr>
<tr>
<td>(\xi_{cd})</td>
<td>Vesic (1973)</td>
<td>(\xi_{qs}-(1-\xi_{qs})/(N_q\tan\phi'))</td>
</tr>
<tr>
<td>(\xi_{qph})</td>
<td>Vesic (1973)</td>
<td>(1+(B/L)\tan\phi')</td>
</tr>
<tr>
<td>(\xi_{qpd})</td>
<td>Vesic (1973)</td>
<td>(1+2\tan(1-\sin\phi')^2(D/B))</td>
</tr>
<tr>
<td>(\xi_{qsd})</td>
<td>Vesic (1973)</td>
<td>(1-0.4(B/L))</td>
</tr>
</tbody>
</table>

\(K_p\gamma\): Passive pressure coefficient  
\(L\): Foundation length

As is expressed in this section, the ultimate axial capacity and settlement of shallow foundation directly relate to the degree of saturation and matric suction of underlying soil. Thus, these parameters need to be accurately calculated for any site condition. The procedure of calculating the site-specific degree of saturation and matric suction is described in the next section.
5.3 NUMERICAL MODELING OF WATER FLOW IN VARIABLY SATURATED SOIL AND VERIFICATION

The Richards equation (Richards 1931), shown in Eq. 5.7, is employed in this study to model the one-dimensional vertical movement of water within the variably saturated soil. This nonlinear partial differential equation derived from Darcy’s law, predicts the spatial and temporal variation of water content and pressure head for the different flux rates in the subsurface area.

\[
\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z} \left[ K \left( 1 + \frac{\partial \psi}{\partial z} \right) \right]
\]  

where \( t \) is the time, \( z \) is the depth from the ground surface, \( \theta \) is the volumetric water content, \( K \) is the unsaturated hydraulic conductivity, \( \psi \) is the pressure head, and \( \frac{\partial \psi}{\partial z} \) is the hydraulic gradient. Although the problem considered in this study is three-dimensional in nature, this one-dimensional model shown above is reasonably accurate for predicting the water flow in the vertical direction. Since the pressure head is considered as the primary variable to be determined using the Richards equation here, the \( \theta \) and \( K \) must be expressed as functions of \( \psi \). The hydraulic conductivity in variably saturated soil is expressed as the product of saturated hydraulic conductivity (\( K_{sat} \)) and relative hydraulic conductivity (\( K_r \)), i.e., \( K = K_{sat} K_r \). A Soil Water Characteristic Curve (SWCC) is then used to express \( K_r \) in terms of \( \psi \). The other parameter in the Richards equation, \( \theta \), is also expressed in term of \( \psi \) using the same SWCC. Among the many SWCCs and corresponding relative hydraulic conductivity functions, the equations
proposed by van Genuchten (1980) were used in this study. The equations of SWCC and corresponding $K_r$ functions are given in Eqs. 5.8 and 5.9, respectively,

\[
\theta(\psi) = \theta_r + \frac{\left(\theta_s - \theta_r\right)}{1 + (\alpha \psi)^n}^{(1-1/n)}
\]

(5.8)

\[
K_r(\psi) = \frac{\left\{1 - (\alpha \psi)^{n-1} \left[1 + (\alpha \psi)^n\right]^{-(1-1/n)}\right\}^2}{\left[1 + (\alpha \psi)^n\right]^{1-1/n/2}}
\]

(5.9)

where $\alpha$ and $n$ are SWCC fitting parameters, $\theta_s$ is saturated water content, and $\theta_r$ is residual water content. Numerous procedures have been developed to solve the Richards equation for a given set of initial and boundary conditions (van Genuchten 1982, Feddes et al. 1988, Celia et al. 1990, Warrick 1991, Zaidel and Russo 1992, Baker 1995, Pan et al. 1996, Romano et al. 1998, Van Dam et al. 2000). In this study, the Finite Volume Method (FVM) was used to solve the Richards equation for various types of boundary conditions. The spatial and temporal discretization and the summary of the solution procedure are given in the following section.

### 5.3.1 Numerical Solution Procedure

To solve the Richards equation, the equation must first be written in term of the pressure head for one-dimensional vertical flow. Thus, $\frac{\partial \theta}{\partial t}$ is expressed as

\[
\frac{\partial \theta}{\partial \psi} \cdot \frac{\partial \psi}{\partial t} = C \cdot \frac{\partial \psi}{\partial t}
\]

and substituted in Eq. 5.10,
\[
C \frac{\partial \psi}{\partial t} = \frac{\partial}{\partial z} \left( K \frac{\partial \psi}{\partial z} + K \right)
\]  

(5.10)

where \( C \) is the specific moisture capacity \((\partial \theta / \partial \psi)\). Next, Eq. 5.10 is integrated with respect to the time \( t \) and depth \( z \) as follows.

\[
\int_{i-1/2}^{i+1/2} \int_{t-1/2}^{t+1/2} C \frac{\partial \psi}{\partial t} dt dz = \int_{t-1/2}^{t+1/2} \int_{i-1/2}^{i+1/2} \frac{\partial}{\partial z} \left( K \frac{\partial \psi}{\partial z} + K \right) dz dt
\]  

(5.11)


\[
\int_{i-1/2}^{i+1/2} \int_{t-1/2}^{t+1/2} C \frac{\partial \psi}{\partial t} dt dz = \int_{t-1/2}^{t+1/2} C (\psi_{i}^{t+\Delta t} - \psi_{i}^{t}) dz = C_{i}^{t+\Delta t} (\psi_{i}^{t+\Delta t} - \psi_{i}^{t}) \Delta z
\]  

(5.12)

The right-hand side is first solved for spatial variation, and then, the integration is discretized into the temporal form, as shown in Eq. 5.13.

\[
\int_{i-1/2}^{i+1/2} \int_{t-1/2}^{t+1/2} \frac{\partial}{\partial z} \left( K \frac{\partial \psi}{\partial z} + K \right) dz dt = \int_{t-1/2}^{t+1/2} \left[ \left( K \frac{\partial \psi}{\partial z} + K \right)_{i+1/2} - \left( K \frac{\partial \psi}{\partial z} + K \right)_{i-1/2} \right] dt =
\]

\[
\left[ \left( K \frac{\psi_{i+1/2}^{t+\Delta t} - \psi_{i-1/2}^{t+\Delta t}}{\Delta z} + K^{t+\Delta t}_{i+1/2} \right) - \left( K \frac{\psi_{i-1/2}^{t+\Delta t} - \psi_{i-1/2}^{t+\Delta t}}{\Delta z} + K^{t+\Delta t}_{i-1/2} \right) \right] \Delta t
\]  

(5.13)

By reassembling both sides, the complete spatial and temporal form of Richards’ equation is expressed in Eq. 5.14.
Considering $m$ as the iteration level and pressure head at iteration $m+1$ as the unknown value and $\Delta z$ as the depth interval. Finally, dividing the Eq. 5.14 by $\Delta z$ and rearranging the formulation, the final form of the Richards equation is written as follow (Eq. 5.15).

\[
C_i^{i+\Delta t,m} \left( \psi_{i}^{i+\Delta t,m+1} - \psi_{i}^{i} \right) \Delta z = \\
\left[ K_{i+\frac{1}{2}}^{i+\Delta t} \psi_{i+\frac{1}{2}}^{i+\Delta t,m+1} - \psi_{i}^{i+\Delta t,m+1} \frac{1}{\Delta z} + K_{i-\frac{1}{2}}^{i+\Delta t} \psi_{i-\frac{1}{2}}^{i+\Delta t,m+1} - \psi_{i}^{i-\Delta t,m+1} \frac{1}{\Delta z} + K_{i+\frac{1}{2}}^{i} \right] \Delta t
\]  

(Eq. 5.15)

5.3.2 Boundary and Initial Conditions

The discretization of the numerical solution of vertical water flow into the subsurface soil is shown in Fig. 1 to illustrate the problem and the boundary conditions. The upper and lower boundary conditions are located on the ground surface and the water table level, respectively. Since the in-flux or out-flux rate varies with different hydrological loads including the precipitation and evapotranspiration during a hydrological cycle, the upper boundary condition needs to be accurately defined for each hydrological condition and applied to the first node. To this end, depending upon the hydrological condition at each time interval, the corresponding flux rate is applied to the right-hand side of the first node of the numerical solution. The effect of both hydrological loads is shown in Eq. 5.16.
\[ K \left( \frac{\partial \psi}{\partial z} + 1 \right) = \begin{cases} \text{Precipitation} = +Q(t)_{\text{in-flux}} \\ \text{Evapotranspiration} = -Q(t)_{\text{out-flux}} \end{cases} \quad \text{at } z = 0 \text{ and } t \geq 0 \quad (5.16) \]

where \( Q(t) \) is the flux rate at the different time of simulation. The numerical implementation of the upper boundary condition (at the first node) is presented below in Eq. 5.17 considering both hydrological loads.

\[
C_{i}^{t+\Delta t,m} \left( \psi_{i}^{t+\Delta t,m+1} - \psi_{i}^{t} \right) = \left[ \pm Q(t+\Delta t)_{\text{top}} + K_{1+1/2}^{t+\Delta t,m+1} \psi_{2}^{t+\Delta t,m+1} - \psi_{1}^{t+\Delta t,m+1} \right] \Delta t \quad (5.17)
\]

In this study, both pressure head and flux boundary conditions are applied at the upper boundary depending upon the intensity of in- or out-flux rate and the surface moisture capacity. On the other hand, for the lower boundary, the pressure head condition is applied and equal to zero. It should be noted that the initial condition is determined based on the site-specific residual condition captured at the beginning of the hydrological cycle.

Figure 5.1. Physical situation and Representative Elementary Volume (REV) for solving Richards’ equation
5.3.3 Model Verification for Given Boundary Conditions

The proposed framework includes two main algorithms, the Richards equation, and SWCC, which together can sort out the coupled geotechnical-hydrological problem. Hence, the validity of the proposed model should be tested through a comparison of the results of the numerical Richards’ equation and the SWCC with either experimental data or other verified model for different hydrological loads. To accomplish that, the generalized solution developed by Celia et al. (1990), which is laid down on a set of experimental data, was used to verify the implemented numerical solution of water flow under a given in-flux condition. In addition to that, since the proposed model is supposed to handle the out-flux condition as well, the results of the proposed method was also verified by an analytical solution of Richards’ equation developed by Warrick et al. (1990) considering the evaporation as the upper boundary condition. The results of the model verification under both hydrological loads are presented in Fig. 5.2.
5.4 DESIGN FRAMEWORK AND APPLICATION TO STUDY SITES

The application of the proposed procedure requires the computation of temporal and spatial variation of matric suction and degree of saturation within the influence zone of the shallow foundation. To have an accurate degree of saturation and matric suction, the upper and lower boundary conditions of the model need to be determined due to the site-specific hydrological loads and water table data. Since these hydrological variables vary with seasonal periods, the extreme hydrological cycle is selected as the worst case scenario of shallow foundation design. Thus, the corresponding hydrological loads during the extreme cycle are considered as the boundary conditions which can cause a higher variation in matric suction and the degree of saturation in the underlying soil. By
applying such boundary conditions, the critical settlement and bearing capacity of the shallow foundation are captured and used as the foundation design values. This way of analysis will lead to a more realistic design approach which relies on the actual site-specific hydrological loads and the related hydrological-geotechnical parameters.

To perform such analysis, first, the site-specific extreme hydrological cycle based on the Standardized Precipitation Evapotranspiration Index (SPEI) (Vicente-Serrano 2010) is determined, and then the corresponding hydrological loads and water table depth during the extreme cycle are considered as the upper and lower boundary conditions. In the next step, the Richards equation solution is employed to compute the temporal and spatial variation of the soil degree of saturation and matric suction of the subsurface soil subjected to extreme hydrological cycle. Afterward, the ultimate bearing capacity and elastic settlement of the foundation are calculated using the average degree of saturation and matric suction within the foundation influence zone during the extreme hydrological cycle. Finally, the critical values of the elastic settlement and ultimate axial capacity are selected as the design values of the shallow foundation. It should be noted that the inherent randomness of shear strength parameters of the soil can also be incorporated into the analysis, but to allow for comparisons between the saturated and variably saturated conditions, the shear strength variables are kept as constants throughout the analysis except the soil unit weight. The soil unit weight changes with varying degrees of saturation computed within the foundation influence zone. The flowchart of the procedure employed in this study is presented in Fig. 5.3.
Coupled Geotechnical-Hydrological Design of Shallow Foundation

Determine Location, Type, Size and Depth of the Shallow Foundation

Determine Site-Specific the Standardized Precipitation Evapotranspiration Index (SPEI) to Identify the Extreme Hydrological Cycle

Determine Site-Specific Hydrological Parameters during the Extreme Hydrological Cycle (Hydrological Loads (Precipitation and Evapotranspiration) and Water Table Data)

Solve Richards’ Equation through Numerical Approach for Site-Specific Hydrological Loads and Water Table over the Selected Extreme Cycle as Model Boundary Conditions

Determine Moisture Content (θ) and Pore-Water Pressure (u_w) Profiles within Foundation Influence Zone over the Selected Extreme Cycle

Calculate Average Matric Suction within Influence Zone

Calculate Average Degree of Saturation within Influence Zone

Calculate Average Soil Unit Weight within Influence Zone

Calculate Average Modulus of Elasticity within Influence Zone

Developed Bearing Capacity and Elastic Settlement Equations for Shallow Foundations Subjected to the Degree of Saturation and Matric Suction

Solve for Ultimate Bearing Capacity and Elastic Settlement

Select the Critical Value of the Ultimate Bearing Capacity and Elastic Settlement and as the Design Values for Shallow Foundation

End

Figure 5.3. Simulation flowchart for coupled hydrological-geotechnical design of shallow foundation

5.4.1 Study Sites’ Characterization

Two sites were selected to demonstrate the proposed procedure and to show the impact of hydrological loads on the ultimate bearing capacity and elastic settlement
through the change of matric suction and degree of saturation. It is worth mentioning that these sites are selected due to their arid climatic condition so that the hydrological loads make higher changes in the geotechnical properties of the underlying soil.

**Albuquerque, NM**

The first case study site was located in Albuquerque, New Mexico, which was selected for its arid climate condition (with Aridity index (AI) = 0.15). The soil type of this region is mostly Sandy Loam. The SWCC parameters of the soil type were taken from the report by Ellithy (2017), and the soil strength parameters of the site were obtained from a geotechnical report by Terracon Consultants, Inc. (2010).

**Austin, TX**

The second case study site was located in Austin, Texas with a semi-arid climatic condition (AI = 0.35). The soil type of this region is mostly Sandy-Clayey Loam. The soil strength parameters of this site were obtained from a geotechnical report provided by Terracon Consultants, Inc. (2013). The SWCC parameters of the soil type were taken from the report by Ellithy (2017).

The van Genuchten model parameters for these two locations are presented in Fig. 5.4. In addition, the selected SWCC and geotechnical parameters of both locations are listed in Table 5.2.
Table 5.2. SWCC and basic geotechnical parameters for the Albuquerque, NM and Austin, TX sites

<table>
<thead>
<tr>
<th>Soil Water Characteristics Curve (SWCC) Fitting Parameters</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Albuquerque, NM</th>
<th>Austin, TX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturated volumetric water content</td>
<td>$\theta_s$</td>
<td>0.410</td>
<td>0.39</td>
<td></td>
</tr>
<tr>
<td>Residual volumetric water content</td>
<td>$\theta_r$</td>
<td>0.065</td>
<td>0.100</td>
<td></td>
</tr>
<tr>
<td>Model parameter (m$^{-1}$)</td>
<td>$\alpha$</td>
<td>1.308</td>
<td>1.663</td>
<td></td>
</tr>
<tr>
<td>Model parameter</td>
<td>$n$</td>
<td>1.89</td>
<td>1.48</td>
<td></td>
</tr>
<tr>
<td>Air entry value (kPa)</td>
<td>$(\mu_r-\mu_w)b$</td>
<td>4</td>
<td>2</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Basic Geotechnical Parameters</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Albuquerque, NM</th>
<th>Austin, TX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight (kN/m$^3$)</td>
<td>$\gamma_d$</td>
<td>17.75</td>
<td>18.85</td>
<td></td>
</tr>
<tr>
<td>Void ratio</td>
<td>$e$</td>
<td>0.491</td>
<td>0.403</td>
<td></td>
</tr>
<tr>
<td>Effective friction angle (deg)</td>
<td>$\phi'$</td>
<td>31</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td>Effective cohesion intercept (kPa)</td>
<td>$c'$</td>
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<td>Plasticity index</td>
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<td>12</td>
<td></td>
</tr>
<tr>
<td>Hydraulic conductivity (cm/hr)</td>
<td>$K_{sat}$</td>
<td>4.39</td>
<td>1.31</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.4. van Genuchten SWCC for the Albuquerque, NM and Austin, TX sites
5.4.2 Site-Specific Boundary Conditions

5.4.2.1 Extreme hydrological cycle based on SPEI index

Determining the extreme hydrological cycle, which includes extreme hydrological loads, is the main step in finding the appropriate boundary conditions for each site in this study. The extreme hydrological cycle contains the highest intensity of precipitation and evapotranspiration as the two primary hydrological loads. Since the extreme historical values of these two loads do not usually occur in the same hydrological cycle, the best way to determine the extreme cycle is to find a cycle showing the highest difference in intensity between the two hydrological loads. To this end, in this study, the Standardized Precipitation Evapotranspiration Index (SPEI) is used as a climatic drought indicator to detect, monitor, and assess the severity of the drought in any area. The SPEI can measure drought severity according to its intensity and duration, and also is able to identify the onset and end of a drought cycle. The SPEI allows comparison of drought severity through time and space which is calculated based on the difference of site-specific precipitation and evapotranspiration which are directly related to the rainfall and air temperature data, respectively. Since the SPEI shows the severity of difference between the precipitation and evapotranspiration as two main hydrological loads discussed in this study, the extreme hydrological cycle is determined by plotting this index over a wide range of time for each specific study site. The gridded SPEI dataset is available from 1901 to 2015 globally, and it is free to access and download from the repository of Spanish National Research Council (CSIC) (Vicente-Serrano 2010).
The SPEI intensity for both sites, Albuquerque and Austin, were extracted using the sites’ geographical coordinate from the dataset downloaded from the CSIS data repository and plotted from 1901 to 2015 (Fig. 5.5). As shown in the figure, the most variation between the negative and positive part of SPEI is occurred during 1998 to 1999 and 2005 to 2006 for Austin, TX and Albuquerque, NM, respectively. These time periods are selected as the extreme hydrological cycles of both study sites in this study. After finding the extreme hydrological cycle, the corresponding site-specific hydrological loads (precipitation and evapotranspiration) and water table depth are required to be determined in order to apply to the boundary conditions.

Figure 5.5. Variation of Standardized Precipitation Evapotranspiration Index (SPEI) for Albuquerque, NM and Austin, TX
5.4.2.2 Hydrological load – upper boundary condition

As is discussed in previous sections, precipitation and evapotranspiration are the main hydrological loads which affect the degree of saturation and matric suction of underlying soil through changing the soil water content and pressure head. These two loads are applied to the upper boundary condition as an in-flux or out-flux model input. The process of extracting each hydrological load data is described below.

5.4.2.2.1 Precipitation

The site-specific precipitation data for the selected extreme hydrological cycle was obtained from the National Climatic Data Center (NCDC) which records daily precipitation values for both study sites.

5.4.2.2.2 Evapotranspiration

Unlike the precipitation, the evapotranspiration (land surface evaporation plus plant transpiration) depends on many other environmental factors such as temperature, daylight time and saturated vapor density. The evapotranspiration can be simply computed based on the Hamon (1961) method in terms of Potential Evapotranspiration (PET) (Eq. 5.18),

\[
PET = 0.1651 \times L_d \times RHOSAT \times KPEC
\]  \hspace{1cm} (5.18)

where \(L_d\) is daytime length, \(T\) is the air average temperature, \(KPEC\) is a calibration coefficient equal to 1, and \(RHOSAT\) is saturated vapor density at a mean temperature calculated using Eq. 5.19,
\[ RHOSAT = 216.7 \frac{ESAT}{T + 273.2} \]  

(5.19)

where \( ESAT \) is the saturated vapor pressure and is calculated using the Eq. 5.20,

\[ ESAT = 6.108e^{\frac{17.27T}{T + 237.3}} \]  

(5.20)

Using the Hamon method, the daily PET for the site-specific extreme hydrological cycle is calculated based on the air temperature which was obtained from the National Oceanic and Atmospheric Administration (NOAA) of NCDC. After the computation of the evapotranspiration, the complete set of hydrological loads over the site-specific extreme hydrological cycle can be constructed to apply as the upper boundary condition of the model. The site-specific hydrological loads, including precipitation and evapotranspiration, for both locations, are shown in Fig. 5.6. As shown in the figure the intensity of the intensity of hydrological loads in Austin is higher than ones for Albuquerque. The intensity ranges between -10 to 180 mm/day for Austin, while this range changes between -5 to 20 mm/day for Albuquerque.
5.4.2.3 Water table - lower boundary condition

The water table depth, as the lower boundary condition, is another factor which affects the matric suction and degree of saturation within the foundation influence zone. The required data was taken from the U.S. Geological Survey (USGS) for the same extreme hydrological cycle determined for both study sites, separately. It should be noted that the pressure head boundary condition is assigned for the lower boundary condition. The site-specific water table depth of both locations is shown in Fig. 5.7 during the selected one-year hydrological cycle. As shown in the figure, the depth of the water table in Austin is deeper than the one for Albuquerque. The water table depth ranges between 40 to 50 m for Austin, while this depth remains almost unchanged for Albuquerque around 16 m.
5.4.3 Computational Platform

Since the numerical solution of the Richards equation for the proposed model is a time consuming process due to the large time span of extreme hydrological cycle, a MATLAB code was developed, parallelized and installed on the Clemson University’s High Performance Computing (HPC) System called Palmetto Cluster, to perform the simulations in a reasonable runtime. In this study, 8 computing nodes (each has 16 cores, 3.1 GHz CPU processor, and 64GB RAM) were employed to run the simulation. This leads to complete the simulation in less than three days. While, running the same simulation on a Personal Computer (PC) (with the configuration of 4 cores, 2.4 GHz CPU, and 16 GB RAM) takes almost two weeks to complete the analysis.
5.5 RESULTS AND DISCUSSION

5.5.1 Parametric Study-Impact of Hydrological Load Duration

In this section, different time durations (1, 2 and 4 days) for a given set of hydrological loads are selected to illustrate the effect of the site-specific SWCC and soil characteristics on the variation of the degree of saturation and matric suction of soil profile for both study sites (Figs. 5.8 and 5.9). As shown in figures, the degree of saturation and matric suction in Austin, TX show higher variation under both hydrological loads (in-flux and out-flux condition) in comparison to Albuquerque, NM. This significant difference is because of the existence of finer soil particles in the Albuquerque site which has a direct effect on the water infiltration or evapotranspiration process and subsequently the water diffusion in subsurface area. It can be seen from the figures that in case of the precipitation, when the time duration increase from 1 to 4 days, water penetrates utmost 1.0 m into the subsurface area for Albuquerque, while this depth is almost 1.6 m for Austin. On the other hand, in case of the evapotranspiration, when the time duration increases from 1 to 4 days, the saturation level of the soil profile is reduced to a depth of 0.6 m below the ground surface for Austin, while this depth is almost 1.0 m for Albuquerque. Although the variation of soil degree of saturation and matric suction profile is higher in Austin rather than Albuquerque, the rate of these variations is greater in Albuquerque due to the higher soil moisture capacity in that region. The higher moisture capacity in Albuquerque is because of the greater amount of finer soil which provides a larger surface area to hold more water. The results of this section help to better
understand the importance of the inherent characteristics of site conditions and hydrological loads on the performance of the shallow foundation through a change of the soil degree of saturation and matric suction.

Figure 5.8. Predicted degree of saturation profile for the given in- and out-flux intensities for the study sites
5.5.2 Performance of Shallow Foundation under Extreme Hydrological Cycle

As is indicated by the findings in the previous section, the intensity and duration of hydrological loads together with site-specific SWCC and soil characteristics have a significant impact on the variation of soil degree of saturation and matric suction. The variation of soil degree of saturation and matric suction subsequently alter the soil strength and stiffness of subsurface area, specifically the area close to the surface. This change, in turn, affects the ultimate bearing capacity and the elastic settlement of the shallow foundation for both study sites. Thus, it can be concluded that unlike the
conventional design approach, the performance of shallow foundation needs to be carried out in a more realistic manner considering the extreme hydrological cycle in order to find the foundation design values. To this end, various strip foundations with width, $B = 1.0, 2.0,$ and $3.0$ m and depths, $D = 0.5, 0.75,$ and $1.0$ m were used to investigate the effect of site-specific extreme hydrological cycle and its corresponding hydrological loads and water table depth on the elastic settlement and ultimate bearing capacity of foundation for both study sites. A uniform load of $400$ kN/m was applied to all the cases studied.

### 5.5.2.1 Average matric suction and degree of saturation

In order to compute the design elastic settlement and ultimate bearing capacity of each foundation, first, the Richards equation was solved to determine the average degree of saturation and matric suction within the foundation influence zone of each case study. The analysis was performed during the site-specific extreme hydrological cycle considering the corresponding hydrological loads and water table depth as the upper and lower boundary conditions for each study case which is discussed in details before. Figs. 5.10 and 5.11 present the average degree of saturation and matric suction of each site during their one-year extreme hydrological cycle determined in Fig. 5.5. In general, the amount of degree of saturation (and matric suction) in Austin, TX is higher (and lower) than Albuquerque, NM. This result is in a good agreement with the arid condition of these two sites. Since the Albuquerque site is located in a severe arid climate condition, the degree of saturation in Albuquerque shows lower values in comparison with ones in Austin. It is found from the figures that the degree of saturation ranges between $35\%$ and $60\%$ in Austin, while this range changes between $20\%$ and $25\%$ in Albuquerque. The
similar trend also is observed in term of the average matric suction. The average matric suction in Austin changes between 50 kPa and 99 kPa while this change is between 108 kPa and 119 kPa in Albuquerque. This significant discrepancy between both sites is due to the higher intensity of the hydrological loads and the presence of coarser soil particles in the Austin site.

![Figure 5.10. Variation of degree of saturation for both sites during one-year extreme hydrological cycle](image)

![Figure 5.11. Variation of matric suction for both sites during one-year extreme hydrological cycle](image)

### 5.5.2.2 Elastic settlement

Changes in matric suction and degree of saturation, due to the site-specific hydrological loads, affect the soil stiffness and subsequently the elastic settlement of the shallow foundation through varying the soil modulus of elasticity. Fig. 5.12 presents the variation of soil modulus of elasticity within the foundation influence zone at a strain level of 1%. It is found that the soil in Albuquerque is stiffer than the one in Austin which
led to a less elastic settlement. According to the figure, the modulus of elasticity ranges between the 10 MPa to 13 MPa in Austin, while this range changes from 14 MPa to 18 MPa in Albuquerque.

![Figure 5.12. Variation of estimated modulus of elasticity for both sites during one-year extreme hydrological cycle](image)

After finding the average modulus of elasticity, the degree of saturation and matric suction within the foundation influence zone of each case study, the amount of elastic settlement can be calculated (Eq. 5.2) for each case to see how the settlement behaves during the extreme hydrological cycle. Fig. 5.13 illustrates the variation of the elastic settlement for both study sites for different foundation sizes. Based on the results, as is expected, the elastic settlement of the foundation in Austin is greater than the ones in Albuquerque. Each case study shows a similar trend which is more close to the variation of the degree of saturation. For both locations, the width ($B$) of the foundation has a greater impact on the settlement in comparison to the depth ($D$). As shown in the figure, the foundation size with higher width value has a lower settlement in comparison to the other foundation sizes. This finding is reasonable since a wide foundation distributes the applied load in a larger surface area and leads to generate less pressure on the ground surface. In terms of the foundation depth, for foundations with the same width
size, higher depth leads higher settlement which is also reasonable since it directly increases the applied load through the foundation weight. It should be noted that the maximum elastic settlement, as the critical design value, for all study cases, occurs in Oct. 1998 and Sep. 2005 for Austin and Albuquerque, respectively.

![Figure 5.13. Elastic settlement variation for (a) Austin, and (b) Albuquerque during the site-specific extreme hydrological cycle](image)

**5.5.2.3 Ultimate bearing capacity**

The amount of ultimate bearing capacity can be calculated (Eq. 5.4) by using an average degree of saturation and matric suction within the foundation influence zone for each case study to compute the site-specific bearing capacity during the extreme hydrological cycle. Fig. 5.14 shows the variation of ultimate bearing capacity in both study locations for different foundation sizes. Based on the results, in general, the
ultimate bearing capacity of the foundation in Austin, TX is greater than that in Albuquerque, NM for each foundation size. It can be seen from the figure that the ultimate bearing capacity is experienced greater changes for Austin compared to Albuquerque where it almost remains unchanged during the extreme hydrological cycle for all foundation sizes. This difference is due to the higher intensity of the hydrological loads and variation of the degree of saturation and matric suction in Austin rather than Albuquerque. The minimum ultimate bearing capacity, as the critical design value, of all study cases, occurs in the same time captured for the critical elastic settlement. In term of the foundation geometry, it should be noted that the depth ($D$) of the foundation has a greater impact on the design values in comparison to the width ($B$) for both locations.

Figure 5.14. Ultimate bearing capacity variation for (a) Austin, and (b) Albuquerque during the site-specific extreme hydrological cycle
5.5.3 Foundation Design Values Determination and Comparison with Deterministic Approach

To assess the impact of the site-specific extreme hydrological cycle including the hydrological loads and water table depth in the design of shallow foundation, a comparison between the proposed method and the deterministic approach, in which the soil is assumed fully saturated, is required. To this end, first, the design settlement and ultimate bearing capacity should be determined based on the proposed method during the extreme hydrological cycle for each study site, separately. A simple way to determine the design values is to use the critical elastic settlement and ultimate bearing capacity of the shallow foundation. Thus, the minimum ultimate bearing capacity and maximum elastic settlement of each case study, which were computed from proposed method, are selected as critical design values to be compared with the conventional design methods for both site locations (Table 5.3). The elastic settlement and ultimate bearing capacity of each shallow foundation were computed using Bowles’ (1987) and Meyerhof’s (1963) general equations (as the conventional design methods), respectively. As shown in Table 5.3, the ultimate axial capacity of each foundation increase by as much as 27% of the conventional method in Austin, while this is utmost 61% for Albuquerque. In case of the settlement, the total elastic settlement of each foundation decreases by almost 35% and 46% at Austin and Albuquerque, respectively. It can be concluded from this comparison that the proposed method which relies on the real site-specific data including hydrological loads, water table depth and SWCC parameters can improve the shallow foundation design.
Table 5.3. Comparison of pile resistances and settlement computed based on proposed and conventional methods in Austin, TX and Albuquerque, NM

<table>
<thead>
<tr>
<th>Width $(B)$ (m)</th>
<th>Depth $(D)$ (m)</th>
<th>Austin, TX</th>
<th>Ultimate bearing capacity (kPa)</th>
<th>Elastic settlement (mm)</th>
<th>Difference (%),[(Unsat-Sat)/Sat]*100%</th>
<th>Ultimate bearing capacity</th>
<th>Elastic settlement</th>
</tr>
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Albuquerque, NM

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<th>Difference (%),[(Unsat-Sat)/Sat]*100%</th>
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5.6 CONCLUSION

This study presented a new method to incorporate the site-specific extreme hydrological cycle and water table depth in the geotechnical design of shallow foundation. The novel procedure proposed in this study investigates the change in ultimate bearing capacity and elastic settlement of shallow foundation due to the site-specific hydrological loads and water table depth.

To illustrate the importance of the proposed method, this method was applied to two sample sites, Austin, TX and Albuquerque, NM, which are located in an arid climatic condition. After finding the site-specific extreme hydrological cycle and its corresponding hydrological loads and water table depth as a model boundary condition...
for both sites, the Richards equation was solved to determine the temporal and spatial variation of the degree of saturation and matric suction in subsurface area. The results show that the degree of saturation and matric suction in Austin, TX has higher variation during the extreme hydrological cycle in comparison to Albuquerque, NM. This significant difference is because of the higher intensity of hydrological loads (precipitation and evapotranspiration) and also the existence of finer soil particles in the Albuquerque site which has a direct effect on the water infiltration or evapotranspiration process in the underlying soil. The same trend is also observed for the soil modulus of elasticity which indicates that the soil stiffness in Albuquerque is higher than the ones in Austin. In the next phase of this study, a parametric study was performed on the shallow foundation geometry to investigate the performance of different shallow foundation sizes under the site-specific extreme hydrological cycle. The results indicated that higher elastic settlement and lower ultimate bearing capacity, as the critical design values of the shallow foundation, occurred at a time when the maximum variation of the degree of saturation is captured during the extreme hydrological cycle. Finally, these critical design values obtained from the proposed method were compared to the ones calculated from the deterministic approach. The results show a higher ultimate bearing capacity and lower elastic settlement for all foundation sizes in both study sites. It can be concluded from this comparison that the proposed method which relies on the real site-specific data including hydrological loads, water table depth and SWCC parameters can improve the predicted shallow foundation design obtained from the conventional methods.
5.7 REFERENCES


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as a tool.” Proc. 5th BIOT Conference on Poromechanics, Vienna, Austria, 1695-1704.


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CHAPTER 6
SUMMARY AND CONCLUSION

6.1 SUMMARY OF DISSERTATION

In this dissertation, a coupled hydrological-geotechnical framework was developed based on the partially saturated soil mechanics principles to understand the impacts of hydrological events on the performance of shallow and deep foundations. Through the proposed framework, the site-specific hydrological loads along with site-specific geotechnical parameters were considered as the model inputs to determine the foundation design values through a probabilistic analysis and/or single extreme hydrological cycle. In both procedures, the one-dimensional Richards equation was numerically solved due to the applied hydrological loads and water table as upper and lower boundary condition to compute the temporal and spatial variation of the degree of saturation and matric suction in subsurface area. Then, the computed average degree of saturation and matric suction within the foundation influence zone were used to calculate the settlement and bearing capacity. Finally, the critical settlement and bearing capacity of the foundation were considered as the design values of foundations.

6.2 MAJOR FINDINGS

The dissertation’s major findings are listed as below:

- The coupled hydrological-geotechnical framework proposed in this study is unique due to its ability to consider the hydrological parameters and water table depth as the additional model inputs in the design of shallow and deep
foundations. This framework can be considered as a beneficial tool to build sustainable and resilient infrastructure systems under hydrological events.

- It was found that the variation of hydrological loads due to site-specific geographical and geotechnical conditions significantly affect the bearing capacity and settlement of foundations.

- The predicted performance of shallow foundation has been improved in terms of settlement and bearing capacity by applying the proposed framework considering both probabilistic analysis and single extreme hydrological cycle.

- The predicted performance of deep foundations (drilled shaft and driven pile) have been improved regarding the settlement and axial capacity by applying the proposed framework using the probabilistic approach.

- It was also found that the fluctuation in water table depth significantly affects the performance of the foundation through changing the average degree of saturation and matric suction within the foundation influence zone.

- Depending on the type of hydrological events, a design procedure needs to be determined based on the proposed design methodologies for having a better assessment of foundation behavior. In the case of heavy rainfall, the probabilistic design approach is more reasonable since the intensity of historical precipitation is a dominant factor in comparison with the event duration. In the case of drought condition, since the duration of the event has more impact on the foundation performance, the analysis based on a single extreme hydrological cycle seems to be more accurate.
6.3 RECOMMENDATIONS FOR FUTURE WORK

- To generalize the proposed approach, the one-dimensional model must be expanded to two- and three-dimensions.

- The proposed framework can be further extended to other important geotechnical systems such as retaining wall and earth slope in order to investigate the performance of these systems under climate events.

- The proposed framework can be further improved to consider the flood event.

- To have a better assessment of foundation behavior under hydrological events, different structural loading conditions such as lateral and dynamic loads are required to be added into the proposed framework.

- The machine learning algorithms can be added to the proposed framework by considering the uncertainty of hydrological and geotechnical parameters in order to have a better prediction of foundation performance under future hydrological events.