RECOMMENDED CITATION

AUTHORS

Venkata A. Sakleshpur
Graduate Research Assistant
Lyles School of Civil Engineering
Purdue University

Monica Prezzi, PhD
Professor of Civil Engineering
Lyles School of Civil Engineering
Purdue University
(765) 494-5034
mprezzi@ecn.purdue.edu
*Corresponding Author*

Rodrigo Salgado, PhD
Charles Pankow Professor of Civil Engineering
Lyles School of Civil Engineering
Purdue University

Mir Zaheer, PE
Geotechnical Design Engineer
Indiana Department of Transportation

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### Title and Subtitle
CPT-Based Geotechnical Design Manual, Volume 2: CPT-Based Design of Foundations—Methods

### Author(s)
Venkata A. Sakleshpur, Monica Prezzi, Rodrigo Salgado, and Mir Zaheer

### Abstract
This manual provides guidance on how to use the cone penetration test (CPT) for site investigation and foundation design. The manual has been organized into three volumes. Volume 1 covers the execution of CPT-based site investigations and presents a comprehensive literature review of CPT-based soil behavior type (SBT) charts and estimation of soil variables from CPT results. Volume 2 covers the methods and equations needed for CPT data interpretation and foundation design in different soil types, while Volume 3 includes several example problems (based on instrumented case histories) with detailed, step-by-step calculations to demonstrate the application of the design methods. The methods included in the manual are current, reliable, and demonstrably the best available for Indiana geology based on extensive CPT research carried out during the past two decades. The design of shallow and pile foundations in the manual is based on the load and resistance factor design (LRFD) framework. The manual also indicates areas of low reliability and limited knowledge, which can be used as indicators for future research.

### Key Words
cone penetration test, soil behavior type, shallow foundation, pile foundation, load and resistance factor design
EXECUTIVE SUMMARY

Introduction

This manual provides guidance on how to use the cone penetration test (CPT) for site investigation and foundation design. The manual has been organized into three volumes.

Volume I covers the execution of CPT-based site investigations, a comprehensive literature review of CPT-based soil behavior type (SBT) charts, and several correlations for the estimation of a soil variable of interest from CPT results. The volume has been organized into two chapters. Chapter 1 details the components of a CPT system, types of CPT equipment, testing procedures and precautions, maintenance of CPT equipment, and planning and execution of a CPT-based site investigation. Chapter 2 presents a compilation of correlations for the estimation of a soil variable of interest from CPT data, and also presents a comprehensive review of the chronological development of the SBT classification systems that have advanced during the past 55 years of CPT history.

Volume II covers the methods and equations needed for CPT data interpretation and foundation design in different soil types. The volume has been organized into four chapters. Chapter 1 provides an introduction to the manual. Chapter 2 presents an overview of Indiana geology, the typical CPT and soil profiles found in Indiana, and the influence of these profiles on CPT-based site variability assessment. Chapter 3 details the methods for estimation of limit bearing capacity and settlement of shallow foundations from CPT data. Chapter 4 describes the methods for estimation of limit unit shaft resistance and ultimate unit base resistance of displacement, non-displacement, and partial displacement piles and pile groups from CPT data. The design of both shallow and pile foundations is based on the load and resistance factor design (LRFD) framework.

Volume III contains several example problems (based on case histories) with detailed, step-by-step calculations to demonstrate the application of the CPT-based foundation design methods covered in Volume II. The volume has been organized into three chapters. Chapter 1 includes example problems for the estimation of optimal spacing between CPT soundings performed in line and distributed in two dimensions using CPT data obtained from the Sagamore Parkway Bridge construction site in Lafayette, Indiana. Chapter 2 contains example problems for the estimation of limit bearing capacity and settlement of shallow foundations using CPT data reported in literature for sites in the US, UK, and Australia. Chapter 3 includes example problems for the estimation of limit unit shaft resistance and ultimate unit base resistance of displacement, non-displacement, and partial displacement piles using CPT data obtained from three sites in Indiana. The predicted foundation load capacities and settlements were found to be in agreement with the measured load test data reported for these sites.

Findings

Not applicable.

Implementation

The CPT-Based Geotechnical Design Manual can be used to train new employees and to facilitate interaction between INDOT engineers, industry, and consultants. Specific implementation items for each volume are listed below.

Volume I

A spreadsheet for the estimation of fundamental soil variables from CPT results was developed. INDOT engineers can use the spreadsheet on a routine basis to interpret CPT data, generate an SBT profile, and obtain the depth profile of a soil property of interest.

Volumes II and III

Spreadsheets for the estimation of optimal spacing between CPT soundings and CPT-based design of shallow and pile foundations were developed. INDOT engineers can use the spreadsheets on a routine basis for the design of transportation infrastructure projects in Indiana.

A relationship between cone resistance $q_c$, corrected SPT blow count $N_{60a}$, and mean particle size $D_{50}$ was developed using data reported by Robertson et al. (1983) and data obtained from 15 sites in Indiana. The relationship can be used to obtain an estimate of $q_c$ for use in a CPT-based foundation design method when only SPT blow counts are available for a site.

A relationship between critical-state friction angle $\phi_c$, mean particle size $D_{50}$, coefficient of uniformity $C_u$, and particle roundness $R$ was developed using test data reported for 23 clean silica sands in the literature. In the absence of direct shear or triaxial compression test results, the relationship can be used to obtain an estimate of $\phi_c$ for poorly-graded, clean silica sands with $D_{50}$, $C_u$, and $R$ values ranging from 0.15–2.68 mm (0.006–0.105 in.), 1.2–3.1, and 0.3–0.8, respectively.
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1. INTRODUCTION

1.1 Background

Site investigation is an important component of every infrastructure project and plays a vital role in project planning, design, and construction. It is akin to diagnosing patients in medicine because a project site’s pathology (i.e., the origin, type, spatial distribution, and properties of soil and rock layers) is evaluated for engineering purposes (Madhav & Abhishek, 2016, 2017). The main goals of a geotechnical site investigation are to: (1) identify soil and rock stratigraphy, (2) establish groundwater level conditions, and (3) estimate geotechnical design parameters (e.g., strength and stiffness). Although site investigations involve both soil and rock characterization, this manual focuses solely on soil investigations performed using the cone penetration test (CPT).

Over the past two to three decades, in situ tests have gained favor over laboratory tests because: (1) in situ tests are generally faster to perform than laboratory tests, and (2) laboratory test results are affected by sample disturbance and represent the properties of only a few points within a stratum. In contrast, in situ tests, particularly the CPT, significantly increase the volume of material investigated at a site and produce more reliable and repeatable data, thus resulting in substantial cost and time savings.

Among available in situ tests, the standard penetration test (SPT) and the cone penetration test (CPT) are the most commonly used tests in practice. The SPT is a crude test that involves driving a standard split-spoon sampler into the ground a distance of 450 mm (18 in.) from multiple blows using a 630 N (140 lb) hammer dropped from a height of 760 mm (30 in.) (Figure 1.1). The number of blows required for the last 300 mm (12 in.) of penetration of the sampler, after an initial seating drive of 150 mm (6 in.), is recorded as the raw SPT blow count \( N_{SPT} \) for the tested depth.

The SPT blow count is affected by energy inefficiencies in the drop hammer system and other factors, such as the effects of the operator, rod length, sampler type, and borehole diameter (Ireland et al., 1970). Although corrections have been proposed to normalize the \( N_{SPT} \) value with respect to these factors (Anderson et al., 2004; Kulhawy & Mayne, 1990; Skempton, 1986), the reliability of the SPT remains quite low as test results are likely to vary between different crews operating the same equipment (Look, 2016; Look et al., 2015) (Figure 1.2). Consequently, the CPT is gradually replacing the SPT as the preferred in situ test for site investigations. The greater availability of powerful CPT rigs has made it easier for engineers to require that CPTs be performed as part of site investigations. Another reason for the increasing reliance on the CPT is the development of sophisticated and reliable foundation design methods based on CPT data.

The CPT is a quasi-static test and is often used as a complement to conventional rotary drilling and sampling methods. The test is performed by pushing a penetrometer having a conical tip with 60° apex angle vertically into the ground at a standard rate of 20 mm/s (0.8 in./s) (ASTM, 2012) (Figure 1.3). The penetrometer is connected to the lowest rod among a string of rods pushed down from a truck-mounted, crawler-mounted, or trailer-mounted rig. The cone penetrometer was originally used to measure only the tip or cone resistance \( q_c \), defined as the vertical force acting on the tip of the penetrometer divided by the base area of the tip. The base area of the cone tip is equal to 1,000 mm² (1.55 in.²) for typical penetrometers that are in compliance with ASTM (2012), although penetrometer sizes in practice can vary greatly.

Over the years, different sensors have been incorporated into the cone to measure sleeve resistance \( f_s \), shear wave velocity \( V_s \), pore water pressure \( u \), and other parameters (Campanella & Weemees, 1990; Mayne & Campanella, 2005; Mitchell, 1988; Robertson et al., 1986). The CPT data is generally recorded at 1-to-5-cm (0.4-to-2-in.) intervals of cone penetration (ASTM, 2012); however, the data can also be recorded at every 0.2 cm (0.08 in.) of cone penetration depending on the level of sophistication of the penetrometer and the data acquisition system (Salgado et al., 2015). The data is directly logged to a field computer in real-time and can be used to estimate geostatigraphy, soil types, water table elevation, and geotechnical design parameters of interest.
Figure 1.2 Comparison of $N_{SPT}$ values obtained: (a) by different crews using the same SPT equipment (adapted from Mayne & Harris, 1993), (b) using safety and auto hammers (adapted from Finno, 1989), and (c) using safety and donut hammers (adapted from Robertson et al., 1983).

Figure 1.3 Overview of the cone penetration test (after ASTM, 2012).

Figure 1.4 shows a typical CPT log, which always contains the cone resistance $q_c$ and sleeve resistance $f_s$ plotted as a function of depth; it may contain more information if additional measurements are made. Sleeve friction or sleeve resistance $f_s$ is defined as the ratio of the shear force acting along the surface of the cylindrical friction sleeve located above the cone tip to the circumferential area of the sleeve. The circumferential area of the sleeve is equal to 15,000 mm$^2$ (23.25 in.$^2$) in the standard cone (ASTM, 2012). Sleeve resistance was originally thought of as being useful for estimating pile shaft resistance; however, by means of the friction ratio $f_s/q_c$, it has more often been used as an indicator of the type of soil through which the cone is advanced (Lunne et al., 1997). In general, a combination of low $q_c$ values and high friction ratio $f_s/q_c$ suggests a clayey soil, whereas for sandy soils, $q_c$ tends to be high and $f_s/q_c$ low (Salgado, 2008). Volume I reviews the charts available in the literature for estimating soil behavior type (SBT) from CPT results.
The seismic piezocone penetration test (SCPTu), a newer version of the CPT, is a hybrid geotechnical-geophysical \textit{in situ} test that provides downhole geophysical measurements of shear wave velocity $V_s$ at 1-m depth intervals in addition to the regular penetration test data obtained at 1-to-5-cm (0.4-to-2-in.) depth intervals (Campanella et al., 1986; Mayne, 2007; Mayne & Campanella, 2005; Robertson et al., 1986). Figure 1.5 shows the results obtained from a SCPTu sounding performed up to a depth of 95 m (312 ft) at the Golden Ears Bridge site in Vancouver, Canada. Such high-quality subsurface data can be efficiently used to delineate the geostatigraphy of a site and obtain the required geotechnical parameters for use in foundation design.

In its simplest application, the CPT offers a quick, expedient, and economical way to characterize the ground conditions at a site. According to Mayne (2007), a 10-m (30-ft)-deep CPT sounding can be completed in about 15–20 minutes, whereas a conventional soil boring takes about 3–6 times longer to complete. Since soil samples are not collected and spoils are not generated during testing, the CPT is less disruptive from an environmental standpoint and thus advantageous when investigating environmentally sensitive areas and potentially contaminated sites where the risk of exposure to hazardous material is high (Campanella & Weemees, 1990; Fukue et al., 2001; McKnight et al., 2015; Mondelli et al., 2010; Walker et al., 2009). The CPT can be performed in most soil types, ranging from soft-to-stiff clays and loose-to-dense sands, and silts, but can be difficult to perform in terrain containing gravels, cobbles, boulders, or other such obstacles to penetration (Han et al., 2019a,b). Nonetheless, the almost continuous CPT data permit clear delineations of soil strata including the thickness and lateral extent of each layer. In addition, the penetration process is amenable to theoretical modeling, even if the level of sophistication of the required analyses is such that it remains a topic of advanced research. The penetration resistance can be either correlated with other geotechnical parameters or used

![Figure 1.4](image1.png) Typical CPT log (Salgado, 2008).

![Figure 1.5](image2.png) Results obtained from a SCPTu sounding performed at the Golden Ears Bridge site in Vancouver, Canada (adapted from Niazi et al., 2010).
directly in design; however, its use in design and interpretation remains a research need. Soil properties used in geotechnical design are often estimated from a limited number of in situ or laboratory tests (due to project budget and time constraints) and are thus subject to uncertainty, raising the question as to how accurately the soil properties derived from these tests represent those of the entire site (Madhira & Sakleshpur, 2018, 2019). Although this uncertainty cannot be eliminated, it can be addressed by quantifying the variability within individual soundings and of clusters of soundings at a site. Because the CPT is a more reliable tool than the SPT, it can be used for both site variability assessment (Salgado et al., 2015, 2019) and load and resistance factor design (LRFD) of foundations (Basu & Salgado, 2012; Han et al., 2015).

1.2 Aim of the Manual

There is a myriad of CPT correlations and CPT-based design protocols in the literature; these correlations and protocols appear in software, producing interpretation results that may be confounding. This leads to confusion among consultants as to which method(s) to use for estimation of soil variables and design of geotechnical structures based on CPT results. This manual does not aim to be an exhaustive review of all that can be done with the CPT or of all the possible ways in which CPT results can be used in geotechnical engineering. The purpose of this manual, written in concise, objective language, is to provide guidance on how to use the CPT specifically for site investigation and foundation design. The primary focus of the manual is on methods that are current, reliable, and demonstrably the best available for Indiana geology based on extensive CPT research carried out during the past two decades. The manual also indicates areas of low reliability and limited knowledge, which can be used as indicators for future research.

1.3 Organization of the Manual

The manual has been organized into three volumes. Volume I contains two chapters—Chapter 1 details the components of a CPT system, types of CPT equipment, testing procedures and precautions, maintenance of CPT equipment, and planning and execution of a CPT-based site investigation. Chapter 2 presents a comprehensive literature review of (a) estimation of soil variables from CPT results and (b) soil behavior type (SBT) charts.

Volume II contains four chapters—Chapter 1 provides an introduction to the manual. Chapter 2 presents an overview of Indiana geology, the typical CPT and soil profiles found in Indiana, and the influence of these profiles on CPT-based site variability assessment. Chapter 3 details the methods for estimation of limit bearing capacity and settlement of shallow foundations from CPT data. Chapter 4 describes the methods for estimation of limit unit shaft resistance and ultimate unit base resistance of displacement, nondisplacement, and partial displacement piles and pile groups from CPT data.

Volume III contains several example problems (based on instrumented case histories) with detailed, step-by-step calculations to demonstrate the application of some CPT-based foundation design methods covered in Volume II.

2. CONSIDERATION OF INDIANA GEOLOGY ON CPT-BASED SITE INVESTIGATIONS

2.1 Overview of Indiana Geology

2.1.1 Bedrock Geology

Indiana's bedrock geology has three important aspects—the first being the topography of the bedrock surface. The bedrock of Indiana has undergone erosion since about 300 million years ago, but it was only during the Ice Age that unconsolidated sediments were deposited over the bedrock due to glacial advances and retreats across the state. The Ice Age, also known as Pleistocene, is a geologic time period that began about two million years ago and ended 10,000 years ago; during this period, the Earth's higher and mid-latitude zones experienced extensive glaciation by large, continental-scale ice sheets (Wilson, 2008). Thus, the bedrock surface is usually not visible in Indiana because nearly two-thirds of the state is covered by glacial material. According to the Indiana Geological and Water Survey (IGWS), Indiana's bedrock is exposed only in the south-central part of the state, which is unglaciated, and in localized areas along the Wabash River—the highest points of the bedrock surface are in Randolph and Wayne counties, while the lowest points are along the Wabash and Ohio Rivers in Posey and Vanderburgh counties.

The types of rocks and their spatial distribution form the second aspect of Indiana's bedrock geology. Figure 2.1 shows the bedrock geologic map of Indiana, which consists of five bedrock units: Pennsylvanian, Mississippian, Devonian, Silurian, and Ordovician units. Each unit or formation is tens to hundreds of feet thick and consists primarily of sedimentary rocks, such as limestone, dolomite, shale, sandstone, and siltstone. Each of these sedimentary rocks weathers at a different rate and produces unique weathering byproducts. For instance, carbonate rocks, such as limestone and dolomite, dissolve slowly in acid rain and snow to produce sinkholes, caves, and other features collectively known as karst (West, 2010; White, 1988). Such soluble rocks having karst or the potential to develop karst features account for about 18% of the land area of the United States (Weary & Doctor, 2014).

Figure 2.2 shows the karst regions in southern Indiana, which include the Mitchell and Muscatatuck Plateaus, the Crawford and Norman Uplands, and the Charlestown Hills area. The Mitchell Plateau in south-central Indiana is a karst plateau developed on
Mississippian carbonates and extends from the eastern part of Owen County down south to the Ohio River in Harrison County and then into Kentucky (Florea et al., 2018; Gray, 2000; Malott, 1922). The Crawford Upland lies to the west of the Mitchell Plateau and is characterized by ridges and valleys developed on shale, sandstone, and carbonate strata of Mississippian age (Florea et al., 2018). Karst features have also been detected along the western margin of the Norman Upland to the east of the Mitchell Plateau as well as in carbonate strata of Silurian and Devonian age in the Muscatatuck Plateau and the Charlestown Hills area in southeastern Indiana (Gray, 2000) (Figure 2.2). Karst presents difficulties and challenges to geotechnical engineers due to the presence of underground cavities that may collapse, forming sinkholes. Figure 2.3 and Figure 2.4 show photographs of sinkholes in Lawrence County and near the Salem Bypass in Washington County, respectively, in Indiana.

The third aspect of Indiana’s bedrock geology is the presence of bends and faults in the stratigraphic units. Figure 2.5 shows the tectonic features of Indiana. The Kankakee Arch and the Cincinnati Arch constitute a broad anticline, which extends from the northwestern to the southeastern part of the state (Rupp, 1991). This anticline is intersected by two faults: the Royal Center Fault and the Fortville Fault. Apart from these two faults, there is the Mt. Carmel Fault (in the Leesville anticline) that extends from Morgan County south through Monroe and Lawrence counties into Washington County, and finally, a concentrated region of faults in the southwestern part of the state called the Wabash Fault Valley System (Ault & Sullivan, 1982; Hildenbrand & Ravat, 1997; René & Stanonis, 1995; Woolery et al., 2018). In general, Indiana is tectonically quiet with practically insignificant movement of the bedrock (Rupp, 1991).

### 2.1.2 Surficial Geology

Figure 2.6 shows the surficial geologic map of Indiana, which can be broadly divided into four regions.
(from north to south) based on the type of deposit encountered. Firstly, large deposits of dune sand, or sand dunes, exist in northern Indiana, particularly along the Lake Michigan shoreline and along the eastern margins of the Wabash and White Rivers (Argyilan et al., 2018; Cressey, 1928; Hill, 1974; Kilibarda & Blockland, 2011; Kilibarda & Shillinglaw, 2014). Secondly, outwash, which is a sorted and stratified mixture of sand and gravel particles transported and deposited by glacial meltwater, exists in northern Indiana and along major river valleys, such as the Eel, Kankakee, Whitewater, Wabash, White, and Ohio Rivers (Logan et al., 1922). Thirdly, glacial till, which is an unsorted, unstratified and heterogeneous mixture of clay-to-boulder size particles deposited by ice, forms flat to hummocky plains in central Indiana (Colgan et al., 2003; Fleming et al., 1993; Gooding, 1973; Loope et al., 2018; Wayne & Thornbury, 1951). These glacial till plains are partly bisected by end moraines, which are long, arcuate ridges of till, in northeastern Indiana (Brown, 2016; Kassab et al., 2017; Wayne, 1965). Finally, thick loess deposits, which contribute to soil fertility, lie east of the Wabash and White Rivers and south of the Wisconsin glacial boundary, as shown in Figure 2.7 (Hall & Anderson, 2000; Kim & Kang, 2013; Shaw, 1915).

Loess is an unstratified, aeolian sediment that consists mostly of silt with small fractions of clay (smectite) and fine sand (quartz/feldspar) along with light carbonate cementation (calcite ≈ 30%) at inter-particle contacts (Mitchell & Soga, 2005). Loess deposits are typically characterized by low water content (≈ 10%), low density (≈ 1.2 g/cm³ or 74.9 lb/ft³), and loose...
metastable fabric (void ratio = 0.67–1.50)—they are strong and incompressible when dry, as evidenced by several stable vertical cliffs found around the world, but are collapsible either with saturation alone or with saturation and loading (Krinitzsky & Turnbull, 1967; Mitchell & Soga, 2005; Rutledge et al., 1996). In addition to the aforementioned soil types, organic soils, such as peat (with organic content > 30%), are commonly found in the Northern Lake Moraine Physiographic Region in northern Indiana and occasionally in central Indiana as well (Wilcox et al., 1986; Wilcox & Simonin, 1988).

2.2 CPT, SPT, and Soil Profiles in Indiana

One of the primary applications of the cone penetration test is stratigraphic profiling. Figure 2.8 shows the distribution of different soil types in Indiana and 10 select locations where CPTs were performed by the Indiana Department of Transportation (INDOT), the United States Geological Survey (USGS), and Purdue University. Table 2.1 summarizes the geographic details of the CPT locations marked in Figure 2.8. The locations were selected from different parts of the
Figure 2.7  Map of southern Indiana showing the distribution of loess deposits (> 1.5 m (5 ft) in thickness) (Source: Gray, n.d.).

Figure 2.8  Pedological map of Indiana showing the CPT locations.
TABLE 2.1
Geographic information of the CPT locations in Indiana

<table>
<thead>
<tr>
<th>Notation</th>
<th>Soil Type</th>
<th>County</th>
<th>Approximate Location Details</th>
<th>Latitude</th>
<th>Longitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Dune sand</td>
<td>Lake</td>
<td>On SR-51/US-6, 900 ft south of I-90</td>
<td>41.5903</td>
<td>-87.2403</td>
</tr>
<tr>
<td>B</td>
<td>Till in hummocky moraine form</td>
<td>Steuben</td>
<td>On SR-4 over Little Turtle Creek</td>
<td>41.5267</td>
<td>-85.1036</td>
</tr>
<tr>
<td>C</td>
<td>Outwash</td>
<td>LaPorte</td>
<td>On US-30 over Turf Farm Ditch</td>
<td>41.4058</td>
<td>-86.7389</td>
</tr>
<tr>
<td>D</td>
<td>Aeolian sand</td>
<td>Newton</td>
<td>On SR-55 over Gregory Ditch</td>
<td>41.0906</td>
<td>-87.3336</td>
</tr>
<tr>
<td>E</td>
<td>Outwash</td>
<td>Tippecanoe</td>
<td>US-52 bridge over Wabash River, Lafayette</td>
<td>40.4511</td>
<td>-86.8929</td>
</tr>
<tr>
<td>F</td>
<td>Glacial till</td>
<td>Clinton</td>
<td>310 ft southwest of INDOT office</td>
<td>40.2777</td>
<td>-86.5342</td>
</tr>
<tr>
<td>G</td>
<td>Glacial till</td>
<td>Madison</td>
<td>On SR-32 over Indian Camp Creek</td>
<td>40.0842</td>
<td>-85.8283</td>
</tr>
<tr>
<td>I</td>
<td>Loess with sand</td>
<td>Knox</td>
<td>On SR-550 over Smalls Creek, 1.57 miles west of SR-67</td>
<td>38.7892</td>
<td>-87.4383</td>
</tr>
<tr>
<td>J</td>
<td>Lacustrine soil</td>
<td>Vanderburgh</td>
<td>On W Delaware St, 2.16 miles west of US-41</td>
<td>37.9840</td>
<td>-87.5816</td>
</tr>
</tbody>
</table>

state to demonstrate the effect of Indiana geology on cone penetration test results.

The raw CPT data collected from each location was post-processed to obtain profiles of cone resistance \( q_c \), sleeve resistance \( f_s \), and friction ratio \( FR = f_s/q_c \). The USGS and INDOT CPT rigs record data at 5 cm depth intervals, while the Purdue CPT rig records data at 2 mm depth intervals (Salgado et al., 2015). The corrected, total cone resistance \( q_t \) was calculated by taking into account the unbalanced pore water pressure acting on opposing sides of both the face and joint annulus of the cone tip (Jamiolkowski et al., 1985; Lunne et al., 1997; Robertson et al., 1986; Salgado, 2008):

\[
q_t = q_c + (1 - a)u_2 \quad \text{(Eq. 2.1)}
\]

where \( q_c \) = measured cone resistance, \( u_2 \) = pore water pressure measured at the shoulder position behind the cone face, and \( a \) = cone area ratio (= 0.8 for the Hogentogler CPT probe (Hogentogler & Co. Inc., 2004)). According to ASTM D5778 (ASTM, 2012), the correction of \( q_c \) to \( q_t \) is particularly important for CPTs in saturated clays, silts, and soils having considerable amount of fines where substantial pore pressures are generated during penetration; however, for CPTs in clean sands, dense to hard geomaterials, and dry soils, the correction may be ignored without significant error. It is assumed hereafter that this correction has been applied whenever it produces nonnegligible changes to \( q_c \), and thus \( q_t \) will not be distinguished from \( q_c \), unless otherwise stated.

A soil profile generation algorithm developed by Ganju et al. (2017) was used to generate stratigraphic profiles from the CPT data obtained at each location. The algorithm requires seven input parameters: depth, corrected cone resistance, sleeve resistance, ground surface elevation, latitude, longitude, and groundwater table depth. The algorithm was implemented for the soil behavior type (SBT) chart proposed originally by Tumay (1985) and modified subsequently by Ganju et al. (2017). The original Tumay (1985) chart was modified in order to (a) minimize ambiguities associated with soil behavior types, and (b) make a clearer distinction between soil intrinsic variables (related closely to soil composition) and soil state variables, such as relative density, stress state, and fabric. Figure 2.9 shows the modified version of the Tumay (1985) SBT chart. In general, a combination of low \( q_c/p_A \) (< 10) and high \( f_s/q_c \) values (> 4%) suggests a clayey soil, whereas a combination of high \( q_c/p_A \) (> 50) and low \( f_s/q_c \) values (< 2%) suggests a sandy soil; where \( p_A = \text{reference stress} = 100 \text{ kPa or 14.5 psi} \).

Table 2.2 summarizes the soil behavior types associated with the modified Tumay (1985) chart. Each soil behavior type that appears in Figure 2.9 is assigned a zone number. For instance, zones 1 to 7 correspond to clays of different rigidities, zone 8 corresponds to

![Figure 2.9 Modified Tumay (1985) SBT chart (Ganju et al., 2017; Salgado et al., 2019).](image-url)
TABLE 2.2  Soil behavior types associated with the modified Tumay (1985) chart

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil Behavior Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive clay</td>
</tr>
<tr>
<td>2</td>
<td>Very soft clay</td>
</tr>
<tr>
<td>3</td>
<td>Soft clay</td>
</tr>
<tr>
<td>4</td>
<td>Medium stiff clay</td>
</tr>
<tr>
<td>5</td>
<td>Stiff clay</td>
</tr>
<tr>
<td>6</td>
<td>Very stiff clay</td>
</tr>
<tr>
<td>7</td>
<td>Sandy clay or silty clay</td>
</tr>
<tr>
<td>8</td>
<td>Clayey silty sand</td>
</tr>
<tr>
<td>9</td>
<td>Clayey sand or silt</td>
</tr>
<tr>
<td>10</td>
<td>Clayey silt</td>
</tr>
<tr>
<td>11</td>
<td>Very dense sand or silty sand</td>
</tr>
<tr>
<td>12</td>
<td>Dense sand or silty sand</td>
</tr>
<tr>
<td>13</td>
<td>Medium dense sand or silty sand</td>
</tr>
<tr>
<td>14</td>
<td>Loose sand or silty sand</td>
</tr>
<tr>
<td>15</td>
<td>Very loose sand or silty sand</td>
</tr>
</tbody>
</table>

Figure 2.10  Modified Robertson (1990) SBT chart (Ganju et al., 2017; Salgado et al., 2019).

TABLE 2.3  Soil behavior types associated with the modified Robertson (1990) chart

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil Behavior Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive fine-grained</td>
</tr>
<tr>
<td>2</td>
<td>Organic clay</td>
</tr>
<tr>
<td>3</td>
<td>Clay to silty clay</td>
</tr>
<tr>
<td>4</td>
<td>Clay silt to silty clay</td>
</tr>
<tr>
<td>5</td>
<td>Sand mixtures: silty sand to sandy silt</td>
</tr>
<tr>
<td>6</td>
<td>Very dense gravelly sand to sand</td>
</tr>
<tr>
<td>7</td>
<td>Dense gravelly sand to sand</td>
</tr>
<tr>
<td>8</td>
<td>Medium dense gravelly sand to sand</td>
</tr>
<tr>
<td>9</td>
<td>Loose gravelly sand to sand</td>
</tr>
<tr>
<td>10</td>
<td>Very loose gravelly sand to sand</td>
</tr>
<tr>
<td>11</td>
<td>Very dense clean sand to silty sand</td>
</tr>
<tr>
<td>12</td>
<td>Dense clean sand to silty sand</td>
</tr>
<tr>
<td>13</td>
<td>Medium dense clean sand to silty sand</td>
</tr>
<tr>
<td>14</td>
<td>Loose clean sand to silty sand</td>
</tr>
<tr>
<td>15</td>
<td>Very loose clean sand to silty sand</td>
</tr>
</tbody>
</table>

Apart from the modified Tumay (1985) chart, a modified version of the Robertson (1990) SBT chart, which distinguishes clean sand from gravelly sand, was also used to generate the SBT profile, particularly for location E in Tippecanoe County. Figure 2.10 shows the modified Robertson (1990) SBT chart according to Ganju et al. (2017). The chart uses values of normalized cone resistance $q_{tn} = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$ and normalized friction ratio $FR_n = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$; where $\sigma_{vo}$ and $\sigma'_{vo} = in situ$ vertical total and effective stresses, respectively, at the depth being considered. As the values of $\sigma_{vo}$ and $\sigma'_{vo}$ depend on the unit weights of the soil layers at the site and the elevation of the groundwater table, the modified Robertson (1990) SBT chart can only be used after the CPT data has been post-processed.

Table 2.3 summarizes the soil behavior types associated with the modified Robertson (1990) chart. Similar to the modified Tumay (1985) chart, each soil behavior type that appears in Figure 2.10 is assigned a zone number. Ganju et al., (2017) further divided the “gravelly sand to sand” region and the “clean sand to silty sand” region of the modified Robertson (1990) chart into five zones each (zones 6 to 10 and 11 to 15 in Table 2.3) based on the relative density, which can be estimated from CPT data using the correlation of Salgado and Prezzi (2007).

A total of 23 CPT soundings were analyzed from locations A–J using the soil profile generation algorithm developed by Ganju et al. (2017). In this algorithm, firstly, an initial soil profile is generated by plotting the $q_c$ and FR values, obtained at each depth during cone penetration, on the selected SBT chart. Secondly, any layer in the initial soil profile with thickness less than or equal to 15 cm (5.9 in.) (or 4.2 cone diameters) is tagged as a thin layer—a layer in which the CPT probe is unable to develop a cone resistance that is representative of that layer. Finally, the initial soil profile is reanalyzed with the objective of merging the thin layers into the adjacent thick layers to obtain the final soil profile. This is done using three sequential approaches: (1) the SBT band approach, (2) the soil group approach, and (3) the average $q_c$ approach, all of which are described in detail by Salgado et al. (2015) and Ganju et al. (2017). The significance of this methodology is that the final...
generated soil profile will not contain layers thinner than 15 cm (5.9 in.). This mitigates the creation of a significantly fragmented soil profile littered with clusters of layers that are too small to be sensed properly by the standard CPT probe.

Apart from the CPT, additional independent sampling may be performed to corroborate the soil profile at a site. However, soil behavior types obtained from SBT charts may not always fully agree with traditional soil classifications based on grain-size distribution and soil plasticity, such as the Unified Soil Classification System (USCS) (ASTM, 2017) or the American Association of State Highway and Transportation Officials (AASHTO, 1991), because of the role of soil fabric and structure (Robertson, 2016). Nonetheless, a qualitative comparison between the SBT profiles generated using the selected SBT chart and the soil profiles obtained from in situ boring logs can be instructive.

To complement the CPT profiles obtained at locations A–J, the corrected SPT blow count \( \frac{q_c}{p_A N_{60}} \) and the ratio \( p_A N_{60} \) are plotted as a function of depth; where \( p_A \) = reference stress (= 100 kPa or 14.5 psi). The SPTs were performed using an automatic trip hammer with an energy ratio of about 80% (Salgado, 2008). As both cone resistance and SPT blow count are essentially penetration resistances, they are closely related. Hence, plots of \( q_c/p_A N_{60} \) versus depth may be useful in case a CPT-based design method needs to be used when only SPT blow counts are available for the site. It should be noted that not all the locations marked in Figure 2.8 have SPT borings completed along with CPT soundings. Also, it is important to note that the SPT borings were not carried out at the exact locations of the CPT soundings but were performed within the same project site. Therefore, the following \( q_c/p_A N_{60} \) plots for each site should be interpreted with caution.

### 2.2.1 Dune/Aeolian Sands

Figure 2.11 and Figure 2.12 show the CPT profiles \( (q_c, f_s, FR) \), the SBT profile generated using the modified Tumay (1985) chart, the SPT \( N_{60} \) and \( q_c/p_A N_{60} \) profiles, and the in situ layer information (with AASHTO group numbers) reported in the boring logs for location A in Lake County and location D in Newton County, respectively. Location A is in the dune sand region of northern Indiana, while location D is slightly further to the south of location A. The stratigraphic profile obtained from the SPT boring log at location A consists of 8 m (26 ft) of medium dense sandy loam followed by 7 m (23 ft) of very loose-to-medium dense sand and 3 m (10 ft) of dense sandy loam. On the other hand, the stratigraphic profile from the SPT boring log at location D consists of 1.5 m (5 ft) of very loose-to-loose sand followed by 12.5 m (41 ft) of medium dense sand.

The numbers mentioned on the SBT profiles, generated using the modified Tumay (1985) chart, correspond to the soil zones listed in Table 2.2. The SBT profiles generated using the modified Tumay (1985) chart for both locations A and D agree qualitatively with the soil profiles obtained from the corresponding SPT boring logs. The soil profiles obtained from the boring logs are based on laboratory testing of soil samples collected at depth intervals of 1.5 m (5 ft), whereas the SBT profiles generated using the modified Tumay (1985) chart are based on nearly continuous CPT measurements at 5 cm (2 in.) depth intervals. Thus, the SBT profiles contain more soil layers than the soil profiles obtained from the boring logs because some of these layers may lie between consecutive SPT sampling intervals. The \( q_c/p_A N_{60} \) values for locations A and D range from about 3 to 8, which is typical for sandy soils based on their mean particle size \( D_{50} \) (Robertson et al., 1983).

#### 2.2.2 Outwash

Figure 2.13 shows the CPT profiles \( (q_c, f_s, FR) \) and the SBT profile generated using the modified Tumay (1985) chart for location C in LaPorte County, while Figure 2.14 shows the CPT profiles \( (q_c, f_s, FR) \), the SBT profile generated using the modified Tumay (1985) chart, the SPT \( N_{60} \) and \( q_c/p_A N_{60} \) profiles, and the in situ layer information (with AASHTO group numbers) reported in the boring log for location E in Tippecanoe County. Location C lies in the outwash region of northern Indiana, while location E is on the bank of the Wabash River near Purdue University. Outwash is a mixture of sand, gravel, cobbles, and boulders that are transported and deposited by glacial meltwater; it may also include some modern river alluvium. The SBT profile at location C consists of multiple layers of loose-to-very dense sand or silty sand (Figure 2.13).

The soil profile reported in the SPT boring log for location E in Tippecanoe County consists of sandy clay loam and loose-to-medium dense sandy gravel in the upper half of the profile and medium dense-to-very dense sand with gravel, cobbles, and boulders in the lower half of the profile (Figure 2.14b). The \( N_{60} \) values range from about 2 to 43, and the \( q_c/p_A N_{60} \) values range from about 3 to as high as 16 due to the presence of gravel, cobbles, and boulders in the soil profile. The modified Tumay (1985) chart includes soil behavior types ranging from clays to clay-silt-sand mixtures to sands of varying states; however, the chart does not clearly distinguish sands from sand-gravel mixtures and gravelly sands. Therefore, a modified version of the Robertson (1990) SBT chart, which distinguishes clean sand from gravelly sand, was also used to generate the SBT profile for location E.

Figure 2.15 compares the SBT profile generated using the modified Robertson (1990) chart with that obtained using the modified Tumay (1985) chart for location E in Tippecanoe County. In order to classify the coarse-grained soil layers at the site based on their relative density (using the Salgado and Prezzi (2007) correlation), the saturated unit weight \( \gamma_{sat} \), the critical-state friction angle \( \phi_c \), and the coefficient of lateral earth pressure at-rest \( K_0 \) of the coarse-grained layers.
were taken as 22.5 kN/m$^3$ (143.2 lb/ft$^3$), 32$, and 0.45, respectively. The SBT profile obtained using the modified Robertson (1990) chart shows layers of very dense and medium dense gravelly sand to sand, indicated by zone numbers 6 and 8, respectively (Table 2.3), between elevations ranging from 149–153 m and 137–143 m and a layer of medium dense gravelly sand to sand at the 128–131 m elevation. In contrast, the SBT profile obtained using the modified Tumay (1985) chart shows layers of very dense sand or silty sand (indicated by zone number 11) at these elevations and does not capture the presence of gravelly material in the profile. The mean particle size $D_{50}$ and gravel content at the site are in the range of 0.4–4.5 mm (0.016–0.18 in.) and 5%–50%, respectively (Han et al., 2019b, 2020). Hence, for sites with high gravel content, the modified Robertson (1990) chart is a better option for generating SBT profiles from CPT data than the modified Tumay (1985) chart. The delineation of gravelly material in the profile using a CPT-based SBT chart has implications in foundation design because the constitutive response of a sand-gravel mixture is different from that of clean sand, for instance, when subjected to shearing.

Figure 2.11 *In situ* test profiles for location A in Lake County: (a) CPT-1 profile ($q_c$, $f_s$, FR) and SBT interpreted from modified Tumay (1985) chart, and (b) $N_{60}$ profile, $(q_s/p_a)/N_{60}$ profile, and soil profile from SPT boring TB-2 (Data source: A. Tilahun, J. Paauwe, & N. Z. Siddiki, personal communication, December 20, 2017).
2.2.3 Glacial Till

Figures 2.16, 2.17, 2.18, and 2.19 show the CPT profiles \((q_c, f_s, FR)\), the SBT profiles generated using the modified Tumay (1985) chart, the SPT \(N_{60}\) and \(q_c/p_A N_{60}\) profiles, and the \textit{in situ} layer information (with USCS/AASHTO group numbers) reported in the boring logs for locations B, F, G, and H in Steuben, Clinton, Madison, and Decatur counties, respectively. These locations are characterized by glacial till deposits, as shown in Figure 2.8. Location B is in northeastern Indiana where the till is in a hummocky moraine form, locations F and G are in central Indiana where the till is mostly in the form of flat plains, and location H is in southeastern Indiana where the till is capped by thin wind-blown silt. The stratigraphic profiles at these locations consist of layers of sandy silty clay, silty sand, and loam with different percentages of sand, silt, and clay. The \(q_c/p_A N_{60}\) values for locations F, G, and H range from 0.5–2.0, 0.5–1.0, and 1.0–3.5, respectively. These ranges are smaller than those reported for the dune/aeolian sand and outwash regions in Sections 2.2.1 and 2.2.2, respectively, due to the presence of smaller particle sizes associated with the soil types illustrated in Figure 2.16 to Figure 2.19.

2.2.4 Loess with Sand

Figure 2.20 shows the CPT profiles \((q_c, f_s, FR)\), the SBT profile generated using the modified Tumay (1985) chart, the SPT \(N_{60}\) and \(q_c/p_A N_{60}\) profiles, and the \textit{in situ} layer information (with AASHTO group numbers) obtained from the SPT boring log for location I in Knox County. This location is in southwestern Indiana,
which is characterized by wind-blown silt deposits. The stratigraphic profile obtained from the SPT boring log consists of 1 m (3 ft) of very loose sand followed by 3 m (10 ft) of very loose-to-loose loam, 2 m (6.5 ft) of very loose-to-medium dense sandy loam, and finally unweathered-to-highly-weathered sandstone at a depth of 16.3–21.0 m (53–69 ft) below the ground surface. These layers are also captured by the CPT-based SBT profile via zone numbers 6–10 (Table 2.2). The \( N_{60} \) values at the site range from about 5 to as high as 80, while the \( q_c/p_A N_{60} \) values range from 1.0 to 4.5.

### 2.2.5 Lacustrine Soil

Figure 2.21 shows the CPT profiles \( (q_c, f_c, FR) \) and the SBT profile generated using the modified Tumay (1985) chart for location J in Vanderburgh County. This location is in southern Indiana, near the border with Kentucky, and is characterized by lacustrine soil. Lacustrine soils form under relatively quiet conditions at the bottom of lakes and typically consist of silt to clay-sized particles. The SBT profile generated using the modified Tumay (1985) chart consists of 4 m (13 ft) of soft-to-very stiff clay and clayey silt underlain by 8 m (26 ft) of medium dense silty sand and 7 m (23 ft) of sandy clay or silty clay.

### 2.3 Correlation Between CPT Cone Resistance and SPT Blow Count

Figure 2.22 shows the correlation between the CPT cone resistance \( q_c \) and the corrected SPT blow count \( N_{60} \) as a function of mean particle size \( D_{50} \). The chart includes data reported by Robertson et al. (1983) and data obtained from 15 sites in Indiana (2 sites each in Hamilton, Tippecanoe, Clinton, and Greene counties, and 1 site each in Jasper, Lake, Newton, Knox, Starke, Dubois, and Carroll counties). Starke, Newton, Jasper, and Lake counties are located in northern Indiana; Hamilton, Tippecanoe, Carroll, and Clinton counties are in central Indiana; and Greene, Knox, and Dubois counties are in southern Indiana. The following expression approximates the trend of the 98 data points plotted in Figure 2.22:

\[
\frac{q_c}{p_A N_{60}} = 6.95 \left( \frac{D_{50}}{D_{ref}} \right)^{0.25} - 0.18 \quad \text{for} \quad 0.001 \leq \frac{D_{50}}{D_{ref}} \leq 10 \quad (\text{Eq. 2.2})
\]

where \( p_A \) = reference stress (= 100 kPa or 14.5 psi), \( D_{50} \) = mean particle size, and \( D_{ref} \) = reference particle size (= 1 mm or 0.0394 in.). The coefficient of determination \( R^2 \) and the standard error (SE) of the regression are 0.89 and 0.77, respectively. Equation 2.2 may be used to obtain an estimate of \( q_c \) for use in a CPT-based foundation design method when only SPT blow counts are available for a site. However, as with any correlation involving the SPT blow count, Eq. 2.2 should be used with caution because of the potential error introduced by the transformation from the SPT blow count (a dynamic resistance) to the CPT cone resistance (a quasi-static resistance). The \( q_c/p_A N_{60} \) ratio estimated using Eq. 2.2 may be decreased by 20%–40%, if needed, to obtain a conservative value of cone resistance. Equation 2.2 can be further improved as additional SPT blow count, cone resistance and \( D_{50} \) data become available in Indiana.
The corrected SPT blow count \( N_{60} \) is expressed as (Salgado, 2008):

\[
N_{60} = C_h C_r C_s C_d N_{SPT}
\]  

(Eq. 2.3)

where \( N_{SPT} \) = measured SPT blow count, \( C_h \) = hammer correction, \( C_r \) = rod length correction, \( C_s \) = sampler correction, and \( C_d \) = borehole diameter correction:

\[
C_h = \begin{cases} 
0.75 & \text{for donut hammer (ER = 45\%)} \\
1.00 & \text{for safety hammer (ER = 60\%)} \\
1.20 & \text{for pin weight hammer (ER = 72\%)} \\
1.33 & \text{for automatic trip hammer (ER = 80\%)} 
\end{cases}
\]  

(Eq. 2.4)

\[
C_r = \begin{cases} 
0.75 & \text{if rod length <4 m (13 ft)} \\
0.85 & \text{if 4 m (13 ft) \leq \text{rod length} <6 m (20 ft)} \\
0.95 & \text{if 6 m (20 ft) \leq \text{rod length} <10 m (33 ft)} \\
1.00 & \text{if rod length \geq 10 m (33 ft)} 
\end{cases}
\]  

(Eq. 2.5)

\[
C_s = \begin{cases} 
1.0 & \text{for liner sampler with liner in place} \\
1.2 & \text{for liner sampler without the liner} 
\end{cases}
\]  

(Eq. 2.6)

\[
C_d = \begin{cases} 
1.00 & \text{for } B = 65 - 115 \text{ mm (2.5 - 4.5 in.)} \\
1.05 & \text{for } B = 150 \text{ mm (6.0 in.)} \\
1.15 & \text{for } B = 200 \text{ mm (8.0 in.)} 
\end{cases}
\]  

(Eq. 2.7)

where ER = energy ratio, and \( B \) = borehole diameter.

Figure 2.14  *In situ* test profiles for location E in Tippecanoe County: (a) CPT-3 profile \((q_c, f_s, FR)\) and SBT interpreted from modified Tumay (1985) chart, and (b) \( N_{60} \) profile, \((q_{c60}/p_{a60})/N_{60}\) profile and soil profile from SPT boring Pier-7 (Data source: A. Tilahun, J. Paauwe, & N. Z. Siddiki, personal communication, December 20, 2017).
Figure 2.15  Comparison of SBT profiles obtained from sounding CPT-5 at location E in Tippecanoe County using: (a) modified Tumay (1985) chart (zone numbers listed in Table 2.2), and (b) modified Robertson (1990) chart (zone numbers listed in Table 2.3).

Figure 2.16  CPT-4 profile ($q_c$, $f_s$, FR) and SBT interpreted from modified Tumay (1985) chart for location B in Steuben County (Data source: A. Tilahun, J. Paauwe, & N. Z. Siddiki, personal communication, December 20, 2017).
2.4 CPT-Based Site Variability Assessment

Soil properties used in geotechnical design are often estimated from a limited number of in situ or laboratory tests (due to project budget and time constraints) and are thus subject to uncertainty, raising the question as to how accurately the soil properties derived from these tests are representative of the entire site (Phoon & Kulhawy, 1999a,b). Although this uncertainty cannot be eliminated, it can be quantified by analyzing the variability within individual CPT soundings and of the collection of soundings performed at a site (Cao & Wang, 2013; Salgado et al., 2015; Xiao et al., 2018). If reasonably quantified, this uncertainty may be used to select appropriate resistance factors for use in load and resistance factor design (LRFD) of foundations and retaining structures (Foye, 2005; Foye et al., 2006a,b, 2009; Kim & Salgado, 2012a,b; Salgado et al., 2011; Salgado & Kim, 2014). For sites with high variability, lower resistance factors could be used to increase the
reliability of the foundation design, whereas for sites with low variability, higher resistance factors could be used to optimize the construction cost. Based on the coefficient of variation (COV) of the average strength parameter (e.g., SPT blow count $N_{SPT}$) of each soil layer at a site, Paikowsky (2004) suggested that site variability can be classified as low (COV < 25%), medium (25% ≤ COV ≤ 40%), or high (COV > 40%). However, the volume of data available for statistical analysis using the SPT is smaller in comparison to the CPT, and thus it is better to use a CPT dataset for site variability assessment.

Salgado et al. (2019) developed the following four-step procedure for CPT-based site variability assessment.

1. Generate the SBT profile from the CPT data using an SBT chart.
2. Quantify vertical variability via the vertical variability index (VVI), which reflects the variability in $q_c$, $f_s$, and soil layering for each CPT sounding.
3. Quantify horizontal variability via the horizontal variability index (HVI), which depends on the cross-correlation between cone resistance logs, cone resistance trend differences, and the spacing between CPT soundings.

Figure 2.18  *In situ* test profiles for location G in Madison County: (a) CPT RB-2 profile ($q_c, f_s, FR$) and SBT interpreted from modified Tumay (1985) chart, and (b) $N_{60}$ profile, ($q_c/p_a)N_{60}$ profile and soil profile from SPT boring TB-2 (Data source: A. Tilahun, J. Paauwe, & N. Z. Siddiki, personal communication, December 20, 2017).
Figure 2.19  *In situ* test profiles for location H in Decatur County: (a) CPT-1 profile ($q_c$, $f_s$, FR) and SBT interpreted from modified Tumay (1985) chart, and (b) $N_{60}$ profile, $(q_s/p_A)/N_{60}$ profile and soil profile from SPT boring TB-1 (Data source: A. Tilahun, J. Paauwe, & N. Z. Siddiki, personal communication, December 20, 2017).

4. Combine both vertical and horizontal variability into an overall site variability rating (SVR) system.

Figure 2.23 shows how to categorize a site as being of low (L), medium (M), or high (H) variability in the vertical and horizontal directions based on whether the site VVI and HVI values fall in the 0%–33%, 33%–66%, or 66%–100% range, respectively. Salgado et al. (2015, 2019) established a site variability rating, defined in terms of a string variable with two characters, each of
which may take the values, L, M, or H, as shown in Figure 2.23. The first letter corresponds to the site VVI, while the second letter corresponds to the site HVI. For instance, if the site VVI and HVI values are 47% and 31%, respectively, the site variability rating is ML, which stands for medium vertical variability and low horizontal variability.

Table 2.4 summarizes the computed vertical and horizontal variability indices for sites in Indiana using the CPT-based site variability assessment algorithm developed by Salgado et al. (2019). The sampling interval for each CPT sounding was at most 5 cm (2 in.), and the sounding depths were in the range of 3–20 m (10–65 ft). Sites A, C, and D have low site VVI values because their SBT profiles consist predominantly of medium dense-to-very dense sands of similar behavior. In contrast, the other sites (B and E–J) have medium-to-high site VVI values because their SBT

Figure 2.20  In situ test profiles for location I in Knox County: (a) CPT-1 profile ($q_c$, $f_s$, FR) and SBT interpreted from modified Tumay (1985) chart, and (b) $N_{60}$ profile, ($q_c/p_a$)/$N_{60}$ profile, and soil profile from SPT boring TB-2 (Data source: A. Tilahun, J. Paauwe, & N. Z. Siddiki, personal communication, December 20, 2017).
profiles consist of sandy, silty, and clayey soils with relatively equal representation; layers of gravelly sand were also observed for site E in Tippecanoe County. Sites B and G in the glacial till areas of Steuben and Madison counties, respectively, have HVI values of 100% due to the presence of soil layers with highly variable $q_c$ values within the depth of interest between soundings. The CPT soundings at site G in Madison County were performed only up to a depth of 3 m (10 ft) because the project involved the replacement of an existing structure and widening of the pavement. Since the HVI value depends on the sounding depth analyzed, the volume of CPT data obtained from the shallow, closely-spaced soundings at site G in Madison County may have been insufficient to render an HVI value that is representative of the site—this may have been another reason for the very high HVI value of 100% obtained for this site. Based on the procedure outlined previously, each site was assigned a qualitative site variability rating (SVR), such as LH for low vertical and high horizontal variability (e.g., site C) and MH for medium vertical and high horizontal variability (e.g., sites B, E to G, and J), as shown in Figure 2.24.

2.5 Optimal Spacing Between CPT Soundings

The cost of a CPT-based geotechnical site investigation is directly proportional to the number of CPT soundings performed, which in turn depends on site geology and variability. The cost of a CPT-based site investigation could be reduced by optimizing the spacing between CPT soundings based on the site variability determined from the soundings already performed at the site. Figure 2.25 shows two CPT soundings, X and Y, that have already been performed at a site; the center-to-center spacing between them is $s_{xy}$.
The optimal spacing ($s_{yc}$)$_{opt}$ between CPT sounding Y and the next sounding Z can be calculated by following these steps (Ganjju et al., 2019; Salgado et al., 2015, 2019):

**Step 1:** Set the analysis (segment) length $L$ as the minimum of the sounding depths of CPT soundings X and Y.

**Step 2:** Determine the number $N$ of cone resistance data points contained within the segment length $L$.

**Step 3:** Calculate the mean cone resistances $\bar{x}$ and $\bar{y}$ of CPT soundings X and Y, respectively, for the segment length considered.

**Step 4:** Calculate the standard deviations $\sigma_x$ and $\sigma_y$ of the $q_c$ values of CPT soundings X and Y, respectively, using:

$$\sigma_x = \sqrt{\frac{1}{N-1} \sum_{i=1}^{N} (x_i - \bar{x})^2} \quad \text{(Eq. 2.8)}$$

$$\sigma_y = \sqrt{\frac{1}{N-1} \sum_{i=1}^{N} (y_i - \bar{y})^2} \quad \text{(Eq. 2.9)}$$

where $x_i$ and $y_i = q_c$ values of the $i^{th}$ data point obtained from CPT soundings X and Y, respectively. The standard deviation of a sample dataset can also be calculated using the STDEV function in Microsoft Excel.

**Step 5:** Estimate the cross-covariance $C_{xy}$ and the cross-correlation coefficient $\rho_{xy}$ between CPT soundings X and Y using:

$$C_{xy} = \frac{1}{N} \sum_{i=1}^{N} (x_i - \bar{x})(y_i - \bar{y}) \quad \text{(Eq. 2.10)}$$

$$\rho_{xy} = \frac{C_{xy}}{\sigma_x \sigma_y} \quad \text{(Eq. 2.11)}$$

The cross-covariance and cross-correlation coefficient of a sample dataset can also be calculated using the functions COVARIANCE.S and CORREL, respectively, in Microsoft Excel. The cross-correlation coefficient $\rho_{xy}$ takes values in the –1 to +1 range. A high cross-correlation coefficient and small $q_c$ trend difference of a CPT pair indicates high correlation and similarity between the two CPTs, and thus low variability in the horizontal direction for the site.

**Step 6:** Calculate the average $q_c$ difference $|\Delta q_c,avg|$ between CPT soundings X and Y using:

$$|\Delta q_c,avg| = \frac{\sum_{i=1}^{N} |x_i - y_i|}{N} \quad \text{(Eq. 2.12)}$$

where $x_i$ and $y_i = q_c$ values of the $i^{th}$ data point obtained from CPT soundings X and Y, respectively, and $N = number of q_c$ data points contained within the segment length $L$.

**Step 7:** Estimate the maximum credible difference $|\Delta q_c,avg|_{max}$ between $q_c$ trends for the segment length considered using:

$$\frac{|\Delta q_c,avg|_{max}}{p_A} = 23.86 \left( \frac{L}{L_R} \right)^{0.46} - 4.30$$

for $1 \leq \frac{L}{L_R} \leq 30 \quad \text{(Eq. 2.13)}$

where $L = analysis (segment) length$, $L_R = reference length (= 1 m or 3.28 ft)$, and $p_A = reference stress (= 100 \text{ kPa or 14.5 psi})$. The maximum credible difference is determined by considering two idealized soil profiles, one with a very soft clay layer throughout, and the other with sand having 85% relative density throughout (Salgado et al., 2019).

**Step 8:** Calculate the values of functions $f_0, f_1,$ and $f_2$ using:

$$f_0 = \min \left[ \frac{|\Delta q_c,avg|}{|\Delta q_c,avg|_{max}} ; 1 \right] \quad \text{(Eq. 2.14)}$$

$$f_1 = \frac{\rho_{xy} + 1}{2} \quad \text{(Eq. 2.15)}$$

$$f_2 = 1 - \exp \left( -0.2S_{s_{xy}} \frac{L}{L_R} \right) \quad \text{(Eq. 2.16)}$$

where $s_{xy} = spacing between CPT soundings X and Y$, and $L_R = reference length (= 1 m or 3.28 ft)$.

**Step 9:** Estimate the horizontal variability index (HVI) for CPT soundings X and Y using:

(continued...)

![Figure 2.23 Site variability rating chart (modified from Salgado et al., 2015).](image)
### TABLE 2.4
Vertical variability index, horizontal variability index, and site variability rating for the sites analyzed

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil Type</th>
<th>County</th>
<th>Number of Soundings</th>
<th>Sounding Depth Analyzed (m)</th>
<th>Sounding ID</th>
<th>VVI (%)</th>
<th>Site VVI (%)</th>
<th>Site HVI (%)</th>
<th>SVR</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Dune sand</td>
<td>Lake</td>
<td>1</td>
<td>19.50</td>
<td>1006751CPT1</td>
<td>30</td>
<td>30</td>
<td>—</td>
<td>MH</td>
<td>Low vertical variability</td>
</tr>
<tr>
<td>B</td>
<td>Till in hummocky moraine form</td>
<td>Steuben</td>
<td>4</td>
<td>6.50</td>
<td>0810115RB2</td>
<td>52</td>
<td>53</td>
<td>100</td>
<td>MH</td>
<td>Medium vertical variability and high horizontal variability</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0810115TB1</td>
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<td></td>
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<tr>
<td>C</td>
<td>Outwash</td>
<td>LaPorte</td>
<td>2</td>
<td>10.15</td>
<td>0101453TB1</td>
<td>27</td>
<td>26</td>
<td>79</td>
<td>LH</td>
<td>Low vertical variability and high horizontal variability</td>
</tr>
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<td>0101453TB2</td>
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<td>15.15</td>
<td>1006752CPT2</td>
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<td>23</td>
<td>—</td>
<td>MH</td>
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</tr>
<tr>
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<td></td>
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<td>40</td>
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<td>0400774CPT5</td>
<td>43</td>
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<tr>
<td>F</td>
<td>Glacial till</td>
<td>Clinton</td>
<td>4</td>
<td>3.17</td>
<td>Frankfort02</td>
<td>69</td>
<td>63</td>
<td>96</td>
<td>MH</td>
<td>Medium vertical variability and high horizontal variability</td>
</tr>
<tr>
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<td></td>
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<td></td>
<td></td>
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<td>54</td>
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<tr>
<td>G</td>
<td>Glacial till</td>
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<td>3</td>
<td>3.00</td>
<td>0101420CPT1</td>
<td>75</td>
<td>57</td>
<td>100</td>
<td>MH</td>
<td>Medium vertical variability and high horizontal variability</td>
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<td></td>
<td>0101420CPT2</td>
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<td>H</td>
<td>Glacial till</td>
<td>Decatur</td>
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<td>69</td>
<td>69</td>
<td>—</td>
<td></td>
<td>High vertical variability</td>
</tr>
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<td>I</td>
<td>Loess with sand</td>
<td>Knox</td>
<td>1</td>
<td>14.40</td>
<td>0800579CPT1</td>
<td>68</td>
<td>68</td>
<td>—</td>
<td></td>
<td>High vertical variability</td>
</tr>
<tr>
<td>J</td>
<td>Lacustrine soil</td>
<td>Vanderburgh</td>
<td>3</td>
<td>19.95</td>
<td>VHC027</td>
<td>43</td>
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<td>MH</td>
<td>Medium vertical variability and high horizontal variability</td>
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<td>VHC033</td>
<td>43</td>
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</tr>
</tbody>
</table>

Note: The site VVI is the average of the individual VVIs of all CPT soundings performed at the site. The first letter in SVR corresponds to the site VVI, while the second letter corresponds to the site HVI. Site HVI values and SVRs could not be assigned for sites where only one CPT sounding was performed.
The horizontal variability index ranges from 0 for a perfectly uniform site to 1 for a highly variable site.

**Step 10:** Compute the optimal spacing \((s_{yz})_{opt}\) between CPT sounding Y and the next sounding Z using:

\[
(s_{yz})_{opt} = (1.5 - \text{HVI}) s_{xy} \quad (\text{Eq. 2.18})
\]

Equation 2.18 shows that if the value of HVI is greater than 0.5, the spacing for the next CPT sounding is decreased, but if the value of HVI is less than 0.5, the spacing for the next CPT sounding is increased.

**Step 11:** If the CPT soundings are not performed in line but are distributed in two dimensions, execute the following substeps.

a. Determine the number of pairs of CPT soundings performed at the site using:

\[
^nC_r = \frac{n!}{(n-r)! r!} \quad (\text{Eq. 2.19})
\]

where \(^nC_r = \text{number of combinations in which } n \text{ objects can be selected } r \text{ at a time, } n = \text{number of CPT soundings already performed at the site, and } r = 2 \text{ (for a pair of CPT soundings). The number of pairs of CPT soundings available at a site can also be calculated using the COMBIN function in Microsoft Excel.}

b. Repeat steps 1 through 9 for all pairs of CPT soundings performed at the site.

c. Calculate the average of the HVI values for all pairs of CPT soundings performed at the site.

d. Substitute the average HVI value for the site into Eq. 2.18 to obtain the new spacing for the next CPT sounding. The next CPT sounding will be at a distance no greater than \((s_{yz})_{opt}\) from any sounding already performed at the site.

The procedure for estimation of optimal spacing between CPT soundings is presented only to provide some guidance. The spacing between CPT soundings in the field may be adjusted based on the level of importance of the structure, knowledge of the site geology, and soil profile variability.

### 2.6 Chapter Summary

In this chapter, an overview of the bedrock and surficial geology of Indiana was presented along with the CPT, SPT and soil profiles obtained from ten different locations across Indiana. About two-thirds of Indiana is covered by sediments that were transported and deposited by glaciers during the Ice Age; the
bedrock surface is visible only in the south-central part of the state. The bedrock geology of Indiana mainly consists of five bedrock units: Pennsylvanian, Mississippian, Devonian, Silurian, and Ordovician units, which in turn consist of sedimentary rocks, such as limestone, dolomite, shale, sandstone, and siltstone. Limestone and dolomite dissolve slowly in water to produce karstic landforms (commonly found in southern Indiana) with underground cavities that may collapse, forming sinkholes. The surface geology of Indiana consists of soils transported by wind, water, or ice: (a) dune and aeolian sands in northern Indiana, (b) outwash in northern Indiana and along major river valleys, (c) glacial till in central Indiana, and (d) loess in southwestern Indiana.

CPT and SPT data were obtained from 10 select sites across Indiana. The data was analyzed to obtain depth profiles of cone resistance $q_c$, sleeve resistance $f_s$, friction ratio (FR), corrected SPT blow count $N_{60}$, and $q_c/p_A N_{60}$. The CPT data was post-processed through a soil profile generation algorithm developed by Ganju et al. (2017) to generate SBT profiles for each site using the modified Tumay (1985) SBT chart. According to this chart, a combination of low $q_c/p_A (< 10)$ and high $f_s/q_c$ values (> 4%) suggests a clayey soil, whereas a combination of high $q_c/p_A (> 50)$ and low $f_s/q_c$ values (< 2%) suggests a sandy soil, where $p_A = $ reference stress ($= 100$ kPa or $14.5$ psi). For each site, the CPT-based SBT profiles compared reasonably well with the corresponding soil profiles obtained from the SPT boring logs. The SBT profiles account for the presence of thin layers, which are otherwise not captured by the soil profiles reported in the SPT boring logs. This is because the SBT profiles are based on nearly continuous CPT measurements at depth intervals of 5 cm (2 in.) or less, whereas the soil profiles obtained from the SPT boring logs are based on laboratory testing of soil samples collected typically at depth intervals of 1.5 m (5 ft). The modified Tumay (1985) chart can be used for generating SBT profiles for all soil types in Indiana, except for gravelly materials, for which the modified Robertson (1990) chart is more appropriate.

A correlation between cone resistance $q_c$, corrected SPT blow count $N_{60}$, and mean particle size $D_{50}$ was developed based on data reported by Robertson et al. (1983) and data obtained from 15 sites in Indiana. The correlation may be used to obtain an estimate of $q_c$ for use in a CPT-based foundation design method when only SPT blow counts are available for a site because CPT-based methods tend to be more reliable. However, as with any correlation involving the SPT blow count, it should be used with caution because of the potential error introduced by the transformation from the SPT blow count (a dynamic resistance) to the CPT cone resistance (a quasi-static resistance). In such cases when only SPT data is available for the site, it may be preferable to use SPT-based methods for design (though not in clay) instead of CPT-based methods.

A CPT-based site variability assessment methodology developed by Salgado et al. (2019) was applied to assess the vertical and horizontal variability of the 10 sites in Indiana. The vertical variability of a CPT sounding was quantified via the vertical variability index (VVI), which reflects the intra-layer variability, the log variability and the COV of the cone resistance of the sounding. The site VVI was taken as the average of the individual VVIs of all CPT soundings performed at a site. The horizontal variability of a site was quantified via the site horizontal variability index (site HVI), which depends on the cross-correlation between cone resistance logs, cone resistance trend differences, and the spacing between CPT soundings. The site VVI and HVI values were combined into an overall site variability rating (SVR) system.

A step-by-step procedure for estimation of optimal spacing between CPT soundings was presented (Table 2.5). However, in order to implement the procedure, data from at least two CPT soundings are needed in advance to estimate the optimal spacing of future CPT soundings performed at a site. The procedure may be further refined through future research, and so the use of this procedure in INDOT construction projects is optional based on the level of familiarity of the engineers with the CPT and the specific site investigation goals of the project under consideration. CPT soundings at the desired spacing may be performed based on the level of importance of the structure, knowledge of the site geology, and soil profile variability.
3.1 Calculation Procedure for Footing Settlement

The total settlement $w$ of an axially-loaded footing can be calculated from CPT results by following these steps.

Step 1: Obtain the site stratigraphy, the groundwater table depth, and the unit weight of the soil in each layer of the profile.

a. Establish the site stratigraphy either from the boring log or by using a CPT-based soil behavior type (SBT) chart (refer to Section 2.2.3 of Volume I) or both if possible.

b. Obtain the depth $z_y$ of the groundwater table from either the boring log or the depth profile of $u_2$ or both if possible, where $u_2$ = pore water pressure measured at the shoulder position behind the cone face (refer to Volume I).

c. Obtain the unit weight of the soil in each layer of the profile whenever soil samples are recovered during the site investigation. In the absence of soil samples, the reader may refer to Section 2.3.3 of Volume I for correlations between the unit weight and CPT data. In general, the saturated unit weight $\gamma_{sat}$ of soil typically ranges from 18–21 kN/m$^3$ (115–135 pcf) for sand, 18.5–22.5 kN/m$^3$ (118–143 pcf) for silty sand, and 15–18 kN/m$^3$ (95–115 pcf) for clay (Salgado, 2008).

Step 2: Set the footing shape (e.g., strip, square, rectangular, or circular), the preliminary geometry (length $L$ and width $B$) of the footing, and the embedment depth $D$ of the footing.

Step 3: Classify the soil in each layer of the profile below the footing as either “sand” or “clay.” For mixed or intermediate soils (i.e., soils containing mixtures of sand, silt, and clay), execute the following substeps.

a. Sand-silt, sand-clay or sand-silt-clay mixtures: Classify these soils as “clay” if fines content FC $\geq 20\%$ and plasticity index PI $\geq 8\%$, otherwise classify them as “sand” (Carraro et al., 2009; Salgado et al., 2000).

b. Sands containing gravel: If a site contains sand layers with gravel content greater than 20%, use the lower-
bound profile of $q_c$, drawn approximately through the valleys of the actual $q_c$ profile, for estimating footing settlement and bearing capacity.

**Note:** In the absence of soil samples, the reader may refer to Section 2.2 of Volume I for estimation of soil behavior type from CPT results.

**Step 4:** Correct the raw $q_c$ data for the pore water pressure generated during cone penetration using (ASTM, 2012):

$$q_c = q_i + (1 - a)t_2$$  \hspace{1cm} (Eq. 3.1)

where $q_i =$ corrected, total cone resistance, $q_c =$ measured cone resistance, $a =$ cone area ratio ($\approx 0.8$ for typical CPT probes), and $t_2 =$ pore water pressure measured at the shoulder position behind the cone face. The pore water pressure correction to the $q_c$ data may be ignored for coarse-grained soils (e.g., sand and gravel) because $q_i$ is approximately equal to $q_c$ in such soils.

**Step 5:** Obtain the footing load and maximum tolerable settlement.

a. Obtain the unfactored structural load $Q$ that will be applied on the footing from the structural engineer.

b. Set the maximum tolerable angular distortion $w_{\text{max}}$ as 1/500 (Skempton & MacDonald, 1956) or other such value specified by a geotechnical code.

c. Set the maximum tolerable settlement $w_{\text{max}}$ of the footing from Table 3.1 or other such value specified by a geotechnical code.

**Step 6:** Calculate the total settlement of the footing.

a. Total settlement of footings in “sand” (Lee et al., 2008; Lee & Salgado, 2002; Schmertmann, 1970; Schmertmann et al., 1978). Execute the following substeps for footings in “sand,” otherwise proceed to step 6(b).

i. Determine the critical-state friction angle $\phi_c$ of sand through one of the following options.

   * Select a $\phi_c$ value between $28^\circ$ and $36^\circ$ for silica sand; sands with rounded, smooth particles with a poorly-graded particle size distribution have values near the low end of this range, while sands with angular, rough particles with a well-graded particle size distribution have values near the high end of this range (refer to Appendix A for additional information if needed).

   * If the mean particle size $D_{50}$, coefficient of uniformity $C_u$, and particle roundness $R$ of the sand are known, estimate the critical-state friction angle using:

   $$\phi_c(\cdot) = 28.3 \left( \frac{D_{50}}{D_{\text{ref}}} \right)^{0.02} (C_u)^{0.08} (R)^{-0.8}$$  \hspace{1cm} (Eq. 3.2)

   where $D_{\text{ref}} =$ reference particle size ($\approx 1$ mm or 0.04 in.), and $\zeta =$ exponent ($= 0.045$). Equation 3.2 is applicable for poorly-graded, clean silica sands with $D_{50} =$ 0.15–2.68 mm (0.006–0.105 in.), $C_u =$ 1.2–3.1, and $R =$ 0.3–0.8. The data used in the development of this equation along with example calculations can be found in Appendix A.

   * If direct shear or triaxial compression test results are available, it is recommended that the critical-state friction angle be determined from such test results.

   ii. Calculate the gross unit load $q_b$ on the footing base (including the loads from the superstructure, the weight of the foundation, and the weight of the backfill when the excavation is backfilled):

   $$q_b = \frac{Q + W_{\text{ftg}} + W_{\text{fill}}}{A}$$  \hspace{1cm} (Eq. 3.3)

   where $Q =$ unfactored column (or wall) load on the footing, $W_{\text{ftg}} =$ weight of the footing ($= \gamma'_{\text{ftg}}A$), $\gamma_c =$ unit weight of concrete ($= 24$ kN/m$^3$ or 150 pcf), $A =$ area of the footing base, $t =$ thickness of the footing, $W_{\text{fill}} =$ weight of the backfill = max[$\gamma_{\text{fill}}A(D - t) ; 0$]. $\gamma_{\text{fill}} =$ unit weight of the backfill, and $D =$ depth of embedment of the footing. If the footing is not backfilled, $W_{\text{fill}} =$ 0. If the thickness of the footing is unknown, an “average” unit weight $\gamma_{\text{avg}}$ may be used for the material above the footing base to calculate the gross unit load $q_b$:

   $$q_b = \frac{Q}{A} + \gamma_{\text{avg}}D = \frac{Q}{A} + \left[ \frac{\gamma_c + \gamma_{\text{fill}}}{2} \right]D$$  \hspace{1cm} (Eq. 3.4)

   iii. Calculate the influence depth $z_{\text{ftg}}$ measured from the footing base using:

   $$\frac{z_{\text{ftg}}}{B} = 2 + 0.4 \left[ \min \left( \frac{L}{B} ; 6 \right) - 1 \right]$$  \hspace{1cm} (Eq. 3.5)

   iv. Calculate the depth $z_{\text{ftg}}$ measured from the footing base at which the strain influence factor peaks using:

   $$\frac{z_{\text{ftg}}}{B} = 0.5 + 0.1 \left[ \min \left( \frac{L}{B} ; 6 \right) - 1 \right]$$  \hspace{1cm} (Eq. 3.6)

   v. Based on the cone resistance profile, divide the soil layers within the influence depth $z_{\text{ftg}}$ below the footing base into sublayers such that the $q_c$ values within each sublayer are either approximately constant or linear with depth so that a representative cone resistance can be assigned to each sublayer.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Isolated Foundations</th>
<th>Mat Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>$15L_R$</td>
<td>$20L_R$</td>
</tr>
<tr>
<td>Clay</td>
<td>$25L_R$</td>
<td>$30L_R$</td>
</tr>
</tbody>
</table>

Note: $L_R =$ reference length ($\approx 1$ m or 39.4 in.). Strip footings are continuous and behave more like mat foundations than isolated foundations.
vi. Estimate the strain influence factor $I_z$ for the sublayer using (Figure 3.1):

$$I_z = \begin{cases} 
I_{z0} + \frac{\gamma' \left( L - I_{z0} \right)}{\gamma'} & \text{for } z_f < z_{fp} \\
\frac{\gamma' \left( z_f - z_{fp} \right)}{\gamma' \left( z_f - z_{fp} \right)} & \text{for } z_{fp} \leq z_f \leq z_{z0} 
\end{cases} \tag{Eq. 3.7}
$$

where $z_f = \text{vertical distance from the footing base to the middle of the sublayer}$, $I_{z0} = \text{strain influence factor at the footing base level}$, and $I_{zp} = \text{peak strain influence factor}$:

$$I_{z0} = \min \left[ 0.1 + 0.0111 \left( \frac{L}{B} - 1 \right) ; 0.2 \right] \tag{Eq. 3.8}$$

$$I_{zp} = 0.5 + 0.1 \sqrt{\frac{q_b - \sigma'_{z0}}{\sigma'_{z0} - \gamma'}} \tag{Eq. 3.9}$$

where $\sigma'_{z0}|_{z=0} = \text{in situ vertical effective stress at the footing base level}$, and $\sigma'_{z0}|_{z=z_{fp}} = \text{in situ vertical effective stress at the depth corresponding to } z_{fp}$.

vii. Determine the coefficient of lateral earth pressure at-rest $K_0$ of the sublayer (refer to Appendix B for guidance).

viii. Estimate the relative density $D_R$ of the sublayer using (Salgado & Prezzi, 2007):

$$D_R(\%) = \frac{\ln \left( \frac{q_c}{q_{c_{in}}} \right) - 0.4947 - 1.041\phi_s - 0.841 \ln \left( \frac{\sigma_{w}}{\gamma'} \right)}{0.0264 - 0.0002\phi_s - 0.0047 \ln \left( \frac{\sigma_{w}}{\gamma'} \right)} \tag{Eq. 3.10}$$

where $q_c = \text{representative cone resistance of the sublayer}$, $\frac{\sigma_{w}}{\gamma'} = \text{in situ horizontal effective stress at the middle of the sublayer}$, and $\sigma'_{z0} = \text{in situ vertical effective stress at the middle of the sublayer}$ (Terzaghi, 1943):

$$\sigma'_{z0} = \sigma_{z0} - \gamma' \tag{Eq. 3.11}$$

where $\sigma_{z0} = \text{in situ vertical total stress at the middle of the sublayer}$, $\gamma'$ = unit weight of water ($= 9.81 \text{kN/m}^3$ or 62.45 psf), $z = \text{depth measured from the ground surface to the middle of the sublayer}$, and $z_{w} = \text{depth of the groundwater table}$.

ix. Estimate the elastic modulus $E$ of the sublayer using:

$$E = \lambda \left( \frac{w}{L_R} \right)^{-0.285} \left( \frac{B}{L_R} \right)^{0.4} \left( \frac{D_R}{100} \right)^{-0.65} \tag{Eq. 3.12}$$

where $w = \text{initial guess value for footing settlement}$ ($= w_{max}$ established in step 5), $B = \text{width or diameter of the footing}$, $L_R = \text{reference length}$ ($= 1 \text{m or 3.28 ft}$), $D_R = \text{relative density of the sublayer (expressed as a percentage)}$, and $\lambda = \text{parameter that accounts for the effects of aging and overconsolidation of sand}$.

Figure 3.1 Strain influence factor $I_z$ versus depth $z_f$ below the footing base (after Salgado 2008; Schmertmann et al., 1978).
x. Compute the total settlement \( w \) of the footing using:

\[
  w = C_1 C_2 \left( q_0 - \sigma'_{vo} \right) \sum_{i=1}^{n} \left( \frac{I_i \Delta z_i}{E_i} \right) \quad (\text{Eq. 3.14})
\]

where \( \Delta z = \) thickness of the sublayer, \( n = \) number of sublayers within the influence depth \( z_0 \) below the footing base, and \( C_1 \) and \( C_2 = \) depth and time factors, respectively:

\[
  C_1 = 1 - 0.5 \left( \frac{\sigma'_{vo} \left| z_0 = 0 \right)}{q_0 - \sigma'_{vo} \left| z_0 = 0 \right)} \right) \quad (\text{Eq. 3.15})
\]

\[
  C_2 = 1 + 0.2 \log \left( \frac{t}{0.1 t_R} \right) \quad (\text{Eq. 3.16})
\]

where \( t_R = \) reference time (\( = 1 \) year), and \( t = \) service life of the superstructure (in the same unit as \( t_R \)).

xi. Compare the value of \( w \) calculated using Eq. 3.14 with the initial guess value assumed in substep (ix). If the two values match, then report the value of \( w \) calculated using Eq. 3.14 as the settlement of the footing. However, if they do not match, return to substep (ix) and use the new value of \( w \) obtained from Eq. 3.14 as the initial guess value for the next iteration (refer to Appendix C for guidance).

b. Total settlement of footings in “clay.” Execute the following substeps for footings in “clay,” otherwise proceed to step 7.

**Immediate settlement of footings in clay (Foye et al., 2008)**

i. Obtain the depth profile of undrained shear strength \( s_u \) below the footing base using (Salgado, 2008):

\[
  s_u = \frac{q_t - \sigma'_{vo}}{N_k} \quad (\text{Eq. 3.17})
\]

where \( q_t = \) corrected, total cone resistance measured under undrained conditions, \( \sigma'_{vo} = \) \textit{in situ} vertical total stress at the depth being considered, and \( N_k = \) cone factor (\( = 9–15 \) as long as the CPT is performed at a penetration rate that is sufficiently high to ensure undrained penetration (refer to Appendix D); soft NC clays tend to have \( N_k \) values near the low end of this range, while stiff OC clays tend to have \( N_k \) values near the high end of this range) (Bisht et al., 2021; Mayne & Peuchen, 2018; Salgado, 2008, 2013, 2014; Salgado et al., 2004).

ii. Average the values of \( s_u \) over a vertical distance of \( B \) below the footing base to obtain a representative undrained shear strength \( s_u \).

iii. Calculate the influence depth \( z_{0\delta} \) below the footing base within which most of the strains develop using:

\[
  \frac{z_{0\delta}}{B} = \min \left[ 1 + 0.111 \left( \frac{L}{B} - 1 \right); 2 \right] \quad (\text{Eq. 3.18})
\]

iv. Obtain the small-strain shear modulus \( G_0 \) profile within the influence depth \( z_{0\delta} \) below the footing base from the results of seismic cone penetration tests (SCPTs) using (Salgado, 2008):

\[
  G_0 = \frac{\gamma_m}{g} \frac{V_s^2}{E} \quad (\text{Eq. 3.19})
\]

where \( \gamma_m = \) unit weight of soil (\( = \gamma_{sat} \) if the soil is saturated), \( g = \) acceleration due to gravity (\( = 9.81 \text{ m/s}^2 \) or \( 32.17 \text{ ft/s}^2 \)), and \( V_s = \) shear wave velocity (refer to Section 2.3.4 of Volume I).

If SCPT results are unavailable, the small-strain shear modulus may be estimated using the following correlation (Viggiani & Atkinson, 1995):

\[
  G_0 = C_g \left( \frac{100 \sigma_{mo}}{p_A} \right)^{n_k} R_0^{m_k} \quad (\text{Eq. 3.20})
\]

where \( C_g, n_k, \) and \( m_k = \) parameters that depend on the plasticity index \( PI; \sigma_{mo} = \) \textit{in situ} mean effective stress at the depth being considered; \( p_A = \) reference stress (\( = 100 \text{ kPa} \) or \( 14.5 \text{ psi} \)); and \( R_0 = \) mean stress-based overconsolidation ratio:

\[
  R_0 = \frac{p'_{pl}}{p'} = \frac{OCR}{1 + 2 K_{0, NC} / \sqrt{OCR}} \quad (\text{Eq. 3.21})
\]

where \( p'_{pl} = \) value of \( p' \) at the intersection of the recompression line with the normal consolidation line in \( v-\ln p' \) space, \( v = \) specific volume (\( = 1 + \rho \)), \( K_{0, NC} = \) coefficient of lateral earth pressure at-rest for normally consolidated soil (\( = 0.50–0.75 \) for NC clay), and \( OCR = \) overconsolidation ratio (refer to Appendix B for guidance).

The parameters \( C_g, n_k, \) and \( m_k \) can be calculated using (Foye et al., 2008; Viggiani & Atkinson, 1995):

\[
  C_g = 37.9 \exp (-0.045 PI) \quad (\text{for PI} > 5\%) \quad (\text{Eq. 3.22})
\]

\[
  n_k = 0.109 \ln (PI) + 0.4374 \quad (\text{for PI} > 5\%) \quad (\text{Eq. 3.23})
\]

\[
  m_k = 0.0015 \ln (PI) + 1.863 \quad (\text{for PI} > 5\%) \quad (\text{Eq. 3.24})
\]

The \textit{in situ} mean effective stress can be calculated using:

\[
  \sigma_{vo} = \frac{1}{k+1} (\sigma_{vo} + k \sigma_{ho}) \quad (\text{Eq. 3.25})
\]

where \( k = 1 \) for plane-strain conditions (e.g., strip footings) and 2 for triaxial conditions (e.g., isolated footings), \( \sigma_{vo} = \) \textit{in situ} vertical effective stress at the depth being considered, \( \sigma_{ho} = \) \textit{in situ} horizontal effective stress at the depth being considered (\( = K_{0, NC} \sigma_{vo} \)), and \( K_0 = \) coefficient of lateral earth pressure at-rest (refer to Appendix B for guidance). The plasticity index \( PI = \) the difference between the liquid limit \( LL \) and the plastic limit \( PL \) of the soil (\( PI = LL - PL \)).

v. Calculate a representative small-strain shear modulus \( G_0 \) by taking the weighted average of the \( G_0 \) values
within the influence depth \( z_{g_0} \) below the footing base:

\[
\hat{G}_0 = \frac{\sum_{i=1}^{n} G_{i}^{avg} H_i}{\sum_{i=1}^{n} H_i} \quad \text{(Eq. 3.26)}
\]

where \( G_{i}^{avg} = \) average small-strain shear modulus of layer \( i \), \( H_i = \) thickness of layer \( i \), and \( n = \) number of clay layers within the influence depth \( z_{g_0} \) below the footing base.

vi. Using trial footing dimensions, estimate the net unit load \( q_{b,net} \) on the footing base:

\[
q_{b,net} = q_b - \gamma_m D
\]

where \( q_b = \) gross unit load on the footing base (including the loads from the superstructure, the weight of the foundation, and the weight of the backfill when the excavation is backfilled; refer to Eqs. 3.3 and 3.4), and \( \gamma_m D = \) total overburden stress at the footing base level.

vii. Obtain the influence factor \( I_q \) from Figure 3.2; \( H = \) thickness of the clay layer below the footing base, and \( B = \) footing width. For circular footings, an equivalent footing width may be obtained by equating the cross-sectional area of the footing with that of an equivalent square.

viii. Compute the immediate settlement \( w_i \) of the footing using:

\[
w_i = I_q \frac{q_{b,net} B}{E_0} \quad \text{(Eq. 3.29)}
\]

Primary consolidation settlement of footings in clay (Skempton & Bjerrum, 1957)

i. Divide the clay layer below the footing base into \( n \) sublayers of thickness \( \Delta z \).

ii. Calculate the vertical stress increment \( \Delta \sigma_v,i \) at the middle of each sublayer caused by the applied load \( Q \) using the 2-to-1 stress distribution rule:

\[
\Delta \sigma_v,i = \begin{cases} 
\frac{Q}{B + z_f} & \text{for strip footings} \\
\frac{4Q}{\pi(B + z_f)} & \text{for circular footings} \\
\frac{Q}{(B + z_f)(L + z_f)} & \text{for rectangular footings}
\end{cases}
\]

\[
\text{(Eq. 3.30)}
\]

where \( z_f = \) vertical distance from the footing base to the middle of the sublayer. \( Q \) takes units of load per unit length for strip footings and units of load for all other footings.

iii. Obtain the initial void ratio \( e_0 \) of the sublayer using the relationship \( e_0 = \frac{w_c G_s}{S} \) where \( w_c = \) water content, \( G_s = \) specific gravity of solids (= 2.60–2.80 for clay), and \( S = \) degree of saturation (= 1 for saturated clay). In the absence of soil samples, the reader may refer to Section 2.3.1 of Volume I for additional information on \( e_0 \).

iv. Estimate the vertical compressive strain \( \Delta \varepsilon_z \) of the sublayer using:

\[
\Delta \varepsilon_z = \begin{cases} 
\frac{c_s}{1 + v} \log \left( \frac{z}{z_0} \right) & \text{if } \sigma'_{i0} = \sigma'_{ip} \text{ and } \sigma'_{i} \geq \sigma'_{ip} \text{ (NC clay)} \\
\frac{1}{1 + v} \left[ C_s \log \left( \frac{z}{z_0} \right) + C_v \log \left( \frac{z_0}{z} \right) \right] & \text{if } \sigma'_{i0} < \sigma'_{ip} \leq \sigma'_{i} \text{ (OC then NC clay)} \\
\frac{c_s}{1 + v} \log \left( \frac{z}{z_0} \right) & \text{if } \sigma'_{i0} \leq \sigma'_{ip} \text{ and } \sigma'_{i} \leq \sigma'_{ip} \text{ (OC clay)}
\end{cases}
\]

\[
\text{(Eq. 3.31)}
\]

where \( \sigma'_{i0} = \) initial (or \( \text{in situ} \)) vertical effective stress at the middle of the sublayer before the stress increment is applied, \( \sigma'_{i} = \) current vertical effective stress at the middle of the sublayer after the stress increment is applied and full primary consolidation has taken place (= \( \sigma'_{i0} + \Delta \sigma_z \)), \( \sigma'_{ip} = \) preconsolidation stress, \( C_s = \) compression index, and \( C_v = \) swelling index.

In the absence of laboratory consolidation test results, the compression index \( C_s \) may be estimated using the following approximate correlation (Wroth & Wood, 1978):

\[
C_s \approx \frac{1}{200} G_s \text{PI} \%
\]

\[
\text{(Eq. 3.32)}
\]

where \( \text{PI} = \) plasticity index (expressed as a percentage). The swelling index \( C_v \) typically ranges from 0.1\( C_s \) to 0.2\( C_s \).

v. Compute the 1D consolidation settlement \( w_{c,1D} \) of the clay layer below the footing base using:

\[
w_{c,1D} = \sum_{i=1}^{n} \Delta \varepsilon_z,i \Delta z_i
\]

\[
\text{(Eq. 3.33)}
\]

where \( \Delta z_i = \) thickness of sublayer \( i \), and \( n = \) number of sublayers.

vi. Compute the primary consolidation settlement \( w_c \) of the footing using:

\[
w_c = \left[ A + \alpha(1 - A) \right] w_{c,1D}
\]

\[
\text{(Eq. 3.34)}
\]

\[
\alpha = \frac{\Delta \sigma_{c1}}{\int_0^H \Delta \sigma_{c1} dz}
\]

\[
\text{(Eq. 3.35)}
\]

where \( A = \) Skempton’s pore pressure parameter (= 0.5–0.75 for NC clay and 0.3–0.5 for OC clay), \( \Delta \sigma_{c1} = \) major principal stress increment, \( \Delta \sigma_{c1} = \) minor principal stress increment, and \( H = \) thickness of the clay layer below the footing base.

Table 3.2 summarizes the values of \( \alpha \) for circular and strip footings as a function of \( H/B \). For square
footings, the value of $a$ for a circular footing with the same cross-sectional area as that of a square footing may be used. For rectangular footings with $0 < B/L < 1$, obtain the value of $a$ by interpolation.

vii. Sum the values of $w_i$ and $w_c$ to obtain the total settlement $w$ of the footing. Note that if significant secondary consolidation is expected at the site, it should be considered together with primary consolidation.

**Step 7: Total settlement check.**

Compare the estimated total settlement $w$ of the footing with the maximum tolerable settlement $w_{max}$ selected in step 5. If $w \leq w_{max}$, the footing design is satisfactory with respect to the serviceability limit state (i.e., excessive settlement). Repeat step 6 to optimize the design if needed. However, if $w > w_{max}$, return to step 6 and revise the footing geometry.

**Step 8: Angular distortion check.**

Execute the following substeps for each pair of adjacent footings at the site.

a. Compute the angular distortion $\alpha$ for the selected footing pair using:

$$\alpha = \frac{\Delta w}{L_{cc}} \quad (\text{Eq. 3.36})$$

where $\Delta w =$ differential settlement, and $L_{cc} =$ span or center-to-center distance between the two footings.
b. Compare the estimated angular distortion \( z \) for the given footing pair with the maximum tolerable angular distortion \( z_{\text{max}} \) selected in step 5. If \( z \leq z_{\text{max}} \), the footing design is satisfactory with respect to the ultimate/serviceability limit state (i.e., excessive differential settlement). If \( z > z_{\text{max}} \), redo the footing design until the maximum tolerable angular distortion criterion is satisfied. If the criterion cannot be satisfied, consider alternative design solutions, such as the use of grade beams; combined footings; replacement of foundation soil with compacted, coarse-grained material; geosynthetic-reinforced foundation bed; mat (or raft) foundations; pile foundations; and piled rafts.

3.2 Calculation Procedure for Limit Bearing Capacity of Footings

The limit unit bearing capacity \( q_{b,L} \) of an axially-loaded footing can be calculated from CPT results by following these steps.

**Step 1:** Determine the nominal or characteristic cone resistance \( q_{c,\text{CAM}} \).

a. Combine the cone resistance profiles obtained from all CPT soundings performed at the site. Note that, for fine-grained soils (e.g., silts and clays), the cone resistance should be corrected for pore water pressure \( u_z \) using Eq. 3.1.

b. Perform a linear regression on the cone resistance data points to obtain the mean trend of the data with depth (Figure 3.3). When performing the regression, consider only those data points that follow the general trend of the \( q_c \) profile and ignore any outliers or regions that contain significant scatter in the data.

c. Draw lines (parallel to the mean trendline) bounding the cone resistance data points, as shown in Figure 3.3.

d. Determine the relationship of cone resistance with depth that is exceeded by 80% of the measurements using (Foye et al., 2006b):

\[
q_{c,\text{CAM}}(z) = E_{q_c}(z) - 0.84 \sigma_{q_c},
\]

(Eq. 3.37)

where \( q_{c,\text{CAM}}(z) \) = conservatively assessed mean (CAM) cone resistance determined using the 80% exceedance criterion (Becker, 1996) (as a function of depth \( z \)), \( E_{q_c}(z) \) = equation of the mean trendline obtained from the regression analysis, and \( \sigma_{q_c} \) = standard deviation of cone resistance (Foye et al., 2006a):

\[
\sigma_{q_c} = \frac{(q_{c,\text{MAX}} - q_{c,\text{MIN}})_{\text{sample}}}{N_x}
\]

(Eq. 3.38)

where \( q_{c,\text{MAX}} \) = value of cone resistance at any depth \( z \) on the upper bound line, \( q_{c,\text{MIN}} \) = value of cone resistance on the lower bound line at the same depth \( z \) at which \( q_{c,\text{MAX}} \) was computed (see Figure 3.3), and \( N_x \) = number of standard deviations of \( q_c \) (obtained from Table 3.3).

**Step 2:** Calculate the limit unit bearing capacity of the footing.

a. Limit unit bearing capacity of footings in “sand.” Execute the following substeps for footings in “sand,” otherwise proceed to step 2(b).

i. Using the values of \( z_a \), \( B \) and \( D \) determined from Section 3.1, calculate the value of the unit weight \( \gamma \) to use in the bearing capacity equation:

\[
\gamma = \begin{cases} 
\gamma_b \text{ if } z_u < D \\
\gamma_b + \left( \frac{z_u - D}{B} \right) (\gamma_m - \gamma_b) \text{ if } D \leq z_u \leq D+B \\
\gamma_m \text{ if } z_u > D+B
\end{cases}
\]

(Eq. 3.39)

where \( z_u \) = depth of the groundwater table, \( B \) = footing width or diameter, \( D \) = depth of embedment of the footing, \( \gamma_m \) = moist unit weight of sand, \( \gamma_b \) = buoyant unit weight of sand \((= \gamma_{sat} - \gamma_w)\), \( \gamma_{sat} \) = saturated unit weight of sand, and \( \gamma_w \) = unit weight of water \((= 9.81 \text{ kN/m}^3 \text{ or } 62.45 \text{ lb/ft}^3)\).

ii. Estimate the relative density \( D_K \) of sand at a depth of \( B/2 \) below the footing base using:

\[
D_K(\%) = \frac{\ln \left( \frac{q_{c,\text{CAM}}}{p_{A}} \right) - 0.4947 - 0.1041 \sigma_{q_c} - 0.841 \ln \left( \frac{z_w}{p_{A}} \right)}{0.0264 - 0.0002 \sigma_{q_c} - 0.0047 \ln \left( \frac{z_w}{p_{A}} \right)}
\]

(Eq. 3.40)

where \( q_{c,\text{CAM}} \) = conservatively assessed mean (CAM) cone resistance at a depth of \( B/2 \) below the footing base (obtained from Eq. 3.37), \( \sigma_{q_c} = \text{in situ} \) vertical effective stress at a depth of \( B/2 \) below the footing base, \( \sigma_{q_c} = \text{in situ} \) horizontal effective stress at a depth of \( B/2 \) below the footing base \( (= K_0 \sigma_{A}) \), \( p_{A} \) = reference stress \((= 100 \text{ kPa or } 14.5 \text{ psi})\), \( \phi_c \) = critical-state friction angle (refer to step 6(a)(ii) of Section 3.1), and \( K_0 \) = coefficient of lateral earth pressure at-rest (refer to Appendix B for guidance).

iii. Calculate the peak friction angle \( \phi_p \) of sand using (Bolton, 1986):

\[
\phi_p = \phi_c + A_\phi \left( \frac{D_K}{100} \right) \left[ Q - \ln \left( \frac{100 \sigma_{mp}}{p_{A}} \right) - R_Q \right]
\]

(Eq. 3.41)

\[
A_\phi = \min \left[ \frac{1}{5} \left( \frac{L}{B} + 4 \right), 5 \right]
\]

(Eq. 3.42)

where \( p_{A} \) = reference stress \((= 100 \text{ kPa or } 14.5 \text{ psi})\), \( Q \) and \( R_Q \) = fitting parameters that depend on the intrinsic characteristics of sand \((Q = 10 \text{ and } R_Q = 1 \text{ for clean silica sand})\), and \( \sigma_{mp} \) = representative mean effective stress (Loukidis, 2006; Salgado, 2008):

\[
\sigma_{mp} = 20p_{A} \left( \frac{\gamma B}{p_{A}} \right)^{0.7} \left( 1 - 0.32 \frac{B}{L} \right)
\]

(Eq. 3.43)

iv. Calculate the shape factors \( s_q \) and \( s_y \) using (Lyamin et al., 2007):

\[
s_q = 1 + 0.098(\phi_p - 1.64) \left( \frac{B}{L} \right)^{0.7 - 0.018\phi_p} \left( \frac{B}{L} \right)^{1 - 0.16(\phi_p)}
\]

(Eq. 3.44)

\[
s_y = 1 + 0.0336(\phi_p - 1) \frac{B}{L}
\]

(Eq. 3.45)
Figure 3.3   Examples of two CPT logs in clay and three CPT logs in sand with mean trendlines and range lines (after Foye et al., 2006a).

### TABLE 3.3
Values of \( N_s \) as a function of sample size \( n \) (after Tippett, 1925)

<table>
<thead>
<tr>
<th>( n )</th>
<th>( N_s )</th>
<th>( n )</th>
<th>( N_s )</th>
<th>( n )</th>
<th>( N_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.128379</td>
<td>12</td>
<td>3.258457</td>
<td>100</td>
<td>5.0152</td>
</tr>
<tr>
<td>3</td>
<td>1.692569</td>
<td>13</td>
<td>3.335982</td>
<td>200</td>
<td>5.492108</td>
</tr>
<tr>
<td>4</td>
<td>2.058751</td>
<td>14</td>
<td>3.406765</td>
<td>300</td>
<td>5.75566</td>
</tr>
<tr>
<td>5</td>
<td>2.325929</td>
<td>15</td>
<td>3.471828</td>
<td>400</td>
<td>5.936396</td>
</tr>
<tr>
<td>6</td>
<td>2.534413</td>
<td>16</td>
<td>3.531984</td>
<td>500</td>
<td>6.073445</td>
</tr>
<tr>
<td>7</td>
<td>2.704357</td>
<td>17</td>
<td>3.587886</td>
<td>600</td>
<td>6.183457</td>
</tr>
<tr>
<td>8</td>
<td>2.847201</td>
<td>18</td>
<td>3.640066</td>
<td>700</td>
<td>6.275154</td>
</tr>
<tr>
<td>9</td>
<td>2.970027</td>
<td>19</td>
<td>3.688965</td>
<td>800</td>
<td>6.353645</td>
</tr>
<tr>
<td>10</td>
<td>3.077506</td>
<td>20</td>
<td>3.734952</td>
<td>900</td>
<td>6.422179</td>
</tr>
<tr>
<td>11</td>
<td>3.172874</td>
<td>50</td>
<td>4.498153</td>
<td>1,000</td>
<td>6.482942</td>
</tr>
</tbody>
</table>

Note: \( n \) = number of cone resistance data points contained within the upper and lower bound lines (see Figure 3.3). For intermediate values of \( n \), the value of \( N_s \) may be obtained by linear interpolation.

For circular footings, the \( s_q \) and \( s_c \) equations should be multiplied by an additional term equal to \( 1 + 0.0025\phi_p \) and \( 1 + 0.002\phi_p \), respectively.

v. Estimate the depth factor \( d_q \) using (Lyamin et al., 2007):  
\[
d_q = 1 + (0.0036\phi_p + 0.393) \left(\frac{D}{H}\right)^{-0.27} \quad (Eq. 3.46)
\]

vi. Calculate the bearing capacity factors \( N_q \) and \( N_c \) using (Loukidis & Salgado, 2011; Reissner, 1924):  
\[
N_q = \frac{1 + \sin \phi_p}{1 - \sin \phi_p} e^{\tan \phi_p} \quad (Eq. 3.47)
\]

\[
N_c = (N_q - 0.6) \tan(1.33\phi_p) \quad (Eq. 3.48)
\]

vii. Compute the limit unit bearing capacity \( q_{UL} \) of the footing using (Lyamin et al., 2007):  
\[
q_{UL} = (s_q d_q q_0 N_q + 0.5(s_c d_c) q_0 BN_c) \quad (Eq. 3.49)
\]

where \( q_0 \) = surcharge (vertical effective stress) at the footing base level, and \( d_c = \) depth factor \((\approx 1)\). For strip footings, the shape factors \( s_q \) and \( s_c \) are equal to 1. Note that additional factors would have to be added to the bearing capacity equation (Eq. 3.49) to account for load inclination, footing base inclination, and ground inclination, as needed.
b. Limit unit bearing capacity of footings in “clay.” Execute the following substeps for footings in “clay,” otherwise proceed to step 3.

i. Determine the undrained shear strength $s_u$ profile below the footing base from CPT results using (Foye et al., 2006a,b; Salgado, 2008):

$$s_u(z) = \frac{q_c,\text{CAM}(z) - \sigma_v(z)}{N_k} \quad \text{(Eq. 3.50)}$$

where $q_c,\text{CAM}(z)$ is conservatively assessed mean (CAM) cone resistance (as a function of depth $z$) corrected for pore water pressure $u_z$, $\sigma_v(z)$ is in situ vertical total stress (as a function of depth $z$), and $N_k$ is cone factor ($= 9–15$ as long as the CPT is performed at a penetration rate that is sufficiently high to ensure undrained penetration (refer to Appendix D); soft NC clays tend to have $N_k$ values near the low end of this range; while stiff OC clays tend to have $N_k$ values near the high end of this range).

ii. Using Eq. 3.50, determine the strength gradient $\rho$ with depth and the undrained shear strength $s_{u0}$ at the footing base level.

iii. Determine the correction factor $F$ from Figure 3.4 based on whether the $s_u$ profile below the footing base resembles profile 1 or profile 2. Profile 1 represents an NC clay deposit with $s_u$ increasing linearly with depth from a nonzero value $s_{u0}$ at the footing base level. Profile 2 represents an NC clay deposit below a certain depth, with the footing base resting on an OC crust for which $s_u$ is constant with depth; $z_f$ is depth measured from the footing base.

iv. Estimate the shape factor $s_{su}$ and depth factor $d_{su}$ using (Salgado, 2008; Salgado et al., 2004):

$$s_{su} = 1 + C_1 \frac{B}{L} \exp \left[ \frac{2.3}{0.353 \left( \frac{\rho B}{s_{u0}} \right)^{0.599}} - 1.3 \right] + C_2 \sqrt{\frac{D}{B}} \quad \text{(Eq. 3.51)}$$

$$d_{su} = 1 + 0.27 \sqrt{\frac{D}{B}} \quad \text{(Eq. 3.52)}$$

where $B = $ footing width, $L = $ footing length, and $C_1$ and $C_2$ are coefficients that depend on the aspect ratio $B/L$ of the footing (Table 3.4).

v. Compute the limit unit bearing capacity $q_{bl}$ of the footing using (Salgado, 2008):

$$q_{bl} = F_{s,eff}d_{su} \left[ 1 + \frac{\rho B}{4s_{u0}N_c} \right] s_{u0}N_c + q_0 \quad \text{(Eq. 3.53)}$$

where $N_c = $ bearing capacity factor ($= 2 + \pi = 5.14$) (Prandtl, 1920, 1921), and $q_0$ is surcharge (vertical total stress) at the footing base level.

3.3 Load and Resistance Factor Design Procedure for Footings

Load and resistance factor design (LRFD) of axially-loaded footings can be done from CPT results by following these steps.

Step 1: Obtain the nominal dead load $DL_n$ and the nominal live load $LL_n$ on the footing from the superstructure design.

Step 2: Set the load factors for dead load and live load, $LF_{DL}$ and $LF_{LL}$, as 1.25 and 1.75, respectively (AASHTO, 2020). These load factors correspond to the Strength I limit state (basic load combination relating to the normal vehicular use of the bridge without wind), as defined by AASHTO (2020). The discussion of other limit states, such as Strength II–V, Extreme Event I and II, Service I–IV, and Fatigue I and II are beyond the scope of the manual—information about these limit states can be found in AASHTO (2020).

Step 3: Calculate the nominal resistance $R_n$ of the footing using:

$$R_n = q_{bl,\text{net}}A \quad \text{(Eq. 3.54)}$$

where $q_{bl,\text{net}} = $ net limit unit bearing capacity of the footing $(= q_{bl} - q_0)$, $q_{bl}$ is limit unit bearing capacity of the footing (obtained from Section 3.2), $q_0$ is surcharge at the footing base level, and $A = $ area of the footing base.

Step 4: Obtain the resistance factor.

Table 3.5 summarizes the resistance factors for load and resistance factor design of footings using the bearing capacity equations (Eqs. 3.49 and 3.53) presented in this chapter, while Table 3.6 summarizes the resistance factors and footing design methods advocated by AASHTO (2020).

Step 5: Verify that the following LRFD inequality is satisfied (Foye et al., 2006b; Salgado, 2008):

$$(RF)R_n \geq LF_{DL}DL_n + LF_{LL}LL_n \quad \text{(Eq. 3.55)}$$

If Eq. 3.55 is satisfied, the footing design is satisfactory with respect to the ultimate limit state (i.e., classical bearing capacity failure). Repeat steps 3 to 5 to optimize the design if needed. However, if Eq. 3.55 is not satisfied, return to step 3 and revise the footing geometry.

Note: The following equation may be used, if needed, to obtain an equivalent factor of safety (FS) for the footing design produced using LRFD (Salgado, 2008):

$$FS = \frac{LF_{DL} + LF_{LL} \left( \frac{L/L_c}{D/L_c} \right)}{(L/L_c + 1)RF} \quad \text{(Eq. 3.56)}$$

where $b_R = $ bias factor $(= R/R_n)$, $R = $ mean resistance of the footing (calculated from $q_{bl}$ using the mean cone resistance profile (Figure 3.3)), and $R_n = $ nominal resistance of the footing (calculated from $q_{bl}$ using the conservatively assessed mean cone resistance $q_{c,\text{CAM}}$ obtained from Eq. 3.37). To obtain a quick estimate of the equivalent factor of safety, the value of the bias factor $b_R$ may be taken as 1.
3.4 Chapter Summary

In this chapter, detailed, step-by-step procedures for computing the total settlement \( w \) and limit unit bearing capacity \( q_{BL} \) of axially-loaded footings from CPT results in sand (silica sand) and clay were presented. Guidelines for footings installed in mixed or intermediate soils, such as sand-silt or sand-clay mixtures, were provided based on the concept of floating versus nonfloating soil fabric.

Methods for estimation of immediate settlement of footings in sand and clay require a representative value of the elastic modulus of the soil below the footing under drained and undrained conditions, respectively. For sands, the ratio of the elastic modulus to the cone resistance is a function of footing settlement level, footing size, and relative density. For clays, the elastic modulus is obtained through the small-strain shear modulus, which can be estimated either from the shear wave velocity (if SCPT results are available) or from the mean effective stress, plasticity index, and OCR.

The method for estimation of primary consolidation settlement of a footing in clay is basically a modification of that used to estimate the one-dimensional consolidation settlement caused by the application of an instantaneous uniform load extending to infinity horizontally; the modification accounts for the three-dimensional effects that arise due to the finite size of the footing. In this method, the main soil variables are initial void ratio, compression index, swelling index, and preconsolidation stress. If significant secondary consolidation is expected at the site, it should be considered together with primary consolidation.

The limit unit bearing capacity of a footing in clay is calculated assuming that the loads are applied rapidly compared to the drainage rate of clay and that the short term is the critical loading condition; therefore, loading takes place under undrained conditions. In contrast, the limit unit bearing capacity of a footing in sand is calculated assuming drained conditions. The main soil variable in the bearing capacity equation is the peak friction angle in the case of sand and the undrained shear strength in the case of clay. The undrained shear strength \( s_u \) can be estimated from CPT results through the cone factor \( N_k \), which typically ranges from 9–15 depending on soil type, stress state and history, and stress path (e.g., triaxial compression versus direct simple shear).

Load and resistance factor design (LRFD) procedures for footings in sand and clay were presented. The nominal resistance of the footing is calculated through a nominal value of cone resistance, which is defined as a conservatively assessed mean (CAM) value that is

Figure 3.4  \( F \) versus \( \rho B/s_{ad} \) for a rough footing base in clay.

<table>
<thead>
<tr>
<th>( B/L \times 10^{-3} )</th>
<th>( C_1 )</th>
<th>( C_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.15</td>
<td>0.163</td>
<td>0.210</td>
</tr>
<tr>
<td>0.50</td>
<td>0.25</td>
<td>0.125</td>
</tr>
<tr>
<td>0.55</td>
<td>0.45</td>
<td>0.125</td>
</tr>
<tr>
<td>0.70</td>
<td>0.65</td>
<td>0.125</td>
</tr>
<tr>
<td>0.85</td>
<td>0.75</td>
<td>0.125</td>
</tr>
</tbody>
</table>
TABLE 3.5
Resistance factors for footings ($D/B \leq 1$) in sand and clay (modified from Foye et al., 2006b)

<table>
<thead>
<tr>
<th>Footing Type</th>
<th>$RF \left[ \beta_T = 3.0 \left( p_{f,T} = 10^{-3} \right) \right]$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sand</td>
</tr>
<tr>
<td>Strip footing</td>
<td>0.25</td>
</tr>
<tr>
<td>Rectangular footing</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Note: $\beta_T =$ target reliability index and $p_{f,T} =$ target probability of failure (a value of $10^{-3}$ means that one in every 1,000 footings would fail). The resistance factors were developed by Foye et al. (2006b) using reliability analysis and they correspond to the CPT-based footing design methods covered in this chapter. The $RF$ values for rectangular footings may also be used for square and circular footings.

TABLE 3.6
Resistance factors for footings in sand and clay (AASHTO, 2020)

<table>
<thead>
<tr>
<th>Method/Soil/Condition</th>
<th>$RF$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theoretical method (Munfakh et al., 2001) for footings in clay</td>
<td>0.50</td>
</tr>
<tr>
<td>Theoretical method (Munfakh et al., 2001) for footings in sand using CPT</td>
<td>0.50</td>
</tr>
<tr>
<td>Semi-empirical methods (Meyerhof, 1956) for footings in sand and clay</td>
<td>0.45</td>
</tr>
<tr>
<td>Plate load test</td>
<td>0.55</td>
</tr>
</tbody>
</table>

Note: The resistance factors were developed using both reliability theory and calibration by fitting to working stress design (WSD) (Allen, 2005). In general, WSD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of about 0.55 to 0.45, respectively (AASHTO, 2020). According to AASHTO (2020), calibration by fitting to WSD controlled the selection of the resistance factor when limited statistical data were available.

exceeded by 80% of the measured $q_c$ data points. The value of $q_{c,CAM}$ depends on the standard deviation of $q_c$, which is estimated from the range of $q_c$ values (i.e., the difference between the maximum and minimum values of $q_c$) contained within the sample dataset. This difference is related to the number of standard deviations of $q_c$, which is a function of the sample size. When using LRFD, it is important to note that the resistance factors are always tied to the specific design methods and equations for which they were developed.

Finally, summary tables for the CPT-based footing design methods covered in this chapter have been prepared so that the methods can be easily referred to when needed. The design methods covered in this chapter are not mandatory for design in INDOT contracts, and other CPT-based methods, some of which are summarized in Table 3.7 to Table 3.10, may be used as deemed appropriate for the site and loading conditions under consideration.
Methods for estimation of footing settlement in sand

<table>
<thead>
<tr>
<th>Reference</th>
<th>Total Settlement ( w )</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lee &amp; Salgado (2002)</td>
<td>( w = C_1 C_2 \left( \frac{q_b - \sigma'<em>{vo}}{E_i} \right) \sum</em>{i=1}^{s} \frac{I_i \Delta z_i}{E_i} )</td>
<td>Schmertmann's method was modified by Lee and Salgado (2002) and Lee et al. (2008) based on results obtained from nonlinear finite element analyses and cavity expansion analyses (using the program CONPOINT) for isolated and strip footings ((B = 1–3 \text{ m})) on silica sand ((D_R = 30%–90%)). It accounts for the effects of aging and overconsolidation of sand on the estimation of a representative elastic modulus (from cone resistance) within the zone of influence of the footing. The method captures the nonlinearity of the footing load-settlement curve caused by the degradation of the elastic modulus of sand with increasing footing settlement level. It can be used to calculate from the cone resistance either the load on a given footing or the area of the footing for a given load corresponding to a user-defined tolerable settlement. The value of ( \lambda ) is equal to 0.38 for young NC silica sand, 0.53 for aged NC silica sand, and 0.91 for OC silica sand.</td>
</tr>
<tr>
<td>Lee et al. (2008)</td>
<td>( C_1 = 1 - 0.5 \left( \frac{\sigma'<em>{vo}}{q_b - \sigma'</em>{vo}} \right) z_f ) ( C_2 = 1 + 0.21 \log \left( \frac{I}{0.1 I_R} \right) )</td>
<td></td>
</tr>
<tr>
<td>Schmertmann (1970)</td>
<td>( I_0 = \left{ \begin{array}{ll} I_B + 2 &amp; \text{if } z_f &lt; z_B \ z_f - 2 I_B &amp; \text{if } z_f \leq z_B \leq z_0 \end{array} \right. )</td>
<td>The equation, originally proposed by Schmertmann et al. (1978), has been rewritten by AASHTO (2020) in a way that requires specific units for certain variables: ( z ) in ft; ( q_b ) in ksi; and ( q_c ) and ( \sigma'_{vo} ) in ksf. The parameters ( z_f, I_0, z_0 ) and ( I_B ) can be determined from the strain influence diagrams provided in either Schmertmann et al. (1978), Salgado (2008), or AASHTO (2020).</td>
</tr>
<tr>
<td>Schmertmann et al. (1978)</td>
<td>( z_f = 2.0 + 0.4 \left[ \min \left( \frac{I}{I_R}, 6 \right) - 1 \right] ; \frac{2a}{B} = 0.5 + 0.1 \left[ \min \left( \frac{I}{I_R}, 6 \right) - 1 \right] )</td>
<td></td>
</tr>
<tr>
<td>AASHTO (2020)</td>
<td>( E = 0.028q_b )</td>
<td></td>
</tr>
<tr>
<td>Schmertmann et al. (1978)</td>
<td>( X = 1.25 ) for ( L/B = 1 ), 1.75 for ( L/B \geq 10 ), and a linearly-interpolated value for ( L/B ) between 1 and 10. The equations for ( C_5 ), ( C_2 ), and ( I_B ) are the same as in the method above. The method was developed by fitting an equation to a database of footing load test results ((122 \text{ footings on noncalcareous sands})) after normalizing the unit load and settlement of the footing with respect to cone resistance and footing size, respectively. The equation is applicable for ( L/B = 1–23 ), ( D/B = 0–2.2 ), and ( q_c = 0.9–21.6 ) MPa. ( (L/B)^{0.345} ) is an influence factor for rectangular footings based on the elasticity theory solution by Giroud (1968).</td>
<td></td>
</tr>
<tr>
<td>Mayne et al. (2012)</td>
<td>( w = C_1 C_2 \left( \frac{q_b - \sigma'<em>{vo}}{E_i} \right) \sum</em>{i=1}^{s} \frac{I_i \Delta z_i}{E_i} )</td>
<td>Mayne &amp; Dasenbrock (2018)</td>
</tr>
<tr>
<td>Mayne &amp; Dasenbrock (2018)</td>
<td>( w = C_1 C_2 \left( \frac{q_b - \sigma'<em>{vo}}{E_i} \right) \sum</em>{i=1}^{s} \frac{I_i \Delta z_i}{E_i} )</td>
<td>MnDOT (Dagger et al., 2018)</td>
</tr>
<tr>
<td>MnDOT (Dagger et al., 2018)</td>
<td>( w = B \left[ \frac{1}{h_s q_c q_{\text{act}}} \left( \frac{L}{B} \right)^{0.345} \right]^2 )</td>
<td></td>
</tr>
</tbody>
</table>

For silts that experience drained loading with no excess pore water pressures being developed, \( h_s = 0.58 \) for clean sand and \( q_{\text{act}} = q_t - \sigma'_{vo} \) where \( q_t = q_s + (1 - a)q_c \). \( q_{\text{act}} \) is an average net cone resistance measured over a vertical distance of \( 1.5B \) below the footing base. For clean sand, the correction of \( q_r \) to \( q_t \) is negligible and since overburden stresses are small, particularly for shallow foundations, \( q_{\text{act}} = q_t = q_c \). For mixed or intermediate soils, \( h_s \) can be determined from the soil behavior type index \( I_c \) as illustrated in Dagger et al. (2018). |
### TABLE 3.7 (Continued)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Total Settlement $w$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gavin et al. (2009)</td>
<td>$w = w_l + w_c$ where $w_l = q_Bk$ for the linear stage ($0 \leq w_l/B \leq w_c/B$), and $w_c = B\left[\frac{q_b}{\left(\frac{B}{k}\right)^{1-n}}\right]^2$ for the nonlinear stage ($w_l/B &lt; w_c/B &lt; 0.03$)</td>
<td>The method approximates the shape of the footing load-settlement curve by an initial linear component (with no modulus degradation) followed by a nonlinear (parabolic) component up to $w_l/B = 10%$. The value of the exponent $n$ was determined by equating the value of $q_b$ obtained from the equation for the nonlinear stage ($w_l/B = 10%$) with a value of $0.2q_{cb}$. The effect of creep is modeled using: $w_c = m\ln\left(\frac{t}{t_R}\right)$; where $m = 0.02\left(\frac{q_{cb}}{q_{c0}}\right)^2$ $w_c = \text{creep component of settlement}$ $m = \text{creep coefficient}$ $t = \text{time elapsed since the application of the load increment}$ $t_R = \text{reference time corresponding to the onset of creep settlement (in the same unit as } t)$ $q_{cb,ult} = \text{value of } q_b \text{ at } w_l/B = 10% = 0.2q_{cb}$</td>
</tr>
<tr>
<td>Lehane (2019)</td>
<td>$q_b = 0.05q_{cb}$ for a short-term relative settlement $w/B$ of $1%$</td>
<td>The equations are based on centrifuge and field load test results of footings in sand. Short-term and long-term (creep) settlement refer to the settlement observed about 1 day and 30 years, respectively, after the application of the load to the footing.</td>
</tr>
<tr>
<td>Liu &amp; Lehane (2021)</td>
<td>$q_b = 0.04q_{cb}$ for a long-term relative settlement $w/B$ of $1%$ $q_{cb}$ is the average cone resistance over the depth of influence $z_\theta$ below the footing base, which is given by: $z_\theta = (B/L_R)^{0.75}$</td>
<td></td>
</tr>
</tbody>
</table>

Note: $C_1$ and $C_2$ = depth and time factors, respectively; $q_b$ = unit load on the footing base; $q'_{\phi}|_{z = 0} = \text{in situ vertical effective stress at the footing base level}$; $I_e = \text{strain influence factor}$; $\Delta z$ = thickness of sublayer; $E$ = elastic modulus; $n$ = number of sublayers; $t_h$ = reference time; $t_s$ = service life of the superstructure (in the same unit as $t_h$); $z_\theta = \text{influence depth measured from the footing base}$; $z_f = \text{vertical distance from the footing base to the middle of the sublayer}$; $I_o = \text{strain influence factor at the footing base level}$; $I_p = \text{peak strain influence factor}$; $z_p = \text{depth measured from the footing base at which the strain influence factor peaks}$; $q'_{\phi}|_{z = z_f} = \text{in situ vertical effective stress at the depth corresponding to } z_f$; $q_c = \text{cone resistance}$; $a = \text{cone area ratio (} \sim 0.8 \text{ for typical CPT probes)}$; $u_2 = \text{pore water pressure measured at the shoulder position behind the cone face}$; $L_R = \text{reference length (} \sim 1 \text{ m or } 3.28 \text{ ft})$; $L = \text{footing length}$; $B = \text{width or diameter of the footing}$; $D_R = \text{relative density (expressed as a percentage)}$; $E_0 = \text{small-strain Young's modulus}[= 2G_{s}(1+\nu_0)]$; $G_0 = \text{small-strain shear modulus}$; $\nu_0 = \text{small-strain Poisson's ratio}$ (0.1–0.2); and $w_l/B = \text{normalized yield settlement level (} \sim 0.05\%)$. |
**TABLE 3.8**

Methods for estimation of bearing capacity of footings in sand

<table>
<thead>
<tr>
<th>Reference</th>
<th>Bearing Capacity</th>
<th>Notes</th>
</tr>
</thead>
</table>
| Bolton (1986) | \( q_{BL} = (s_d d_q) q_o N_c + 0.5(s_d d_q) Q_{BN} \) | The equations for the shape factors \( s_d \) and \( s_p \), and depth factor \( d_q \) are based on the results of rigorous lower- and upper-bound limit analyses of circular, rectangular, and strip footings in sand. For a strip footing \((B/L = 0)\) placed on the surface \((D/B = 0)\) of a sand deposit, the shape and depth factors reduce to a value of 1. For circular footings, the \( s_q \) and \( s_p \) equations should be multiplied by an additional term equal to 1 + 0.0025\( \phi_p \) and \( 1 + 0.002 \phi_p \), respectively.
| Lyamin et al. (2007) | \( q_s = 1 + 0.098 s_p - 1.64 \left( \frac{D}{B} \right) \left( \frac{B}{L} \right) \left( \frac{q_{c}}{c} \right) \) | The equation for the bearing capacity factor \( N_c \) fits almost perfectly the exact values of \( N_c \) obtained by Martin (2005) using the method of characteristics or slip-line method, even for very low values of friction angle. The parameter \( A_p \) is equal to 3 for triaxial conditions \((e.g.,\) square and circular footings with \( L/B = 1 \)) and 5 for plane-strain conditions \((e.g.,\) elongated rectangular footings with \( L/B \geq 7 \) and strip footings). For rectangular footings with \( 1 < L/B < 7 \), \( A_p \) is interpolated between 3 and 5 using \( A_p = \frac{1}{3} \left( \frac{L}{B} + 8 \right) \).
| Loukidis & Salgado (2011) | \( q_s = 1 + 0.033 s_p - 0.939 \left( \frac{D}{B} \right) \left( \frac{B}{L} \right) \left( \frac{q_{c}}{c} \right) \) | The parameter \( A_p \) is equal to 3 for triaxial conditions \((e.g.,\) square and circular footings with \( L/B = 1 \)) and 5 for plane-strain conditions \((e.g.,\) elongated rectangular footings with \( L/B \geq 7 \) and strip footings). For rectangular footings with \( 1 < L/B < 7 \), \( A_p \) is interpolated between 3 and 5 using \( A_p = \frac{1}{3} \left( \frac{L}{B} + 8 \right) \).
| Salgado (2008) | \( q_s = 1 + 0.098 s_p - 1.64 \left( \frac{D}{B} \right) \left( \frac{B}{L} \right) \left( \frac{q_{c}}{c} \right) \) | The equation for the bearing capacity factor \( N_c \) fits almost perfectly the exact values of \( N_c \) obtained by Martin (2005) using the method of characteristics or slip-line method, even for very low values of friction angle. The parameter \( A_p \) is equal to 3 for triaxial conditions \((e.g.,\) square and circular footings with \( L/B = 1 \)) and 5 for plane-strain conditions \((e.g.,\) elongated rectangular footings with \( L/B \geq 7 \) and strip footings). For rectangular footings with \( 1 < L/B < 7 \), \( A_p \) is interpolated between 3 and 5 using \( A_p = \frac{1}{3} \left( \frac{L}{B} + 8 \right) \).

**Meyerhof (1956)**

**AASHTO (2020)**

\( q_{BL} = \frac{q_{oL} B}{40} \left( C_{wq} D + C_{vq} \right) \)

\( q_{oL} \) is an average cone resistance measured over a vertical distance of \( B \) below the footing base.

\( q_{oL} = \frac{h_i \Delta q_B \left( \frac{L}{B} \right)^{0.5} \left( \frac{D}{B} \right)^{-0.345}}{5} \)

\( h_i = 0.58 \) and \((w/B)_{\text{max}} = 12\%\) for clean sand

\( q_{i,\text{net}} = q_i - \sigma_{i,v} \) where \( q_i = q_{i,\text{net}} + (1 - \rho_i) \)

\( q_{i,\text{net}} \) is an average net cone resistance measured over a vertical distance of 1.5\( B \) below the footing base. For clean sand, the correction of \( q_i \) to \( q_{i,\text{net}} \) is negligible and since overburden stresses are small, particularly for shallow foundations, \( q_{i,\text{net}} = q_i = q_c \).

This is an empirical method with \( B \) and \( D \) in the units of ft and \( q_{oL} \) in ksf.

**Mayne & Woeller (2014)**

**Mayne & Dasenbrock (2018)**

**MnDOT (Dagger et al., 2018)**

\( q_{i,\text{net}} = q_i - \sigma_{i,v} \) where \( q_i = q_{i,\text{net}} + (1 - \rho_i) \)

\( q_{i,\text{net}} \) is an average net cone resistance measured over a vertical distance of 1.5\( B \) below the footing base. For clean sand, the correction of \( q_i \) to \( q_{i,\text{net}} \) is negligible and since overburden stresses are small, particularly for shallow foundations, \( q_{i,\text{net}} = q_i = q_c \).

The method was developed by fitting an equation to a database of footing load test results \((31\) footings in \(13\) silica sands and \(11\) footings in \(4\) silt deposits) after normalizing the unit load and settlement of the footing with respect to cone resistance and footing size, respectively.

The footing geometries consisted of 29 square, 7 rectangular \((\text{nearly square})\), and 6 circular footings with \( B = 0.5-6.1\) m and \( D = 0-2.35\) m.

The sands were of different age \((\text{recent}, \text{Holocene}, \text{Pleistocene})\) and geologic origin \((\text{alluvial}, \text{marine}, \text{glaciofluvial}, \text{deltic}, \text{aeolian}, \text{and residual})\) with \( D_{50} = 0.1-0.4\) mm and \( q_o = 0.9-10.7\) MPa.

For silts that experience drained loading with no excess pore water pressures being developed, \( h_i = 1.12 \) and \((w/B)_{\text{max}} = 10\%\). For mixed or intermediate soils, \( h_i \) can be determined from the soil behavior type index \( I_s \) and \((w/B)_{\text{max}} \) can be interpolated based on the value of \( h_i \).

(Continued)
TABLE 3.8  
(Continued)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Bearing Capacity</th>
<th>Notes</th>
</tr>
</thead>
</table>
| Lehane (2019)              | $q_b,ult = 3q_c,b$  
$\alpha = 0.16$ for 10% relative settlement  
$q_a$ is the average cone resistance over the depth of influence $z_{fp}$ below the footing base, which is given by: $z_{fp}/L_R = (B/L_R)^{0.7}$ | Lehane (2013) compiled a database of load test results for 47 footings in sand with $B = 0.25–3$ m, $D = 0.1–1.6$ m, and $q_s = 3.5–14.5$ MPa. The equation for $q_b,ult$ predicts 80% of the footing load test results to within 25% of the measured $q_b,ult$ values at 10% relative settlement. Liu and Lehane (2021) proposed the following equation to obtain lower-bound estimates of $q_b,ult$ corresponding to a long-term (creep) $w/B$ ratio of 10%: $q_b,ult = 0.1q_c,b (long term)$  
$\alpha$ was determined based on the observed load-settlement response ($q_b/q_c,b$ versus $w/B$) of model and full-scale, square footings in sand. $q_c,b$ is the average cone resistance over the depth of influence $z_{fp}$ below the footing base, which is given by: $z_{fp}/L_R = (B/L_R)^{0.75}$ |
| Gavin et al. (2009)         | $q_b,ult = 3q_c,b$  
$\alpha = 0.2$ for 10% relative settlement  
$q_a$ is the average cone resistance over the depth of influence $z_{fp}$ below the footing base, which is given by: $z_{fp}/L_R = (B/L_R)^{0.75}$ | The value of $\alpha$ was determined based on the observed load-settlement response ($q_b/q_c,b$ versus $w/B$) of model and full-scale, square footings in sand. |
| Lee & Salgado (2005)        | $q_b,ult = 3q_c,b$  
$q_a$ is an average cone resistance measured over a vertical distance of $B$ below the footing base. The value of $\alpha$ for 20% relative settlement can be obtained from the table provided by Lee and Salgado (2005) as a function of $D_f$, $K_\theta$, and $B$. | The method was developed based on results obtained from nonlinear finite element analyses and cavity expansion analyses (using the program CONPOINT) for circular footings ($B = 1–3$ m) on Ottawa sand ($D_f = 30%–90%$). The equation is also applicable for square footings so long as an equivalent area is considered. However, for footing shapes other than circular or square, introduction of shape factors would be required. |

Note: $q_{db} =$ limit unit bearing capacity (i.e., the unit load at which the footing plunges into the ground), $q_s =$ surcharge (vertical effective stress) at the footing base level, $\gamma =$ unit weight of soil below the footing base, $B =$ footing width, $s_f$ and $s_c =$ shape factors, $d_q$ and $d_c =$ depth factors, $N_q$ and $N_c =$ bearing capacity factors, $z_w =$ depth of the groundwater table, $\gamma_m =$ moist unit weight of soil, $\gamma_b =$ buoyant unit weight of soil, $\gamma_p =$ peak friction angle, $L =$ footing length, $D =$ depth of embedment of the footing, $\phi_c =$ critical-state friction angle, $D_f =$ relative density, $Q$ and $R_Q =$ fitting parameters that depend on the intrinsic characteristics of sand ($Q = 10$ and $R_Q = 1$ for clean silica sand), $q_c, CAM =$ conservatively assessed mean (CAM) cone resistance at a depth of $B/2$ below the footing base (Eq. 3.37), $\sigma'_{ho} =$ in situ vertical effective stress at a depth of $B/2$ below the footing base, $\sigma'_{ho} =$ in situ horizontal effective stress at a depth of $B/2$ below the footing base (= $K_\theta\sigma'_{ho}$), $K_\theta =$ coefficient of lateral earth pressure at-rest, $P_A =$ reference stress (= 100 kPa or 14.5 psi), $q_{db,ult} =$ ultimate unit bearing capacity (mobilized at a given relative settlement $w/B$), $w =$ footing settlement, $a =$ cone area ratio (= 0.8 for typical CPT probes), $u_2 =$ pore water pressure measured at the shoulder position behind the cone face, and $L_R =$ reference length (= 1 m or 3.28 ft).
### TABLE 3.9
Methods for estimation of footing settlement in clay (Foye et al., 2008; Salgado, 2008; Skempton & Bjerrum, 1957)

<table>
<thead>
<tr>
<th>Immediate Settlement ( w_i )</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_i = I_q \frac{q_{b,net} B}{E_0} )</td>
<td>The method was developed based on results obtained from nonlinear finite element analyses of strip, square, and rectangular footings on clay. The influence factor ( I_q ) is determined from Figure 3.2 as a function of ( H/B, \frac{q_{b,net}}{s_y} ), and footing geometry; where ( q_{b,net} = q_b - \gamma_{s} D ) and ( s_y ) = average undrained shear strength over a vertical distance of ( B ) below the footing base. The undrained shear strength profile can be obtained from CPT results using ( s_y = \frac{(q_v - \sigma_{vc})/N_c}{V} ); where ( q_v = q_v + (1 - a)k D ) and ( N_c ) = cone factor (= 9–15); soft NC clays tend to have ( N_c ) values near the low end of this range, while stiff OC clays tend to have ( N_c ) values near the high end of this range. The small-strain shear modulus ( G_0 ) is averaged over the influence depth ( z_{i0} ) below the footing base to obtain ( G_0 ), where ( G_0^{\text{avg}} ) = average small-strain shear modulus of layer ( i ), ( H_i ) = thickness of layer ( i ), and ( n ) = number of clay layers within the influence depth ( z_{i0} ) below the footing base. Parameters ( C_p, n_c, ) and ( m_c ) depend on the plasticity index ( P_c ) (Viggiani &amp; Atkinson, 1995): ( C_p = 37.9 \exp(-0.045 PI) ) for ( PI &gt; 5% ); ( n_c = 0.129 \ln(PI) + 0.4374 ) for ( PI &gt; 5% ); ( m_c = 0.0015 PI + 0.1863 ) for ( PI &gt; 5% ).</td>
</tr>
<tr>
<td>( E_0 = 2(1 + v)G_0 )</td>
<td></td>
</tr>
<tr>
<td>( G_0 = \sum \frac{G_0^{\text{avg}} H_i}{\sum H_i} )</td>
<td></td>
</tr>
<tr>
<td>(</td>
<td>\sigma_{a0} = \frac{1}{k+1} \left( \sigma_a + k \sigma_{a0} \right) )</td>
</tr>
<tr>
<td>(</td>
<td>w_{iD} = \sum \Delta \epsilon_{i,j} \Delta z_i )</td>
</tr>
<tr>
<td>(</td>
<td>\Delta \epsilon_{i,j} = \frac{C_i}{1 + \epsilon_0} \left[ \frac{C_1 \log \left( \frac{\sigma_{a0}}{\sigma_{a0}} \right) + C_2 \log \left( \frac{\sigma_{a0}}{\sigma_{a0}} \right)}{\sigma_{a0}} \right] )</td>
</tr>
<tr>
<td>(</td>
<td>\sigma_{a0} = \sigma_{a0} + \Delta \epsilon_{i,j} )</td>
</tr>
</tbody>
</table>

### Consolidation Settlement \( w_c \)

\( w_c = [4 + z(1 - A)] w_{iD} \); where \( w_{iD} = \sum \Delta \epsilon_{i,j} \Delta z_i \)

\[ \Delta \epsilon_{i,j} = \begin{cases} C_1 \log \left( \frac{\sigma_{a0}}{\sigma_{a0}} \right) & \text{for NC clay} \\ C_1 \log \left( \frac{\sigma_{a0}}{\sigma_{a0}} \right) + C_2 \log \left( \frac{\sigma_{a0}}{\sigma_{a0}} \right) & \text{for OC then NC clay} \\ C_1 \log \left( \frac{\sigma_{a0}}{\sigma_{a0}} \right) & \text{for OC clay} \end{cases} \]

\[ \sigma_i = \sigma_i + \Delta \sigma_i \]

**Notes:** \( I_q \) = influence factor; \( q_{b,net} = \) net unit load on the footing base; \( q_b = \) gross unit load on the footing base; \( H = \) thickness of the clay layer below the footing base; \( B = \) footing width; \( D = \) depth of embedment of the footing; \( \gamma_{s} = \) corrected, total core resistance measured under undrained conditions; \( q_v = \) core resistance; \( a = \) cone area ratio (= 0.8 for typical CPT probes); \( \psi_p = \) pore water pressure measured at the shoulder position behind the cone face; \( \sigma_{a0} = \text{in situ } \) vertical total stress at the depth being considered; \( E_0 = \) representative small-strain Young’s modulus; \( v = \) Poisson’s ratio (= 0.5 for undrained conditions); \( \gamma_{s} = \) unit weight of soil; \( g = \) acceleration due to gravity (9.81 m/s² or 32.17 ft/s²); \( Y_c = \) shear wave velocity (refer to Section 2.3.4 of Volume I); \( \sigma_{a0} = \) reference stress (= 100 kPa or 145 psi); \( \sigma_{a0} = \text{in situ } \) mean effective stress at the depth being considered; \( \sigma_i = \text{in situ } \) horizontal effective stress at the depth being considered (= \( K_0 \sigma_{a0} \)); \( K_0 = \) coefficient of lateral earth pressure at-rest; \( k = 1 \) for plane-strain conditions (e.g., strip footings) and 2 for triaxial conditions (e.g., isolated footings); \( R_0 = \) mean stress-based overconsolidation ratio; \( K_{0,NC} = \) coefficient of lateral earth pressure at-rest for normally consolidated soil (= 0.5–0.75 for NC clay); OCR = overconsolidation ratio; \( w_{iD} = \) one-dimensional consolidation settlement; \( A = \) Skempton’s pore pressure parameter (= 0.5–0.75 for NC clay and 0.3–0.5 for OC clay); \( \sigma_{i0} \) and \( \sigma_i = \text{initial (in situ) } \) and current vertical effective stresses, respectively, at the depth being considered; \( \Delta \sigma_i = \) vertical stress increment; \( \sigma_{i0} = \) preconsolidation stress; \( e_0 = \) initial void ratio; \( \Delta z = \) thickness of the sublayer; \( n = \) number of sublayers; and \( \Delta \epsilon_{i,j} = \) vertical compressive strain.
### TABLE 3.10
Methods for estimation of bearing capacity of footings in clay

<table>
<thead>
<tr>
<th>Reference</th>
<th>Bearing Capacity</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Davis &amp; Booker (1973)</td>
<td>( q_{bl} = F_{s,ud} \left( 1 + \frac{\rho B}{4s_{ud}/N_c} \right) s_{ud}/N_c + q_0 )</td>
<td>The equation for the shape factor ( s_{ud} ) is based on a least-squares fit to the values of ( s_{ud} ) obtained from the computer program ABC (Martin, 2004), which is based on the method of characteristics.</td>
</tr>
<tr>
<td>Salgado et al. (2004)</td>
<td>( s_{ud} = 1 + C_1 \frac{B}{L} \left( \exp \left[ 0.353 \left( \frac{\rho R}{s_{ud}} \right)^{0.59} \right] - 1.3 \right) + C_2 \sqrt{\frac{D}{B}} )</td>
<td>The equations for the depth factor ( d_{ud} ) is based on the results of rigorous lower- and upper-bound limit analyses of footings embedded in clay.</td>
</tr>
<tr>
<td>Salgado (2008)</td>
<td>( d_{ud} = 1 + 0.27 \sqrt{\frac{D}{B}} )</td>
<td>The equation for the depth factor ( d_{ud} ) is based on the results of rigorous lower- and upper-bound limit analyses of footings embedded in clay.</td>
</tr>
<tr>
<td>Mayne &amp; Woeller (2014)</td>
<td>( q_{b,all} = h_s q_{c,net} \left( \frac{w}{B} \right)<em>{\text{max}} \left( \frac{L}{B} \right)</em>{\text{max}}^{-0.345} )</td>
<td>The method was developed by fitting an equation to a database of footing load test results (12 footings in 6 intact clays and 11 footings in 5 fissured clays) after normalizing the unit load and settlement of the footing with respect to cone resistance and footing size, respectively. The footing geometries consisted of 13 square, 1 rectangular, and 9 circular footings with ( B = 0.4-5.0 ) m. For mixed or intermediate soils, ( h_s ) can be determined from the soil behavior type index ( I_s ) and ( (w/B)_{\text{max}} ) can be interpolated based on the value of ( h_s ).</td>
</tr>
<tr>
<td>MnDOT (Dagger et al., 2018)</td>
<td>( \begin{align*} h_s &amp;= 1.47 \text{ and } (w/B)<em>{\text{max}} = 7% \text{ for fissured clay} \ h_s &amp;= 2.70 \text{ and } (w/B)</em>{\text{max}} = 4% \text{ for intact clay} \ q_{c,net} &amp;= q_s - \sigma_0 ; \text{ where } q_s = q_c + (1 - a)u_2 \ q_{c,net} &amp;= \text{an average net cone resistance measured over a vertical distance of } 1.5B \text{ below the footing base} \end{align*} )</td>
<td>The method is based on load-settlement data ( (q_s/q_c, L/B) ) compiled from undrained footing load tests in 5 clays. ( q_{bd} ) is the average cone resistance (corrected for pore water pressure ( u_s )) over a vertical distance of ( B ) below the footing base.</td>
</tr>
<tr>
<td>Lehane (2019)</td>
<td>( q_{b,ul} \approx 0.45 q_{b,all} )</td>
<td>The method is based on load-settlement data ( (q_s/q_c, L/B) ) compiled from undrained footing load tests in 5 clays. ( q_{bd} ) is the average cone resistance (corrected for pore water pressure ( u_s )) over a vertical distance of ( B ) below the footing base.</td>
</tr>
</tbody>
</table>

Note: \( q_{bl} \) = limit unit bearing capacity (i.e., the unit load at which the footing plunges into the ground), \( F \) = correction factor (Figure 3.4), \( s_{ud} \) = shape factor, \( d_{ud} \) = depth factor, \( q_{c, CAM} \) = conservatively assessed mean (CAM) cone resistance (corrected for pore water pressure \( u_s \)), \( \rho \) = rate of increase of undrained shear strength \( s_u \) with depth, \( s_{ud} \) = undrained shear strength at the footing base level, \( N_c \) = bearing capacity factor (= 2 + \( \pi \approx 5.14 \)), \( q_0 \) = surcharge (vertical total stress) at the footing base level, \( B \) = footing width, \( L \) = footing length, \( C_1 \) and \( C_2 \) = coefficients that depend on the aspect ratio \( B/L \) of the footing (Table 3.4), \( q_{b,ul} \) = ultimate unit bearing capacity (mobilized at a given relative settlement \( w/B \)), \( q_{b,all} \) = allowable bearing pressure, \( q_{c,net} \) = corrected, total cone resistance measured under undrained conditions, \( q_c \) = cone resistance, \( a \) = cone area ratio (= 0.8 for typical CPT probes), \( u_2 \) = pore water pressure measured at the shoulder position behind the cone face, \( \sigma_0 \) = in situ vertical total stress at the depth being considered, and \( L_{ref} \) = reference length (= 1 m or 3.28 ft).
4. CPT-BASED DESIGN OF PILE FOUNDATIONS

Piles can be classified into three categories based on the changes caused to the state of in situ soil during their installation: (1) nondisplacement piles (e.g., drilled shafts), (2) partial-displacement piles (e.g., H-piles and open-ended pipe (OEP) piles), and (3) full-displacement piles (e.g., closed-ended pipe (CEP) piles). A pile derives its load-carrying capacity by two mechanisms: (a) shaft resistance, which is the friction or adhesion along the pile shaft with the surrounding soil, and (b) base resistance, which is the compressive resistance at the contact of the pile base with the underlying soil. Shaft resistance is fully mobilized for small pile head settlements (on the order of 0.25%–1% of the pile diameter), whereas complete mobilization of pile base resistance requires large pile head settlements (on the order of 15%–25% of the pile diameter) (Salgado, 2008).

4.1 Calculation Procedure for Limit Shaft Capacity of Single Piles

The limit shaft capacity $Q_{u,L}$ of a single, isolated, axially-loaded pile can be calculated from CPT results by following these steps.

**Step 1:** Obtain the site stratigraphy, the groundwater table depth, and the unit weight of the soil in each layer of the profile.

a. Establish the site stratigraphy either from the boring log or by using a CPT-based soil behavior type (SBT) chart (refer to Section 2.2.3 of Volume I) or both if possible.

b. Obtain the depth $z_w$ of the groundwater table from either the boring log or the depth profile of $u_2$ or both if possible, where $u_2 =$ pore water pressure measured at the shoulder position behind the cone face (refer to Volume I).

c. Obtain the unit weight of the soil in each layer of the profile whenever soil samples are recovered during the site investigation. In the absence of soil samples, the reader may refer to Section 2.3.3 of Volume I for correlations between the unit weight and CPT data. In general, the saturated unit weight $\gamma_{sat}$ of soil typically ranges from 18–21 kN/m$^3$ (115–135 pcf) for sand, 18.5–22.5 kN/m$^3$ (118–143 pcf) for silty sand, and 15–18 kN/m$^3$ (95–115 pcf) for clay (Salgado, 2008).

**Step 2:** Select the pile type and decide the pile length.

a. Set the pile type and the embedment length $L$ of the pile based on the soil profile at the site.

b. If a competent bearing layer, such as dense sand, stiff clay, or rock, exists at a reasonable depth from the ground surface, embed the pile base in the bearing layer to ensure that the contribution of that layer toward the base resistance can be realized.

**Step 3:** Classify the soil in each layer that is in contact with the pile as either “sand” or “clay.” For mixed or intermediate soils (i.e., soils containing mixtures of sand, silt, and clay), execute the following substeps.

a. Sand-silt, sand-clay or sand-silt-clay mixtures: Classify these soils as “clay” if fines content FC $\cong 20\%$ and plasticity index PI $\cong 8\%$, otherwise classify them as “sand” (Carraro et al., 2009; Salgado et al., 2000).

b. Sands containing gravel: If a site contains sand layers with gravel content greater than 20%, use the lower-bound profile of $q_c$ drawn approximately through the valleys of the actual $q_c$ profile, for estimating the pile capacity (Ganju et al., 2020; Han et al., 2019b, 2020).

**Note:** In the absence of soil samples, the reader may refer to Section 2.2 of Volume I for estimation of soil behavior type from CPT results.

**Step 4:** Correct the raw $q_c$ data for the pore water pressure generated during cone penetration using (ASTM, 2012):

$$q_t = q_c + (1-a)u_2$$

(Eq. 4.1)

where $q_t =$ corrected, total cone resistance, $q_c =$ measured cone resistance, $a =$ cone area ratio ($\approx 0.8$ for typical CPT probes), and $u_2 =$ pore water pressure measured at the shoulder position behind the cone face. The pore water pressure correction to the $q_c$ data may be ignored for coarse-grained soils (e.g., sand and gravel) because $q_t$ is approximately equal to $q_c$ in such soils.

**Step 5:** Using the cone resistance values obtained from step 4, divide the soil layers in contact with the pile shaft into sublayers, as shown in Figure 4.1. The sublayers should satisfy the following criteria.

a. The cone resistance values within each sublayer should be either approximately constant or linear with depth so that a representative cone resistance, indicated by the grey vertical bars in Figure 4.1, can be assigned to each sublayer.

b. The sublayer should consist of the same soil type, i.e., either “sand” or “clay.”

**Step 6:** Calculate the in situ vertical effective stress $\sigma_{v,0}$ at the middle of each sublayer using (Terzaghi, 1943):

$$\sigma_{v,0} = \sigma_{v,0} - u_0$$

(Eq. 4.2)

where $\sigma_{v,0} =$ in situ vertical total stress at the middle of the sublayer, $u_0 =$ hydrostatic pore water pressure at the middle of the sublayer $= \max\{\gamma_w(z - z_w); 0\}$, $\gamma_w =$ unit weight of water ($= 9.81$ kN/m$^3$ or 62.45 pcf), $z_w =$ depth measured from the ground surface to the middle of the sublayer, and $z_w =$ depth of the groundwater table.

**Step 7:** Calculate the limit unit shaft resistance of pile segments in contact with “sand” sublayers. Execute the following substeps if the sublayer is “sand,” otherwise proceed to step 8.

a. Calculate the in situ horizontal effective stress $\sigma_{h,0} (= K_0\sigma_{v,0})$ at the middle of the sublayer, where $K_0 =$ coefficient of lateral earth pressure at-rest (refer to Appendix B for guidance).

b. Determine the critical-state friction angle $\phi_c$ of the sublayer through one of the following options.
Select a $\phi_c$ value between 28° and 36° for silica sand; sands with rounded, smooth particles with a poorly-graded particle size distribution have values near the low end of this range, while sands with angular, rough particles with a well-graded particle size distribution have values near the high end of this range (refer to Appendix A for additional information if needed).

If the mean particle size $D_{50}$, coefficient of uniformity $C_U$, and particle roundness $R$ of the sublayer are known, estimate the critical-state friction angle using:

$$\phi_c^{(\xi)} = 28.3 \left( \frac{D_{50}}{D_{ref}} \right)^{\zeta} (C_U)^{2z} (R)^{-3}$$

(Eq. 4.3)

where $D_{ref}$ = reference particle size (= 1 mm or 0.04 in.), and $\zeta$ = exponent (= 0.045). Equation 4.3 is applicable for poorly-graded, clean silica sands with $D_{50}$ = 0.15–2.68 mm (0.006–0.105 in.), $C_U$ = 1.2–3.1, and $R$ = 0.3–0.8. The data used in the development of this equation along with example calculations can be found in Appendix A.

If direct shear or triaxial compression test results are available, it is recommended that the critical-state friction angle be determined from such test results.

c. Set the critical-state interface friction angle $\delta_c$ of the sublayer.

i. For precast concrete piles, set $\delta_c/\phi_c = 0.95$.

ii. For cast-in-place concrete piles, set $\delta_c/\phi_c = 1.00$.

iii. For steel piles, set $\delta_c/\phi_c = 0.80$–0.85. If the $D_{50}$ and $C_U$ values of the sand are known, obtain the value of $\delta_c/\phi_c$ from Figure 4.2.

d. H-piles in “sand”. Following the Imperial College pile design method (ICPDM) (Jardine et al., 2005), compute the limit unit shaft resistance $q_{uL}$ of the pile segment in contact with a sand sublayer using:

$$q_{uL} = (F_{load} \sigma_{r' +} + \Delta \sigma_{r'd}) \tan \delta_c$$

(Eq. 4.4)

where $F_{load}$ = factor that accounts for loading direction (= 0.8 for tension and 1.0 for compression), $\sigma_{r' +}$ = local radial effective stress acting on the pile segment after installation, and $\Delta \sigma_{r'd}$ = increase in local radial effective stress associated with constrained dilation during pile loading.
\[
\sigma'_m = 0.029 q_l \left( \frac{\sigma'_0}{p_A} \right)^{0.13} \left( \max \left( \frac{h}{A_p} \sqrt{\frac{2}{\pi}} \right) \right)^{-0.38} \quad (\text{Eq. 4.5})
\]

\[
\Delta' = 2q_l [0.0203 + 0.00125 \eta] - 1.216 \times 10^{-6} \eta^2 \quad (\text{Eq. 4.6})
\]

\[
\eta = \frac{p_A}{\sqrt{\frac{\sigma'_0}{p_A}}}
\]

where \( p_A \) is reference stress (100 kPa or 14.5 psi), \( h \) is vertical distance from the middle of the sublayer to the pile base, \( \Delta r \) is radial displacement of soil during pile loading (0.02 mm (0.8 mil) for lightly rusted steel piles), and \( A_p \) is area of the pile base (refer to Table 4.2).

d. Drilled shafts, CEP and OEP piles in “sand”: Following the Purdue pile design method (PPDM) (Han et al., 2017, 2019b), compute the limit unit shaft resistance \( q_{UL} \) of the pile segment in contact with a sand sublayer using:

\[
q_{UL} = \begin{cases} 
F_{load} K \sigma'_0 \tan \delta_s & \text{for CEP piles} \\
K \sigma'_0 \tan \delta_s & \text{for drilled shafts} \\
K (1 - 0.66 \text{PLR}) \sigma'_0 \tan \delta_s & \text{for OEP piles}
\end{cases}
\]

where \( F_{load} \) is factor that accounts for loading direction (0.5–0.6 for tension (Galvis-Castro et al., 2019) and 1.0 for compression), PLR = plug length ratio, and \( K = \) lateral earth pressure coefficient:

\[
K = \begin{cases} 
0.2 + \frac{0.01 q_l}{p_A} - 0.2 \exp \left( -0.14 h / L_R \right) & \text{for CEP and OEP piles} \\
0.67 K_0 \exp \left( 0.3 / \sqrt{K_0 - 0.4} \right) D_R / 100 \left. 1.5 - 0.35 \ln \left( \frac{\sigma'_0}{p_A} \right) \right|_{p_A} & \text{for drilled shafts}
\end{cases}
\]

where \( p_A \) is reference stress (100 kPa or 14.5 psi), \( h \) is vertical distance from the middle of the sublayer to the pile base, \( L_R = \) reference length (1 m or 3.28 ft), and \( D_R = \) relative density (expressed as a percentage):

\[
D_R(\%) = \frac{\ln \frac{q_l}{p_A} - 0.4947 - 0.1041 \phi_s - 0.841 \ln \left( \frac{\sigma'_0}{p_A} \right)}{0.0264 - 0.0002 \phi_s - 0.0047 \ln \left( \frac{\sigma'_0}{p_A} \right)}
\]

(Eq. 4.10)

For OEP piles, the plug length ratio (PLR) used in the equation for \( q_{UL} \) is that measured at the specific depth where \( q_{UL} \) is calculated. If the PLR is not measured, it can be approximated using the same equation (Eq. 4.29) provided for the incremental filling ratio (IFR).

**Step 8:** Calculate the limit unit shaft resistance of pile segments in contact with “clay” sublayers. Execute the following substeps if the sublayer is “clay,” otherwise proceed to step 9.

a. Select a \( \phi_s \) value between 15° and 30° for clay; high-plasticity clays with high smectite and clay contents tend to have values near the low end of this range, while low-plasticity clays with low smectite and clay contents tend to have values near the high end of this range (refer to Table E.1 of Appendix E). If laboratory shear test results (e.g., triaxial compression) are available, it is recommended that the critical-state friction angle be determined from such test results.

b. Select a \( \phi_{min} \) value between 5° and 15° for clay (refer to Appendix E for guidance). If ring shear test results are available, it is recommended that the minimum residual-state friction angle be determined from such test results.

c. CEP piles and drilled shafts in “clay” (PPDM):

i. Determine the undrained shear strength \( s_u \) of the sublayer from CPT results using (Salgado, 2008):

\[
s_u = \frac{q_l - \sigma'_0}{N_k}
\]

(Eq. 4.11)

where \( q_l = \) corrected, total cone resistance measured under undrained conditions, \( \sigma'_0 = \) in situ vertical total stress at the middle of the sublayer, and \( N_k = \) cone factor (= 9–15 as long as the CPT is performed at a penetration rate that is sufficiently high to ensure undrained penetration (refer to Appendix D); soft NC clays tend to have \( N_k \) values near the low end of this range, while stiff OC clays tend to have \( N_k \) values near the high end of this range) (Bisht et al., 2021; Mayne & Peuchen, 2018; Salgado, 2008, 2013, 2014; Salgado et al., 2004).

ii. Following the Purdue pile design method (PPDM) (Basu et al., 2009, 2014; Chakraborty et al., 2013), compute the limit unit shaft resistance \( q_{UL} \) of the pile segment in contact with a clay sublayer using:

\[
q_{UL} = 2 A_1
\]

(Eq. 4.12)

\[
x = \begin{cases} 
A_1 + (1 - A_1) \exp \left[ - \left( \frac{\sigma'_0}{p_A} \right) (\phi_s - \phi_{min}) \right] & \text{for CEP piles} \\
A_1 + (1 - A_1) \exp \left[ - \left( \frac{\sigma'_0}{p_A} \right) (\phi_s - \phi_{min}) \right] & \text{for drilled shafts}
\end{cases}
\]

(Eq. 4.13)

\[
A_1 = \begin{cases} 
0.75 & \text{for } \phi_s - \phi_{min} \leq 5 \degree \\
0.43 & \text{for } \phi_s - \phi_{min} \geq 12 \degree \\
0 & \text{for } \phi_s - \phi_{min} \geq 12 \degree
\end{cases}
\]

for CEP piles

\[
A_1 = \begin{cases} 
0.75 & \text{for } \phi_s - \phi_{min} \leq 5 \degree \\
0.40 & \text{for } \phi_s - \phi_{min} \geq 12 \degree
\end{cases}
\]

for drilled shafts

(Eq. 4.14)

\[
A_2 = \frac{0.55 + 0.43 \ln \left( \frac{s_u}{\sigma'_0} \right)}{0.55 + 0.30 \ln \left( \frac{s_u}{\sigma'_0} \right)}
\]

(Eq. 4.15)

where \( p_A \) is reference stress (100 kPa or 14.5 psi). For \( 5 \degree < \phi_s - \phi_{min} < 12 \degree \), obtain the value of \( A_1 \) by interpolation.
d. OEP piles and H-piles in “clay” (ICPDM).

i. Obtain the overconsolidation ratio (OCR) of the sublayer (refer to Appendix B and Section 2.3.7 of Volume I for guidance). If laboratory consolidation test results (e.g., oedometer test or constant rate of strain (CRS) test) are available, it is recommended that the OCR be determined from such test results.

ii. Estimate the sensitivity \( S_i \) of the sublayer using:

\[
S_i = \frac{\sigma_{uw}}{\sigma_{dr}}
\]  

(4.16)

where \( \sigma_{uw} \) = “undisturbed” or \textit{in situ} undrained shear strength of the sublayer (refer to step (8(c)(ii))). The remolded undrained shear strength \( \sigma_{dr} \) of the sublayer may be estimated using the following approximate correlation (Wroth, 1979): 

\[
\frac{\sigma_{dr}}{p_A} \approx 0.017 \times 10^{2(1 - Li)}
\]  

(4.17)

where \( p_A \) = reference stress (= 100 kPa or 14.5 psi), \( Li \) = liquidity index (= (wc – PL)/PI), wc = water content, PI = plasticity index (= LL – PL), LL = liquid limit, and PL = plastic limit. In the absence of soil samples, the reader may refer to Sections 2.3.10.5 and 2.3.10.6 of Volume I for additional information on \( \sigma_{dr} \) and \( S_i \), respectively.

iii. Estimate the lateral earth pressure coefficient \( K \) of the sublayer (Jardine et al., 2005):

\[ K = [2.2 + 0.016OCR - 0.87 \log S_i] \]  

OCR\( ^{0.42} \left( \max \left[ \frac{h}{R} \right] ^{0.20} \right) \]  

(4.18)

Step 9: Repeat steps 7 and 8 to obtain the limit unit shaft resistance \( q_{UL} \) for each “sand” and “clay” sublayer in contact with the pile shaft.

Step 10: Compute the limit shaft capacity \( Q_{UL} \) of the pile using:

\[ Q_{UL} = \sum_{i=1}^{n} q_{UL,i}A_{si} \]  

(4.23)

where \( A_{si} \) = pile shaft area interfacing with sublayer \( i \) (Table 4.1), and \( n \) = number of sublayers in contact with the pile shaft.

### TABLE 4.1 Expressions for \( A_{si} \) for different pile cross-sections

<table>
<thead>
<tr>
<th>Pile Cross-Section</th>
<th>Pile Shaft Area ( A_{si} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circle</td>
<td>( \pi B^2/4 )</td>
</tr>
<tr>
<td>Square</td>
<td>( B^2 )</td>
</tr>
<tr>
<td>Rectangle</td>
<td>( B_w B_l )</td>
</tr>
<tr>
<td>H-section(^1)</td>
<td>( 2(b_f + d) \tau_f ) + ( 2X_p + \tau_f ) (( d - 2\tau_f )) for H-piles, ( \tau_f ) = thickness of flange; and ( \tau_w ) = thickness of web. (^1)For H-piles, ( X_p = b_f/8 ) if ( b_f/2 &lt; (d - 2\tau_f) &lt; b_f ) and ( X_p = b_f^2/16(d - 2\tau_f) ) if ( (d - 2\tau_f) \geq b_f ) (De Beer et al., 1980; Jardine et al., 2005).</td>
</tr>
</tbody>
</table>

Note: \( B = \) pile diameter (or width in the case of a square pile); \( B_w \) and \( B_l \) = width and length, respectively, of the cross-section of a rectangular pile (in plan); \( b_f = \) width of flange; \( d = \) depth of H-section; and \( \Delta X_i = \) thickness of sublayer \( i \).
4.2 Calculation Procedure for Ultimate Base Capacity of Single Piles

The ultimate base capacity \( Q_{b,ult} \) of a single, isolated, axially-loaded pile can be calculated from CPT results by following these steps.

**Step 1:** Estimate the average cone resistance \( q_{cb} \) at the pile base.

a. Execute the following substeps, depending on the pile design method, to estimate the average cone resistance \( q_{cb} \) at the pile base.

i. For the Purdue pile design method (PPDM), calculate the value of \( q_{cb} \) by averaging the cone resistance over a vertical distance within 1\( B \) above and 2\( B \) below the pile base.

ii. For the Imperial College pile design method (ICPDM), calculate the value of \( q_{cb} \) by averaging the cone resistance over a vertical distance within 1.5\( B \) above and 1.5\( B \) below the pile base.

*Note:* If the soil within the averaging zone is clay, use the corrected, total cone resistance \( q_t \) (Eq. 4.1), instead of \( q_c \).

b. If the pile base is embedded in a competent (strong) but thin layer (e.g., dense sand or stiff clay) below which there happens to be a weak layer (e.g., loose sand or soft clay), then execute the following substeps to estimate the average cone resistance \( q_{cb} \) at the pile base.

i. From the cone resistance profile, determine the representative cone resistances, \( q_{cw} \) and \( q_{cs} \), of the weak and strong layers, respectively.

ii. Estimate the sensing distance \( H_s \) using (Xu, 2007; Xu & Lehane, 2008):

\[
H_s = 1.41 - 2.52 \ln \left( \frac{q_{cw}}{q_{cs}} \right) \quad \text{(Eq. 4.24)}
\]

The sensing distance is the vertical distance from the layer interface at which the cone resistance first starts changing as the cone moves toward it (Salgado, 2014; Tehrani et al., 2018).

iii. Determine the vertical distance \( H \) from the pile base to the interface between the strong and weak layers.

iv. If \( H \leq H_s \), calculate the value of \( q_{cb} \) using the following equations (Xu & Lehane, 2008):

\[
q_{cb} = q_{cw} + \left( 1 - \frac{q_{cw}}{q_{cs}} \right) \exp \left\{ - \exp \left[ A_1 + A_2 \left( \frac{H}{B} \right) \right] \right\} \quad \text{(Eq. 4.25)}
\]

\[
A_1 = \min \left[ -0.22 \ln \left( \frac{q_{cw}}{q_{cs}} \right) + 0.11; 1.5 \right] \quad \text{(Eq. 4.26)}
\]

\[
A_2 = \min \left[ -0.11 \ln \left( \frac{q_{cw}}{q_{cs}} \right) - 0.79; -0.2 \right] \quad \text{(Eq. 4.27)}
\]

However, if \( H > H_s \), the base resistance of the pile will not be affected much by the presence of the underlying weak layer (Xu, 2007); therefore, we can calculate the value of \( q_{cb} \) from step 1(a). Note that piles should be sufficiently embedded in a strong, competent layer, whenever possible, to avoid serviceability issues.

**Step 2:** Calculate the ultimate unit base resistance \( q_{b,ult} \) of the pile.

a. For piles bearing in “sand,” calculate the ultimate unit base resistance \( q_{b,ult} \) of the pile using (Han et al., 2017, 2019b; Jardine et al., 2005; Lehane et al., 2005):

\[
q_{b,ult} = \begin{cases} 
q_b & \text{for H-piles (ICPDM)} \\
(1 - 0.0058D_b)q_b & \text{for CEP piles (PPDM)} 
\end{cases}
\]

\[
d_b = \left\{ 62.9 \left( \frac{D_b}{100} \right)^{1.83} \left( \frac{\sigma_{ap}}{p_a} \right)^{0.4} \right\} \quad \text{for drilled shafts (PPDM)}
\]

\[
\min \left[ 0.21 \left( \frac{I_{FR}}{I_{FR}} \right)^{-1.2}; q_b; 0.6q_b \right] \quad \text{for OEP piles (PPDM)}
\]

\[
\text{Eq. (4.28)}
\]

\[
\text{IFR} \approx \min \left[ 1; \left( \frac{B_i}{1.5L_R} \right)^{0.2} \right] \quad \text{Eq. (4.29)}
\]

where IFR = incremental filling ratio, \( B_i \) = inner diameter of OEP pile, and \( L_R \) = reference length (= 1 m or 39.4 in.). Equation 4.29 can be used to estimate the IFR if plug length measurements are unavailable, but if they are available, then average the IFR over the last 3\( B \) of pile driving. The relative density \( D_b \) of the bearing layer can be estimated from CPT results using (Salgado & Prezzi, 2007):

\[
D_b(\%) = -0.4947 - 0.1041\phi_v - 0.841\ln \left( \frac{\sigma_{ap}}{p_a} \right) - 0.0264 - 0.0002\phi_v - 0.0047\ln \left( \frac{\sigma_{ap}}{p_a} \right)
\]

\[
\text{Eq. (4.30)}
\]

where \( \sigma_{ap} = \text{in situ horizontal effective stress (} = K_0\sigma_{v0} \) at a depth of \( L + (B/2) \), \( \sigma_v = \text{in situ vertical effective stress at a depth of } L + (B/2), p_a = \text{reference stress (} = 100 \text{kPa or } 14.5 \text{ psi}), \phi_v = \text{critical-state friction angle (refer to step 7(b) of Section 4.1), and } K_0 = \text{coefficient of lateral earth pressure at-rest (refer to Appendix B for guidance).}

b. For piles bearing in “clay,” calculate the ultimate unit base resistance \( q_{b,ult} \) of the pile using (Jardine et al., 2005; Salgado, 2006, 2008):

\[
q_{b,ult} = \begin{cases} 
q_{cb} & \text{for H-piles (ICPDM)} \\
10s_u & \text{for CEP piles (PPDM)} \\
cos q_{cb} & \text{for OEP piles (ICPDM)} \\
9.6s_u & \text{for drilled shafts (PPDM)}
\end{cases}
\]

\[
\text{Eq. (4.31)}
\]

where \( s_u = \text{undrained shear strength of the bearing layer, estimated from CPT results using (Salgado, 2008):}

\[
s_u = \frac{q_{cb} - \sigma_{v0}}{N_k}
\]

\[
\text{Eq. (4.32)}
\]

where \( \sigma_{v0} = \text{in situ vertical total stress at a depth of } L + (B/2), N_k = \text{cone factor (} \approx 9–15 \text{ as long as the CPT is performed at a penetration rate that is sufficiently high to ensure undrained penetration (refer to Appendix D); soft
NC clays tend to have \( N_k \) values near the low end of this range, while stiff OC clays tend to have \( N_k \) values near the high end of this range, and \( c_k = \) coefficient (\( = 0.4 \) if Eq. 4.33 is satisfied and 1.0 otherwise):

\[
\frac{B_i}{d_i} + 0.45 \frac{Q_{b,ult}}{p_A} < 36 \quad (\text{Eq. 4.33})
\]

where \( B_i = \) inner diameter of OEP pile, \( d_i = \) cone diameter, and \( p_A = \) reference stress (\( = 100 \) kPa or 14.5 psi).

**Step 3:** Multiply the ultimate unit base resistance \( q_{b,ult} \) obtained from step 2 with the pile base area \( A_b \) to obtain the ultimate base capacity \( Q_{b,ult} \) of the pile:

\[
Q_{b,ult} = q_{b,ult} A_b \quad (\text{Eq. 4.34})
\]

Table 4.2 summarizes the expressions for \( A_b \) for different pile cross-sections. For OEP piles bearing in sand (PPDM), calculate the value of \( A_b \) using the gross cross-sectional area \((\pi B_i^2/4)\) of the pile base. For OEP piles bearing in clay (ICPDM), calculate the value of \( A_b \) using the gross cross-sectional area \((\pi B_i^2/4)\) of the pile base if Eq. 4.33 is satisfied, otherwise use the annulus area of steel.

**Step 4:** Compute the ultimate load capacity \( Q_{ult} \) of the pile using:

\[
Q_{ult} = Q_{sL} + Q_{b,ult} \quad (\text{Eq. 4.35})
\]

where \( Q_{sL} = \) limit shaft capacity of the pile, and \( Q_{b,ult} = \) ultimate base capacity of the pile. The ultimate pile load capacity \( Q_{ult} \) obtained from Eq. 4.35 corresponds to a pile head settlement \( \delta \) equal to 10% of the pile diameter \( B \). For piles of noncircular cross-section (e.g., H-piles), an equivalent pile diameter may be obtained by equating the cross-sectional area of the pile with that of an equivalent circle.

### 4.3 Load and Resistance Factor Design Procedure for Single Piles

Load and resistance factor design (LRFD) of a single, isolated, axially-loaded pile can be done from CPT results by following these steps.

**Step 1:** Obtain the nominal dead load \( DL^n \) and the nominal live load \( LL^n \) on the foundation from the superstructure design.

**Step 2:** Set the load factors for dead load and live load, \( LF_{DL} \) and \( LF_{LL} \), as 1.25 and 1.75, respectively (AASHTO, 2020). These load factors correspond to the Strength I limit state (basic load combination relating to the normal vehicular use of the bridge without wind), as defined by AASHTO (2020). The discussion of other limit states, such as Strength II–V, Extreme Event I and II, Service I–IV, and Fatigue I and II are beyond the scope of the manual—information about these limit states can be found in AASHTO (2020).

**Step 3:** Obtain the nominal limit shaft capacity \( Q_{sL}^n \) and the nominal ultimate base capacity \( Q_{b,ult}^n \) of the pile by following the steps outlined in Sections 4.1 and 4.2, respectively.

**Step 4:** Obtain the resistance factors.

a. **Purdue pile design method (PPDM):** Table 4.3 summarizes the PPDM resistance factors, \( RF_b \) and \( RF_s \), for the pile shaft and base resistances, respectively, based on the selected pile type and the predominant soil type at the site. The resistance factors may be adjusted as deemed necessary for sites with high soil variability in the vertical and horizontal directions. Further research is needed to develop PPDM resistance factors for OEP piles in sand.

b. **Imperial College pile design method (ICPDM):** Table 4.4 summarizes the ICPDM resistance factors for driven piles in sand and clay. The resistance factors may be adjusted as deemed necessary for sites with high soil variability in the vertical and horizontal directions.

c. **AASHTO:** Table 4.5 and Table 4.6 summarize the resistance factors advocated by AASHTO (2020) for drilled shafts and driven piles, respectively, in sand and clay.

**Step 5:** Verify that the following LRFD inequality is satisfied (Basu & Salgado, 2012; Foye et al., 2009):

\[
RF_s Q_{sL}^n + RF_b Q_{b,ult}^n \geq LF_{DL} DL^n + LF_{LL} LL^n \quad (\text{Eq. 4.36})
\]

If Eq. 4.36 is satisfied, the pile design is satisfactory for the selected target probability of failure. Repeat steps 3 to 5 to optimize the design if needed. However, if Eq. 4.36 is not satisfied, return to step 3 and revise the pile geometry.

*Note:* The following equation may be used, if needed, to obtain a factor of safety (FS) based on the Working Stress Design (WSD) method (Han et al., 2015):

\[
RF_b Q_{b,ult}^n + RF_s Q_{sL}^n \geq LF_{DL} DL^n + LF_{LL} LL^n \quad (\text{Eq. 4.36})
\]

### Table 4.3

PPDM resistance factors for drilled shafts and CEP piles in sand and clay (modified from Han et al., 2015)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Predominant Soil Type at the Site</th>
<th>( p_{f,T} = 10^{-4} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drilled shaft</td>
<td>Sand</td>
<td>0.70</td>
</tr>
<tr>
<td>Drilled shaft</td>
<td>Clay</td>
<td>0.65</td>
</tr>
<tr>
<td>CEP pile</td>
<td>Sand</td>
<td>0.30</td>
</tr>
<tr>
<td>CEP pile</td>
<td>Clay</td>
<td>0.65</td>
</tr>
</tbody>
</table>

*Note:* The resistance factors were developed by Han et al. (2015) based on results obtained from Monte Carlo simulations. For layered clay deposits (soft over stiff layers), the values of \( RF_b \) and \( RF_s \) should be decreased by 25% and 20%, respectively.

Notation: \( p_{f,T} \) = target probability of failure (a value of \( 10^{-4} \) means that one in every 10,000 piles would fail).
### TABLE 4.4
ICPDM resistance factors for driven piles in sand and clay (modified from Kim et al., 2011; Kim & Lee, 2012)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Predominant Soil Type at the Site</th>
<th>$RF_b$</th>
<th>$RF_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEP and OEP pile</td>
<td>Sand</td>
<td>0.56</td>
<td>0.45</td>
</tr>
<tr>
<td>CEP and OEP pile</td>
<td>Clay</td>
<td>0.58</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Note: The resistance factors were developed by Kim et al. (2011) and Kim and Lee (2012) based on results obtained from reliability analyses performed using the first-order reliability method (FORM). The $RF$ values listed in Table 4.4 are the lowest among the values reported by Kim et al. (2011) and Kim and Lee (2012) for different combinations of $\frac{DL}{LL}$ and $\frac{Q_{ult}}{Q_n}$. These values may also be used for H-piles as the design equations are similar to those for CEP and OEP piles.

Notation: $\beta_T$ = target reliability index and $p_{f,T}$ = target probability of failure (a value of $2 \times 10^{-4}$ means that one in every 5,000 piles would fail).

### TABLE 4.5
Resistance factors for drilled shafts in sand and clay (AASHTO, 2020)

<table>
<thead>
<tr>
<th>Method/Condition</th>
<th>Predominant Soil Type at the Site</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$-method (Brown et al., 2010)</td>
<td>Clay</td>
<td>$RF_{b}$ = 0.40, $RF_{s}$ = 0.45</td>
</tr>
<tr>
<td>$\beta$-method (Brown et al., 2010)</td>
<td>Sand</td>
<td>$RF_{b}$ = 0.50, $RF_{s}$ = 0.55</td>
</tr>
<tr>
<td>Static load test (compression)</td>
<td>Sand/Clay</td>
<td>$RF_{b}$ = 0.70, $RF_{s}$ = 0.70</td>
</tr>
</tbody>
</table>

Note: The resistance factors were developed based on statistical analysis of load test data combined with reliability theory (Paikowsky et al., 2004), fitting to allowable stress design (ASD), or both (Allen, 2005). For piles subjected to uplift (tension), the resistance factor $RF$ is equal to 0.35 for the $\alpha$-method, 0.45 for the $\beta$-method, and 0.60 for pile design based on static load test results.

### TABLE 4.6
Resistance factors for driven piles in sand and clay (AASHTO, 2020)

<table>
<thead>
<tr>
<th>Method/Condition</th>
<th>Predominant Soil Type at the Site</th>
<th>Resistance Factor $RF$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT method (Nottingham &amp; Schmertmann, 1975)</td>
<td>Sand/Clay</td>
<td>0.50</td>
</tr>
<tr>
<td>Static load test (compression)</td>
<td>Sand/Clay</td>
<td>0.75–0.80$^1$</td>
</tr>
</tbody>
</table>

Note: The resistance factors were developed based on statistical analysis of load test results combined with reliability theory (Paikowsky et al., 2004), fitting to allowable stress design (ASD), or both (Allen, 2005). For piles subjected to uplift (tension), the resistance factor $RF$ is equal to 0.40 for the CPT method and 0.60 for pile design based on static load test results. Since a single value for the resistance factor was provided by AASHTO (2020), this value may be used for both the shaft and base components (i.e., $RF = RF_b = RF_s$).

$^1$Additional information can be found in AASHTO (2020), including resistance factors for conditions when dynamic tests are performed on the piles.

where $C_n = nominal\ capacity$, and $D_n = nominal\ demand$.

### 4.4 Load and Resistance Factor Design Procedure for Pile Groups

When the axial load from the superstructure exceeds the resistance offered by a single pile, as is the case for foundations of skyscrapers, bridge piers and abutments, power plants, and offshore oil platforms, it becomes necessary to install multiple piles as a group to support the load. Load and resistance factor design (LRFD) of axially-loaded pile groups can be done from CPT results by following these steps.

**Step 1:** Obtain the nominal dead load $DL_n$ and the nominal live load $LL_n$ on the foundation from the superstructure design.

**Step 2:** Set the load factors for dead load and live load, $LF_{DL}$ and $LF_{LL}$, as 1.25 and 1.75, respectively (AASHTO, 2020). These load factors correspond to the Strength I limit state (basic load combination relating to the normal vehicular use of the bridge without wind), as defined by AASHTO (2020). The discussion of other
limit states, such as Strength II–V, Extreme Event I and II, Service I–IV, and Fatigue I and II are beyond the scope of the manual—information about these limit states can be found in AASHTO (2020).

Step 3: Obtain the nominal limit shaft capacity \( Q_{s,i}^{L,L,i} \) and the nominal ultimate base capacity \( Q_{bult,i}^{L,L,i} \) of a single pile in the group by following the steps outlined in Sections 4.1 and 4.2, respectively.

Step 4: Set the pile center-to-center spacing \( s_{cc} \) and the configuration (or layout) of the pile group.

Step 5: LRFD of pile groups in “sand.”

Execute the following substeps if the pile group is installed in a soil profile that consists predominantly of “sand,” otherwise proceed to step 6.

a. Determine the average (representative) relative density of the sand layer(s) crossed by the pile group (using Eq. 4.10) and the relative density of the bearing layer in which the pile group is embedded (using Eq. 4.30).

b. For small drilled shaft groups (e.g., \( 1 \times 2, 1 \times 3, \) and \( 2 \times 2 \) groups) (Figure 4.3a), the efficiencies \( \eta_{s,i} \) and \( \eta_{b,i} \) for the shaft and base resistances, respectively, are equal to 1.0 for a pile head settlement of 30 mm (1.2 in.). For a large, drilled shaft group (e.g., \( 4 \times 4 \) group) (Figure 4.3b), refer to Table 4.7 for the values of \( \eta_{s,i} \) and \( \eta_{b,i} \). Further research is needed to develop rigorous values of \( \eta_{s,i} \) and \( \eta_{b,i} \) for driven pile groups in sand; in the meantime, the same values for drilled shaft groups may also be used for driven pile groups if deemed appropriate. Alternatively, Table 4.8 and Table 4.9 summarize the efficiencies advocated by AASHTO (2020) for drilled shaft groups and driven pile groups, respectively, in sand.

c. Obtain the resistance factors, \( RF_s \) and \( RF_b \), for the pile shaft and base resistances, respectively, from step 4 of Section 4.3.

d. Verify that the following LRFD inequality is satisfied (Han et al., 2015):

\[
RF_s \left[ \sum_{i=1}^{n_p} \eta_{s,i} Q_{s,i}^{L,L,i} \right] + RF_b \left[ \sum_{i=1}^{n_p} \eta_{b,i} Q_{bult,i}^{L,L,i} \right] \geq LF_{FD} DL^n + LF_{LL} LL^n \quad \text{(Eq. 4.38)}
\]

where \( n_p \) = number of piles in the group. If Eq. 4.38 is satisfied, the pile group design is satisfactory for the selected target probability of failure. Repeat steps 3 to 5 to optimize the design if needed. However, if Eq. 4.38 is not satisfied, return to step 3 and revise the design.

Note: The following equation may be used, if needed, to obtain a factor of safety (FS) based on the Working Stress Design (WSD) method:

\[
FS = \frac{C^n}{D^n} = \frac{\sum_{i=1}^{n_p} \eta_{s,i} Q_{s,i}^{n,n,i} + \sum_{i=1}^{n_p} \eta_{b,i} Q_{bult,i}^{n,n,i}}{DL^n + LL^n} \quad \text{(Eq. 4.39)}
\]

where \( C^n \) = nominal capacity, and \( D^n \) = nominal demand.

Step 6: LRFD of pile groups in “clay.”

Execute the following substeps if the pile group is installed in a soil profile that consists predominantly of “clay.”

i. For small drilled shaft groups (e.g., \( 1 \times 2, 1 \times 3, \) and \( 2 \times 2 \) groups) (Figure 4.3a), the efficiencies \( \eta_{s,i} \) and \( \eta_{b,i} \) for the shaft and base resistances, respectively, are equal to 1.0 for a pile head settlement of 30 mm (1.2 in.). For a large drilled shaft group (e.g., \( 4 \times 4 \) group) (Figure 4.3b), refer to Table 4.10 for the values of \( \eta_{s,i} \) and \( \eta_{b,i} \). Further research is needed to develop rigorous values of \( \eta_{s,i} \) and \( \eta_{b,i} \) for driven pile groups in clay; in the meantime, the same values for drilled shaft groups may also be used for driven pile groups if deemed appropriate. Alternatively, Table 4.11 summarizes the efficiencies advocated by AASHTO (2020) for drilled shaft groups and driven pile groups in clay.

ii. Obtain the resistance factors, \( RF_s \) and \( RF_b \), for the pile shaft and base resistances, respectively, from step 4 of Section 4.3.

iii. Verify that the following LRFD inequality is satisfied (Han et al., 2015):
TABLE 4.7
Shaft and base efficiencies for a large (4 × 4) drilled shaft group in sand for \( s_{cc} = 2B \) (Han et al., 2015; Han, Salgado, 2019)

<table>
<thead>
<tr>
<th>Pile Head Settlement ( w )</th>
<th>Efficiency ( \eta_{b,i} )</th>
<th>Efficiency ( \eta_{s,i} )</th>
<th>Efficiency ( \eta_{c,i} )</th>
<th>Efficiency ( \eta_{b,i} )</th>
<th>Efficiency ( \eta_{s,i} )</th>
<th>Efficiency ( \eta_{c,i} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 mm (1.2 in.)</td>
<td>Center Pile: 1.14</td>
<td>Center Pile: 0.90</td>
<td>Corner Pile: 0.80</td>
<td>Center Pile: 0.93</td>
<td>Center Pile: 0.78</td>
<td>Corner Pile: 0.74</td>
</tr>
<tr>
<td></td>
<td>Side Pile: 0.63</td>
<td>Side Pile: 1.01</td>
<td>Corner Pile: 1.06</td>
<td>Side Pile: 0.94</td>
<td>Side Pile: 1.25</td>
<td>Corner Pile: 1.01</td>
</tr>
<tr>
<td>50 mm (2.0 in.)</td>
<td>Center Pile: 1.28</td>
<td>Center Pile: 0.96</td>
<td>Corner Pile: 0.81</td>
<td>Center Pile: 1.16</td>
<td>Center Pile: 0.86</td>
<td>Corner Pile: 0.77</td>
</tr>
<tr>
<td></td>
<td>Side Pile: 0.80</td>
<td>Side Pile: 1.19</td>
<td>Corner Pile: 1.16</td>
<td>Side Pile: 1.23</td>
<td>Side Pile: 1.51</td>
<td>Corner Pile: 1.04</td>
</tr>
</tbody>
</table>

Note: The value of 50 mm (2 in.) for the pile head settlement is based on the tolerable settlement criteria for frame structures and bridges. Settlements beyond 50 mm (2 in.) would lead to serviceability issues, while those approaching 100 mm (4 in.) would lead to structural damage (Bozozuk, 1978). For intermediate values of \( w \) and \( D_R \), the values of \( \eta_{s,i} \), and \( \eta_{c,i} \) can be obtained by linear interpolation.

Notation: \( R = \) pile diameter, \( s_{cc} = \) pile center-to-center spacing, \( \eta_{s,i} = \) efficiency for shaft resistance of the \( i^{th} \) pile in the group, and \( \eta_{b,i} = \) efficiency for base resistance of the \( i^{th} \) pile in the group.

TABLE 4.8
Efficiencies for small and large driven shaft groups in sand (AASHTO, 2020)

<table>
<thead>
<tr>
<th>Group Configuration</th>
<th>( s_{cc} )</th>
<th>Special Conditions</th>
<th>( \eta_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single row (e.g., 1 × 2 and 1 × 3 groups)</td>
<td>2( B )</td>
<td>—</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>( \geq 3B )</td>
<td>—</td>
<td>1.00</td>
</tr>
<tr>
<td>Multiple row (e.g., 2 × 2 and 4 × 4 groups)</td>
<td>2.5( B )</td>
<td>—</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>3( B )</td>
<td>—</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>( \geq 4B )</td>
<td>—</td>
<td>1.00</td>
</tr>
<tr>
<td>Single and multiple rows</td>
<td>( \geq 2B )</td>
<td>Pressure grouting is used along the sides of the pile to restore lateral stress losses caused by pile installation, and the pile base is pressure grouted</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Note: For intermediate values of \( s_{cc} \), the value of \( \eta_i \) can be obtained by linear interpolation. For pile groups bearing on a strong soil layer of limited thickness overlying a weak deposit, the nominal resistance of the pile group is taken as the lesser of (a) the sum of the individual nominal resistances of each pile in the group, and (b) the nominal resistance of the pile group against block failure, with consideration to the punching of the pile group into the underlying weak layer (AASHTO, 2020).

Notation: \( B = \) pile diameter, \( s_{cc} = \) pile center-to-center spacing, and \( \eta_i = \) efficiency of the \( i^{th} \) pile in the group \((= \eta_{s,i} = \eta_{b,i})\).
iii. Estimate the limit unit base resistance \( q_{bL} \) of the pile group using (Salgado, 2008; Skempton, 1951):

\[
q_{bL} = 5s_u \left(1 + 0.2 \frac{B_g}{L_{g,i}} \right) \left(1 + \frac{L}{12B_g} \right) \quad \text{(Eq. 4.41)}
\]

where \( s_u \) = undrained shear strength at a depth of \( L + (B_g/3) \), and \( L = \) pile embedment length.

d. Set both the shaft and base resistance factors, \( RF_s \) and \( RF_b \), as equal to 0.60 for driven pile groups and 0.55 for drilled shaft groups (AASHTO, 2020).

ev. Verify that the following LRFD inequality is satisfied:

\[
RF_s \left[ 2(B_g + L_d) L_{q,sL} \right] + RF_b \left[ B_g L_d q_{bL} \right] \\
\geq LF_{DL} DL^n + LF_{LL} L^n 
\]

Note: The following equation may be used, if needed, to obtain a factor of safety (FS) based on the Working Stress Design (WSD) method.

\[
FS = \frac{C^n}{D^n} = \frac{2(B_g + L_d) L_{q,sL} + B_g L_d q_{bL}}{DL^n + LL^n} \quad \text{(Eq. 4.43)}
\]

where \( C^n = \) nominal capacity, and \( D^n = \) nominal demand.

### TABLE 4.10
Shaft and base efficiencies for a large \((4 \times 4)\) drilled shaft group in NC clay for \( s_{cc} = 2B \) (Han et al., 2015)

<table>
<thead>
<tr>
<th>Pile Head Settlement ( w )</th>
<th>Center Pile</th>
<th>Side Pile</th>
<th>Corner Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 mm (1.2 in.)</td>
<td>( \eta_{b,i} )</td>
<td>0.96</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td>( \eta_{s,i} )</td>
<td>0.38</td>
<td>0.77</td>
</tr>
<tr>
<td>50 mm (2.0 in.)</td>
<td>( \eta_{b,i} )</td>
<td>1.02</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>( \eta_{s,i} )</td>
<td>0.46</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Note: The value of 50 mm (2 in.) for the pile head settlement is based on the tolerable settlement criteria for frame structures and bridges. Settlements beyond 50 mm (2 in.) would lead to serviceability issues, while those approaching 100 mm (4 in.) would lead to structural damage (Bozozuk, 1978). For intermediate values of \( w \), the values of \( \eta_{s,i} \) and \( \eta_{b,i} \) can be obtained by linear interpolation. Further research is needed to develop rigorous values of \( \eta_{s,i} \) and \( \eta_{b,i} \) for pile groups in OC clay, but until then, the same values for NC clay may also be used for OC clay.

Notation: \( B = \) pile diameter, \( s_{cc} = \) pile center-to-center spacing, \( \eta_{s,i} = \) efficiency for shaft resistance of the \( i^{th} \) pile in the group, and \( \eta_{b,i} = \) efficiency for base resistance of the \( i^{th} \) pile in the group.

### TABLE 4.11
Efficiencies for small and large drilled shaft and driven pile groups in clay (AASHTO, 2020)

<table>
<thead>
<tr>
<th>Group Configuration</th>
<th>( s_{cc} )</th>
<th>Condition</th>
<th>( \eta_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single and multiple rows</td>
<td>( 2.5B )</td>
<td>Pile cap is not in firm contact with the ground and the soil at the ground surface is soft</td>
<td>0.65</td>
</tr>
<tr>
<td>Single and multiple rows</td>
<td>( \geq 6B )</td>
<td>Same as above</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Note: For intermediate values of \( s_{cc} \), the value of \( \eta_i \) can be obtained by linear interpolation. If the pile cap is not in firm contact with the ground but the soil is stiff, \( \eta_i = 1.0 \). If the pile cap is in firm contact with the ground, \( \eta_i = 1.0 \).

Notation: \( B = \) pile diameter, \( s_{cc} = \) pile center-to-center spacing, and \( \eta_i = \) efficiency of the \( i^{th} \) pile in the group \((= \eta_{b,i} = \eta_{b,i})\).

---

**Figure 4.4** Schematic of a \( 3 \times 4 \) pile group with parameters \( L_{g}, B_g, \) and \( L \) in (a) plan view and (b) 3D view (Salgado, 2008).
c. If Eqs. 4.40 and 4.42 are satisfied, the pile group design is satisfactory with respect to the ultimate limit states of individual pile failure and block failure, respectively. Repeat steps 3, 4 and 6 to optimize the design if needed. However, if either Eq. 4.40 or Eq. 4.42 is not satisfied, return to step 3 and revise the design.

4.5 Chapter Summary

In this chapter, detailed, step-by-step procedures for computing the limit shaft capacity $Q_{sL}$ and the ultimate base capacity $Q_{b,ult}$ of a single, isolated, axially-loaded pile from CPT results in sand (silica sand) and clay were presented. The limit unit shaft resistance $q_d$ and the ultimate unit base resistance $q_{b,ult}$ of a pile designed using the PPDM depend on the critical-state friction angle $\phi_c$, and relative density $D_R$ in the case of sands, and the undrained shear strength $s_u$, friction angles, $\phi_c$ and $\phi_r$, in the case of clays; $\phi_{r,min}$ = minimum residual-state friction angle. The undrained shear strength $s_u$ can be estimated from CPT results through the cone factor $N_k$, which typically ranges from 9–15 depending on soil type, stress state and history, and stress path (e.g., triaxial compression versus direct simple shear). In addition to some of these variables, the ICPDM relies on other key parameters, such as residual interface friction angle $\delta_r$, sensitivity $S_r$, and overconsolidation ratio OCR in the case of clays. For base resistance calculations, both the PPDM and the ICPDM average the cone resistance $q_c$ around the pile base according to some formula and relate the unit base resistance $q_{b,ult}$ of the pile to the representative (average) cone resistance $q_{cb}$, which serves as a proxy for the limit unit base resistance $q_{b,L}$ of the pile.

Guidelines for piles installed in mixed or intermediate soils, such as sand-silt-clay mixtures and gravelly sand, were provided. In addition, load and resistance factor design (LRFD) procedures for single piles and pile groups were presented, and potential areas for future research have been indicated. When using LRFD, it is important to note that the resistance factors are always tied to the specific design methods and equations for which they were developed.

Summary tables for the CPT-based pile design methods covered in this chapter have been prepared so that the methods can be easily referred to when needed. The design methods covered in this chapter are not mandatory for design in INDOT contracts, and other CPT-based methods, some of which are summarized in Table 4.12 to Table 4.25, may be used as deemed appropriate for the site and loading conditions under consideration. Pile design methods that rely solely on the measured values of cone resistance to the exclusion of other information that may be available at design time will not be as accurate as methods that consider all the available information. In this sense, they are, in fact, less conservative. The inclusion of key intrinsic and state variables known to control the mechanical response of soil during shearing, such as relative density and stress state in the case of sands, and undrained shear strength, critical-state friction angle, and minimum residual-state friction angle in the case of clays, improves the capability of a CPT-based design method to predict the ultimate load capacity of a pile. These variables can be determined using the guidance and relationships provided in the manual. Also, pile design methods that consider the effect of soil plugging during the installation of open-ended pipe piles are expected to provide more realistic estimates of pile capacity than methods that do not consider this effect. The capacities mobilized by a closed-ended pipe pile and an open-ended pipe pile are different (Han et al., 2019b; Paik et al., 2003), and pile design methods that do not differentiate between these pile types do not consider installation effects on pile capacity.

Shaft degradation is a process by which the unit shaft resistance at a given depth along the pile decreases as the pile is driven down further from that depth (Lehane et al., 1993; Randolph, 2003; Randolph et al., 1994; White & Lehane, 2004). This degradation, however, is not properly accounted for in the purely direct CPT-based pile design methods (i.e., methods that rely only on CPT data to the exclusion of other variables). Furthermore, because of greater variability in sleeve resistance measurements (among other issues), $f_s$ is not a reliable parameter for use in foundation design (Schneider et al., 2008), which is why the modern pile design methods (e.g., PPDM, ICPDM, UWAPDM, and UPDM) rely instead on the cone resistance $q_c$, among other variables, and contain a shaft resistance degradation term in the design equations. The PPDM, ICPDM, UWAPDM, and UPDM are based on the 10% relative settlement criterion, i.e., the methods predict the ultimate load capacity of the pile corresponding to a pile displacement equal to 10% of the pile diameter (except for certain cases, such as floating piles in soft clay, where the limit load is achieved after relatively small settlements (Basu & Salgado, 2014)).

A final note is in order regarding the use of the cone resistance to obtain other soil parameters of interest. The cone resistance is a single measurement, but it depends on more than one variable. For example, in simple terms, the cone resistance $q_c$ in sand depends on two state variables—relative density $D_R$ and in situ horizontal effective stress $\sigma_{ho}$ (White & Lehane, 2004)—and one intrinsic variable—critical-state friction angle $\phi_c$. The cone resistance can be used to estimate $D_R$ if the other two variables ($\sigma_{ho}$ and $\phi_c$) are known, but it cannot be used to determine all three variables. This needs to be kept in mind as engineers may be tempted to obtain the values of more than one variable from $q_c$, which is a single measurement. Interpreting CPT results can be likened to solving a system of equations: the number of equations must be equal to the number of unknowns to be determined. If only one measurement is available, we cannot determine multiple independent variables from that one measurement.
4.5.1 Design Methods for Nondisplacement Piles (Drilled Shafts) in Sandy and Clayey Soils

TABLE 4.12
PPDM equations for the unit shaft and base resistances for nondisplacement piles (drilled shafts) in sand and clay

<table>
<thead>
<tr>
<th>Soil Type and References</th>
<th>Limit Unit Shaft Resistance $q_{ul}$</th>
<th>Ultimate Unit Base Resistance $q_{ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (Han et al., 2017)</td>
<td>$q_{ul} = K \sigma_0 \tan \delta$</td>
<td>$q_{ult} = 62 \rho_d \left( \frac{D_r}{100} \right)^{1.83} \left( \frac{\sigma_0}{\rho_d} \right)^{0.4}$</td>
</tr>
<tr>
<td></td>
<td>$K = 0.67 \exp(0.3 \sqrt{K_0 - 0.4}) \exp\left( \frac{D_r}{100} \left[ 1.5 - 0.35 \ln \left( \frac{\sigma_0}{\rho_d} \right) \right] \right)$</td>
<td></td>
</tr>
<tr>
<td>Clay (Chakraborty et al., 2013; Salgado, 2006)</td>
<td>$q_{ul} = 2 \sigma_0$</td>
<td>$q_{ult} = 9.6 \sigma_0$</td>
</tr>
<tr>
<td></td>
<td>$x = \left( \frac{\sigma_o}{\sigma_0} \right)^{0.05} \left{ A_1 + (1 - A_1) \exp \left[ - \left( \frac{\sigma_o}{\rho_0} \right) \left( \phi_0 - \phi_{r, \text{min}} \right) \right] \right}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_1 = 0.75$ for $\phi_0 - \phi_{r, \text{min}} \geq 5^\circ$, 0.40 for $\phi_0 - \phi_{r, \text{min}} \geq 12^\circ$ and a linearly interpolated value for $5^\circ &lt; \phi_0 - \phi_{r, \text{min}} &lt; 12^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_2 = 0.4 + 0.3 \ln \left( \frac{\sigma_0}{\sigma_0} \right)$</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method predicts the ultimate load capacity $Q_{ult}$ of the pile corresponding to a pile head settlement $w$ equal to 10% of the pile diameter $B$. The equation for the ultimate unit base resistance $q_{ult}$ is applicable for $L/B < 50$. The method is intended to estimate the shaft resistance in clay after dissipation of the excess pore water pressure generated during pile installation. The relative density $D_r$ and undrained shear strength $s_u$ can be estimated from CPT results using the equations provided in the chapter.

Notation: PPDM = Purdue pile design method, $\rho_d$ = reference stress (= 100 kPa or 14.5 psi), $K$ = coefficient of lateral earth pressure, $\sigma_0$ = in situ vertical effective stress at the depth being considered, $\phi_0$ = critical-state interface friction angle (which, for drilled shafts, is equal to the internal critical-state friction angle $\phi_i$ of the soil), $\phi_{r, \text{min}} = \phi_{r, \text{min}} \text{in situ}$ horizontal effective stress at the depth being considered ($= K_0 \sigma_0$), $K_0$ = coefficient of lateral earth pressure at-rest (Appendix B), and $\phi_{r, \text{min}} = \phi_{r, \text{min}} \text{in situ}$ horizontal effective stress at the depth being considered (Appendix E).

TABLE 4.13
MnDOT equations (Modified UniCone method) for the unit shaft and base resistances for nondisplacement piles (drilled shafts) in sand and clay (Dagger et al., 2018)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Limit Unit Shaft Resistance $q_{ul}$</th>
<th>Ultimate Unit Base Resistance $q_{ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>$q_{ul} = q_E \theta_l \theta_m \left( 10^{0.732L} - 3.605 \right)$</td>
<td>$q_{ult} = q_E \left( 10^{0.325L} - 1.218 \right)$</td>
</tr>
<tr>
<td></td>
<td>$L = \sqrt{\left( 3.47 - \log Q_{sim} \right)^2 + \left( 1.22 + \log F_c \right)^2}$</td>
<td>$L_c$ is calculated using the same set of equations</td>
</tr>
<tr>
<td></td>
<td>$Q_{sim} = \left( \frac{q_t - \sigma_0}{\sigma_0} \right) \frac{p_d}{\sigma_0}$ and $F_t = \frac{f_s}{q_t - \sigma_0} \times 100%$</td>
<td>as those in the estimation of $q_{ul}$</td>
</tr>
<tr>
<td></td>
<td>$n = \min \left( 0.381 L_c + 0.05 \left( \frac{q_t}{\sigma_0} \right) - 0.15, 1 \right)$</td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>Use the same equation as for sand</td>
<td>Use the same equation as for sand</td>
</tr>
</tbody>
</table>

Note: The method predicts the maximum load capacity $Q_{max}$ of the pile (i.e., the maximum load applied on the piles considered in the database). For most (> 90%) of the pile load tests considered in the database, the value of $Q_{max}$ was nearly equal to the value of $Q_{sim}$ based on the 10% relative settlement criterion (i.e., the load corresponding to a pile head settlement $w$ equal to 10% of the pile diameter $B$). The following adjustment was proposed to estimate $Q_{ult}$ from $Q_{sim}$: $Q_{ult} = 0.986Q_{max}$ (Niazi & Mayne, 2016).

The value of the exponent $n$ is approximately equal to 1 for clay, 0.75 for silt, and 0.5 for sand. For mixed or intermediate soils, iterative calculations are needed to determine the value of $L$. For the first iteration, the method recommends the use of $n = 1$ to obtain an initial value of $L$ at the depth being considered. In the next iteration, this initial value of $L$ is used to update the value of $n$, which is then used to obtain a new value of $L$. The process is repeated until the value of $L$ converges, which is generally after the third cycle. Additional information on sensitive clays can be found in Niazi and Mayne (2016).

The representative cone resistance $q_{ult}$ for base resistance calculation is $q_E$, averaged over a vertical distance of $B$ below the pile base (Dagger et al., 2018).

Notation: MnDOT = Minnesota Department of Transportation, $B$ = pile diameter, $q_E$ = effective cone resistance ($= q_t - u_2$); $q_t$ = corrected total cone resistance; $f_s$ = sleeve resistance; $u_2$ = pore water pressure measured at the shoulder position behind the cone face; $L$ = soil behavior type index; $Q_{sim}$ = normalized cone resistance; $F_t$ = normalized friction ratio; $\sigma_0$ and $\sigma_{r, \text{in situ}}$ = in situ vertical total and effective stresses, respectively, at the depth being considered; $p_d$ = reference stress (= 100 kPa or 14.5 psi); $\theta_m$ = coefficient for pile type (= 0.84 for drilled shafts); $\theta_l$ = coefficient for loading direction (= 0.85 for tension and 1.11 for compression); and $\theta_{l, \text{in situ}}$ = coefficient for loading procedure (= 1.09 for constant rate of penetration test and 0.97 for maintained load test).
4.5.2 Design Methods for Displacement Piles in Sandy Soil

TABLE 4.14
PPDM equations for the unit shaft and base resistances for displacement piles driven in sand (modified from Han et al., 2019b)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{sL}$</th>
<th>Ultimate Unit Base Resistance $q_{b,ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{sL} = F_{load}K\sigma_0\tan \delta_c$</td>
<td>$q_{b,ult} = (1-0.0058D_R)q_{cb}$</td>
</tr>
<tr>
<td></td>
<td>$K = K_{min} + (K_{max} - K_{min}) \exp \left( -\frac{2h}{L_R} \right)$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$K_{max} = \frac{0.01(q_{cb}/p_4)}{\sqrt{\sigma_{04}/p_4}}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$K_{min} = 0.2$ and $\alpha = 0.14$</td>
<td></td>
</tr>
<tr>
<td>Open-ended pipe pile</td>
<td>$q_{sL} = K(1-0.66PLR)\sigma_0\tan \delta_c$</td>
<td>$q_{b,ult} = \min \left[ 0.21(IFR)^{-\frac{1}{2}}q_{cb}; 0.6q_{cb} \right]$</td>
</tr>
<tr>
<td></td>
<td>$K$ and $K_{max}$ take the same formulae as above,</td>
<td>$IFR \approx \min \left[ 1; \left( \frac{B_i}{1.5L_R} \right)^{0.27} \right]$</td>
</tr>
<tr>
<td></td>
<td>with $K_{min} = 0.2$ and $\alpha = 0.14$</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method predicts the ultimate load capacity $Q_{ult}$ of the pile corresponding to a pile head settlement $w$ equal to 10% of the pile diameter $B$. The method considers open-ended pipe piles in sand to behave as fully-plugged piles during static loading. Accordingly, the ultimate base capacity $Q_{b,ult}$ of an open-ended pipe pile is calculated using the gross cross-sectional area ($\pi B^2/4$) of the pile base. The exponential term in the equation for $K$ accounts for shaft resistance degradation due to pile driving.

For open-ended pipe piles, the plug length ratio (PLR) used in the equation for $q_{sL}$ is that measured at the specific depth where $q_{sL}$ is calculated. If the PLR is not measured, it can be approximated using the same equation provided for the IFR. IFR is the incremental filling ratio averaged over the last 3B of pile driving; if not measured, it can be estimated using the equation provided.

The representative cone resistance $q_{cb}$ for base resistance calculation is $q_c$ averaged from 1B above to 2B below the pile base.

Notation: PPDM = Purdue pile design method, $F_{load}$ = factor that accounts for loading direction (≈ 0.5–0.6 for tension and 1.0 for compression), $p_4$ = reference stress (= 100 kPa or 14.5 psi), $L_R$ = reference length (= 1 m or 39.4 in.), $K = \text{coefficient of lateral earth pressure}$, $\sigma_0 = \text{in situ vertical effective stress at the depth being considered}$, $\delta_c = \text{critical-state interface friction angle (Figure 4.2)}$, $h = \text{vertical distance from the pile base to the depth being considered}$, $B_i = \text{inner diameter of open-ended pipe pile}$, $\sigma_{04} = \text{in situ horizontal effective stress at the depth being considered}$ (= $K_0\sigma_0$), $K_0 = \text{coefficient of lateral earth pressure at-rest (Appendix B)}$, $q_c = \text{cone resistance}$, and $D_R = \text{relative density (estimated from CPT results using Eq. 4.30)}$. 

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### TABLE 4.15
ICPDM equations for the unit shaft and base resistances for displacement piles driven in sand (Jardine et al., 2005)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{L}$</th>
<th>Ultimate Unit Base Resistance $q_{b,ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{L} = (F_{load} \sigma_{c} + \Delta \sigma_{c}) \tan \delta_{c}$</td>
<td>$q_{b,ult} = \max \left[0.3; 1 - 0.5 \log \left(\frac{B}{\pi} \right)\right] q_{b}$</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{c} = 0.029q_{f} \left(\frac{\sigma_{0}}{\mu} \right)^{0.13} \left(\max \left[h \left(\frac{R}{h} \right) \right] \right)^{-0.38}$ and $\Delta \sigma_{c} = \frac{2G \Delta r}{R}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$G = q_{s} \left[0.0203 + 0.00125 \eta - 1.216 \times 10^{-6} \eta^{3} \right]^{-1}$ and $\eta = \frac{q_{c}/P_{a}}{\sigma_{\mu}/P_{a}}$</td>
<td></td>
</tr>
<tr>
<td>Open-ended pipe pile</td>
<td>Use the same equations as for closed-ended pipe pile but with an equivalent pile radius $R$ given by:</td>
<td>The pile responds as a plugged pile during static loading if:</td>
</tr>
<tr>
<td></td>
<td>$R = \sqrt{R_{o}^{2} - R_{i}^{2}}$</td>
<td>$\frac{B_{i}}{L_{R}} &lt; 0.2(D_{R} - 30)$ or $\frac{B_{o}}{d_{c}} &lt; 0.083 \sqrt{P_{a}}$</td>
</tr>
<tr>
<td></td>
<td>For piles in tension, the value of $q_{L}$ is decreased further by 10%.</td>
<td>Response as a plugged pile during static loading:</td>
</tr>
<tr>
<td></td>
<td>$q_{b,ult} = \max \left[0.15; 0.5 - 0.25 \log \left(\frac{B}{\pi} \right) \left(1 - \frac{R_{i}^{2}}{R_{o}^{2}} \right)\right] q_{b}$</td>
<td>$Q_{b,ult} = q_{b,ult} \pi R_{o}^{2}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Response as an unplugged pile during static loading:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$q_{ann,ult} = q_{b,ult}$ and $Q_{b,ult} = q_{ann,ult} \pi (R_{o}^{2} - R_{i}^{2})$</td>
</tr>
<tr>
<td>H-pile</td>
<td>Use the same equations as for closed-ended pipe pile but with an equivalent pile radius $R$ given by:</td>
<td>$q_{b,ult} = q_{b}$</td>
</tr>
<tr>
<td></td>
<td>$R = \sqrt{A_{h} \pi}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_{h} = 2b_{f} t_{f} + (2X_{p} + t_{w})(d - 2t_{j})$</td>
<td>$q_{b,ult} = 0.7q_{b}$</td>
</tr>
<tr>
<td></td>
<td>$X_{p} = b_{j}/8$ if $b_{j}/2 &lt; (d - 2t_{j}) &lt; b_{j}$ and $X_{p} = b_{j}/2[16(d - 2t_{j})]$ if $(d - 2t_{j}) \geq b_{j}$</td>
<td></td>
</tr>
<tr>
<td>Square or rectangular pile</td>
<td>Use the same equations as for closed-ended pipe pile but with an equivalent pile radius $R$ given by:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R = \sqrt{A_{h} \pi}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_{h} = B_{x} B_{y}$ where $B_{x}$ and $B_{y}$ are width and length, respectively, of the pile cross-section (in plan)</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method predicts the ultimate load capacity $Q_{ult}$ of the pile corresponding to a pile head settlement $w$ equal to 10% of the pile diameter $B$. In addition, the method is intended to predict the pile capacity measured 10 days after driving for “virgin” piles (i.e., piles that have not been load-tested). The representative cone resistance $q_{cb}$ for base resistance calculation is $q_{c}$ averaged from 1.5 $B$ above to 1.5 $B$ below the pile base.

Notation: ICPDM = Imperial College pile design method, $F_{load} = \text{factor that accounts for loading direction} (= 0.8 \text{ for tension and 1.0 for compression})$, $\Delta r = \text{radial displacement of soil during pile loading} (= 0.02 \text{ mm or 0.8 mil for lightly rusted steel piles})$, $p_{a} = \text{reference stress} (= 100 \text{ kPa or 14.5 psi})$, $L_{R} = \text{reference length} (= 1 \text{ m or 39.4 in.})$, $\sigma_{c} = \text{local radial effective stress acting on the pile segment after installation}$, $\Delta \sigma_{c} = \text{increase in local radial effective stress associated with constrained dilation during pile loading}$, $\sigma_{\mu} = \text{in situ vertical effective stress at the depth being considered}$, $\delta_{c} = \text{critical-state interface friction angle}$, $B_{i} = \text{inner diameter of open-ended pipe pile}$, $d_{c} = \text{cone diameter}$, $R = \text{pile radius}$, $h = \text{vertical distance from the pile base to the depth being considered}$, $q_{c} = \text{cone resistance}$, $D_{R} = \text{relative density}$, $R_{o} = \text{outer radius of open-ended pipe pile}$, $R_{i} = \text{inner radius of open-ended pipe pile}$, $A_{h} = \text{area of pile base}$, $G = \text{shear modulus}$, $b_{f} = \text{width of flange}$, $d = \text{depth of H-section}$, $t_{f} = \text{thickness of flange}$, $t_{w} = \text{thickness of web}$, and $q_{ann,ult} = \text{ultimate unit annulus resistance}$. 

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TABLE 4.16
UWAPDM equations for the unit shaft and base resistances for displacement piles driven in sand (Lehane et al., 2005)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{ul}$</th>
<th>Ultimate Unit Base Resistance $q_{ub,ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{ul} = \frac{f_l}{f_s} \left( \frac{\sigma_n + \Delta \sigma_{id}}{\sigma_n} \right) \tan \delta_{c}$</td>
<td>$q_{ub,ult} = 0.6 q_{ul}$</td>
</tr>
<tr>
<td></td>
<td>$\sigma_n = 0.03 q_{ul} \left( \max \left( \frac{h}{B} \right) \right)^{-0.5}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\Delta \sigma_{id} = \frac{4G \Delta r}{B}$ and $G = 185 \left( \frac{q_{ul}}{p_A} \right)^{-0.75}$</td>
<td></td>
</tr>
</tbody>
</table>

Open-ended pipe pile

|                               | $q_{ul} = \frac{f_l}{f_s} \left( \frac{\sigma_n + \Delta \sigma_{id}}{\sigma_n} \right) \tan \delta_{c}$ | $q_{ub,ult} = (0.15 + 0.45 A_{rb}) q_{ub}$      |
|                               | $\sigma_n = 0.03 q_{ul} \left( A_{rb} \right)^{0.3} \left( \max \left( \frac{h}{B} \right)^{0.2} \right)$ | $A_{rb} = 1 - \text{FFR} \left( \frac{B}{R} \right)^2$ |
|                               | $\Delta \sigma_{id} = \frac{4G \Delta r}{B}$ and $G = 185 \left( \frac{q_{ul}}{p_A} \right)^{-0.75}$ | FFR is the final filling ratio, which is defined as the average incremental filling ratio measured over the final 3B of pile driving; if not measured, it can be roughly approximated by using the same equation for the IFR. |
|                               | IFR is the average incremental filling ratio measured over the final 20B of pile driving; when plug length measurements are not available, it can be estimated using: |                                                  |
|                               | IFR $\approx \min \left[ 1, \left( \frac{B}{1.5L_B} \right)^{0.23} \right]$ |                                                  |

Note: The method predicts the ultimate load capacity $Q_{ub}$ of the pile corresponding to a pile base settlement equal to 1% of the pile diameter (Lehane et al., 2007; Xu, 2007; Xu et al., 2008). In addition, the method is intended to predict the pile capacity measured 10–20 days after driving. The method considers open-ended pipe piles in sand to behave as fully-plugged piles during static loading. Accordingly, the ultimate base capacity $Q_{ub,ult}$ of an open-ended pipe pile is calculated using the gross cross-sectional area ($\pi B^2/4$) of the pile base.

The representative cone resistance $q_{rb}$ for base resistance calculation is $q_r$ averaged using the Dutch technique (Figure 4.5): $q_{rb} = 0.5(q_{1a} + q_{1b})$, with $q_{1a} = 0.5(q_{1a} + q_{1b})$, $q_{1a}$ is average of the $q_r$ values over a vertical distance of $\lambda B$ below the pile base, $q_{1b}$ is average of the $q_r$ values over a vertical distance of $\lambda B$ below the pile base following a minimum path rule, and $B$ is average of the $q_r$ values over a vertical distance of $8B$ above the pile base following a minimum path rule. The value of $q_{1a}$ is calculated for different $\lambda$ values ranging from 0.7 to 4.0, and the minimum value of $q_{1a}$ obtained is used in the calculation of $q_{ub}$. Additional information about the computation of $q_{1a}$ and $q_{1b}$ can be found in Schmertmann (1978).

In the absence of plug length measurements, the value of the IFR may also be estimated using: IFR $= \tanh(0.3(B/d)^{0.2})$ (Lehane, 2019). The FFR can be roughly approximated by using the same equation for the IFR.

Note: UWAPDM = University of Western Australia pile design method, $f_{lf} / f_{ls}$ = ratio of tension to compression capacity ($= 0.75$ for tension and 1.0 for compression), $\Delta r$ = radial displacement of soil during pile loading ($= 0.02$ mm or 0.8 mil for lightly rusted steel piles), $p_A$ = reference stress ($= 100$ kPa or 14.5 psi), $L_B$ = reference length ($= 1$ m or 39.4 in.), $\sigma_n$ = local radial effective stress acting on the pile segment after installation, $\Delta \sigma_{id}$ = increase in local radial effective stress associated with constrained dilation during pile loading, $\sigma_{cr}$ = critical-state interface friction angle, $A_{rb}$ = effective shaft area ratio, $A_{rb} = \text{in situ}$ vertical effective stress at the depth being considered, $\delta_c$ = critical-state interface friction angle, $A_{rb}$ = effective shaft area ratio, $B_r$ = inner diameter of open-ended pipe pile, $B_{eff} = \text{effective pile diameter}, d_c$ = cone diameter, $h = \text{vertical distance from the pile base to the depth being considered}, q_r = \text{cone resistance}, G = \text{shear modulus}.$

TABLE 4.17
AASHTO equations for the unit shaft and base resistances for displacement piles driven in sand (AASHTO, 2020; Nottingham & Schmertmann, 1975)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{ul}$</th>
<th>Limit Unit Base Resistance $q_{ub}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{ul} = \left{ \begin{array}{ll} 0.125 K_{fs} \left( \frac{B}{R} \right) &amp; \text{for } 0 \leq z \leq 8B \ K_{fs} &amp; \text{for } 8B \leq z \leq L \end{array} \right. \right.$</td>
<td>$q_{ub} = q_{ub,h}$</td>
</tr>
</tbody>
</table>

Note: The representative cone resistance $q_{rh}$ for base resistance calculation is $q_r$ averaged using the Dutch technique (Figure 4.5): $q_{rb} = 0.5(q_{1a} + q_{1b})$, with $q_{1a} = 0.5(q_{1a} + q_{1b})$, $q_{1a}$ is average of the $q_r$ values over a vertical distance of $\lambda B$ below the pile base, $q_{1b}$ is average of the $q_r$ values over a vertical distance of $\lambda B$ below the pile base following a minimum path rule, and $q_{1b}$ is average of the $q_r$ values over a vertical distance of $8B$ above the pile base following a minimum path rule. The value of $q_{1a}$ is calculated for different $\lambda$ values ranging from 0.7 to 4.0, and the minimum value of $q_{1a}$ obtained is used in the calculation of $q_{ub}$. Additional information about the computation of $q_{1a}$ and $q_{1b}$ can be found in AASHTO (2020).

Note: $K_{fs}$ = correction factor (estimated from the chart provided by AASHTO (2020) as a function of $L_B$, penetrometer type (electrical versus mechanical), and pile material (steel, concrete, or timber)), $f_s$ = sleeve resistance, $L$ = embedded length of the pile, $B$ = width or diameter of the pile, $z$ = depth measured from the ground surface, and $q_r$ = cone resistance.
Figure 4.5 Dutch technique for estimation of $q_{cb}$ (modified from Schmertmann, 1978).

TABLE 4.18
MnDOT equations (Modified UniCone method) for the unit shaft and base resistances for displacement piles driven in sand (Dagger et al., 2018)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{UL}$</th>
<th>Ultimate Unit Base Resistance $q_{b,ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{UL} = q_{E} \theta_{pt} \sigma_{n0} (10^{0.732L} - 3.605)$</td>
<td>$q_{b,ult} = (10^{0.325L} - 1.218) q_{cb}$</td>
</tr>
<tr>
<td>Open-ended pipe pile</td>
<td>$L = \sqrt{(3.47 - \log Q_{tn})^2 + (1.22 + \log F_r)^2}$</td>
<td>$I_r$ is calculated using the same set of equations as those in the estimation of $q_{UL}$.</td>
</tr>
<tr>
<td>H-pile</td>
<td>$Q_{tn} = \left( \frac{q_t - \sigma_0}{p_{at}} \right) \left( \frac{p_{at}}{\sigma_n} \right)^n$ and $F_r = \frac{f_r}{q_t - \sigma_0} \times 100%$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$n = \min \left[ 0.381 I_r + 0.05 \left( \frac{\sigma_n}{p_{at}} \right) - 0.15 ; 1 \right]$</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method predicts the maximum load capacity $Q_{max}$ of the pile (i.e., the maximum load applied on the piles considered in the database). For most (> 90%) of the pile load tests considered in the database, the value of $Q_{max}$ was nearly equal to the value of $Q_{ult}$ based on the 10% relative settlement criterion (i.e., the load corresponding to a pile head settlement $w$ equal to 10% of the pile diameter $B$). The following adjustment was proposed to estimate $Q_{ult}$ from $Q_{max}$: $Q_{ult} = 0.986 Q_{max}$ (Niazi & Mayne, 2016).

The value of the exponent $n$ is approximately equal to 1 for clay, 0.75 for silt, and 0.5 for sand. For mixed or intermediate soils, iterative calculations are needed to determine the value of $I_r$. For the first iteration, the method recommends the use of $n = 1$ to obtain an initial value of $I_r$ at the depth being considered. In the next iteration, this initial value of $I_r$ is used to update the value of $n$, which is then used to obtain a new value of $I_r$. The process is repeated until the value of $I_r$ converges, which is generally after the third cycle.

The ultimate base capacity $Q_{b,ult}$ of an open-ended pipe pile is calculated using the gross cross-sectional area ($\pi B^2/4$) of the pile base. The representative cone resistance $q_{cb}$ for base resistance calculation is $q_{E}$ averaged over a vertical distance of $B$ below the pile base (Dagger et al., 2018).

Notation: MnDOT = Minnesota Department of Transportation, $B$ = pile diameter, $q_{E}$ = effective cone resistance ($= q_{t} - u_2$); $q_t$ = corrected total cone resistance; $f_r$ = sleeve resistance; $u_2$ = pore water pressure measured at the shoulder position behind the cone face; $I_r$ = soil behavior type index; $Q_{tn}$ = normalized cone resistance; $F_r$ = normalized friction ratio; $\sigma_n$ and $\sigma_0$ = in situ vertical total and effective stresses, respectively, at the depth being considered; $p_{at}$ = reference stress (= 100 kPa or 14.5 psi); $\theta_{pt}$ = coefficient for pile type (= 1.13 for driven piles); $\theta_{rc}$ = coefficient for loading direction (= 0.85 for tension and 1.11 for compression); and $\theta_{rate}$ = coefficient for loading procedure (= 1.09 for constant rate of penetration test and 0.97 for maintained load test).
TABLE 4.19
UPDM equations for the unit shaft and base resistances for displacement piles driven in sand (Lehane et al., 2020)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{uL}$</th>
<th>Ultimate Unit Base Resistance $q_{b,ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{uL} = \frac{f_{1}}{f_{c}} \left( \sigma_{\gamma} + \Delta \sigma_{\gamma} \right) \tan \delta_{c}$</td>
<td>$q_{b,ult} = 0.5q_{b,b}$</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{\gamma} = \frac{q_{c} q}{44} \left( \max \left( \frac{h}{B} \right) \right)^{0.4}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\Delta \sigma_{\gamma} = 0.1q_{c} \left( \frac{d_{s}}{B} \right)^{-0.33}$</td>
<td></td>
</tr>
<tr>
<td>Open-ended pipe pile</td>
<td>$q_{uL} = \frac{f_{1}}{f_{c}} \left( \sigma_{\gamma} + \Delta \sigma_{\gamma} \right) \tan \delta_{c}$</td>
<td>$q_{b,ult} = (0.12 + 0.38A_{b})q_{b,b}$</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{\gamma} = \frac{q_{c} q}{44} \left( A_{n} \right)^{0.1} \left( \max \left( \frac{h}{B} \right) \right)^{0.4}$</td>
<td>$A_{b} = 1 - \text{FFR} \left( \frac{B}{B} \right)^{2}$</td>
</tr>
</tbody>
</table>
|                        | $\Delta \sigma_{\gamma} = 0.1q_{c} \left( \frac{d_{s}}{B} \right)^{-0.33}$                        | FFR is the final filling ratio, which is defined as the average incremental filling ratio measured over the final 3B of pile driving; if not measured, it can be roughly approximated by using the same equation for the PLR.

Note: The method predicts the ultimate load capacity $Q_{ub}$ of the pile corresponding to a pile base settlement equal to 10% of the pile diameter. In addition, the method is intended to predict the pile capacity measured 14 days after driving.

The method considers open-ended pipe piles in sand to behave as fully-plugged piles during static loading. Accordingly, the ultimate base capacity $Q_{ub,ult}$ of an open-ended pipe pile is calculated using the gross cross-sectional area ($\pi B^{2}/4$) of the pile base.

For piles installed in relatively homogeneous sands, the representative cone resistance $q_{c}$ for base resistance calculation is $q_{c}$ averaged from 1.5B above to 1.5B below the pile base. For piles installed in highly variable soil profiles (i.e., when $q_{c}$ varies significantly in the vicinity of the pile base), $q_{c}$ can be either taken as 1.2$q_{k,Dutch}$ or estimated using the procedure developed by Boulanger and DeJong (2018); $q_{k,Dutch}$ is $q_{c}$ averaged using the Dutch technique (Schmertmann, 1978). For open-ended pipe piles, $B$ is replaced by $B_{eff} = B(A_{b}B)^{0.75}$ in the calculation of $q_{b,b}$.

Notation: UPDM = Unified pile design method, $f_{c}/f_{1} = \text{ratio of tension to compression capacity} (\geq 0.75$ for tension and 1.0 for compression), $\sigma_{\gamma} = \text{local radial effective stress acting on the pile segment after installation}$, $\Delta \sigma_{\gamma} = \text{increase in local radial effective stress associated with constrained dilation during pile loading}$, $\sigma_{\gamma} = \text{in situ vertical effective stress at the depth being considered}$, $\delta_{c} = \text{critical-state interface friction angle}$ ($\approx 29^\circ$ in the absence of laboratory interface shear test results), $A_{n} = \text{effective shaft area ratio}$, $A_{b} = \text{effective base area ratio}$, $B = \text{inner diameter of open-ended pipe pile}$, $B_{eff} = \text{effective pile diameter}$, $d_{s} = \text{cone diameter}$, $h = \text{vertical distance from the pile base to the depth being considered}$, and $q_{c} = \text{concrete resistance}$.

4.5.3 Design Methods for Displacement Piles in Clayey Soil

TABLE 4.20
PPDM equations for the unit shaft and base resistances for displacement piles driven in clay (Basu et al., 2009; Salgado, 2008)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{uL}$</th>
<th>Ultimate Unit Base Resistance $q_{b,ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{uL} = 2s_{u}$</td>
<td>$q_{b,ult} \approx \begin{cases} 10s_{u} &amp; \text{for short-term resistance} \ 12s_{u} &amp; \text{for long-term resistance} \end{cases}$</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{\gamma} = A_{1} + (1 - A_{1}) \exp \left[ - \frac{\sigma_{\gamma}}{P_{A}} \left( \phi_{c} - \phi_{c,\text{min}} \right) t_{c} \right]$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_{1} = 1.28 \left( \frac{s_{u}}{\sigma_{\gamma}} \right)^{-0.05} A_{1} + (1 - A_{1}) \exp \left[ - \frac{\sigma_{\gamma}}{P_{A}} \left( \phi_{c} - \phi_{c,\text{min}} \right) t_{c} \right]$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_{1} = 0.75$ for $\phi_{c} - \phi_{c,\text{min}} \leq 5^\circ$; 0.43 for $\phi_{c} - \phi_{c,\text{min}} \geq 12^\circ$ and a linearly interpolated value for $5^\circ &lt; \phi_{c} - \phi_{c,\text{min}} &lt; 12^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_{1} = 0.55 + 0.43 \ln \left( \frac{s_{u}}{\sigma_{\gamma}} \right)$ and $A_{1} = 0.64 + 0.40 \ln \left( \frac{s_{u}}{\sigma_{\gamma}} \right)$</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method predicts the ultimate load capacity $Q_{ub}$ of the pile corresponding to a pile head settlement equal to 10% of the pile diameter $B$.

Short-term resistance refers to the resistance available immediately after pile installation (corresponding to zero dissipation of excess pore water pressure). Long-term resistance refers to the resistance available after dissipation of the excess pore water pressure generated during pile installation.

Notation: PPDM = Purdue pile design method, $P_{A} = \text{reference stress} \geq 100 \text{ kPa or } 14.5 \text{ psi}$, $\sigma_{\gamma} = \text{in situ vertical effective stress at the depth being considered}$, $\phi_{c} = \text{critical-state friction angle}$, $\phi_{c,\text{min}} = \text{minimum residual-state friction angle}$ (Appendix E), and $s_{u} = \text{undrained shear strength}$ (estimated from CPT results using the equations provided in the chapter).
TABLE 4.21
ICPDM equations for the unit shaft and base resistances for displacement piles driven in clay (Jardine et al., 2005)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance $q_{uL}$</th>
<th>Ultimate Unit Base Resistance $q_{u,ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{uL} = F_{\text{load}} K \sigma'_v \tan \delta_v$</td>
<td>$q_{u,ult} = \begin{cases} 0.8q_{uL} &amp; \text{for undrained loading} \ 1.3q_{uL} &amp; \text{for drained loading} \end{cases}$</td>
</tr>
<tr>
<td></td>
<td>$K = [2.2 + 0.016 \text{OCR} - 0.87 \Delta p] \text{OCR}^{0.42} \left( \max \left[ \frac{h}{R}, 8 \right] \right)^{-0.20}$</td>
<td></td>
</tr>
<tr>
<td>Open-ended pipe pile</td>
<td>Use the same equations as for closed-ended pipe pile but with an equivalent pile radius $R$ given by:</td>
<td>$\Delta p = \log_{10} S_i$ and $S_i = \frac{S_u}{S_0}$</td>
</tr>
<tr>
<td></td>
<td>$R = \sqrt{R_o^2 - R_i^2}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_b = 2b_0 t_f + (2X_p + t_w)(d - 2t_f)$</td>
<td>$q_{u,ult} = \begin{cases} 0.45q_{uL} &amp; \text{for undrained loading} \ 0.65q_{uL} &amp; \text{for drained loading} \end{cases}$</td>
</tr>
<tr>
<td></td>
<td>$X_p = b_t/8$ if $b_t/2 &lt; (d - 2t_f) &lt; b_t$, and $X_p = b_t/8d - 2t_f)$ if $(d - 2t_f) \geq b_t$</td>
<td>$Q_{b,ult} = q_{u,ult} \pi \left( R_0^2 - R_i^2 \right)$</td>
</tr>
<tr>
<td></td>
<td>$R = \sqrt{A_b / \pi}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$q_{u,ult} = 0.7q_{uL}$</td>
<td></td>
</tr>
<tr>
<td>Square or rectangular pile</td>
<td>Use the same equations as for closed-ended pipe pile but with an equivalent pile radius $R$ given by:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_b = B_wB_t$, where $B_w$ and $B_t$ = width and length, respectively, of the pile cross-section (in plan)</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method predicts the ultimate load capacity $Q_{u,ult}$ of the pile corresponding to a pile head settlement $w$ equal to 10% of the pile diameter $B$. In addition, the method is intended to estimate the shaft resistance after dissipation of the excess pore water pressure generated during pile installation. The representative cone resistance $q_{uL}$ for base resistance calculation is $q_t$ averaged from 1.5$B$ above to 1.5$B$ below the pile base.

The residual interface friction angle $\delta_v$ can be determined from the results of ring shear interface tests performed for the applicable value of normal effective stress (Ramsey et al., 1998). If such test results are unavailable, it is possible to estimate the value of $\delta_v$ by recognizing that it varies with the normal effective stress $\sigma'_v$ acting on the pile shaft, which, for production piles, is typically rough, so that $\delta_v$ is approximately equal to $\phi_v$. Note that $\sigma'_v$ in the context of pile shaft resistance calculation is the horizontal effective stress $\sigma'_h$ on the pile operative at the time of shearing: $\sigma'_h = F_{\text{load}} K \sigma'_v$.

Notation: ICPDM = Imperial College pile design method, $F_{\text{load}} = 0.8$ regardless of the loading direction, $p_A = \text{reference stress (100 kPa or 14.5 psi)}, L_R = \text{reference length (-1 m or 39.4 in.)}, q_t = \text{corrected total cone resistance}, \sigma'_v = \text{in situ vertical effective stress at the depth being considered}, A_b = \text{area of pile base}, B_i = \text{inner diameter of open-ended pipe pile}, d_i = \text{cone diameter}, R = \text{pile radius}, h = \text{vertical distance from the pile base to the depth being considered}, \text{OCR} = \text{overconsolidation ratio}, \sigma_u = \text{undrained shear strength}, \Delta p_v = \text{relative void index at yield in c-log $\sigma'_v$ space}, S_i = \text{sensitivity}, S_w = \text{remolded undrained shear strength}, LI = \text{liquidity index} = (wc - PL)/PL, wc = \text{water content}, PL = \text{plastic limit}, PI = \text{plasticity index}, R_o = \text{outer radius of open-ended pipe pile}, R_i = \text{inner radius of open-ended pipe pile}, b_f = \text{width of flange}, d = \text{depth of H-section}, t_f = \text{thickness of flange}, t_w = \text{thickness of web},$ and $q_{u,ult} = \text{ultimate unit annulus resistance.}$
Schmertmann, 1975

The residual interface friction angle \(\delta_r\) can be determined from the results of ring shear interface tests performed for the applicable value of normal effective stress (Ramsey et al., 1998). If such test results are unavailable, it is possible to estimate the value of \(\delta_r\) by recognizing that it varies with the normal effective stress \(\sigma'\) acting on the pile shaft, which, for production piles, is typically rough, so that \(\delta_r\) is approximately equal to \(\phi_r\). Note that \(\sigma'\), in the context of pile shaft resistance calculation, is the horizontal effective stress \(\sigma'_h\) on the pile operative at the time of shearing: \(\sigma'_h = 0.23q\max(h/R;1)^{-0.2}(q/\sigma'_{90})^{0.15}\).

Note: UWAPDM = University of Western Australia pile design method, \(q_t = \) corrected, total cone resistance, \(\sigma'_{90} = \) in situ vertical effective stress at the depth being considered, \(R = \) pile radius, \(h = \) vertical distance from the pile base to the depth being considered, \(R_o = \) outer radius of open-ended pipe pile, and \(R_i = \) inner radius of open-ended pipe pile.

### TABLE 4.22
UWAPDM equations for the unit shaft and base resistances for displacement piles driven in clay (Lehane, 2019; Lehane et al., 2013)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance (q_{u_L})</th>
<th>Ultimate Unit Base Resistance (q_{b,ul})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>(q_{u_L} = \frac{0.23q\max(h/R;1)^{-0.2}}{\tan\delta_r} ) or (q_{u_L} = 0.055q\max(h/R;1)^{-0.2})</td>
<td>(q_{b,ul} = 0.5q_{u_L}) for undrained loading</td>
</tr>
<tr>
<td>Open-ended pipe pile</td>
<td>Use the same equations as for closed-ended pipe pile but with an equivalent pile radius (R) given by:</td>
<td>Response as a plugged pile during static loading: (q_{b,ul} = 0.5q_{u_L}) for undrained loading</td>
</tr>
<tr>
<td></td>
<td>(R = \sqrt{R_o^2 - R_i^2})</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method is intended to estimate the shaft resistance after dissipation of the excess pore water pressure generated during pile installation (Lehane, 2019; Lehane et al., 2017). Two equations were proposed for the limit unit shaft resistance \(q_{u_L}\) and the second one was reported by Lehane et al. (2013) to be slightly more reliable than the first. The ultimate base capacity \(Q_{b,ul}\) of an open-ended pipe pile is calculated using the gross cross-sectional area \((\pi R^2/4)\) of the pile base.

### TABLE 4.23
AASHTO equations for the unit shaft and base resistances for displacement piles driven in clay (AASHTO, 2020; Nottingham & Schmertmann, 1975)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Limit Unit Shaft Resistance (q_{u_L})</th>
<th>Limit Unit Base Resistance (q_{b,L})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pile</td>
<td>(q_{u_L} = \begin{cases} 0.125K_sf_s(z/B) &amp; \text{for } 0 \leq z \leq 8B \ K_sf_z &amp; \text{for } 8B \leq z \leq L \end{cases} )</td>
<td>(q_{b,L} = \min(0.5q_{u,L}, 0.5q_{u,L} + q_{b,1a}))</td>
</tr>
</tbody>
</table>

Note: The representative cone resistance \(q_{b,1}\) for base resistance calculation is \(q_t\) averaged using the Dutch technique (Figure 4.5): \(q_{b,1} = 0.5(q_{b,1} + q_{b,2})\), with \(q_{b,1} = 0.5(q_{b,1a} + q_{b,1b})\), \(q_{b,2} = \) average of the \(q_t\) values over a vertical distance of \(\lambda B\) below the pile base, \(q_{b,1b} = \) average of the \(q_t\) values over a vertical distance of \(\lambda B\) below the pile base following a minimum path rule, and \(q_{b,2} = \) average of the \(q_t\) values over a vertical distance of \(8B\) above the pile base following a minimum path rule. The value of \(q_{b,1}\) is calculated for different \(\lambda\) values ranging from 0.7 to 4.0, and the minimum value of \(q_{b,1}\) obtained is used in the calculation of \(q_{b,1}\). Additional information about the computation of \(q_{b,1}\) and \(q_{b,2}\) can be found in AASHTO (2020).

Note: \(K_s = \) correction factor [estimated from the chart provided by AASHTO (2020) as a function of \(f_s\) and pile material (steel, concrete, or timber)], \(f_s = \) sleeve resistance, \(L = \) embedded length of the pile, \(B = \) width or diameter of the pile, \(z = \) depth measured from the ground surface, and \(q_t = \) corrected, total cone resistance (Eq. 4.1).
TABLE 4.24
MnDOT equations (Modified UniCone method) for the unit shaft and base resistances for displacement piles driven in clay (Dagger et al., 2018)

<table>
<thead>
<tr>
<th>Pile Type and Reference</th>
<th>Unit Shaft Resistance</th>
<th>Ultimate Unit Base Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{uL} = q_{c} \theta_{f} \theta_{s} \theta_{L}$ ($10^{0.722L - 3.605}$)</td>
<td>$q_{ultL} = (10^{0.225L - 1.181}) q_{c}$</td>
</tr>
<tr>
<td>Open-ended pipe pile H-pile</td>
<td>$I_{L} = \sqrt{(3.47 - \log Q_{c})^{2} + (1.22 + \log F_{c})^{2}}$</td>
<td>$I_{ult} = \sqrt{(3.47 - \log Q_{ultc})^{2} + (1.22 + \log F_{ultc})^{2}}$</td>
</tr>
<tr>
<td></td>
<td>$Q_{ult} = \left( \frac{q_{ultc} - \sigma_{0}}{p_{A}} \right) \left( \frac{p_{A}}{p_{A}} \right)^{n} \frac{\sigma_{0}}{p_{A}}$</td>
<td>$Q_{ultult} = \left( \frac{q_{ultultc} - \sigma_{0}}{p_{A}} \right) \left( \frac{p_{A}}{p_{A}} \right)^{n} \frac{\sigma_{0}}{p_{A}}$</td>
</tr>
<tr>
<td></td>
<td>$n = \min \left[ 0.381L + 0.05 \left( \frac{q_{ultc}}{p_{A}} \right) - 0.15 ; 1 \right]$</td>
<td>$n = \min \left[ 0.381L + 0.05 \left( \frac{q_{ultultc}}{p_{A}} \right) - 0.15 ; 1 \right]$</td>
</tr>
</tbody>
</table>

Note: The method predicts the maximum load capacity $Q_{max}$ of the pile (i.e., the maximum load applied on the piles considered in the database). For most (> 90%) of the pile load tests considered in the database, the value of $Q_{max}$ was nearly equal to the value of $Q_{ult}$ based on the 10% relative settlement criterion (i.e., the load corresponding to a pile head settlement $\eta$ equal to 10% of the pile diameter $B$). The following adjustment was proposed to estimate $Q_{ult}$ from $Q_{max}$: $Q_{ult} = 0.986 Q_{max}$ (Niazi & Mayne, 2016).

The value of the exponent $n$ is approximately equal to 1 for clay, 0.75 for silt, and 0.5 for sand. For mixed or intermediate soils, iterative calculations are needed to determine the value of $I_{L}$. For the first iteration, the method recommends the use of $n = 1$ to obtain an initial value of $I_{L}$ at the depth being considered. In the next iteration, this initial value of $I_{L}$ is used to update the value of $n$, which is then used to obtain a new value of $I_{L}$. The process is repeated until the value of $I_{L}$ converges, which is generally after the third cycle. Additional information on sensitive clays can be found in Niazi and Mayne (2016).

The ultimate base capacity $Q_{ultult}$ of an open-ended pipe pile is calculated using the gross cross-sectional area ($\pi B^{2}/4$) of the pile base. The representative cone resistance $q_{ultc}$ for base resistance calculation is $q_{c}$ averaged over a vertical distance of $B$ below the pile base (Dagger et al., 2018).

Notation: MnDOT = Minnesota Department of Transportation, $B$ = pile diameter, $q_{c}$ = effective cone resistance ($= q_{t} - u_{z}$); $q_{t}$ = corrected, total cone resistance; $f_{s}$ = sleeve resistance; $u_{z}$ = pore water pressure measured at the shoulder position behind the cone face; $I_{c}$ = soil behavior type index; $Q_{ult}$ = normalized cone resistance; $F_{c}$ = normalized friction ratio; $\sigma_{0}$ and $\sigma_{0}'$ = in situ vertical total and effective stresses, respectively, at the depth being considered; $p_{A}$ = reference stress ($= 100$ kPa or 14.5 psi); $\theta_{f}$ = coefficient for pile type ($= 1.13$ for driven piles); $\theta_{s}$ = coefficient for loading direction ($= 0.85$ for tension and 1.11 for compression); and $\theta_{ultult}$ = coefficient for loading procedure ($= 1.09$ for constant rate of penetration test and 0.97 for maintained load test).

TABLE 4.25
NDOT equations for the unit shaft and base resistances for displacement piles driven in clay (Song et al., 2019)

<table>
<thead>
<tr>
<th>Pile Type and Reference</th>
<th>Unit Shaft Resistance</th>
<th>Unit Base Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Closed-ended pipe pile</td>
<td>$q_{L} = \frac{q_{t}}{60}$</td>
<td>$q_{b} = 0.54 q_{b}$</td>
</tr>
<tr>
<td>Precast prestressed concrete pile (modified from de Ruiter and Beringen, 1979)</td>
<td>$q_{L} = \min \left( m \frac{f_{s,avg}}{m}\right)$; $0.72 p_{A}$</td>
<td>$q_{b} = \min \left( 0.5q_{b}; 150 p_{A} \right)$</td>
</tr>
<tr>
<td>$m^{*} = 0.45 + 8.55 \exp \left( -0.09 f_{s,avg} \right)$</td>
<td>$m^{*} = 0.45 + 8.55 \exp \left( -0.09 f_{s,avg} \right)$</td>
<td></td>
</tr>
<tr>
<td>$f_{s,avg} = \frac{\sum_{i=1}^{n} f_{s} \Delta z_{i}}{\sum_{i=1}^{n} \Delta z_{i}}$</td>
<td>$f_{s,avg} = \frac{\sum_{i=1}^{n} f_{s} \Delta z_{i}}{\sum_{i=1}^{n} \Delta z_{i}}$</td>
<td></td>
</tr>
</tbody>
</table>

Note: The method is applicable to fine-grained Nebraska soils and predicts the pile capacity that would be obtained from dynamic load tests performed using the pile driving analyzer (PDA) at the end of initial driving and post-processed using the signal matching program CAPWAP (Case Pile Wave Analysis Program).

In the de Ruiter and Beringen (1979) method, the representative cone resistance $q_{c}$ for base resistance calculation is $q_{t}$ averaged using the Dutch technique (Figure 4.5). In the Tumay and Fakhroo (1982) method, $q_{b}$ is calculated in a manner similar to the Dutch technique: $q_{b} = 0.5(q_{1}+q_{2})$, with $q_{1} = 0.5(q_{1a}+q_{1b})$. $q_{1a}$ = average of the $q_{1}$ values over a vertical distance of $4B$ below the pile base, $q_{1b}$ = average of the $q_{1}$ values over a vertical distance of $4B$ below the pile base following a minimum path rule, and $q_{2}$ = average of the $q_{1}$ values over a vertical distance of $8B$ above the pile base following a minimum path rule.

Notation: NDOT = Nebraska Department of Transportation, $m^{*} = \text{modified friction coefficient}$, $f_{s,avg} = \text{weighted-average sleeve resistance}$, $f_{s} = \text{sleeve resistance of soil layer } i$, $\Delta z_{i} = \text{thickness of soil layer } i$, $n = \text{number of soil layers in contact with the pile shaft}$, $p_{A} = \text{reference stress}$ ($= 100$ kPa or 14.5 psi), $B = \text{pile diameter}$, and $q_{t}$ = corrected, total cone resistance (Eq. 4.1).


Madhira, M., & Sakleshpur, V. A. (2019). Mining geotechnical parameters from the ground. 8th Annual Praphulla Kumar Lecture (pp. 1–35). Indian Geotechnical Society (Kochi Chapter).


APPENDICES

Appendix A. Critical-State Friction Angle of Sand

Appendix B. OCR and $K_0$ of Soil

Appendix C. Iterative Scheme for Footing Settlement in Sand

Appendix D. Penetration Rate Effect on Cone Resistance

Appendix E. Residual-State Friction Angle of Clay
APPENDIX A. CRITICAL-STATE FRICTION ANGLE OF SAND

The critical-state friction angle $\phi_c$ is simply the friction angle that a given soil has at critical state. It is independent of soil state (i.e., relative density and confining stress) but depends on particle size (e.g., $D_{50}$), morphology (e.g., roundness $R$ and sphericity $S$), mineralogy (e.g., silicates versus carbonates), and gradation (e.g., coefficient of uniformity $C_U$) (Han et al., 2018; Salgado, 2008). The value of $\phi_c$ for a silica sand typically ranges from $28^\circ$–$36^\circ$; sands with rounded, smooth particles with a poorly-graded particle size distribution have values near the low end of this range, while sands with angular, rough particles with a well-graded particle size distribution have values near the high end of this range (Salgado, 2008). In contrast, the value of $\phi_c$ for a carbonate sand typically ranges from $37^\circ$–$44^\circ$ (Altuhafi et al., 2016; Coop & Lee, 1993; Salgado, 2008).

A.1 Roundness

Roundness is a measure of sharpness of the particle corners (Figure A.1). It is defined as the ratio of the average radius of curvature of the corners of a 2D projection of the particle to the radius $r_{ins}$ of the largest inscribed circle for the same projection (Wadell, 1932):

$$ R = \frac{\sum_{i=1}^{N} r_i}{N} $$

(Eq. A.1)

where $r_i =$ radius of curvature of corner $i$ of the particle, and $N =$ number of particle corners. Table A.1 summarizes the different roundness classes proposed by Powers (1953).

![Figure A.1 Definition of roundness for a 2D projected outline of a particle (Hryciw et al., 2016; Wadell, 1932).]
Table A.1 Classification of particles based on roundness (Powers, 1953)

<table>
<thead>
<tr>
<th>Roundness Class</th>
<th>Roundness Interval</th>
<th>Mean Roundness¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very angular</td>
<td>0.12–0.17</td>
<td>0.14</td>
</tr>
<tr>
<td>Angular</td>
<td>0.17–0.25</td>
<td>0.21</td>
</tr>
<tr>
<td>Subangular</td>
<td>0.25–0.35</td>
<td>0.30</td>
</tr>
<tr>
<td>Subrounded</td>
<td>0.35–0.49</td>
<td>0.41</td>
</tr>
<tr>
<td>Rounded</td>
<td>0.49–0.70</td>
<td>0.59</td>
</tr>
<tr>
<td>Well-rounded</td>
<td>0.70–1.00</td>
<td>0.84</td>
</tr>
</tbody>
</table>

¹Geometric mean

A.2 Sphericity

Sphericity is a measure of the extent to which a particle resembles the shape of a sphere. Particle sphericity has been defined in several ways in the literature (Mitchell & Soga, 2005; Rodríguez et al., 2012); three widely used definitions are detailed below.

1. **Diameter sphericity** $S_D$: It is defined as the ratio of the diameter $D_c$ of a circle having the same area as the projected 2D area of the particle to the diameter $D_{cir}$ of the smallest circle circumscribed about the 2D projection of the particle (Wadell, 1933):

   $$ S_D = \frac{D_c}{D_{cir}} \quad \text{(Eq. A.2)} $$

2. **Width-to-length ratio sphericity** $S_{WL}$: It is defined as the ratio of the width $d_2$ to the length $d_1$ of the particle (Zheng & Hryciw, 2015):

   $$ S_{WL} = \frac{d_2}{d_1} \quad \text{(Eq. A.3)} $$

   The length $d_1$ and width $d_2$ of the particle are defined as the largest and smallest dimensions, respectively, of a rectangle enclosing the particle; the selected rectangle is the one with the largest possible dimension circumscribing the particle. The reciprocal of the width-to-length ratio sphericity is commonly referred to as the elongation ratio.

3. **Perimeter sphericity** $S_P$: It is defined as the ratio of the perimeter $P_c$ of a circle having the same area as the projected 2D area $A$ of the particle to the projected perimeter $P$ of the particle (Altuhafi et al., 2013):

   $$ S_P = \frac{P_c}{P} = \frac{2\sqrt{\pi A}}{P} \quad \text{(Eq. A.4)} $$

   Figure A.2 illustrates the definitions of diameter sphericity $S_D$ and width-to-length ratio sphericity $S_{WL}$. Figure A.3 shows a chart developed by Krumbein and Sloss (1951) with 20 reference particle silhouettes having roundness and sphericity values ranging from 0.1–0.9 and 0.3–0.9, respectively, in increments of 0.2. If access to digital, computer-based tools, such as ImageJ and MATLAB, is limited, the chart can be used to estimate particle roundness and sphericity by comparing the shapes of individual particles viewed under a microscope with the
reference particle silhouettes given in the chart. The sphericity obtained from the Krumbein and Sloss (1951) chart is the width-to-length ratio sphericity $S_{WL}$ (Zheng & Hryciw, 2015).

![Diagram](image)

Figure A.2 Illustrations of (a) diameter $D_{cir}$ of the smallest circle circumscribed about the 2D projection of the particle, and (b) length $d_1$ and width $d_2$ of the particle.

![Chart](image)

Figure A.3 Chart for estimating roundness and sphericity (Krumbein & Sloss, 1951).

### A.3 Silica Sand Database

Table A.2 summarizes the intrinsic parameters of 23 clean silica sands reported in the literature. The parameters include mean particle size $D_{50}$, coefficient of uniformity $C_U$, roundness $R$, sphericity $S$, minimum void ratio $e_{min}$, maximum void ratio $e_{max}$, and critical-state friction angle

**Table A.2**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean $D_{50}$</td>
<td>0.20 mm</td>
</tr>
<tr>
<td>Coefficient $C_U$</td>
<td>1.5</td>
</tr>
<tr>
<td>Roundness $R$</td>
<td>0.8</td>
</tr>
<tr>
<td>Sphericity $S$</td>
<td>0.7</td>
</tr>
<tr>
<td>Minimum $e_{min}$</td>
<td>0.25</td>
</tr>
<tr>
<td>Maximum $e_{max}$</td>
<td>0.35</td>
</tr>
<tr>
<td>Friction angle</td>
<td>30 degrees</td>
</tr>
</tbody>
</table>
\( \phi \) in triaxial compression. All the sands are poorly-graded, except FS Ohio SW, which is classified as well-graded according to the Unified Soil Classification System (USCS) (ASTM, 2012). The number designations for some of the uniform sands (e.g., Ottawa 20–30) listed in Table A.2 indicate the sieve numbers between which the sand particles were retained. The \( D_{50}, C_U, \) and \( R \) values for the sands are in the range of 0.15–2.68 mm (0.006–0.105 in.), 1.2–7.9, and 0.3–0.8, respectively. Although different researchers have defined particle sphericity in different ways for the sands listed in Table A.2, the \( S \) values were found to lie within a relatively narrow range of 0.65–0.90 regardless of the definition used. Zheng and Hryciw (2016) also found the \( S \) values to lie within a similar range for the sands considered in their database. They reasoned that sand particles are usually bulky in nature and that slender, elongated sand particles are rarely found in practice because such particles are susceptible to breakage.
Table A.2 Intrinsic parameters of 23 clean silica sands reported in the literature

<table>
<thead>
<tr>
<th>Sand</th>
<th>Gradation</th>
<th>Morphology</th>
<th>Packing</th>
<th>Strength</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$D_{50}$ (mm)</td>
<td>$C_U$</td>
<td>$R$</td>
<td>$S$</td>
<td>$\varepsilon_{\text{min}}$</td>
</tr>
<tr>
<td>FS Ohio 6–10</td>
<td>2.68</td>
<td>1.31</td>
<td>0.43</td>
<td>0.86</td>
<td>0.66</td>
</tr>
<tr>
<td>FS Ohio 10–16</td>
<td>1.59</td>
<td>1.30</td>
<td>0.44</td>
<td>0.83</td>
<td>0.65</td>
</tr>
<tr>
<td>FS Ohio 16–20</td>
<td>1.01</td>
<td>1.25</td>
<td>0.40</td>
<td>0.78</td>
<td>0.66</td>
</tr>
<tr>
<td>FS Ohio 20–40</td>
<td>0.63</td>
<td>1.42</td>
<td>0.39</td>
<td>0.82</td>
<td>0.62</td>
</tr>
<tr>
<td>FS Ohio 50–100</td>
<td>0.23</td>
<td>1.56</td>
<td>0.35</td>
<td>0.82</td>
<td>0.63</td>
</tr>
<tr>
<td>FS Ohio Coarse</td>
<td>1.50</td>
<td>2.00</td>
<td>—</td>
<td>—</td>
<td>0.45</td>
</tr>
<tr>
<td>FS Ohio Fine</td>
<td>0.35</td>
<td>2.00</td>
<td>—</td>
<td>—</td>
<td>0.48</td>
</tr>
<tr>
<td>FS Ohio SW</td>
<td>1.04</td>
<td>7.90</td>
<td>—</td>
<td>—</td>
<td>0.37</td>
</tr>
<tr>
<td>Fontainebleau NE34</td>
<td>0.21</td>
<td>1.53</td>
<td>0.45</td>
<td>0.75^2</td>
<td>0.51</td>
</tr>
<tr>
<td>Fraser River</td>
<td>0.30</td>
<td>2.40</td>
<td>0.43</td>
<td>0.83</td>
<td>0.68</td>
</tr>
<tr>
<td>Ham River</td>
<td>0.30</td>
<td>1.59</td>
<td>0.45</td>
<td>0.65^2</td>
<td>0.59</td>
</tr>
<tr>
<td>Lausitz</td>
<td>0.25</td>
<td>3.09</td>
<td>0.51</td>
<td>—</td>
<td>0.44</td>
</tr>
<tr>
<td>Leighton Buzzard</td>
<td>0.78</td>
<td>1.27</td>
<td>0.75</td>
<td>0.80^2</td>
<td>0.51</td>
</tr>
<tr>
<td>Longstone</td>
<td>0.15</td>
<td>1.43</td>
<td>0.30</td>
<td>0.65^2</td>
<td>0.61</td>
</tr>
<tr>
<td>M31</td>
<td>0.28</td>
<td>1.54</td>
<td>0.62</td>
<td>0.70^2</td>
<td>0.53</td>
</tr>
<tr>
<td>Monterey No. 0</td>
<td>0.38</td>
<td>1.58</td>
<td>—</td>
<td>0.89^3</td>
<td>0.53</td>
</tr>
<tr>
<td>Ohio Gold Frac</td>
<td>0.62</td>
<td>1.60</td>
<td>0.43</td>
<td>0.83</td>
<td>0.58</td>
</tr>
<tr>
<td>Ottawa Graded</td>
<td>0.31</td>
<td>1.89</td>
<td>0.80^4</td>
<td>0.90^4</td>
<td>0.49</td>
</tr>
<tr>
<td>Ottawa 20–30</td>
<td>0.72</td>
<td>1.18</td>
<td>0.72</td>
<td>0.88</td>
<td>0.50</td>
</tr>
<tr>
<td>Q-Rok^4</td>
<td>0.63</td>
<td>1.50</td>
<td>0.40</td>
<td>0.73</td>
<td>0.70</td>
</tr>
<tr>
<td>Sacramento River</td>
<td>0.30</td>
<td>1.80</td>
<td>—</td>
<td>0.88^3</td>
<td>0.53</td>
</tr>
<tr>
<td>Ticino</td>
<td>0.58</td>
<td>1.50</td>
<td>0.40</td>
<td>0.80^2</td>
<td>0.57</td>
</tr>
<tr>
<td>Toyoura</td>
<td>0.17</td>
<td>1.70</td>
<td>0.35</td>
<td>0.65^2</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Note: $D_{50}$ = mean particle size, $C_U$ = coefficient of uniformity (= $D_{50}/D_{10}$), $\varepsilon_{\text{min}}$ = minimum void ratio, $\varepsilon_{\text{max}}$ = maximum void ratio, $R$ = roundness, $S$ = diameter sphericity $SD$ (unless otherwise indicated), and $\phi_c$ = critical-state friction angle in triaxial compression (unless otherwise indicated). The properties of INDOT No. 4 sand, which is a backfill material typically used for retaining wall construction in Indiana, are: $D_{50} = 0.85$ mm, $C_U = 4.58$, $R = 0.72$, $SWL = 0.72$, $\varepsilon_{\text{min}} = 0.29$, $\varepsilon_{\text{max}} = 0.54$, and $\phi_c = 38.0$° in direct shear (Rahman et al., 2020).

1 Obtained from direct shear test results.
3 Perimeter sphericity $SR$ (Altuhafi et al., 2013).
4 Unpublished research.
A.4 Simple Correlation

In the absence of direct shear (DS) or triaxial compression (TXC) test results, a simple approach to critical-state friction angle estimation is to use an equation of the form:

\[
\phi_c = C_1 \left( \frac{D_{50}}{D_{\text{ref}}} \right)^{C_2} (C_U)^{C_3} (R)^{C_4}
\]  
(Eq. A.5)

where \(D_{\text{ref}}\) = reference particle size (= 1 mm or 0.04 in.); and \(C_1, C_2, C_3, \) and \(C_4\) = regression coefficients. The values of \(C_1, C_2, C_3, \) and \(C_4\) were obtained by performing a least squares regression in Microsoft Excel. The following equation was found to fit the \(\phi_c\) values reported in Table A.2 quite well:

\[
\phi_c (\degree) = 28.3 \left( \frac{D_{50}}{D_{\text{ref}}} \right)^{\zeta} (C_U)^{2\zeta} (R)^{-3\zeta}
\]  
(Eq. A.6)

where \(\phi_c\) = critical-state friction angle in triaxial compression, and \(\zeta\) = exponent (= 0.045). The adjusted coefficient of determination \(r^2\), mean absolute error, and mean absolute percentage error are 0.89, 0.4°, and 1.3%, respectively. The adjusted \(r^2\) is a modified version of \(r^2\) that has been adjusted for the number of independent variables considered in the model. Equation A.6 is applicable for poorly-graded, clean silica sands with \(D_{50} = 0.15–2.68\) mm (0.006–0.105 in.), \(C_U = 1.2–3.1\), and \(R = 0.3–0.8\); however, it should be used with caution for (a) well-graded sands with \(C_U \geq 6\), (b) sands with \(D_{50}, C_U\) and \(R\) values that lie outside these ranges, and (c) sands with plastic or non-plastic fines greater than 5%. Equation A.6 could be further improved through future research.

Figure A.4 compares the critical-state friction angle predicted using Eq. A.6 with that obtained from TXC test results for the poorly-graded, clean silica sands listed in Table A.2. The differences between the predicted and measured \(\phi_c\) values are within 1°. The value of \(\phi_c\) predicted using Eq. A.6 may be decreased by a degree or two, if needed, to obtain a conservative estimate for use in foundation design. However, we re-emphasize that laboratory direct shear or triaxial compression test results provide the best means for estimating the critical-state friction angle of sands, particularly those that contain plastic or non-plastic fines greater than 5% (Carraro et al., 2009; Murthy et al., 2007).
Critical-state friction angle obtained from TXC tests (°)

Figure A.4 Comparison of critical-state friction angles obtained from Eq. A.6 and TXC tests on poorly-graded, clean silica sands.

To evaluate the performance of Eq. A.6 in an unbiased manner, a blind test was performed on two additional, poorly-graded, clean silica sands—Nerlerk sand and Fujian sand; these sands were not used in the development of Eq. A.6. The properties of Nerlerk sand are: $D_{50} = 0.23$ mm (0.009 in.), $C_U = 1.56$, $R = 0.43$, $S_{WL} = 0.75$, $e_{\text{min}} = 0.66$, $e_{\text{max}} = 0.89$, and $\phi_c = 30^\circ$ in triaxial compression (Sladen et al., 1985); the values of $R$ and $S_{WL}$ are based on Krumbein and Sloss (1951). The properties of Fujian sand are: $D_{50} = 0.40$ mm (0.016 in.), $C_U = 1.53$, $R = 0.55$, and $\phi_c = 30.8^\circ$ in triaxial compression (Yang & Wei, 2012). The critical-state friction angle of Nerlerk sand and Fujian sand obtained from Eq. A.6 is shown below.

**Nerlerk Sand**

$$\phi_c = 28.3 \left( \frac{D_{50}}{D_{\text{ref}}} \right)^\zeta \left( C_U \right)^{2\zeta} (R)^{-3\zeta} = 28.3 \times \left( \frac{0.009}{0.04} \right)^{0.045} \times (1.56)^{0.09} \times (0.43)^{-0.135} = 30.9^\circ$$

**Fujian Sand**

$$\phi_c = 28.3 \left( \frac{D_{50}}{D_{\text{ref}}} \right)^\zeta \left( C_U \right)^{2\zeta} (R)^{-3\zeta} = 28.3 \times \left( \frac{0.016}{0.04} \right)^{0.045} \times (1.53)^{0.09} \times (0.55)^{-0.135} = 30.6^\circ$$
The difference between the predicted and measured $\phi_c$ value is equal to 0.9° for Nerlerk sand and 0.2° for Fujian sand.

**A.5 Procedure for Estimation of $\phi_c$ from Intrinsic Soil Variables**

In the absence of direct shear or triaxial compression test results, the critical-state friction angle $\phi_c$ of a poorly-graded, clean silica sand may be estimated from intrinsic soil variables by following these steps.

1. Perform a sieve analysis test and obtain the particle-size distribution curve.
2. Determine the mean particle size $D_{50}$ and the coefficient of uniformity $C_U (= D_{60}/D_{10})$ from the particle-size distribution curve.
3. Determine the dominant particle size of the sand (i.e., the sieve size with the maximum percentage by mass of particles retained on the sieve).
4. Select a reasonable number of random particles (say 25 particles) from those retained on the sieve identified in step 3 and place them in an orderly fashion on a flat surface (e.g., glass slide). The number of random particles may be increased or decreased depending on how variable the morphology is from one particle to the next.
5. Execute one of the following methods, based on the desired level of sophistication, to determine particle roundness and sphericity.

*Method 1 (Visual)*

a. Observe the particles through a microscope.

b. Compare the observed shapes of the particles against the reference particle silhouettes given in the chart by Krumbein and Sloss (1951) (Figure A.3).

c. Determine the roundness $R$ and sphericity $S$ of each particle and average the values for all the particles selected.

*Method 2 (Computational)*

a. Observe the particles through a microscope and obtain high-resolution images of the particles using a digital camera attached to the microscope.


c. Determine the roundness $R$ and sphericity $S$ of each particle and average the values for all the particles selected.

APPENDIX B. OCR AND $K_0$ OF SOIL

B.1 Overconsolidation Ratio (OCR)

Laboratory consolidation tests, such as the oedometer test or the constant rate of strain (CRS) test, provide the best means of determining the OCR of clays. In addition, the OCR may be known from the site history (e.g., if soil was previously removed or structures were demolished at the site), or it may be deduced from geologic considerations or from in situ testing observations. A preliminary estimate of the OCR of clay can be obtained from CPT results using the following approximate correlation (Ladd et al., 1977; Salgado, 2008; Wroth, 1984):

\[
OCR = \max \left\{ \left[ \frac{(s_u/\sigma'_v)_\text{OC}}{(s_u/\sigma'_v)_\text{NC}} \right]^{1.25}; 1 \right\} \approx \max \left\{ \left[ \frac{q_{\text{in}}/N_k}{(s_u/\sigma'_v)_\text{NC}} \right]^{1.25}; 1 \right\} \quad (\text{Eq. B.1})
\]

where \( (s_u/\sigma'_v)_\text{OC} \) = normalized undrained shear strength of an OC clay; \( (s_u/\sigma'_v)_\text{NC} \) = normalized undrained shear strength of the same clay when normally consolidated (\( \approx 0.2–0.3 \) for most clays); \( q_{\text{in}} \) = normalized cone resistance (= \( q_t - \sigma_{\text{v0}}/\sigma'_v \)); \( q_t \) = corrected, total cone resistance measured under undrained conditions (Eq. 2.1); \( \sigma_{\text{v0}} \) and \( \sigma'_v \) = in situ vertical total and effective stresses, respectively, at the depth being considered; and \( N_k \) = cone factor (\( \approx 9–15 \) as long as the CPT is performed at a penetration rate that is sufficiently high to ensure undrained penetration (refer to Appendix D); soft NC clays tend to have \( N_k \) values near the low end of this range, while stiff OC clays tend to have \( N_k \) values near the high end of this range) (Bisht et al., 2021; Mayne & Peuchen, 2018; Salgado, 2008, 2013, 2014; Salgado et al., 2004). An average \( N_k \) value of 12 may be used in Eq. B.1 to obtain a preliminary estimate of the OCR.

The normalized undrained shear strength \( (s_u/\sigma'_v)_\text{NC} \) of an NC clay can be estimated using (Wroth, 1984):

\[
\left( \frac{s_u}{\sigma'_v} \right)_\text{NC} \approx \begin{cases} 
\frac{1.7 \sin \phi_c}{3 - \sin \phi_c} & \text{for CIUC conditions} \\
\frac{\sin \phi_c}{2a} \left( \frac{a^2 + 1}{2} \right)^\Lambda & \text{for CK}_0\text{UC conditions (Eq. B.2)}
\end{cases}
\]

where CIUC = isotropically-consolidated undrained triaxial compression, CK\( _0 \text{UC} = K_0 \)-consolidated undrained triaxial compression, \( \Lambda \) = plastic volumetric strain ratio (\( \approx 0.8 \)), and \( \phi_c \) = critical-state friction angle (\( \approx 15^\circ–30^\circ \) for most clays; high-plasticity clays with high smectite and clay contents tend to have values near the low end of this range, while low-plasticity clays with low smectite and clay contents tend to have values near the high end of this range (refer to Table E.1 of Appendix E)). An alternative expression that provides conservative estimates of \( (s_u/\sigma'_v)_\text{NC} \) for both CIUC and CK\( _0 \text{UC} \) test conditions is \( (s_u/\sigma'_v)_\text{NC} = \phi_c/100 \) (Kulhawy & Mayne, 1990; Salgado, 2008).

The OCR (\( = \sigma'_vp/\sigma'v \)) of sand may be evaluated based on the geologic history of the site, where \( \sigma'_vp \) = preconsolidation stress, which is the maximum vertical effective stress ever
experienced by the soil, and $\sigma'_v$ = current vertical effective stress. The reader may also refer to Section 2.3.7 of Volume I for additional information on the OCR.

**B.2 Coefficient of Lateral Earth Pressure At-Rest $K_0$**

The coefficient of lateral earth pressure at-rest $K_0$ of soil can be determined using (Brooker & Ireland, 1965):

$$K_0 = K_{0,NC}\sqrt{OCR}$$  \hspace{1cm} (Eq. B.4)

where $K_{0,NC}$ = value of $K_0$ if the soil is normally consolidated (= 0.40–0.50 for NC sand, with dense sands tending to have lower values and loose sands having higher values, and 0.50–0.75 for NC clay) (Salgado, 2008; Salgado & Prezzi, 2007), and OCR = overconsolidation ratio, which is equal to 1 for NC soil and greater than 1 for OC soil. The reader may also refer to Section 2.3.9 of Volume I for additional information on $K_0$. 
APPENDIX C. ITERATIVE SCHEME FOR FOOTING SETTLEMENT IN SAND

Because the representative elastic modulus of each sublayer is a function of footing settlement, an iterative scheme is needed if we wish to generate a load-settlement curve for a given footing geometry. Figure C.1 shows the iterative scheme proposed to achieve this objective. An initial guess value for \( w = w_{\text{max}} \) established in step 5(c) of Section 3.1 is first chosen, and the representative elastic modulus of each sublayer is then calculated using Eq. 3.12. Next, the footing settlement computed using Eq. 3.14 is compared with the initial guess value. If the convergence criterion of \( 10^{-5} \) is satisfied, the value of \( w \) obtained from Eq. 3.14 is reported as the footing settlement corresponding to the load acting on the footing. However, if the convergence criterion is not satisfied, the footing settlement obtained from Eq. 3.14 is used as the initial guess value for \( w \) in the next iteration. A convergence criterion of \( 10^{-5} \) was found to be adequate with respect to accuracy and computational time, and convergence was typically achieved within a few iterations. The iterative scheme can be used to obtain the load-settlement curve of the footing up to a footing settlement \( w \) equal to 10% of the footing size \( B \); however, it should not be used to estimate the limit unit bearing capacity \( q_{BL} \) of the footing (i.e., the unit load on the footing base that causes the footing to plunge into the ground). The iterations can be performed in Microsoft Excel either by going to File → Options → Formulas and selecting Enable iterative calculation in the Calculation options tab or by using the Solver tool. Note that parameters \( D_R, E/q_e, \) and \( I_z \) should be calculated for each sublayer within the influence depth \( z_f \) below the footing base.
Start

Obtain $q_c$ profile from CPT sounding

Input $L$, $B$, $D$, $K_0$, $\gamma_m$, $\phi_c$, $Q$, $w_{\text{max}}$

Calculate $z_f$, $z_{fp}$, $I_0$, $I_{zp}$, $I_z$ using Eqs. (3.5)–(3.9)

Calculate $D_R$ using Eq. (3.10)

Choose initial guess for $w$ [$= w_{\text{old}} = w_{\text{max}}$ (say 25 mm or 1 in.)]

Calculate $E/q_c$ using Eq. (3.12)

Calculate $w$ ($= w_{\text{new}}$) using Eq. (3.14)

Input $w_{\text{old}} = w_{\text{new}}$

Check if $\frac{|w_{\text{old}} - w_{\text{new}}|}{w_{\text{old}}} \leq 10^{-5}$

Yes

Footing design is satisfactory with respect to the serviceability limit state

End

No

Modify $L$ or $B$

Check if $w_{\text{new}} \leq w_{\text{max}}$

Yes

No

End

Figure C.1 Iterative scheme for estimation of footing settlement in sand using CPT results.
APPENDIX D. PENETRATION RATE EFFECT ON CONE RESISTANCE

Cone penetration at the standard rate of 2 cm/s (0.8 in./s) is fully drained for clean sand and fully undrained for pure clay. However, for soils containing mixtures of sand, silt, and clay, cone penetration at the standard rate of 2 cm/s (0.8 in./s) may take place under partially drained conditions depending on the ratios of these three broad particle size groups and the fabric of the soil. According to Kim et al. (2008, 2006), the undrained cone resistance is expected to be measured in CPTs performed with the standard cone \( (d_c = 35.7 \text{ mm or 1.4 in.}) \) at the standard rate \( (v = 2 \text{ cm/s or 0.8 in./s}) \) in soils having coefficient of consolidation \( c_v \) values less than roughly \( 10^{-4} \text{ m}^2/\text{s} \) (0.15 in.\(^2\)/s). However, if the \( c_v \) value of the soil is greater than about \( 10^{-4} \text{ m}^2/\text{s} \) (0.15 in.\(^2\)/s), the CPT sounding should be performed at a faster rate so that the normalized penetration rate \( V = \frac{vd_c}{c_v} \) is greater than 10 (Salgado & Prezzi, 2014). This approach would be the easiest way to ensure that cone penetration in mixed or intermediate soils takes place under undrained conditions. However, as this is still a topic of ongoing research, the implementation of this approach is optional and not mandatory in INDOT construction projects. The alternative would be to attempt to interpret the results of a CPT sounding actually performed under partial drainage conditions; however, there are no reliable methods for doing that at the present time. The coefficient of consolidation can be determined from the results of laboratory consolidation tests or CPT pore pressure dissipation tests (DeJong & Randolph, 2012), as discussed in Sections 1.3.6 and 2.3.14 of Volume I. Dissipation tests are valuable in clayey soils and they should be done whenever engineers judge that the value of the information obtained from the test justifies the expense for the site being investigated.

Volume I of the manual includes a synthesis of the work done by researchers on the aspect of penetration rate vis-à-vis the drainage conditions. The methodology proposed by DeJong et al. (2013) to address partial drainage conditions during cone penetration in intermediate soils is provided in Section 1.3.7 of Volume I. However, this methodology has not been standardized or formally adopted in practice.
APPENDIX E. RESIDUAL-STATE FRICTION ANGLE OF CLAY

The residual shear strength $\tau_r$ of clay is the product of the normal effective stress $\sigma'$ on the shearing plane and the tangent of the residual-state friction angle $\phi_r$, which in turn depends on the value of $\sigma'$, the clay mineralogy, the clay fraction (CF), and the magnitude and rate of shear displacement. According to Skempton (1985), the shear displacements needed for an intact clay with CF $\leq 30\%$ and $\sigma' < 600$ kPa to attain residual-state friction angles of $\phi_r$ and $\phi_r + 1^\circ$ range from 100–500 mm (4–20 in.) and 30–200 mm (1.2–8.0 in.), respectively. Based on the clay fraction of the soil, different residual-state shearing mechanisms are possible, resulting in different values of $\phi_r$ (Lupini, 1980; Lupini et al., 1981). Based on Skempton’s observations on the variation of $\phi_r$ with the clay fraction of sand-bentonite mixtures tested in ring shear, Salgado (2006) proposed the following equation for $\phi_r$ of clay-silt-sand mixtures as a function of the clay fraction at a given stress level:

$$\phi_r = \phi_r|_{\text{pure clay}} + \left(\frac{\phi_{r,\text{mix}} - \phi_r|_{\text{pure clay}}}{27\%}\right) \left[52\% - \text{CF(\%)\right] \right}$$

(Eq. E.1)

where $\phi_{c,\text{mix}} = \text{critical-state friction angle of the clay-silt-sand mixture, and } \phi_r|_{\text{pure clay}} = \text{residual-state friction angle of the clay fraction of the mixture.}$ For CF $\leq 25\%$, the bulky sand/silt particles are likely to control the behavior of the mixture and thus $\phi_r = \phi_{c,\text{mix}}$, whereas for CF $\geq 52\%$, the platy/tube-like/needle-like clay particles are likely to control the behavior of the mixture and thus $\phi_r = \phi_r|_{\text{pure clay}} \approx 5^\circ, 10^\circ, \text{ and } 15^\circ$ for montmorillonite, illite, and kaolinite clay minerals, respectively (Skempton, 1985). For intermediate values of CF between 25% and 52%, $\phi_r$ lies between $\phi_{c,\text{mix}}$ and $\phi_r|_{\text{pure clay}}$.

Besides the clay fraction and mineralogy, the residual-state friction angle $\phi_r$ also depends on the magnitude of the normal effective stress $\sigma'$ acting on the shearing plane; $\phi_r$ decreases nonlinearly with increasing $\sigma'$ (Figure E.1) because a larger normal stress forces greater realignment of clay particles in the direction of shearing. Soils with high clay fraction (CF $\geq 52\%$) and high smectite content, such as London clay, exhibit a significant drop in $\phi_r$ with increasing $\sigma'$, while soils with low clay fraction (CF $\leq 25\%$) and low smectite content may not exhibit any residual behavior. Following the work by Maksimović (1989), $\phi_r$ can be expressed in terms of $\sigma'$ using (Salgado, 2006):

$$\phi_r = \phi_{r,\text{min}} + \frac{\phi_c - \phi_{r,\text{min}}}{\sigma' - \sigma'_{\text{median}}}$$

(Eq. E.2)

where $\sigma' = \text{normal effective stress on the plane of shearing, } \phi_{r,\text{min}} = \text{minimum residual-state friction angle (attained at large normal effective stress), } \phi_c = \text{critical-state friction angle, and } \sigma'_{\text{median}} = \text{value of } \sigma' \text{ at which the friction angle is equal to the average of } \phi_{r,\text{min}} \text{ and } \phi_c$. At very large stresses, $\phi_r$ reaches an absolute minimum, denoted by $\phi_{r,\text{min}}$. For $\sigma'$ on the shearing plane approaching zero, $\phi_r$ approaches the critical-state friction angle $\phi_c$ due to the negligible reorientation of the clay particles in the absence of a normal stress forcing this reorientation to happen.
Figure E.1 Residual-state friction angle $\phi_r$ versus normal effective stress $\sigma'$ on the shearing plane (Salgado, 2006).

Table E.1 summarizes the values of $\phi_c$ and $\phi_{r,\text{min}}$ of some well-known soils in the literature, such as Lower Cromer till, Boston blue clay, San Francisco bay mud, London clay, and Weald clay as a function of their CF and PI values. Although Lower Cromer till is a glacial till composed of sand (> 50%), clay (= 14%–20%), and almost no silt (Gens, 1982), it has been considered in the literature to behave like a “clay” but with no residual behavior. Boston blue clay is a low-plasticity, insensitive, marine clay, composed of illite and quartz (Terzaghi et al., 1996), and does not exhibit any residual behavior (Ladd & Edgers, 1972). San Francisco bay mud is a highly-plastic silt containing a large amount of clay-size particles (montmorillonite and illite), organic substances, shell fragments, and traces of sand (Bonaparte, 1982). London clay is composed of illite, kaolinite, montmorillonite, and quartz (Gasparre, 2005); both San Francisco bay mud and London clay exhibit residual strength with sustained shearing beyond the critical state. Figure E.2 illustrates the fit of Eq. E.2 to ring shear test data for Weald clay. The fit was done by first estimating the value of $\phi_c$ in triaxial compression (Parry, 1960) and then finding the values of $\sigma'_{\text{median}}$ and $\phi_{r,\text{min}}$ that minimize the sum of least squares.
Table E.1 Critical-state and residual-state strength data for clayey soils reported in the literature

<table>
<thead>
<tr>
<th>Soil</th>
<th>Mineralogy</th>
<th>CF (%)</th>
<th>PI (%)</th>
<th>A</th>
<th>$\phi_c$ (°)</th>
<th>$\phi_{r,\text{min}}$ (°)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boston Blue Clay</td>
<td>Illite, quartz</td>
<td>35</td>
<td>13.1</td>
<td>0.37</td>
<td>32.4(^1)</td>
<td>—</td>
<td>Ladd &amp; Varallyay (1965)</td>
</tr>
<tr>
<td>London Clay</td>
<td>Kaolinite, illite, montmorillonite, quartz</td>
<td>53–62</td>
<td>42–45</td>
<td>0.73–0.79</td>
<td>21.3</td>
<td>9.4(^2)</td>
<td>Bishop et al. (1971); Gasparre (2005); Nishimura (2005)</td>
</tr>
<tr>
<td>Lower Cromer Till</td>
<td>Illite, calcite, quartz</td>
<td>14–20</td>
<td>10–12</td>
<td>0.60–0.71</td>
<td>30.0</td>
<td>—</td>
<td>Dafallas et al. (2006); Gens (1982); Lupini et al. (1981)</td>
</tr>
<tr>
<td>San Francisco Bay Mud</td>
<td>Illite, montmorillonite</td>
<td>47</td>
<td>47</td>
<td>1.00</td>
<td>28.9(^1)</td>
<td>16.2</td>
<td>Kirkgard &amp; Lade (1991); Meehan (2006)</td>
</tr>
<tr>
<td>Weald Clay</td>
<td>Illite, kaolinite, illite-</td>
<td>52</td>
<td>33</td>
<td>0.63</td>
<td>20.9</td>
<td>8.3(^3)</td>
<td>Akinlotan (2017); Bishop et al. (1971); Parry (1960)</td>
</tr>
</tbody>
</table>

Note: CF = clay fraction, PI = plasticity index, $A$ = activity (= PI/CF), $\phi_c$ = critical-state friction angle in triaxial compression, and $\phi_{r,\text{min}}$ = minimum residual-state friction angle in ring shear.

\(^1\) Extrapolated value corresponding to 30% axial strain (Chakraborty, 2009).

\(^2\) Value corresponds to blue London clay at Wraysbury (CF = 57%, PI = 43%, $A = 0.75$). For brown London clay at Walthamstow (CF = 53%, PI = 42%, $A = 0.79$), $\phi_{r,\text{min}} = 7.5^\circ$ (Bishop et al., 1971).

\(^3\) Obtained from the fit of Eq. E.2 to ring shear test data reported by Bishop et al. (1971).
Effective normal stress $\sigma'$ (ksf)

Test data (Bishop et al. 1971)
Fit using Eq. (E.2)

$\phi_c = 20.9^\circ$
$
\phi_{r,\text{min}} = 8.3^\circ$

$\sigma'_{\text{median}} = 51 \text{ kPa (1 ksf)}$

Figure E.2 Fit of Eq. E.2 to ring shear test data for Weald clay.
About the Joint Transportation Research Program (JTRP)

On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,600 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

Free online access to all reports is provided through a unique collaboration between JTRP and Purdue Libraries. These are available at http://docs.lib.purdue.edu/jtrp.

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