

Calibration of Geotechnical Resistance Factor For Axially Loaded Steel Piles Driven in Alberta And Related Cost-Benefit Analysis

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Dedication

To my beloved family, my father Reza Roshani, my mother Fahimeh Naderi, lovely wife Morvarid Kardan and the soul of my grandfather Mirza Ali Naderi.

Abstract

Driven steel open-ended piles are used as foundations to support structures by transferring loads to the surrounding soil and/or deeper, more competent strata. These foundations are commonly employed in oil & gas, and infrastructure projects across Alberta. During the design phase, their capacity is typically estimated using semi-empirical equations based on static analysis methods and soil mechanics principles. However, uncertainties in pile capacity prediction persist, primarily due to site characterization, soil behavior, and construction quality. To address these uncertainties and ensure adequate safety under working loads, a Geotechnical Resistance Factor (GRF), as recommended by part of LRFD design approach, is applied to the calculated pile capacity. In Canada, based on the updated Canadian Foundation Engineering Manual (CEFM) for sites with a typical level of geotechnical understanding and where neither Pile Dynamic Analysis (PDA) nor static load testing is performed, a GRF with a value of 0.4 is recommended.

However, several studies have indicated that using a uniform GRF without accounting for regional soil and bedrock variability often results in overly conservative designs. Similarly, applying a single GRF value to both shaft and toe capacities, while neglecting pile setup effects, further compounds this conservatism. Recalibration of GRFs is therefore necessary to better reflect these variables. This study aims to recalibrate GRF values regionally within Alberta for open-ended driven steel piles that have been axially loaded with the use of advanced statistical methods, PDA test data collected across the province and related geotechnical investigation data. The regional calibrated GRFs have been obtained initially for five (5) regions of Alberta (NE, NW, SE, SW, C). These values have been further recalibrated considering pile setup effects and differentiate value for shaft and toe capacities. Additionally, a cost-benefit analysis has been used to evaluate how much optimized GRFs can significantly reduce pile length requirements in design and cost in construction. Results show that locally calibrated GRFs yield the greatest savings about 20% compared to 10% for regionally calibrated values and 15% for soil-type-based calibrations. Incorporating pile setup effects into locally calibrated GRFs further enhances cost efficiency, with potential savings ranging from 23% to 35%, depending on soil conditions and calibration methodology.

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Chapter 1

General Introduction

1.1 Background

Driven steel open ended pile foundations are commonly used in Canada for supporting heavy structures in oil & gas infrastructure and bridge projects, particularly in the Alberta Region. (Bedair, 2013; Belbas, 2013). Pile capacity can be estimated using three analytical methods: static analysis, dynamic analysis, and dynamic formulas. The static analysis, developed empirically or semi-empirically from soil investigation data, is widely recommended by the Canadian Foundation Engineering Manual (CFEM 2023). However, the predicted pile capacity using the empirical-based static analysis method often differs from the actual capacities observed during pile driving. This discrepancy is attributed to the uncertainties encountered in geotechnical engineering. The highest level of uncertainty during construction is associated with soil properties estimation, the driving equipment, and the procedures used. Several factors can impact the estimated pile capacity, including soil disturbance during driving, soil-pile interaction, changes in pile capacity over time due to setup or relaxation, and the use of different static analysis methods. These uncertainties can lead to unacceptably high probabilities of service failure, which is why Geotechnical Resistance Factors (GRFs) are used in driven pile design (Jabo 2014).

For a chosen static analysis method, pile capacity may be achieved using the Load and Resistance Factor Design (LRFD) methodology. The LRFD approach offers several advantages for deep foundation design (Hamilton and Murff 1992). Its primary benefit is handling uncertainties in design parameters through a rational framework based on probability theory, leading to a consistent degree of reliability. As a result, the LRFD provides a uniform design approach for the entire structure (superstructure and substructure), enhancing the

overall design and construction process (Abdelsalam et al. 2010). In the LRFD method, different load types and combinations are multiplied by load factors, while resistances are multiplied by GRFs, ensuring the factored loads do not exceed the factored resistances. The GRF values can vary depending on the method used to estimate or measure pile capacity. Lower GRF values are assigned to empirical-based methods, while higher GRF values are assigned to field testing methods. Over the past decades, local LRFD calibrations have been conducted to reduce the conservatism built into the design process and to reflect local construction experiences and practices. For example, R. Skirrow and Wong (2008) performed a cost-benefit analysis using locally calibrated GRF values on several pile tests. They indicated that using the PDA (Pile Dynamic Analysis) testing procedure could reduce the relative cost of driven piles by up to 26%. Their analysis considered the following assumptions: a) 10% of the piles were tested using PDA, b) a GRF value of 0.5 as recommended in the Canadian Foundation Manual was used, and c) a total of 80 driven steel pipes were used during construction. If a more accurate GRF value, such as 0.6 associated with PDA testing, had been used, a further reduction in construction costs of about 32% could have been achieved. Jabo (2014) demonstrated that the GRF value at the USA national level produced resistance factors lower than those generated from data for piles driven in geological settings like Arkansas. Using these low resistance factors in bridge design resulted in oversized piles, more piles in a bridge foundation, or poor agreement between design and in-situ capacity values, leading to a significant increase in cost for the entire structure. This study aims to improve the recommended GRF value for the province of Alberta, incorporating PDA testing using a probabilistic procedure, thereby increasing factored pile capacity and reducing costs. Several case studies in Alberta are used in this research to demonstrate the methodology. The GRF value for axial capacity is calibrated using site-specific geotechnical data and PDA/Static test results along with several probabilistic methods. A Cost-Benefit Analysis, along with sensitivity analysis, is then carried out using the calibrated GRF values to obtain an accurate cost reduction estimate.

1.2 Statement of Problem

LRFD was initially introduced in Canada in 1979 with the publication of the Ontario Highway Design Code and its commentary. Goble R. investigated the LRFD concept for pile foundation design as early as 1980. Following the publication of the second (1983) and third (1992) editions of the Ontario Highway Bridge Design Code, more engineers began adopting the LRFD approach over the traditionally used Allowable Stress Design (ASD) method. In the 1992 code, the recommended values for static or dynamic analyses were

uniform and relatively low. Barker et al.'s extensive research in 1991 led to the development of LRFD for bridge foundation design, culminating in NCHRP Report 343, which laid the foundation for the 1997 AASHTO Bridge Design Specification. In 2006, Rahman et al. suggested, based on static and dynamic load test data from highway bridge projects, that resistance factors for pile foundation designs could be significantly increased. Honjo et al. (2002) employed the First-Order Reliability Method (FORM) for calculating partial factors for axially loaded piles and found that these factors depend on both the distribution of resistance and the uncertainties in soil parameters. Following extensive research in this area, AASHTO mandated the implementation of LRFD for all bridge structures, including foundations, beginning in 2007.

The CEFM (2023) began differentiating resistance factors between individual static or dynamic analyses. A lower GRF value (0.4) for typical site was assigned to semi-empirical methods, while a higher value (0.6) was allocated to static load test methods. CEFM (2023) allowed also to increase GRF for approximately 10% to 15% if there is high understanding from site conditions. Reliable estimation of GRF values using field testing methods can potentially allow designers to reduce the number and length of piles by employing higher GRF values (Skirrow and Wang 2008). To enhance the design and reliability of foundation piles, many researchers (e.g., Ng et al. 2012; Zhang 2004; Roling et al. 2011) suggest that GRF values listed in CFEM 2006, or other building codes may be conservative. The core hypothesis behind calibrating GRF values using the LRFD approach involves quantifying uncertainties through probabilistic methods such as the First Order Second Moment (FOSM), First Order Second Moment (FORM), or Monte Carlo Simulation (MCS). This aims to achieve engineered designs with consistent reliability levels (probability of failure) (Dithinde 2007). In Alberta, where soils and bedrock might differ significantly, the suggested GRF value based on CFM is overly conservative. The primary objectives of this research are to improve the agreement between pile capacity during design and monitoring phases and to enhance the current design and acceptance protocol through local calibration of resistance factors for driven steel piles in the Alberta region. The study involves evaluating the most used design and acceptance approaches for driven piles to unify them; collecting high-quality data on driven piles in Alberta; and performing robust statistical analyses on all collected data, leading to refined and higher resistance factors, specifically using PDA testing. The calibration of GRF values for pile design in the Alberta region will be achieved using field soil investigation data, PDA/static load test results, and three different MCS, FOSM, FORM reliability analyses. The calibration process begins with analyzing PDA test outcomes at the End of Drive (EOD) state. Subsequently, the influence of the following two scenarios is examined to enhance the precision of the calibrated results: Subsequently, the influence of the following two scenarios is examined to enhance the precision of the calibrated results:

1. Refine and define the calibrated GRF values individually by distinguishing between the shaft and toe capacities of the pile.

2. Enhance the calibrated GRF values by considering the effect of pore water pressure dissipation over time.

Finally, a cost-benefit analysis will be completed for different scenarios to demonstrate how improved GRF values for each scenario can reduce construction costs.

1.3 Research Objectives

The primary objectives of the current research are summarized as follows:

1. **Database Creation:** Develop a comprehensive database containing PDA test results, static test results, related pile design, geotechnical investigation data, and pile monitoring logs. This database will be compiled from various infrastructure projects and the oil and gas industry in Alberta, Canada. The aim is to evaluate and summarize the geotechnical properties for each project, including pile test results and other relevant data.

2. **Calibration of GRF Values using PDA results at End of Drive:** Calibrate the GRF values for different semi-empirical methods as suggested by the CFEM 2006. This calibration will be conducted as well using three different reliability analyses: Monte Carlo Simulation (MCS), First Order Second Moment (FOSM), and First Order Reliability Method (FORM). The calibration will cover different zones of Alberta based on its geological map, which is divided into six zones: Southwestern Alberta (SW), Southeastern Alberta (SE), Central Alberta (C), Northwestern Alberta (NW), Northeastern Alberta (NE), and the Rocky Mountains and Foothills (RM&F).

3. **Improve Calibrated GRF values by Defining Independent GRF for Shaft and Toe Capacity:** Perform calibration of the GRF values for semi-empirical methods, assuming independent values for the shaft and toe capacity of driven piles. This process will again utilize the three reliability analyses (MCS, FOSM, FORM) for different zones of Alberta.

4. **Improve Calibrated GRF values Considering Setup Time's Effect:** Evaluate how the dissipation of pore water pressure over time influences these calibrated GRF values.

5. **Cost-Benefit Analysis:** Perform a cost-benefit analysis to demonstrate how improved GRF values for each scenario can lead to reduced construction costs. This analysis will depend on the number of piles, their size, and the requirements for PDA testing.

1.4 Organization of Proposed Thesis

The organization of this thesis is illustrated in Figure 1.1. The thesis is structured into five chapters, each designed to facilitate understanding and effectively present the key findings. It is important to note that chapters three, four, five, and six are written in a research paper format, containing sections like abstract, introduction, materials and methods, results and discussion, conclusion, and references. This paper-based format will result in some repetition, as each paper must be standalone and self-contained. The scope of this thesis encompasses the following five categories:

Second chapter: This chapter provides a brief overview of the theoretical and technical background information related to pile foundation design, semi-empirical methods in literature, LRFD and ASD approaches, probabilistic methods, mathematical equations for GRF values in various codes, and summarizing GRF values for pile foundations at several codes. Additionally, this chapter reveals the relevance of the adopted approach to achieve the defined research scope.:

Third chapter: The methodology used to collect all necessary information for the research program is detailed in this chapter. It includes a summary of data collected during several field tests, detailing the testing setup used in field studies to measure single pile bearing capacity and the methodology to use these results to calibrate the GRF. Additionally, in this chapter, a detailed discussion on the methods to access the database is provided.

Fourth chapter (*Status: Paper has been published in ASCE-Journal of Risk and Uncertainty Analysis*): In this chapter, the GRFs for the static analysis methods were calibrated using a well-known probabilistic technique called Monte Carlo simulation (MCS). Calibrated GRF values have been recommended for the design of pile bearing capacity based on the soil type, cohesive fine content along the pile length, different empirical methods, and geological region of Alberta. The results showed that regional calibration of GFR based on the local database resulted in higher values of resistance factors than those recommended in national codes, leading to a more accurate and cost-effective design procedure.

Fifth chapter (*Status: Paper has been published in International Geotechnical Journal*): The main aim of this study is to improve calibrated GRF results in the fourth chapter considering separate GRFs for the toe and shaft resistances of pile foundations using different reliability analysis methods. Two criteria called "70% rule" and "R ratio" were used to evaluate the results.

Sixth chapter (*Status: Paper has been published in Deep Foundation Institute Journal*): The primary objective of this chapter is to assess the impact of time

on the recommended GRFs by several codes. To achieve this, the study employs a well-established probabilistic technique known as Monte Carlo Simulation (MCS) to quantify the influence of time, referred to as "setup time". To enhance the assessment of setup time's effect, the recorded bearing capacities are collected during two distinct time points: after the completion of pile installation or the end of drive (EOD) condition, and after a specific setup time at the Beginning-of-Restrike (BOR) condition.

Seventh chapter: This chapter discusses the cost savings and design optimizations that can be achieved with improved calibrated GRF values resulting from previous sections and includes a cost-benefit analysis using various nomograms.

Eight chapter: This chapter also succinctly summarizes the main conclusions from the different studies undertaken throughout this thesis and links them with the overall research scope. It also provides recommendations for future studies on determining more precise GRF values for deep foundations.

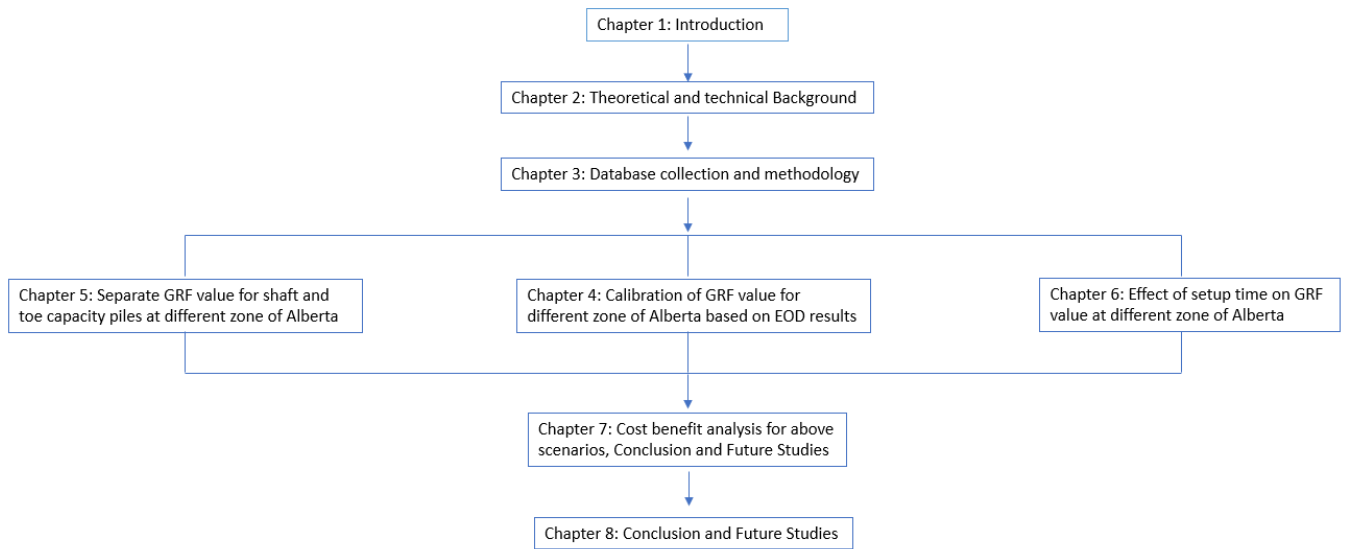


Figure 1.1: Organization of the thesis

1.5 References

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Chapter 2

Technical and theoretical background

2.1 Pile Foundations Overview

Pile foundations, integral structural elements fabricated from timber, concrete, or steel, are engineered to transmit loads from superstructures to deeper, more stable soil layers or bedrock. This load transference mechanism is primarily realized in two distinct forms: lateral load distribution along the pile shaft, known as friction or floating piles, and direct load application at the pile tip for endbearing piles. Despite the differentiation based on the primary load transfer mechanism, it is pertinent to acknowledge that all piles inherently engage in a synergistic load bearing action, combining side resistance and end bearing to optimize structural support. The multifaceted utility of pile foundations extends to several critical functions, including (Das 2010, Duncan and Buchingnani 1976):

- Redistribution of superstructural loads across or through the soil strata, accommodating both vertical and lateral forces concurrently.
- Counteraction against uplift or overturning forces, which is paramount for substructures like basement mats beneath aquifers or for sustaining bridge foundations subjected to lateral forces such as hydrodynamic pressures.
- Compaction of loose, cohesionless soil deposits through a combination of volumetric displacement and vibratory driving techniques, thereby enhancing soil density. This application may also involve temporary piles, which can be extracted post compaction.
- Stabilization of spread or mat footings on marginal soils or those overlaying compressible layers, effectively mitigating settlement and enhancing soil rigidity to control vibrational amplitudes and resonance frequencies beneath machinery foundations.

- Augmentation of safety factors for bridge abutments and piers, especially in scenarios with potential scouring risks (Ghosn and Moses, 1998; Grigorian 1997; Hannigan et al., 1997).

- Support for offshore structures by channeling surface loads through aquatic environments into subaqueous soil layers, necessitating design considerations for both vertical (including buckling) and lateral forces.

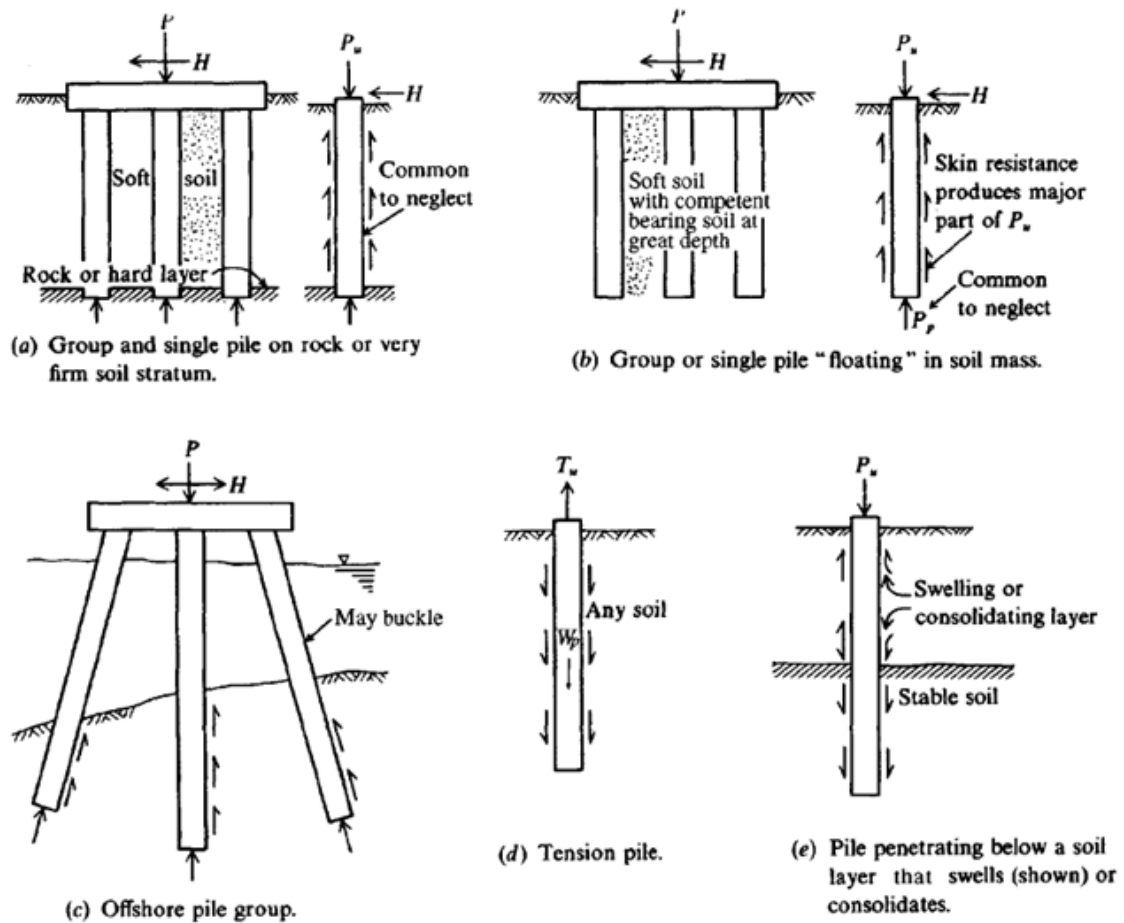


Figure 2.1: Typical pile configurations (Bowles, 1996)

2.1.1 Bearing Capacity

Pile foundations function as conduits, channeling loads from superstructures to deeper, more stable soil layers. This load transition can manifest through the shaft of friction piles

via a vertical load distribution mechanism, or through the point of end bearing piles, where load application is direct. **Figure 2.2** delineates these pile categories—friction and end bearing. The mechanism of load distribution is quantifiable through static load testing, facilitated by affixing strain gauges at various depths along the pile shaft, as depicted in **Figure 2.3**. Here, the axial load measurement is represented by with and denoting the pile shaft resistance and total pile resistance, respectively. In both scenarios, the determination of bearing capacity necessitates a synthesis of shaft and toe resistance. The formula to ascertain the axial capacity of an individual pile is articulated as:

$$R = Q_s + Q_b - W_p \quad (2.1)$$

Where: R = axial capacity of a single pile, Q_s = shaft capacity of pile, Q_b = base capacity of pile, W_p = pile weight. The calculation of each axial pile capacity component is achievable through the application of established soil mechanics principles.

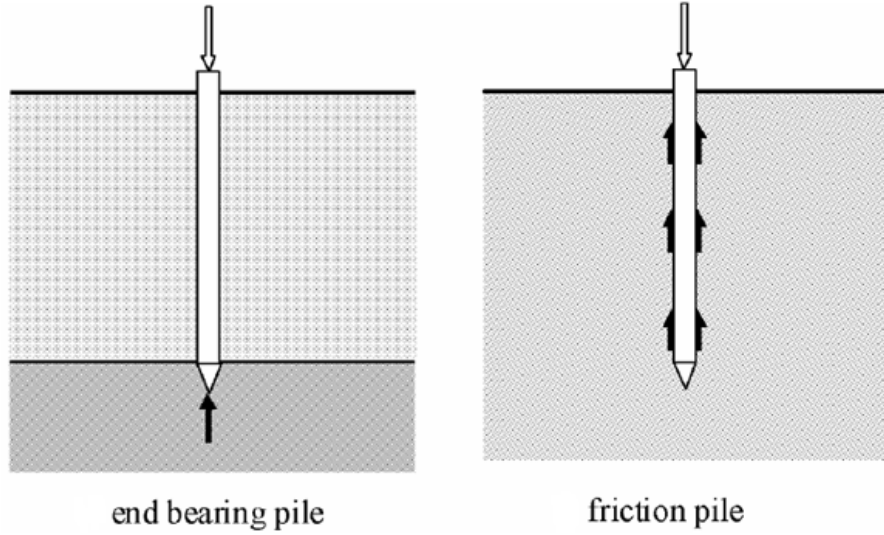


Figure 2.2: Pile classification based on load transfer mechanism (Madabhushi et al. 2010)

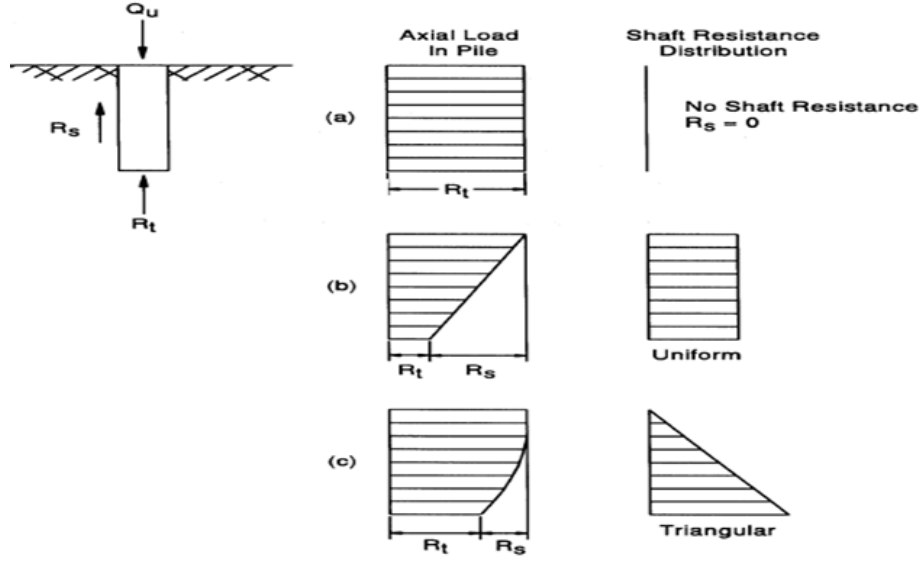


Figure 2.3: Typical load transfer profiles (National Highway Institute, 2010)

2.1.2 Pile Shaft Capacity

The calculation of pile shaft capacity necessitates the integration of shear stress across the pile's surface area, delineated by the formula:

$$Q_s = 2\pi r \times \int_s^L \tau_s \quad (2.2)$$

where: r = pile radius, L = pile length, τ_s = shear stress along the pile shaft. The concept of ultimate skin resistance emerges from the differential movement or relative slip between the pile and surrounding soil. This slip, defined by the variance in shaft strain due to axial loading compared to the soil strain induced by the transfer of load via skin resistance, escalates along the pile shaft with increased load application. Initially, in the upper segments of the pile, slip reaches a threshold skin resistance, prompting load transfer to lower segments and eventually to the pile tip, thereby engaging it to bear the load. Optimal pile load capacity is attainable under conditions of rapid pile penetration. Conversely, slower penetration rates result in increased point load but diminish slip resistance, ultimately settling at a value below the maximum, as elucidated by Bowles (1996).

2.2 Determination of Pile Capacity using Most Common Semi-Empirical Methods

The estimation of axial pile capacity, particularly for piles driven into different types of soils, presents substantial challenges within foundation design, marked by significant uncertainty, especially under unsaturated conditions (Taylan & Vanapalli, 2012; Fredlund et al., 1996). Notably, discrepancies often arise between empirical observations and established design guidelines. For example, while design principles may posit a gradual decrease in the rate of pile capacity increase with depth to achieve threshold values of shaft friction and end bearing capacity beyond a critical depth, empirical data suggest that peak values typically occur near the pile tip, with the lowest values observed near the surface (Hansen, 1956; Hansen, 1966). This divergence underscores the complexity of accurately predicting pile behavior in situ, highlighting the need for a nuanced understanding of soil-pile interaction dynamics that may deviate from conventional design expectations.

2.2.1 Coarse Soils:

The β Method: An Effective Stress Analysis for Pile Shaft Friction

The β method employs an effective stress framework to calculate unit shaft friction at any depth z along the pile. This is expressed as:

$$q_s = \sigma'_v K_s \tan \delta = \beta \sigma'_v \quad (2.3)$$

Where: K_s = mean lateral earth coefficient, δ = internal friction angle of soil, σ'_v = vertical effective stress along the pile length. The determination of K_s indicative of the soil's lateral restraint in its natural state, hinges on multiple variables including the degree of over consolidation, soil shear resistance angle, initial stress conditions, soil compressibility, and pile installation method, alongside the pile's material, dimensions, and geometry. For bored piles, K_s is typically equated to the at rest earth pressure coefficient, K_0 whereas for driven piles, it is posited to be double K_0 (FHWA-SA-91-042, 1992). Factors influencing δ encompass the average soil particle size, pile surface texture (contingent upon the pile material), installation technique, and normal stress at the pile-soil interface. The value of δ generally ranges between 0.5 and 1.0. **Table 2-1** illustrates typical values for the combined shaft resistance factor, β .

Table 2.1: Range of β coefficients (CEFM, 2006)

Soil Type	Cast-in-Place Piles	Driven Piles
Loose sand	0.2-0.4	0.3-0.8
Medium sand	0.3-0.5	0.6-1.0
Dense sand	0.4-0.7	0.8-1.2

The effective stress method is tailored to characterize the long term drained shear strength conditions, emphasizing the pivotal role of the soil’s effective friction angle, ϕ' , in these analyses. Consequently, within this framework, it’s essential to utilize the effective soil friction angle for accurate modeling. The computation of unit toe resistance, q_t , is thus formulated on the basis of this principle, ensuring that the analysis reflects the true drained shear strength of the soil over extended periods. This approach underscores the importance of considering the effective stress parameters for a comprehensive understanding of soil behavior under long term loading conditions, enabling more precise and reliable foundation design. The unit toe resistance, q_t can be computed as:

$$q_t = N_t \sigma'_t \quad (2.4)$$

Where: N_t = bearing capacity factor, σ'_t = vertical effective stress at pile toe. For unsaturated soils, Vanapalli and Taylan (2011b) have proposed modifications to the conventional *beta* methods to estimate the shaft bearing capacity of single piles, which is a function of matric suction with the same above-mentioned principles.

Table 2.2: Range of N_t coefficients (CEFM, 2023)

Soil Type	Cast-in-Place Piles	Driven Piles
Loose sand	20-30	30-80
Medium sand	30-60	50-120
Dense sand	50-100	100-120

Meyerhof Method Based on Standard Penetration Test (SPT) Data:

This technique is pivotal for the initial estimation of pile capacity within sandy soil contexts, drawing upon empirical correlations between Standard Penetration Test (SPT) results and static pile load testing outcomes (Meyerhof, 1995). These correlations emerge from extensive analysis of various pile load test outcomes across numerous sandy soil deposits. Meyerhof (1976) introduced this method, which stands out for its simplicity and

ease of application. However, due to its inherent limitations, it is recommended to employ this method solely for preliminary estimations, rather than for final design purposes (FHA, 1997). Meyerhof's (1976) methodology advocates for utilizing the average corrected SPT resistance value, N' , expressed in blows per 300 mm, calculated over the pile's embedded length. The formula proposed to deduce the average unit shaft resistance, q_s , for driven displacement piles in kilopascals is:

$$q_s = 2\overline{N'} \leq 100kPa \quad (2.5)$$

In case of driven non-displacement pile:

$$q_s = 1\overline{N'} \leq 100kPa \quad (2.6)$$

Meyerhof (1976) proposed that the toe resistance, q_t , for driven piles which are embedded in sand can be computed in kPa by:

$$q_t = 400\overline{N'_o} + \frac{(40\overline{N'_B} - 40\overline{N'_o})D_B}{b} \leq 400\overline{N'_B} \quad (2.7)$$

where: $\overline{N'_o}$ = average corrected SPT N' value for the stratum overlying the bearing stratum. $\overline{N'_B}$ = average corrected SPT N' value of the bearing stratum, D_B = pile embedded depth into the bearing stratum in meters, b = pile diameter in meters. When the embedded depth into the bearing stratum goes to 10 pile diameters, the limiting value $400\overline{N'_B}$ is expected. (FHA, 1997).

Nordlund Method:

The Nordlund method, established in 1963, derives from meticulous field observations and the conceptualization of piles as tapered structures. The development of this method incorporated extensive data from multiple pile load tests, providing a robust empirical foundation for its application. However, it's pertinent to note that this method is not advised for piles exceeding widths of 600 mm. Nordlund articulated that the calculation of shaft resistance is influenced by a constellation of factors, including:

- The soil friction angle, which contributes to the lateral resistance offered by the soil.
- The sliding surface friction angle, influencing the interface resistance between the pile and the soil.
- The taper of the pile, affecting the pile's interaction with the surrounding soil and its capacity to displace soil.

- The effective unit weight of the soil, a critical parameter in determining the soil's contribution to pile resistance.
- The total length of the pile, which impacts the pile's load bearing capacity.
- The minimum pile perimeter, influencing the contact area between the pile and the soil.
- The volume of soil displaced by the pile, reflecting the pile's impact on the surrounding soil structure.

This method emphasizes the integral role of these parameters in defining the shaft resistance of piles, underscoring the complexity and variability inherent in pile-soil interaction mechanisms. The proposed equation for the Nordlund method for ultimate shaft capacity is:

$$Q_S = \sum_{d=0}^D K_\delta C_F p_d \left(\frac{\sin(\delta + \omega)}{\cos \omega} \right) C_d \Delta d \quad (2.8)$$

Where: d = depth, D = embedded pile length, K_δ = coefficient of lateral earth pressure at depth d , C_F = correction factor for K_δ when $\neq \phi$, p_d = effective overburden pressure at the center of the depth increment d , δ = friction angle between the pile and the soil, ϕ = soil friction angle, ω = angle of pile taper from vertical, C_d = pile diameter at depth d , Δd = length of pile segment. The mentioned parameters are displayed in **Figure 2.4**. Also, this method can be utilized for piles of uniform cross section by assuming the ω as zero in the equation. Nordlund (1963) suggested using the following equation for estimating the ultimate toe capacity, Q_t :

$$Q_t = \alpha_t N'_q A_t p_t \quad (2.9)$$

Where: α_t = dimensionless factor (dependent on pile depth-width relationship), N'_q = bearing capacity factor, A_t = pile toe area, p_t = effective overburden pressure at the pile toe. Nordlund recommended using a limiting value of 150 kPa for effective overburden pressure, p_t . The α_t coefficient and the bearing capacity factor, N'_q , can be obtained by using **Figures 2-4** and **2-6**, respectively.

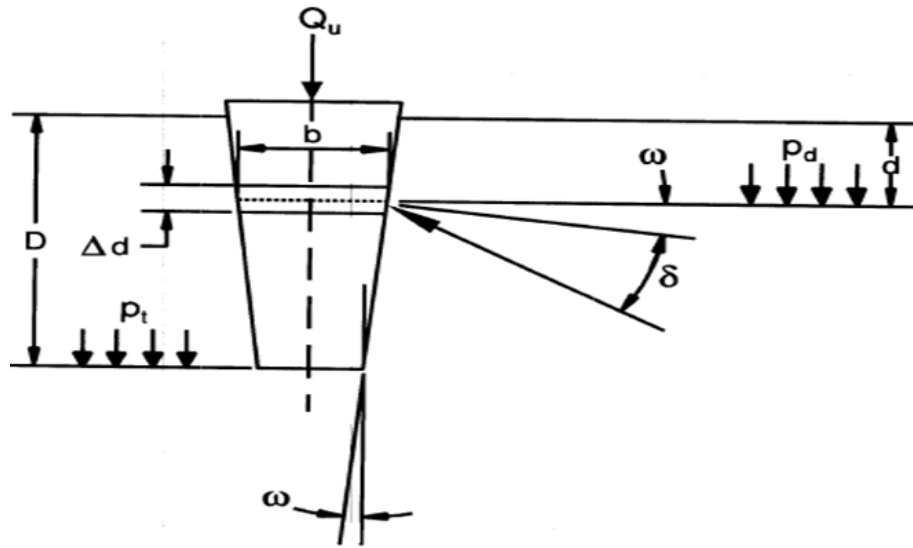


Figure 2.4: Nordlund's elements for pile bearing capacity (FHA, 1997)

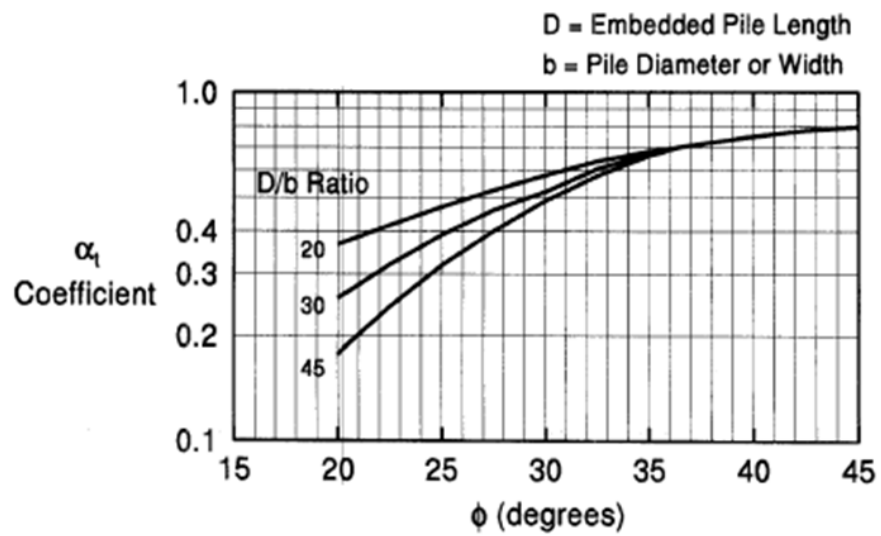


Figure 2.5: Chart for estimating α_t coefficient (FHA, 1997)

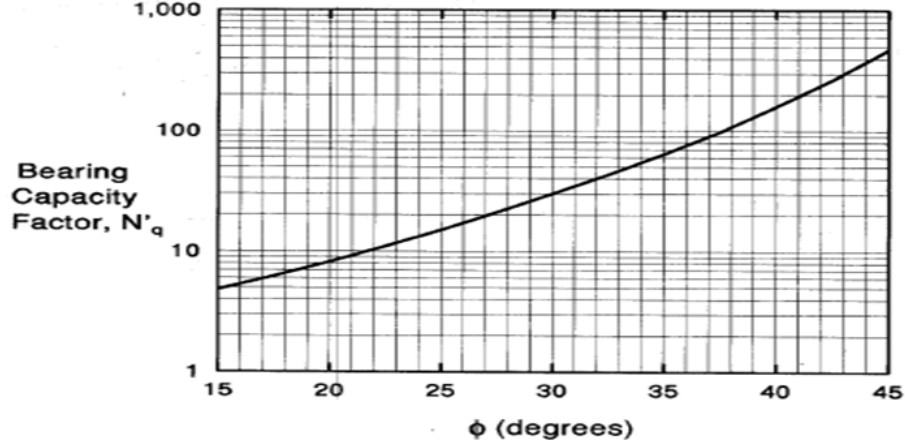


Figure 2.6: Chart for estimating bearing capacity factor, N'_q (FHA, 1997)

Nottingham and Schmertmann Method

Developed by Nottingham and Schmertmann in 1975, this method leverages Cone Penetration Test (CPT) data to estimate pile bearing capacity. Given the CPT's efficacy in mimicking model pile data, its outcomes are instrumental in the design process, especially for sites where cone sounding can achieve depths comparable to those intended for driven piles. This approach is particularly valued for its direct use of in-situ test results, providing a detailed profile of soil resistance characteristics. Nottingham and Schmertmann advocate for the following formula to calculate the ultimate shaft resistance, R_s :

$$R_s = K \left[\frac{1}{2} (\bar{f}_s A_s)_{0 \rightarrow 8b} + (\bar{f}_s A_s)_{8b \rightarrow D} \right] \quad (2.10)$$

where: K = ratio of unit pile shaft resistance to unit cone sleeve friction as a function of the full penetration depth, (D) , $\bar{f}_s$ = average unit sleeve friction over the depth interval indicated by subscript, A_s = pile-soil surface area over the depth interval, b = pile width or diameter, D = embedded pile length, 0 to $8b$ = range of depths for segment from ground surface to a depth of $8b$, $8b$ to D = range of depths for segment from a depth equal to $8b$ to the pile toe. The coefficient K can be obtained from **Figure 2-7**.

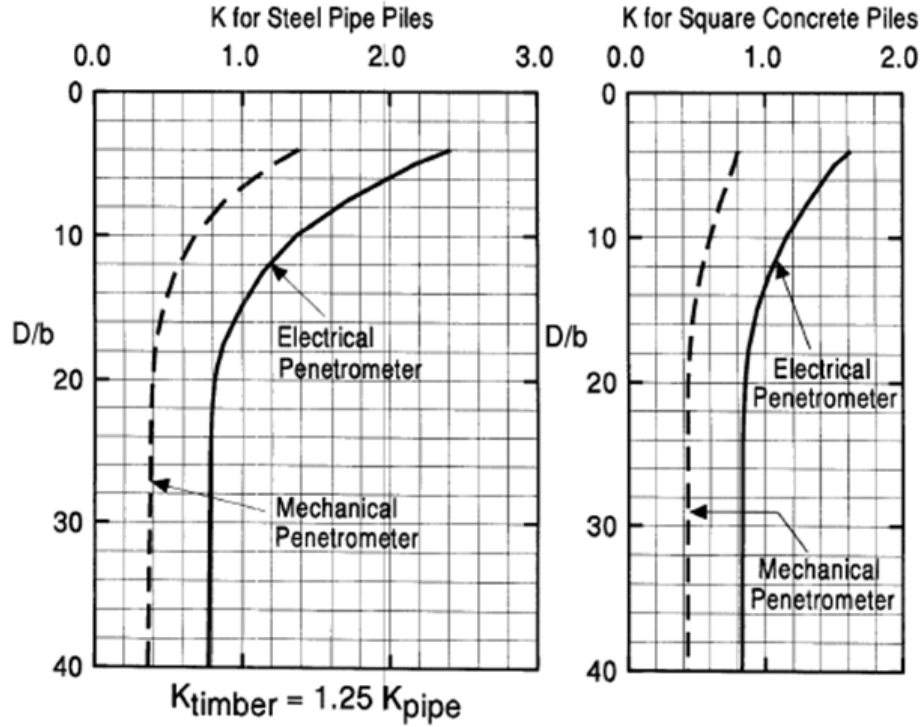


Figure 2.7: Penetrometer design curves for pile side friction (FHA, 1997)

To accurately determine the pile toe capacity utilizing this method, reference to **Figure 2.8** is essential. The procedure mandates a weighted analysis of the cone tip resistance value, encompassing a range from 8 pile diameters above the pile toe to a depth of 3.75 pile diameters below the pile toe. This is done to identify the lower cone tip resistance, q_c , within this specified depth interval. The acceptable range for q_c is stipulated to be between 5000 to 15000 kPa. The calculation involves determining q_{c1} , the average cone tip resistance over a distance of x_b below the pile toe, following a designated path (1-2-3). This includes summing q_c values both in the downward (path 1-2) and upward (path 2-3) trajectories. Actual q_{c1} values are utilized along path 1-2, while the minimum path rule is applied along path 2-3. The computation of q_{c1} is performed for x values ranging from 0.7 to 3.75 below the pile toe, selecting the smallest q_{c1} value derived. Subsequently, q_{c2} represents the average q_c over 8 pile diameters above the pile toe (path 3-4), adhering to the minimum path rule as executed in the q_{c1} analysis. Here, b denotes the pile width or diameter, and D signifies the embedded pile length.

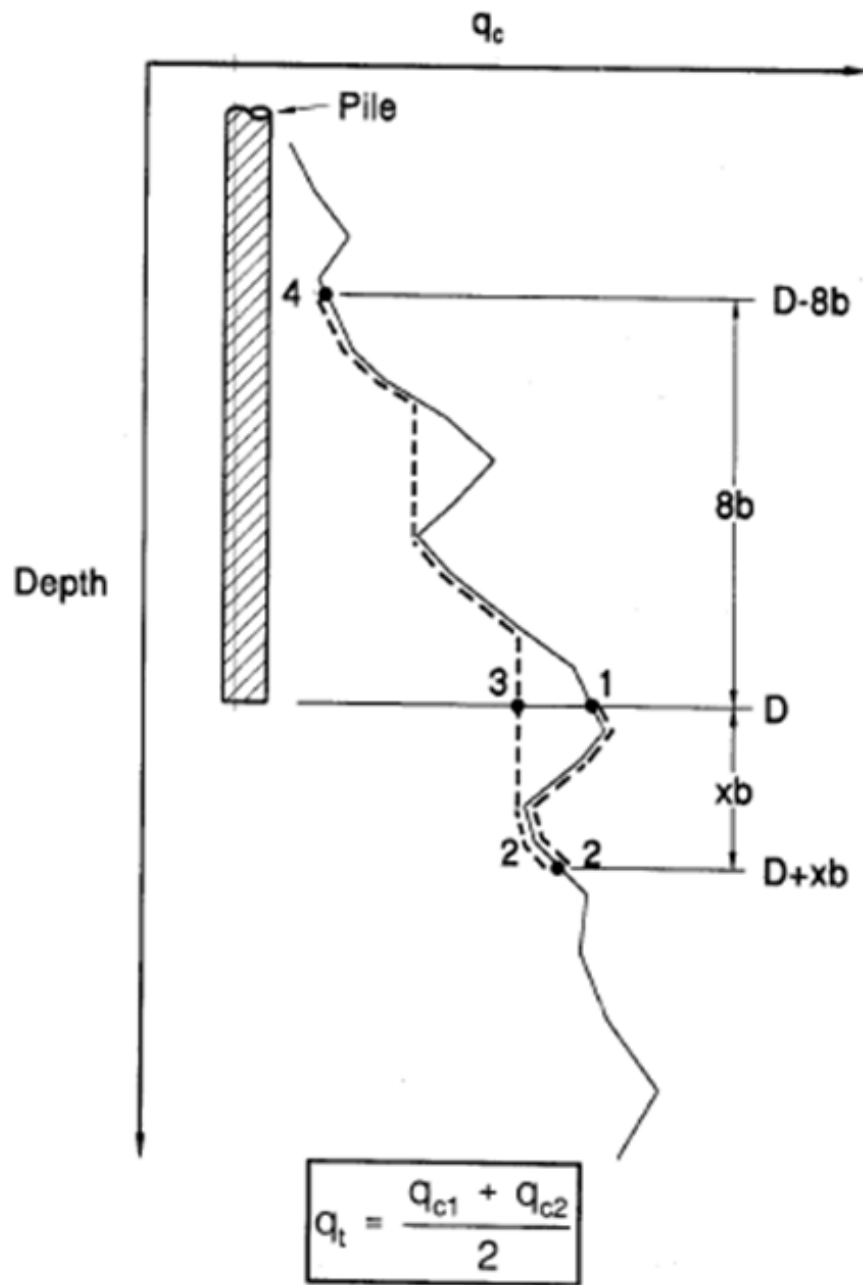


Figure 2.8: Illustration of Nottingham and Schmertman procedure for estimating pile toe capacity (FHA, 1997)

Eslami and Fellenius Method

This methodology is formulated on the premise of soil profiling, specifically tailored for harnessing cone penetrometer (both CPT and CPTu) data in the context of pile design. A comprehensive database amalgamating CPT/CPTu data alongside boring logs, sampling records, laboratory tests, and standard soil characteristics from a multitude of case studies was meticulously assembled. This database was methodically categorized into five principal soil types, namely:

- Sensitive and Collapsible Clay and/or Silt
- Clay and/or Silt
- Silty Clay and/or Clayey Silt
- Sandy Silt and/or Silty Sand
- Sand and/or Sandy Gravel

Data points from the database were systematically plotted on a Begemann-type profiling chart. Subsequently, envelopes encapsulating each soil type category were delineated to facilitate soil type identification and characterization. These delineated envelopes, instrumental in soil type discrimination, are depicted in **Figure 2.9**. Effective cone resistance defined by:

$$Q_E = q_t - U_2 \quad (2.11)$$

Where: Q_E = “effective” cone resistance, q_t = cone resistance corrected for pore water pressure on shoulder, u_2 = pore pressure measured at cone shoulder. The Q_E value has been empirically validated as a consistent metric for evaluating soil responses, particularly in the context of pile shaft and pile toe resistances (Eslami, 1996; Eslami & Fellenius, 1995; 1996; 1997). The Eslami-Fellenius chart, recognized for its simplicity and efficiency, eliminates the need for adjustments to estimated effective and total stresses. This tool is specifically designed for the analysis of soil types through CPTu data profiling.

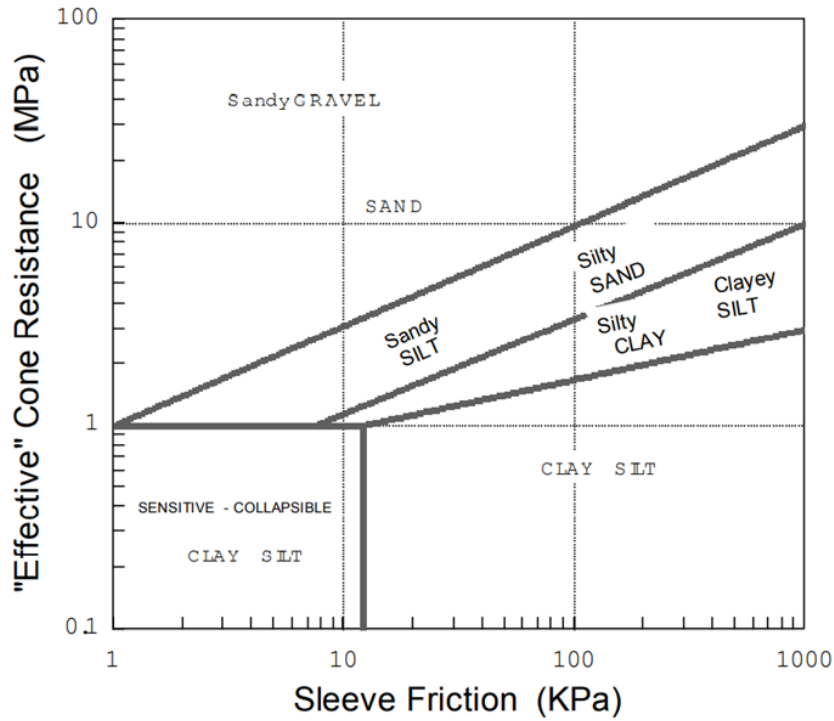


Figure 2.9: The Eslami-Fellenius profiling chart (Eslami and Fellenius, 1997)

The chart facilitates the delineation of soil types by categorizing them into major soil fractions such as clay, silt, sand, and gravel. While international and North American standards concur on the demarcation of these primary soil categories, discrepancies arise in the nomenclature adjustments based on the presence of constituents other than the dominant soil fraction. This nuance underscores the importance of context-specific interpretations when applying soil-type names in geotechnical analysis and design.

Laboratoire Central des Ponts et Chaussees (LCPC) Method: The LPC method was developed by Bustamante and Gianeselli (1982), by considering the soil type, pile type, and level of cone tip resistance. In this method, categories are categorized into 3 groups, A, B, and C, which are tabulated in Table 2-3. By using Table 2-3 and determining the pile group, the pile shaft resistance design curve can be obtained from Table 2-4. Consequently, **Figures 2.10** can be used to gain the shaft resistance value.

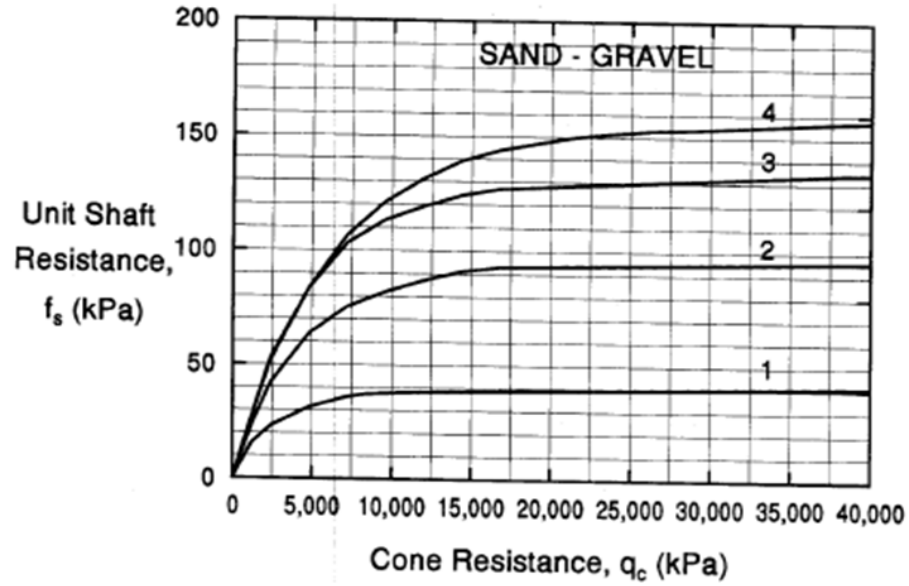


Figure 2.10: Maximum unit shaft resistance curves for LCPC method (FHA, 1997)

Table 2.3: Driven pile type categories for LPC method (FHA, 1997)

Pile Type	Pile Description	Pile Installation Procedure
A	Driven prefabricated	Reinforced or prestressed concrete pile installed by driving or vibro-driving.
B	Concrete piles.	Pile made of steel only and driven in place: H pile, pipe pile or any shape obtained by welding sheet-pile sections.
C	Driven steel piles.	Made of hollow cylinder elements of lightly reinforced concrete assembled by pre stressing before driving. Each element is generally 1.5 to 3 m long and 0.7 to 0.9 m in diameter; the thickness is approximately 0.15 m. The piles are driven open end-ed.

Table 2.4: Curve selection based on pile type and insertion procedures (FHA, 1997)

Curve No.	q_c kPa	Pile Type	Comments on insertion procedure
1	< 3500	A, B, C	
2	> 3500	A, B, C	For fine sands. Since steel piles can lead to very small values of f_s in such soils, use curve 1 unless higher values can be based on load test results. For concrete piles, use curve 2 for fine sands of $q_c > 7500 \text{ kPa}$.
3	< 7500	A, B	For coarse gravelly sand or gravel only. For concrete piles, use curve 4 if it can be justified by a load test.
4	> 7500	A	Only for Coarse gravelly sand and gravel and, if justified, by load test.

The unit toe resistance is computed by using the cone bearing factor, K_c , which is found in Table 2-5. This factor should be multiplied by the average cone resistance, q_c , in a range of one pile diameter below the pile toe.

Table 2.5: Cone bearing capacity factors for LPC method (FHA, 1997)

Soil type	Cone bearing factor, K_c
Sand-gravel	0.375

2.2.2 Cohesive Soils:

The determination of pile shaft capacity in clay-dominated soils typically leverages empirical methodologies, utilizing both total and effective stress analyses. Additionally, correlations derived from Cone Penetration Test (CPT) outcomes prove beneficial. The effective stress approach, deemed more analytically cogent for such soil contexts, is particularly pertinent when addressing undrained soil shear strength (S_u) values exceeding 25 kPa. This approach, while fundamentally similar to that employed for sandy soils, necessitates distinct adjustments in the values of β (shaft resistance factor) and N_t (bearing capacity factor) to account for the differential soil mechanics properties between clay and sand. Thus, the effective stress methodology underscores the nuanced consideration required for accurately estimating pile shaft capacity in varied soil types, emphasizing tailored parameter values reflective of the specific soil characteristics.

The α Method

This method is a total stress method which deals with undrained soil shear strength, S_u . In α method the shaft resistance is considered independent of the effective overburden pressure and the unit shaft resistance can be stated as a function of an empirical adhesion factor, α . The unit shaft resistance, f_s , can be computed by:

$$f_s = c_a = \alpha c_u \quad (2.12)$$

Where: C_a = shear stress between the pile and soil at failure, α = empirical adhesion factor, C_u = average undrained shear strength of undisturbed clay along the pile shaft.

The empirical adhesion reduction factor, crucial for calculating pile shaft capacity, is significantly affected by various factors including the clay's nature and strength, pile dimensions, installation technique, and temporal influences. This method, though straightforward due to its foundation on laboratory undrained shear strength tests, exhibits considerable variability in the relationship between adhesion and undrained shear strength. To accurately determine the adhesion factor, reference to **Figure 2.11** is recommended, except in instances where the site conditions align with scenarios depicted in **Figure 2.12**. Specifically:

- **Figure 2.12 (a)** pertains to driven piles passing through a sand or gravel layer into stiff clay, where the granular material's incursion into the clay anticipates higher adhesion factor values. As pile penetration into the clay layer increases, the influence of the granular stratum diminishes, leading to a decrease in the adhesion factor.
- **Figure 2.12 (b)** applies to piles driven through a soft clay layer into a stiffer clay layer below, where the driving process drags soft clay into the stiff layer, reducing the adhesion factor for the latter. To enhance the adhesion factor, deeper pile penetration is necessary.
- **Figure 2.12 (c)** is relevant for piles driven directly into stiff clay without overlying strata. In such cases, a gap often forms between the pile and soil along the upper shaft, reducing shaft resistance. Deeper penetration can mitigate this gap's impact, thereby increasing the adhesion factor.

Vanapalli and Taylan (2011a) propose modifications to the traditional α methods for estimating the shaft bearing capacity of single piles. These modifications incorporate the principles outlined above and include the influence of matric suction, offering a refined

approach to accurately determining pile shaft capacity in clay soils. For clays which have the undrained shear strength less than 100 kPa, the bearing capacity is entirely governed by shaft resistance (Azzouz et al. 1990). The case of soft soils can be treated in the same manner. The adhesion factor ranges from 0.5 to 1.0 (CFEM 2006).

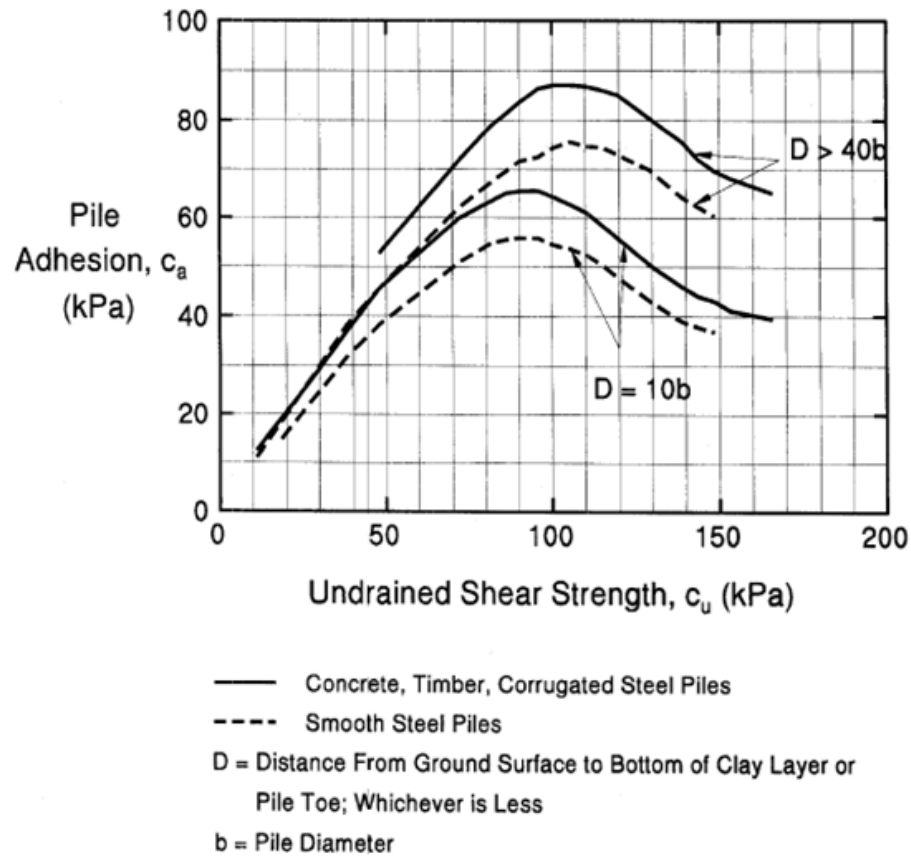


Figure 2.11: Adhesion values for piles in clay soil (FHA, 1997)

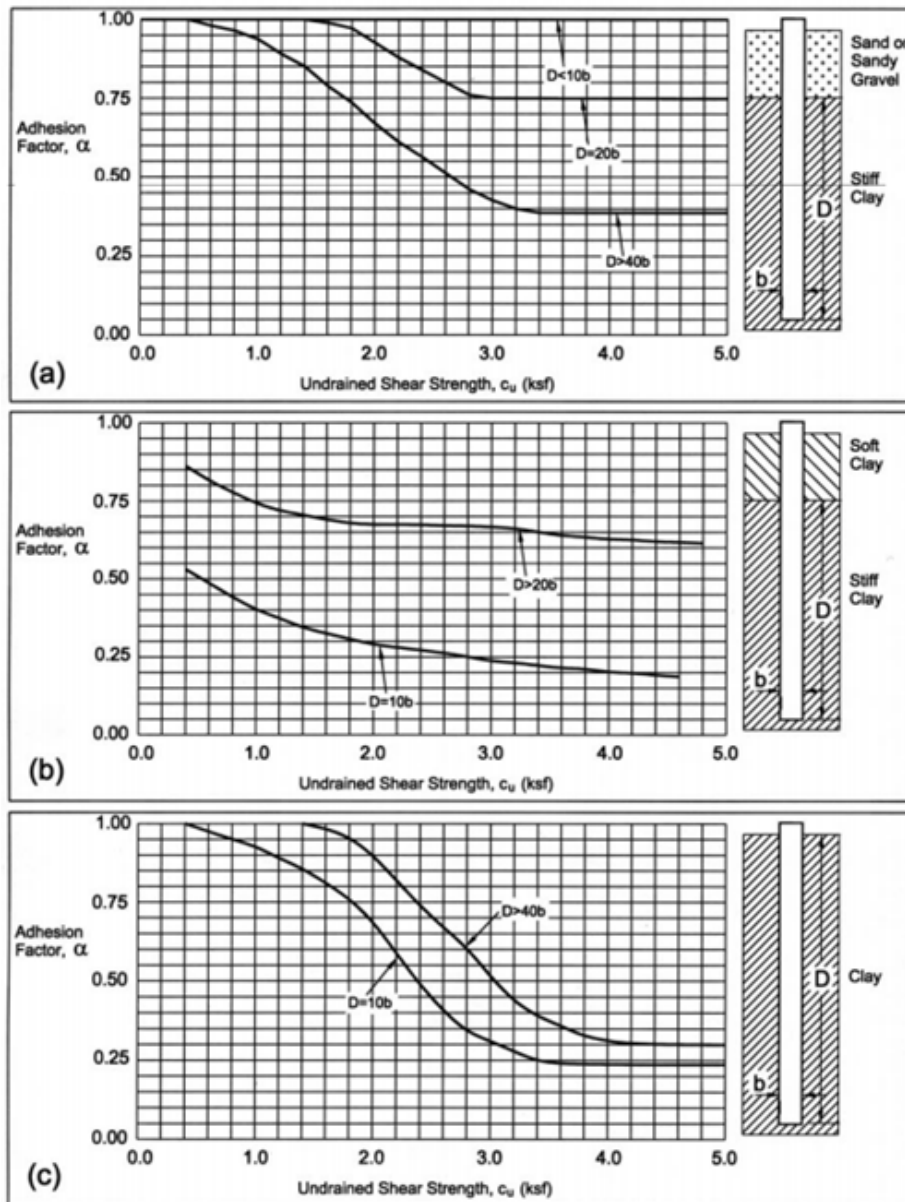


Figure 2.12: Adhesion factor for piles drive in clay (FHA, 1997)

The β Method

The β -method is a semi-empirical approach used to evaluate the axial capacity of piles embedded in cohesive soils. It is based on the principle that the unit shaft resistance mobilized along the pile–soil interface can be expressed as a function of the effective vertical stress in the soil and an empirical dimensionless coefficient, β . This approach is widely used because it provides a simplified yet robust representation of soil pile interaction, particularly in the context of the Load and Resistance Factor Design (LRFD) framework. Table 2-6 illustrates the common values for combined shaft resistance factor, β for cohesive soils.

Table 2.6: Range of β coefficients (FHA, 1997)

Soil Type	ϕ'	β
Clay	25-30	0.23-0.40

The λ Method

Vijayvergiya and Focht (1972) proposed a combined method to obtain the shaft resistance of a pile embedded in over-consolidated clay. The lambda method is a mixture of the effective and total stress methods. The given equation is as below:

$$f_s = \lambda(\bar{q} + 2s_u) \quad (2.13)$$

Where: λ = coefficient, $\bar{q}$ = effective overburden pressure to the average depth of pile segment, S_u = undrained shear stress. In the total stress method, the toe resistance may be computed by:

$$q_t = N_t s_u \quad (2.14)$$

Where: q_t = toe resistance, N_t = a bearing capacity coefficient which is a function of the pile diameter, S_u = minimum undrained shear strength of the clay at pile toe. Common values of N_t are illustrated in Table 2-7.

Table 2.7: Common values for coefficient N_t (CFEM, 2006)

Pile toe diameter	Bearing capacity coefficient, N_t
Smaller than 0.5 m	9
0.5 m to 1 m	7
Larger than 1m	6

By using the effective stress method, the unit toe resistance, q_t can be computed as:

$$q_t = N_t \sigma'_t \quad (2.15)$$

Where: N_t = bearing capacity factor, σ'_t = vertical effective stress at pile toe.

Table 2.8: Caption

Soil Type	ϕ'	N_t
Clay	25-30	3-30

Nottingham and Schmertmann Method

The Cone Penetration Test (CPT) is highly regarded as an effective analog for pile behavior, making the correlation between CPT outcomes and pile load test data a valuable asset in the design process. The Nottingham and Schmertmann method, predicated on these CPT correlation equations, provides a structured approach to integrating empirical data into pile design, particularly for piles embedded in clay.

In 1975, Nottingham and Schmertmann introduced an equation specifically tailored to calculate shaft resistance in clay-based pile installations. This formula underscores the method's reliance on direct CPT data, reinforcing the test's utility in capturing the nuanced interactions between pile surfaces and clay soils. By leveraging such correlations, designers can achieve a more accurate and informed estimation of pile shaft resistance, thereby enhancing the reliability and effectiveness of pile foundation systems in clayey environments.

$$R_s = \alpha' \overline{f_s} A_s \quad (2.16)$$

Where: α' = ratio of pile shaft resistance to cone sleeve friction, $\overline{f_s}$ = average unit sleeve friction, A_s = pile-soil surface area over f_s depth interval, the values of α' can be obtained from Figure 2-13 which varies as a function of sleeve friction, f_s . In order to estimate the pile toe capacity by this method, **Figure 2-14** can be used. In this method, the cone tip resistance value has to be weighted, from 8 pile diameters above the pile toe, to as much as 3.75 pile diameters below the pile toe, to find the lower cone tip resistance, q_c within the depth range. The recommended limit value for q_c , is between 5000 to 15000 kPa. q_{c1} = average q_c over a distance of x_b below the pile toe (path 1-2-3). Sum q_c values in both the downward (path 1-2) and upward (path 2-3) directions. Use actual q_c values along path 1-2 and the minimum path rule along path 2-3. Compute q_{c1} for x -values from 0.7 to 3.75 below the pile toe and use the minimum q_{c1} value obtained. q_{c2} = average q_c

over a distance of $8b$ above the pile toe (path 3-4). Use the minimum path rule as for path 2-3 in q_c computations, b = pile width or diameter, D = embedded pile length.

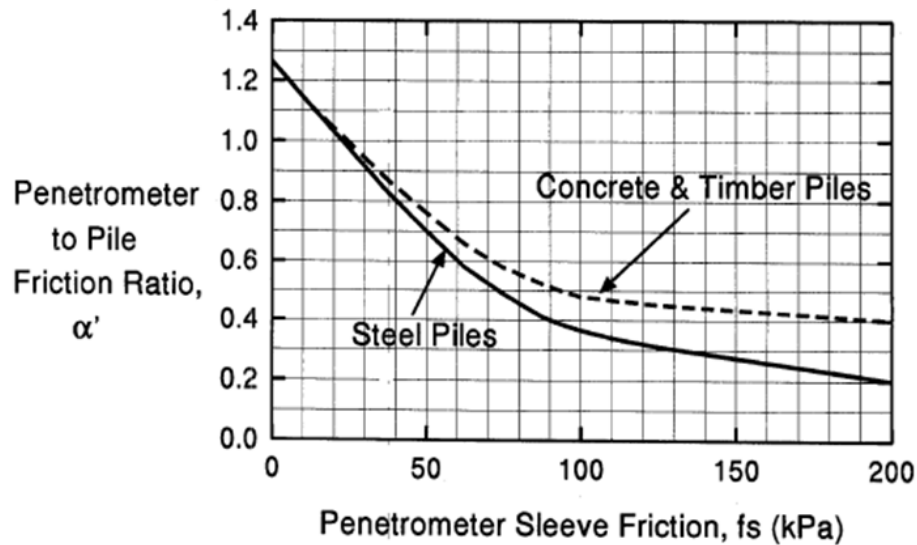


Figure 2.13: Design curve for pile side friction in clay (FHA, 1997)

Laboratoire des Ponts et Chaussees (LPC) Method

Figure 2.15 serves as a pivotal reference for determining shaft resistance values in cohesive soil conditions. The figure includes design curves 1 and 2, specifically tailored for cone resistance (q_c) values of 700 kPa and 1200 kPa, respectively. For situations where q_c falls within the range of 700 to 1200 kPa, it is advisable to employ interpolation techniques to accurately deduce the shaft resistance value. This interpolation ensures that the estimated shaft resistance aligns closely with the specific q_c value observed, facilitating a more precise and tailored approach to pile design in cohesive soils.

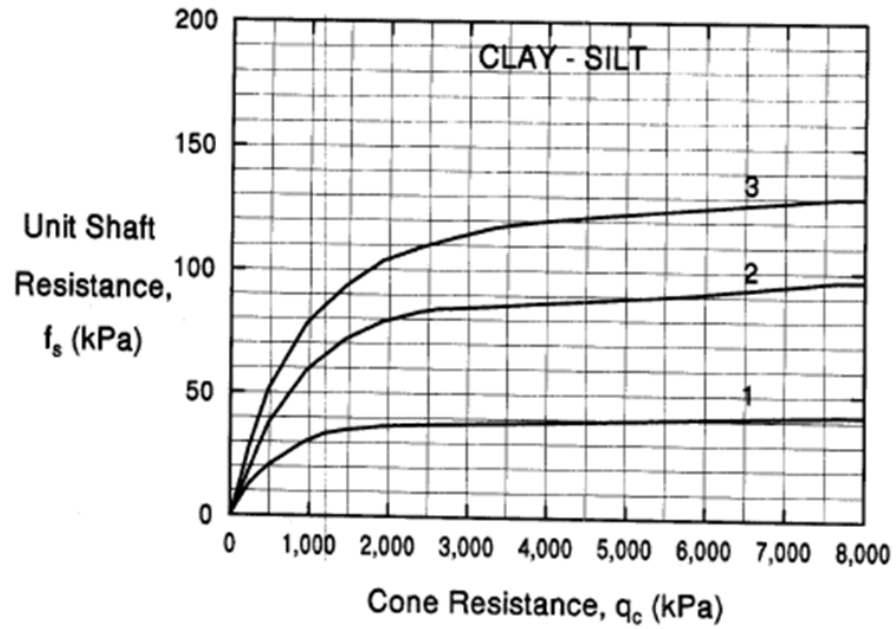


Figure 2.14: Maximum unit shaft resistance curves for LPC method (FHA, 1997)

Table 2.9: Driven pile type categories for LPC method (FHA, 1997)

Pile Type	Pile Description	Pile Installation Procedure
A	Driven prefabricated	Reinforced or prestressed concrete pile installed by driving or vibro-driving.
B	Concrete piles.	Pile made of steel only and driven in place: H pile, pipe pile or any shape obtained by welding sheet-pile sections.
C	Driven steel piles.	Made of hollow cylinder elements of lightly reinforced concrete assembled by pre stressing before driving. Each element is generally 1.5 to 3 m long and 0.7 to 0.9 m in diameter; the thickness is approximately 0.15 m. The piles are driven open ended.

Table 2.10: Curve selection based on pile type and insertion procedures (FHA, 1997)

Curve No.	q_c kPa	Pile Type	Comments on insertion procedure
1	< 700	A, B, C	
2	< 1200	A, B, C	For all steel piles, experience shows that, in plastic soil, f_s is often as low as curve 1. Therefore, use curve 1 in plastic soils when no previous load test data is available. For all driven concrete piles use curve 3 in low plasticity soils with sand or sand and gravel layers or containing boulders, and when $q_c > 2500 \text{ kPa}$.
3	> 1200	A, B	For all driven concrete piles use curve 3 in low plasticity soils with sand or sand and gravel layers or containing boulders, and when $q_c > 2500 \text{ kPa}$.

Bustamante and Gianselli (1983) proposed the LPC method. In the LPC method, the unit toe resistance can be calculated by using the cone bearing factor, K_c , which is found in Table 2-11. This factor should be multiplied by the average cone resistance, q_c , in a range of one pile diameter below the pile toe. The unit cone resistance is gained in the zone ranging 1.5 D above and below the pile toe. (Abu-Farsakh et al., 2008)

$$q_t = K_c q_c \quad (2.17)$$

Table 2.11: Cone bearing capacity factors for LPC method (FHA, 1997)

Soil type	Cone bearing factor, K_c
Clay-silt	0.600

2.3 Determination of Pile Capacity using Field Tests

2.3.1 Pile Dynamic Analysis

Dynamic pile testing has transitioned from an exclusive research tool to a routine procedure widely adopted across the construction industry. With a proven track record spanning over five decades, its cost-effectiveness and reliability have been firmly established, leading to widespread adoption by contractors, consultants, and government bodies. Many have invested in acquiring the necessary equipment to assess pile installation integrity or evaluate the bearing capacity of foundation elements. Alternatively, some opt for outsourcing

this testing to specialized service providers. The methodologies underpinning dynamic pile testing are comprehensively outlined in the ASTM D4945 standard, titled "Test Method for High Strain Dynamic Testing of Deep Foundations." This standard, along with various construction specifications and building codes (such as the International Building Code (IBC) and the American Association of State Highway and Transportation Officials (AASHTO) guidelines in the USA), often mandates or recommends dynamic testing. These regulations typically advocate for "signal matching," broadly known under the proprietary name CAPWAP. CAPWAP stands as the industry benchmark for capacity and stress analysis derived from dynamic test data. Adoption of progressive codes like the Load and Resistance Factor Design (LRFD) methodology, as advocated by AASHTO, highlights the economic benefits of dynamic testing. LRFD offers more favorable resistance factors, thereby incentivizing the use of dynamic testing through potential cost savings, diminished risk, and overall lower foundation expenses when juxtaposed with traditional static analysis or less sophisticated dynamic formula methods.

2.3.2 PDA testing

The Pile Dynamic Analysis (PDA) was originally designed to determine the static capacity of piles, offering an innovative approach that aligns well with static load test results. Over time, its application has broadened, frequently reducing or even replacing the need for static testing due to its reliable correlation with static load tests. For capacity evaluation using the PDA, results must be meticulously analyzed, often in conjunction with CAPWAP analysis or supplementary static tests (Linkins and Rausche, 2004). The utility of PDA in capacity computation encompasses several domains:

1. **Design Efficiency:** On large scale projects, PDA testing offers a cost effective strategy to assess pile capacity relative to penetration depth and time. This can enable higher design capacities for given driving resistances, potentially reducing pile numbers or penetration depths, thereby achieving significant cost reductions. It also facilitates the estimation of soil resistance distribution, including skin friction and end bearing.
2. **Understanding Setup and Relaxation Effects:** Testing immediately post driving and after a delay allows for monitoring soil strength changes and their impact on pile capacity. Acknowledging time dependent soil effects is crucial for accurately determining the service load capabilities of piles. Significant setup can lead to cost savings by reducing the number or length of piles required.

3. **Supplementing Static Tests:** In scenarios where static load tests are impractical, the PDA serves as a valuable tool, supplementing traditional geotechnical methods and wave equation analysis. It can replace certain static tests, offering time and cost efficiencies, and assist in determining optimal penetration or driving criteria for piles due for static testing.
4. **Construction Control and Quality Assurance:** Selective testing of production piles provides a measure of capacity across the site, ensuring quality control. For larger sites, periodic testing can offer a comprehensive overview of site conditions, with special attention to piles exhibiting unusual driving characteristics.

The dynamic nature of soil pile interaction, particularly in fine grained soils, underscores the PDA's relevance. Pile driving typically generates excess pore pressures, reducing immediate pile capacity by lowering effective stress on the pile. However, as these pressures dissipate, effective soil stresses and, consequently, axial pile capacity increase—a phenomenon known as soil setup. Most projects experience capacity benefits from this process, alongside potential effects of soil aging or arching from lateral installation movements. Conversely, relaxation observed in specific conditions like weathered shale showcases the complexity of predicting long term pile capacity based solely on initial driving or short-term restrike tests. CAPWAP analysis measurements taken with the PDA on the pile, usually near the pile top, during an impact can and should be evaluated by the Case Method. This method allows for a quick and simple calculation of soil resistance, integrity and stress values in closed form which means that limiting assumptions such as pile uniformity must be made. CAPWAP combines the measurements of the Case Method with a wave equation type numerical analysis. Since the pile top motion and forces have been measured it is then not necessary to model the hammer and driving system. Using the measurements as the input function, the soil model can be adjusted so that the computed response matches the measured response, and thus also the process is referred to as “signal matching”. Such a numerical analysis approach of the measurements yields a complete set of results for complex pile and soil systems.

THE CAPWAP SOIL MODEL

CAPWAP uses a modified Smith approach to calculate the resistance acting on a pile segment i during the dynamic event (CAPWAP Manual, 2014). The displacement and velocity of each pile segment relative to the soil is the basis for computing the soil resistance forces (Note: in the Smith approach, the soil surrounding the soil-pile interface is considered fixed; an extension to that fixed soil assumption is CAPWAP's radiation damping model,

which gives the soil one degree of freedom). The soil model consists of an elasto-plastic spring and a linear dashpot (Figure 2-16) described by three parameters at every soil segment k : ultimate resistance R_{uk} , quake q_k , and viscous damping factor J_k . The J_k damping factor can be estimated using a test device for simulating the dynamic response of the pile-clay interface in the laboratory and the test programme (Chin and Seidel 2004, Linkins et al., 1992). The total static bearing capacity R_u of the pile is the sum of all R_{uk} . The total (static plus dynamic) resistance force R_k at soil segment k is computed from

$$R_k = R_{sk} + R_{dk} \quad (2.18)$$

Where R_{sk} and R_{dk} are the time varying static and dynamic soil resistance forces at soil segment k which may act against the motion of pile segment i . Soil resistance forces may act at each embedded pile segment. However, since the pile segments are usually rather short and because of the limited resolution of the analysis approach, it may be sufficient to have one soil resistance element at the bottom pile segment for end bearing and one shaft resistance element at every second pile segment for the portion of the pile embedded in the soil (this better matches the resolution of the rise time – time from initial impact to the first peak in velocity). The soil resistance R_k which acts on the pile segment i can be calculated using the following equations (before rebound occurs):

$$R_k = k_s k_{ui} + J_k(v_i) \quad \text{for } u_i < q_k \quad (2.19)$$

$$R_k = R_{uk} + J_k(v_i) \quad \text{for } u_i \geq q_k \quad (2.20)$$

Where: v_i - velocity of the pile segment i ; u_i - displacement of the pile segment i ; k_{sk} - the stiffness of the soil in the pile-soil interface at soil segment k ; R_{uk} - the ultimate static resistance at soil segment k ; q_k – the quake at soil segment k , $q_k = R_{uk} / k_{sk}$.

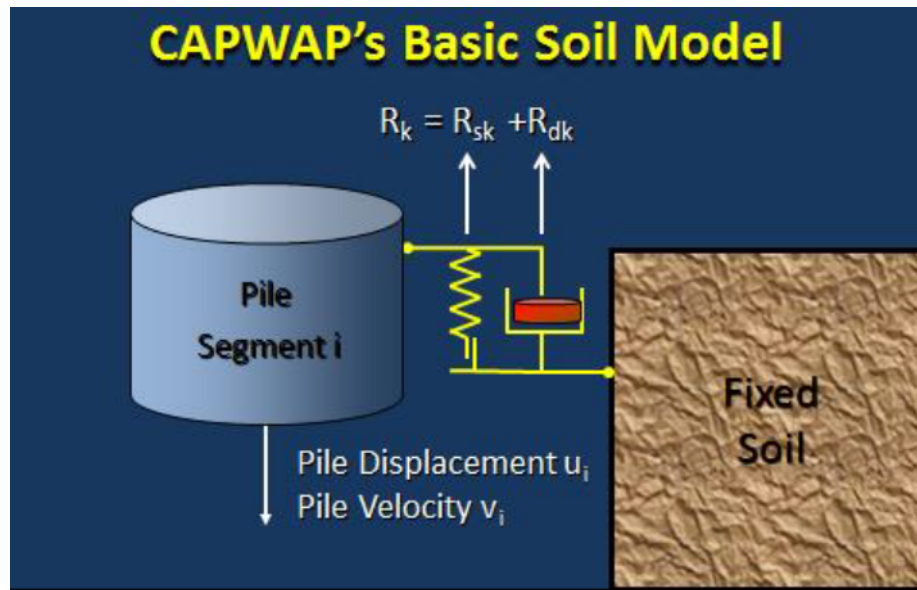


Figure 2.15: The CAPWAP soil resistance model is based on Smith with modifications (CAPWAP Manual, 2014)

CAPWAP Limitations

The efficacy of CAPWAP analysis in predicting the static behavior of piles is contingent upon a set of critical assumptions, which are essential for ensuring accuracy in its application (Linkins and Rausche, 2004; Linkins et al., 1996; Rausche et al., 2008). These assumptions encompass:

- a. Quality of Field Data:** The foundation of CAPWAP analysis lies in the acquisition of high-quality data from field measurements, ensuring the reliability of the analysis outcomes.
- b. Soil Resistance Model:** The adopted soil resistance model must be realistic, offering the capability to distinguish between static and dynamic resistance components effectively.
- c. Pile Representation:** Accurate modeling of the pile within the analysis is crucial, including its geometric and material properties.
- d. Hammer Energy:** The energy delivered by the hammer must be adequate to induce a significant permanent displacement of the pile, a key factor in dynamic analysis.
- e. Signal Match Quality:** Achieving a satisfactory match between measured and calculated signals is pivotal for the validation of analysis results.
- f. Long term Static Behavior Assessment:** For evaluations aimed at understanding

long-term static behavior, dynamic pile tests should be conducted during a restrike phase, following an appropriate waiting period post-installation, to accurately reflect the pile’s capacity at the time of testing.

The soil model utilized in CAPWAP shares many similarities with the model developed by Smith (1960), which has undergone extensive validation across numerous projects. Despite its widespread use, the Smith model is somewhat simplistic, given the complex and variable nature of soil properties, both globally and within individual sites. To address these challenges and enhance the model’s applicability, CAPWAP incorporates extensions to the basic Smith model. These modifications aim to (a) achieve closer alignment between measured and calculated variables at the pile top, and (b) surmount the aforementioned limitations, thereby improving the model’s predictive accuracy and reliability in diverse soil conditions.

2.3.3 Static Load testing

Static Static Load Tests are considered the premier method for determining the capacity of foundation piles, offering unparalleled accuracy in evaluating how a pile responds to applied loads. These tests not only determine the ultimate failure load but also examine a pile’s capacity to bear loads without excessive or ongoing movement. The primary goal is to verify that the constructed pile’s load-bearing capacity exceeds the design’s nominal resistance requirements, addressing factors such as compression, tension, and lateral forces (Fellenius, 2001a). Performing these tests in conjunction with Pile Dynamic Analysis (PDA) provides the best insights, allowing Geoengineers to adopt a more rational approach to foundation design (Caltrans, 2016). Static load testing is particularly recommended for piles placed in soils with varied geological features or those of lower quality. These tests help confirm or refine design assumptions, including adjustments in tip elevation. Although they yield more reliable data compared to pile driving formulas—and can show that driven piles might safely surpass formula-based capacity estimates—they are often less popular during the design phase due to their high cost and time-consuming process (Caltrans, 2016).

Test Procedure and Standards

The test involves applying axial loads to a pile’s top via one or more hydraulic jacks, adhering to established standards and procedures (e.g., ASTM D-1143, FHWA-SA-91-042, 1992; CEFM, 2023). These tests can be executed during either the design or construction stages, contingent on the project’s QA/QC requirements or foundational design parameters. For

smaller projects, the relative cost of testing can be a significant consideration against the overall foundation expense.

Loading Procedures

Two primary loading methodologies are utilized in static load testing:

- **Maintained Load (ML):** This approach involves applying load increments, each a substantial fraction of the maximum anticipated load, maintained until pile movement stabilizes below 0.01 inch per hour over two hours. The final load is sustained for 24 hours, with a Quick Test variant involving smaller increments and fixed time intervals for rapid assessment.
- **Constant Rate of Penetration (CRP):** This method entails continuously loading the pile to maintain a constant rate of head motion, with movement readings taken every 30 seconds. The pile's capacity is determined by the higher of two failure criteria: the first sets the allowable design capacity based on a specified settlement post-rebound, while the second employs the Davisson offset limit load criterion to define pile capacity or failure (FHWA, 2006).

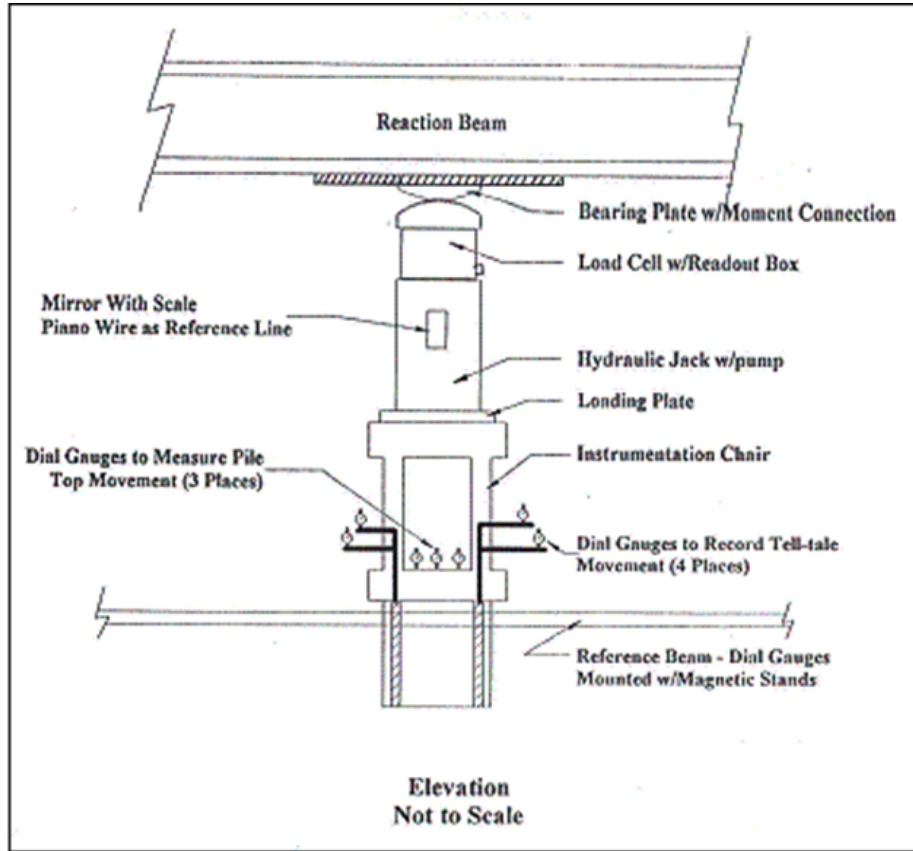


Figure 2.16: Typical static load test arrangement showing instrumentation (FHWA, 2006)

The ultimate load is interpreted as the point at which the displacement of the pile head meets a limit that is offset to the elastic compression line of the pile. For piles less than 61 cm in diameter, the limit is defined by the following linear relationship:

$$S_f = S_e + (0.38 + 0.008D) \quad (2.21)$$

Where: S_f = Movement of pile head (cm), D = Pile diameter or width (cm), S_e = Elastic compression of total pile length (cm). The elastic compression in this case refers to the pile deflection that would occur if 100 percent of the applied load was transferred to the toe of the pile (i.e., zero shaft friction), and is given by the following equation:

$$S_e = QL/AE \quad (2.22)$$

Where: Q = Applied load. L = Total length of pile, A = Cross-sectional area of the pile, E = Modulus of elasticity of the pile. The average load in the pile at the midpoint between two telltale locations was estimated from the elastic shortening of the pile using the following equation:

$$Q_{\text{avg}} = AE \frac{(D_1 - D_2)}{\Delta L} \quad (2.23)$$

D_1 = Deflection at upper telltale location, D_2 = Deflection at lower telltale location, ΔL = Distance between the upper and lower telltale locations. The specifications require that the elastic modulus be determined via compression tests performed on representative concrete samples (ASTM C 469-87a). However, this method is not applicable to the concrete-filled steel pipe piles. LI YI (2004) summarized these effects as “ASTM (1987) suggests the clear distance between the test pile and the reaction pile(s) or cribbing shall be at least five times the butt diameter or diagonal dimension of the test pile, but not less than 2.5m; it also notes that factors such as type and depth of reaction, soil conditions, and magnitude of loads should be considered. Weltman et. al. (1978) consider a distance from the face of the test pile of 1.0m should be appropriate in the kentledge reaction system while in the tension pile reaction system, at least $8d$ (diameter of the pile) would be entailed, whereas 3 to $4d$ is employed and a lower limit of 2.0m is recommended in practice. It is noted that the significant interaction between the test pile and reaction system within 3 times diameters of the test pile is common sense. The effects of interaction between reaction piles and the test pile have been examined theoretically with the elastic method by Poulos and Davis (1968), Poulos and Mattes (1975), and Letotske et al. (1997). To minimize the error, a correction factor, F_c , is defined. It may be seen that in the range of spacings between the test and reaction piles commonly used (2.5 to 4 diameters), F_c may be 2 or even greater. The error becomes more severe for stiffer, more slender piles. In this case, the interaction is generally much less, and consequently, large values of F_c do not occur at normal spacing unless the piles are relatively slender and compressible (Fellenius 2009). Both cases suggest that the usual spacing of about three diameters may result in significant under-measurement of the settlement of the test pile. Zheng (1999) made a nonlinear analysis considering the small strain stiffness variation for soil on the influence of the rectangular-shaped kentledge cribbage on the test pile. Zheng (1999) has analyzed pile load tests using two reaction piles under working load in non-homogeneous London clay with soil stiffness proportional to depth, in which parametric studies were conducted to illustrate the influence of the pile diameter D and the L/D (L is the length of the pile) etc., on the interaction factor ratio β (the ratio of the interaction factor α_2 for two piles at a spacing of ‘ $2S$ ’ over the interaction factor α_1 for two piles at a spacing of ‘ S ’) in different

soils.” **Figure 2-18** indicates plots of pile head load movement relationships for the three loading procedures.

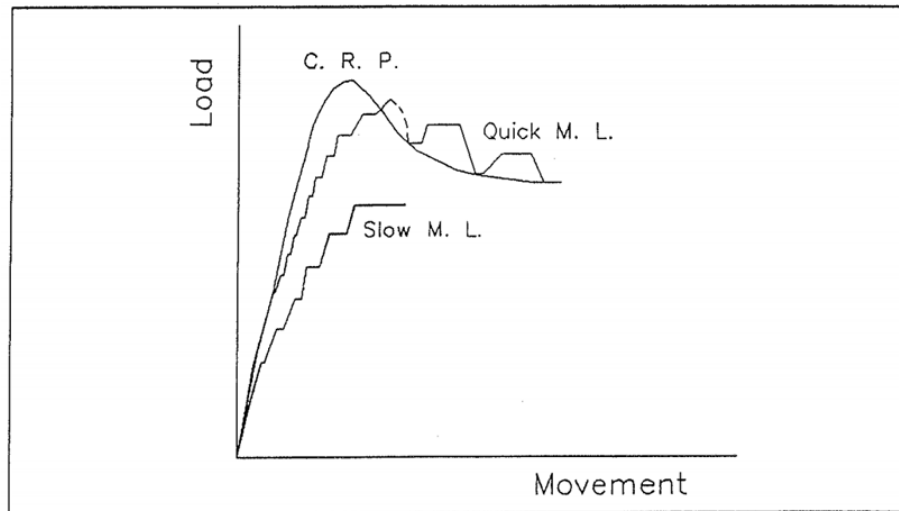


Figure 2.17: Comparison of Typical Load-Movement Behaviour for Three test procedures

The problems and limitations of static load testing can be categorized as follows:

- Difficulties arising from incorrectly performed tests and problems inherent in the static test itself. The cost also will increase as load test increase, you may need bigger beam and frame.
- Safety limitations: the large loads involved in a static test store large amount of elastic energy. if something fails, serious injury of testing personnel is possible.
- Problems in measurement of load and movement: Load measurement is very difficult to precisely measured. The most common means of load measurement is to read the jack pressure. This can produce large error due to jack friction. Felennius et. al (1999) indicated error of 15
- Training: The personnel that doing this type of test needs to have adequate training
- Pile failure: Pile failure may not be happened in some cases. research (Dennis and Olson, 1983; Wysocky and long 1994) indicated that testing has not been done professionally in 15 to20

- Offshore: Static load test may not be possible in case of offshore projects such as oil and gas or in deep water river
- Not Enough information: Static load test can't provide any information regarding hammer, termination criteria and time required to drive the pile. Depends on hammer energy, pile tension may be varied.
- Pile damage: if there is any damage on installed pile, static load test can't provide any information.
- Long term behaviour of soil: effect of long-term behaviour of soil due to pore water pressure dissipation on bearing capacity can't be found in static load testing (Chang 2001).

2.4 Design Criteria

2.4.1 General

Over the years, geotechnical design procedures have evolved to ensure that designs meet performance requirements in the face of uncertainties. The below items need to be considered to define the design approach for pile foundation:

- **Functionality of structural elements:** Making decision on, investigation approaches, site selection, and design methods may be affected by the type, purpose and function of the structure. The goal structure should be designed based on the failure consequences, which can be listed as loss of lives and property, economic damages either local or national.
- **Serviceability:** There are two main categories for structure or foundation failure definition, an actual collapse or a Structure or a functional failure. Functional failure may take place as excessive deflection, unacceptable differential movements, excessive vibration, and premature deterioration due to environmental parameters. As a result, factor of safety against the collapse is not sufficient and the effects of settlement and vibration on functional performance should be considered.
- **Soil-Structure Interaction:** The functional aspect of the structure and the economic significance of the structure will govern the design method, the analysis, type of the foundation, the pile test, and the analytical models for the pile and the structure.

- **Construction Limitations:** Installation of the pile foundation is another critical issue that must be done under high levels of cooperation between the design engineer and field crews (US Army Corps of Engineers, 1997).

2.4.2 Work Stress Design

In the early 1800's, the work stress design (WSD) or allowable stress design (ASD) has been the traditional design basis in geotechnical engineering worldwide. Safety is achieved by restricting the applied loads to values less than the ultimate geotechnical resistance divided by a factor of safety using the mathematical relationship of the general form (Dithinde, 2007):

$$\frac{R}{FS} \geq \sum Q \quad (2.24)$$

In which: R = ultimate geotechnical resistance; FS = factor of safety; $\sum Q$ = summation of load effects (dead and live loads). Two alternative definitions of the factor of safety emerge and are defined as follows (Dithinde, 2007):

- Meanfactorofsafety= $\frac{\bar{R}}{\bar{Q}}$
- Nominalfactorofsafety= $\frac{R_n}{Q_n}$

In geotechnical design, the factor of safety is generally applied to the resistance side on account that uncertainties in soil parameters are the largest uncertainties affecting geotechnical design. The values of the global factors of safety for various geotechnical structures have been developed from previous experience with similar structures under similar conditions (Terzaghi, 1942; Becker, 1996; Becker et al. 2005). They reflect experience and the consequences of failure. In geotechnical design, Belidor (1776) and Coulomb (1729) who both suggested placing a value of 1.23 on the width of retaining walls determined from earth pressure theory, and later. Meyerhof (1995) recommended a range of 2-3 on the ultimate bearing capacity of foundations. FS guarantees that the foundation or structure has reserve capacity against collapse in case of a given load. Selecting the proper factor of safety requires a high level of knowledge about the design parameters. For each component of the bearing capacity, a proper factor of safety can be assigned:

$$R_a = \frac{Q_s}{FS_s} + \frac{Q_b}{FS_b} \quad (2.25)$$

Table 2.12: Ranges of global factor of safety for geotechnical design (After Terzaghi and Peck, 1948, 1967)

Failure type	Item	Factor of Safety
Shearing	Earthwork	1.3-1.5
	Earth Retaining Structure	1.5-2
	Foundations	2-3
Seepage	Uplift heave	1.5-2
	Exit gradient, piping	2-3
Ultimate pile loads	Load tests	1.5-2
	Dynamic formulates	3

Where: R_a = allowable pile capacity, FS_s = pile shaft factor of safety, FS_b = pile base factor of safety. Or the allowable pile capacity can be calculated by using a single value factor of safety:

$$R_a = \frac{R}{FS} \quad (2.26)$$

A normal range for the factor of safety that is chosen by design engineers is between 2.0 and 3.0, depending on design parameter uncertainties. (Bowles, 1997).

2.4.3 Limit state design

In general, two main categories of limit states are normally considered. These include (Hansen, 1956; Simpson et al., 1981):

- The ultimate limit state and
- Serviceability limit state.

Ultimate limit states are associated with the total or partial collapse of the structure (e.g., strength, ultimate bearing capacity, overturning, sliding, etc). It concerns the safety of the structure and its contents including people. Typical examples include loss of equilibrium of a part or all of a structure as a rigid body leading to overturning, sliding, etc. Conversely, serviceability limit states correspond to those conditions beyond which specific requirements of the structure or structural element are no longer met.

Different type of Limit state design

In the last 10 years, there has been a trend to use probabilistic methods for Limit State design for geotechnical problems (Dithinde, 2007; Randolph et al., 1994). In this theory, probability provides the basis for measuring this ability. In a probabilistic setting, the reliability of an engineered system is the probability of its satisfactory performance for a specific period under specific service conditions (i.e. the probability of survival) (Ebeling, 1997). Mathematically, the relationship between the probability of failure (Pf) and reliability (R) is expressed as follows:

$$P_f + R = 1 \quad (2.27)$$

Complete probabilistic design is difficult to apply reliably and properly for geotechnical problems as it lacks statistically viable information (Borden 1981). These types of approaches are also time-consuming and expensive, which makes them impractical. Because of these difficulties, simpler based design has been developed (Probabilistic Level I and II) (CEFM, 2023; Burland et al., 1978).

Load and Resistance Factor Design (LRFD) approach

The basic hypothesis of the LRFD is quantifying the uncertainties based on probabilistic approaches, which aims to achieve engineered designs with consistent levels of reliability (or probability of failure) (Dithinde, 2007). In the LRFD approach, different load types and combinations are multiplied by load factors, while resistances are multiplied by resistance factors, where the factored loads should not exceed the factored resistances. The first geotechnical LRFD manual was developed in 1989 by the American Association of State Highway and Transportation Officials (AASHTO). The FHWA released the FHWA Manual for the Design of Bridge Foundations in early 1990 (FHWA, 1997), followed by the National Cooperative Highway Research Program (NCHRP) Report 343 by Barker et al. (1991). A simple definition of failure is when the factored loads exceed the factored capacities. The basic equation of the LRFD-based design can be expressed as follows (Dithinde, 2007):

$$\phi R_n \geq \sum \alpha_i S_i \quad (2.28)$$

Where: ϕR_n is the factored geotechnical resistance, ϕ is the geotechnical resistance factor, R_n is the ultimate geotechnical resistance determined through engineering analysis (Semi-equation) using field data or performance data and it represents the geotechnical

engineer's best estimate and has considered all the factors influencing resistance, $\sum \alpha_i S_i$ is the summation of factored overall load effects for a given load combination condition, α_i is the load factored corresponding to a particular load, S_i is the specified load component of the overall load effects ; and i represents various types of loads such as dead load, live load, wind load, etc. The values for load factor (α_i) and ϕ are various and can be obtained from applicable codes such as NBCC 2005, CHBDC 2024, AASHTO 2007, EN 1990, EN 1997 etc. The resistance factor, however, is less than one and accounts for variabilities in geotechnical parameters (Ellingwood et al. 1982a , 1982b). Assuming independent variables for load effects and resistance which is the case of static loading, and consider nominal values for load effect and resistivity effect as below:

$$R_n = \frac{\bar{R}}{K_r} \quad (2.29)$$

$$S_n = \frac{\bar{S}}{K_s} \quad (2.30)$$

Where: K_r is the ratio of mean value to nominal value for resistance, K_s is the ratio of mean value to specified value for load effects. The K_r is greater than or equal to one and K_s values are less than one. These two factors are referred to as bias factors in literature. The magnitude of the load and resistance factors, and the difference between R and Q , are determined such that the probability of failure, P_f , that Q is greater than R is acceptably small (Allen et al. 2005). **Figure 2-19** illustrates the principle, in this case for two normal distributions.

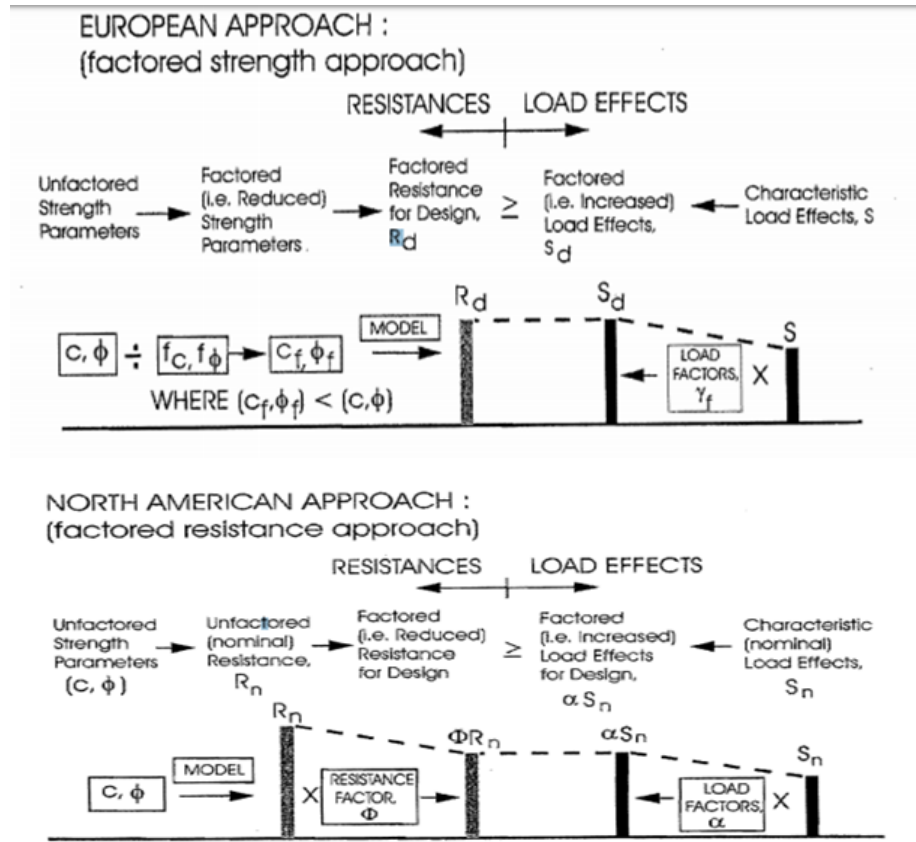


Figure 2.18: Comparison of limit states design approaches for ultimate limit states (Becker 1996)

Figure 2-21 illustrates the relationship between β and the probability of failure P_f . This relationship is developed from the eq below using the Microsoft Excel Function NORMSDIST, which returns the standard normal cumulative distribution function (CDF) value for a given value of β , as shown below (Allen et al. 2005):

$$P_f = 1 - \text{NORMSDIST}(\beta) \quad (2.31)$$

Figure 2-20 applies to a normally distributed function, g . The more the limit state function value, g , departs from a normal distribution, the more approximate the relationship shown in **Figure 2-21** becomes. However, the individual variables used to calculate g do not necessarily need to be normally distributed. Probability density functions (PDFs) and

cumulative distribution functions (CDFs) are used throughout the calibration process to statistically characterize the random variables used in the calibration process.

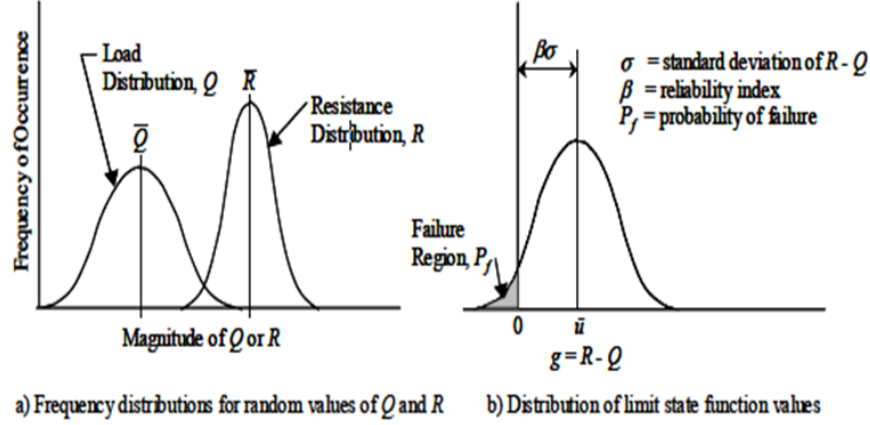


Figure 2.19: Probability of failure and reliability index (Withiam, et al.,1998).

Although values of ϕ may differ between codes in various countries, load factors are typically in the range of 0.85 to 1.3 for dead loads and in the range of 1.5 to 2.0 for live and environmental loads. A load factor of α_i less than 1.0 for dead loads is used when the dead load component contributes to the resistance against overturning, uplift, and sliding. Typical values of α_i range from about 0.3 to 0.9, depending on ground type, method of calculating resistance, and class of structure such as foundation type or retaining structure. The recommended resistance factor per CEFM (2023) is as below; in these tables, the reliability index of 2.8 to 3.5 is considered, which is generally consistent with the design of structures.

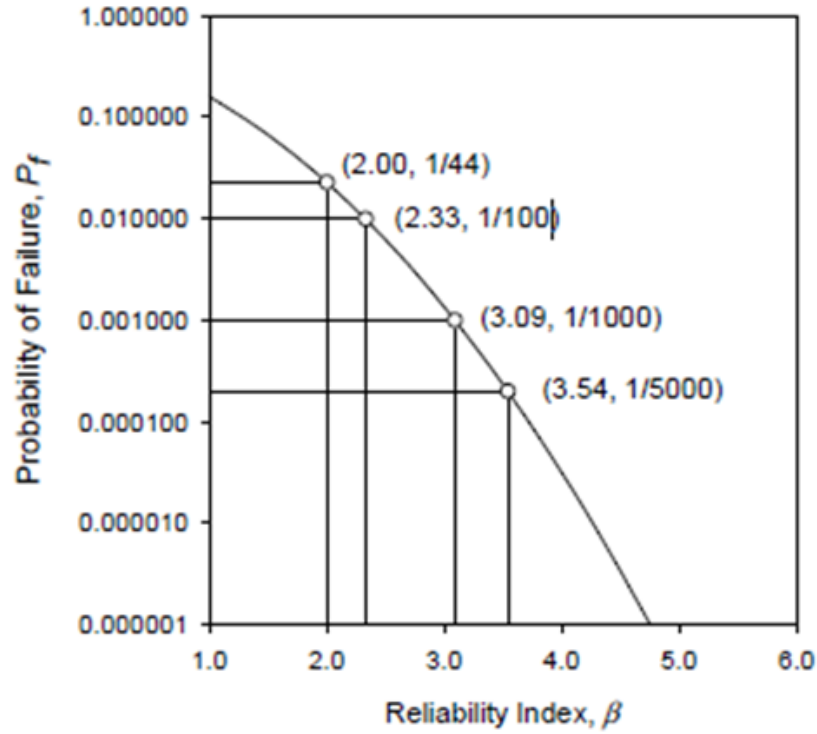


Figure 2.20: Relationship between β and P_f (Allen et. al. 2005).

The AASHTO Code (1997 and 2004) or API (1989) specifies many more resistance factors than are provided by CHBDC and NBCC. The 2006 as well as the current 2007 AASHTO-LRFD bridge design specifications depend on the NCHRP Report 507 by Paikowsky et al. (2004) and the FHWA-NHI report by Allen (2005).

LRFD Challenges for Pile Design

LRFD approaches have been developed in offshore engineering (e.g., Tang, 1993; Hamilton and Murff, 1992), general foundation design (e.g., Kulhawy and Mayne, 1990), and pile design for transportation structures (Barker et al., 1991; O'Neill, 1995). The *LRFD Bridge Design Specifications* (AASHTO, 2012) states that the ultimate resistance (R_n), multiplied by a resistance factor (ϕ), which thus becomes the factored resistance (R_r), must be greater than or equal to the summation of loads (Q_i) multiplied by corresponding load factors (γ_i) and a modifier (η_i). For strength limit states:

$$R_r = \phi R_n \geq \sum \gamma_i \eta_i Q_i \quad (2.32)$$

Where:

$$\eta_i = \eta_D \eta_R \eta_I > 0.95 \quad (2.33)$$

Where η_i = factors to account for, η_D = effects of ductility, η_R = redundancy, and η_I = operational importance.

The specifications provide the following equations for determining the factored bearing resistance of piles, Q_R :

$$Q_R = \phi Q_n = \phi_q Q_{\text{ult}} = \phi_{qp} Q_p + \phi_{qs} Q_s \quad (2.34)$$

For which:

$$Q_p = q_p A_p \quad (2.35)$$

$$Q_s = q_s A_s \quad (2.36)$$

Where: ϕ_q = resistance factor for the bearing resistance of a single pile specified for methods that do not distinguish between total resistance and the individual contributions of tip resistance and shaft resistance, Q_{ult} = bearing resistance of a single pile, Q_p = pile tip resistance, Q_s = pile shaft resistance, q_p = unit tip resistance of pile, q_s = unit shaft resistance of pile, A_s = surface area of pile shaft, A_p = area of pile tip, ϕ_{qp} , ϕ_{qs} = resistance factors for tip and shaft resistance, respectively.

The resistance factors for compression vary between 0.45 and 0.70. Primary sources of geotechnical uncertainties can be classified into three categories:

1. Inherent variability,
2. Measurement uncertainties, and
3. Transformation uncertainties (Kulhawy & Phoon, 2002).

The first source results largely from the natural geologic processes that produced and continually modify the soil mass in-situ. The second source arises from:

- Sampling methods used to obtain soil/rock specimens (e.g., a standard penetration test, SPT),
- Field or laboratory testing techniques used to evaluate soil/rock properties (e.g., SPT blow count or triaxial tests), and
- Measurements of soil/rock properties in the field and laboratory, which produce random errors typical of all measurements (Withiam et al., 1998).

In general, tests that are highly operator-dependent and have complicated procedures will have greater variability than those with simple procedures and little operator dependency (Kulhawy & Trautmann, 1996).

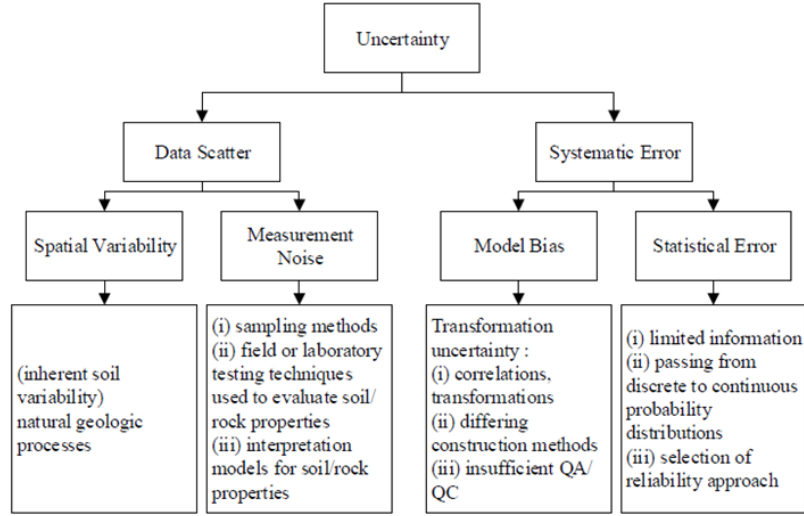


Figure 2.21: Chart Summarizing Different Sources of Uncertainty in Geotechnical Reliability (Jabo, 2014)

2.4.4 Reliability Analysis and Calibration Criteria

Reliability Targets

The selection of the target reliability index, β_T , is an important step in the calibration process. One of the most important selection criteria is past practice. The value of β implied by the safety factor, FS , prescribed in past or current work stress design (WSD)

specifications can be used as a starting point to establish the β_T value. If the safety factor from previous WSD practice has been proven from experience to consistently produce safe designs, the safety level can be assumed to be adequate or even higher than what is needed [?]. This assumption allows for the selection of the target value of β , which can be calculated as follows (Nowak and Collins, 2000):

$$\beta = \sqrt{\frac{\bar{R} - \bar{Q}}{\sigma_R^2 + \sigma_Q^2}} \quad (2-61)$$

Where: $\bar{R}$ = the mean of the resistance R , $\bar{Q}$ = the mean of the load Q , σ_R = the standard deviation of the resistance R , σ_Q = the standard deviation of the load Q . The selection of the target β to estimate the load and resistance factors depends on the desired probability of failure, P_f . In general, strength limit state resistance factors for structural design are derived to produce a β value of 3.5 ($P_f \approx 1$ in 5,000) for structural components. However, past geotechnical design practice has resulted in an effective β value for foundations of approximately 3.0, or $P_f \approx 1$ in 1,000 (Withiam, et al., 1998).

β Value for Geotechnical Components

Moses and Verma (1987) suggested target reliability levels for calibrating bridge codes (e.g., AASHTO Specifications). For bridge spans of less than 100 ft, a β_T of 2.5 to 2.7 is suggested for redundant bridges and a β_T of 3.5 for nonredundant bridges. Barker et al. (1991) indicate that values of β between 1.5 and 2.8 are generally obtained for the lognormal procedure. Thus, a target value of β between 2.5 and 3 may be appropriate. However, piles are typically used in groups. The failure of one pile does not necessarily imply failure of the pile group. Due to this redundancy in pile groups, it is suggested that the target reliability index for driven piles can be reduced from 2.5–3.0 to a value between 2.0 and 2.5 (Banerjee and Driscoll, 1976; Hussein et al., 2002). Reliability indices for driven piles are summarized in the table below.

Reliability Indices for Driven Piles

Zhang et al. (2001) used a first-order reliability method to evaluate the reliability of axially loaded pile groups designed using the traditional concept of group efficiency. Group effects and system effects were identified as the major causes of the significantly greater observed reliability of pile foundations compared to the calculated reliability of single piles. Group effect relates to the combined action of any number of piles versus a single pile (Banerjee

Table 2.13: Reliability Indices for Driven Piles (Barker et al., 1991)

Dead to Live Load Ratio	Target Reliability (β_T) - Lognormal	Target Reliability (β_T) - Advanced
1.00	1.6–2.8	1.6–3.0
3.69	1.7–3.1	1.8–3.3

and Driscoll 1976). A system effect relates to the contribution of superstructure stiffness to load distribution and resistance. The calculated probability of failure of pile groups was found to be 1 to 4 orders of magnitude smaller than that of single piles, depending on the significance of system effects (changing the system bias factor λ_s from 1 to 2 (Comodromos, and Anagnostopoulos, 2003)). Based on their study, Zhang et al. (2001) state that the target reliability index, β_T , for achieving a specified reliability level should differ for an isolated single pile (β), an isolated pile group (β_{TG}), and a pile system (β_{TS}). They give the following recommendations based on their research (Sabdelsalam, 2014):

- A β_{TG} value of 3.0 requires a β of 2.0–2.8 if no system effects are considered.
- A β_{TG} value of 3.0 requires a β of 1.7–2.5 if a system effect factor of 1.5 is considered.

An additional aspect to the increased reliability of deep foundations can be obtained from the limited data available regarding the loads, which arrive at the piles during their service. Tang et al. (1994) followed the response of drilled shafts during construction loading and found that, while 44% to 67% of the design load was measured at the pile's top, only 6% to 13% of the design load arrived at the tip in the rock socket. In the design of drilled shafts, the friction or the end bearing are often being neglected, especially in rock sockets. This practice and the observed values suggest that piles are often underutilized (over conservative), a fact contributing to the reliability of pile foundations, which rarely fail. These facts, while recognized, cannot be considered when assigning a target reliability value until more data are available and relevant load factors can be directly developed for foundations.

Load Conditions and Load Factors:

The use of a load combination that includes lateral loading may at times be the restrictive loading condition for deep foundations design. Pile lateral capacity is usually controlled by service limit state, and as such, was excluded from the scope of the present study,

which focuses on the axial capacity of single piles/drilled shafts. The load combination for strength I was therefore applied in its primary form as shown in the following limit state (Sabdelsalam, 2014):

$$Z = R - D \quad (2.37)$$

Where: R = strength or resistance of pile, D = dead load and LL = vehicular live loads. The probabilistic characteristics of the random variables D and LL are assumed to be those used by AASHTO (Nowak, 1999a) with the following load factors and lognormal distributions (bias and COV) for live and dead loads, respectively:

$$\gamma_L = 1.75 \quad \lambda_{QL} = 1.15 \quad COV_{QL} = 0.2$$

$$\gamma_D = 1.25 \quad \lambda_{QD} = 1.05 \quad COV_{QD} = 0.1$$

2.4.5 Calibration Techniques

First Order Second Moment (FOSM)

First Order Second Moment (FOSM) is a closed form solution and a probabilistic reliability method that produces a resistance (φ) factor for a given reliability index (β). It is based on the lower order terms from a Taylor series expansion of the performance function to approximate the variance of the performance function (Baecher and Christian 2005). The FOSM is a first-order expansion about the mean value and a linear approximation of the second moment (the variance) (Patev 1995). FOSM has had a presence in geotechnical engineering since 1969 when it was originally proposed by Benjamin and Cornell (1970). It was subsequently used by Barker et al. (1991) for NCHRP Report 343. Scott and Salgado (2003) indicated the lognormal distribution better represent and models the transient loads and fully characterizes it by its first two moments.

The FOSM method has made two assumptions to derive a closed form solution. The first assumption is that the moments of failure can be estimated when the mean values of the variables are known (Baecher and Christian 2005). The second is an assumption about the type of probability distribution which describes the data. When this distribution is known, the probability of failure and the reliability index can be computed (Baecher and Christian 2005). While closed form solutions are easy to apply, they can be too simplistic for foundation design (Phoon and Kulhawy 1996).

First Order Reliability Method (FORM)

The First Order Reliability Method (FORM) was first proposed for use in 1974 by Hasofer and Lind (Baecher and Christian 2005) to address the limitations of the FOSM by identifying a geometric interpretation of the reliability index (β) and not relying on assumptions or extrapolations. The reliability index (β) is the measured distance in dimensionless space between the mean of the limit state function or performance function as $g = R - Q$ and the failure function ($g = 0$) (Baecher and Christian 2005). To be consistent with the current structural code and the load factors to which it leads, it is necessary for calibration of resistance factors for deep foundations to use FORM (Nowak, 1999b). Following Ayyub and Assakkaf (1999), and Ayyub (1999), the present project calibrates LRFD partial safety factors using FORM, as developed by Hasofer and Lind (1974). FORM can be used to assess the reliability of a pile with respect to specified limit states and provides a means for calculating partial safety factors φ and γ_i for resistance and loads, respectively, against a target reliability level, β_T . FORM requires only first and second moment information on resistances and loads (i.e., means and variances) and an assumption of distribution shape (e.g., normal, lognormal, etc.). In design practice, there are usually two types of limit state: ultimate limit state and serviceability limit state. Each can be represented by a performance function of the form:

$$g(X) = g(X_1, X_2, \dots, X_n) \quad (2.38)$$

in which $X = (X_1, X_2, \dots, X_n)$ is a vector of basic random variables of strengths and loads. The performance function $g(X)$, often called the limit state function, relates random variables to either the strength or serviceability limit state. The limit is defined as $g(X) = 0$, implying failure when $g(X) < 0$ (Figure below). The reliability index, β , is the distance from the origin of the space of basic random variables to the failure surface at the most probable point on that surface, that is, at the point on $g(X) = 0$ at which the joint PDF of X is greatest. This is sometimes called the design point and is found by an iterative solution procedure (Thoft-Christensen and Baker, 1982). The relationship of the limit states can also be used to back-calculate representative values of the reliability index, β , from current design practice.

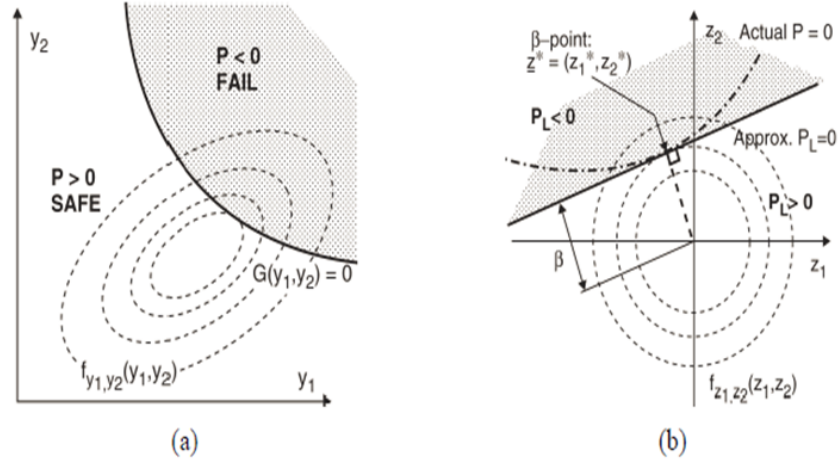


Figure 2.22: Presentation of (a) General Reliability Problem with Constrained Nonlinear Optimization in Original Physical Space, and (b) Solution using FORM with an Approximate Linear Limit State Function in Standard Space (Phoon, 2008).

Monte Carlo Simulation

Monte Carlo Simulation (MCS) is a form of experimental mathematics that provides probabilistic or deterministic solutions to problems. The MCS method has become AASHTO's preferred calibration tool and is recommended for all AASHTO related calibrations of LRFD factors (Paikowsky et al. 2010). The MCS method generates random values of load (Q) and resistance (R) based on the mean, COV, and distribution of the sample. The limit state function, $g = Q - R$ or $g = \ln(Q/R)$, is formed with the random values and evaluated for failure. Failure is defined when $g \leq 0$. A predetermined quantity of simulations is executed with the random values, and a tally of the total times failure occurred is divided by the number of simulations, giving the probability of failure (pf). Calibration Results Summary in Literature

Using ASD Fitting Calibration

NCHRP Report 343 by Barker, et al. (1991) is the source of the AASHTO (2004) LRFD resistance factors for driven pile foundation design. This report only evaluated the reliability of static analysis methods in predicting the bearing capacity of driven piles. While the report did not evaluate dynamic prediction methods, it included a parameter, λ_v , which

accounted for the effect of quality control procedures during pile driving, such as the use of the ENR (Engineering News-Record) driving formula for pile capacity acceptance. Calibrations using reliability theory were performed for the λ -Method (Vijayvergiya & Focht Jr., 1972), the CPT Method (Nottingham & Schmertmann, 1975, Es-lami and Fellenius 1995), the SPT Method (Meyerhof, 1976), the β -Method (Esrig & Kirby, 1979), and the α -Method (Tomlinson, 1987, 1994, 1957). In general, Barker’s calibrations attempted to replicate the design capacities that matched those created by using Allowable Stress Design (ASD) procedures. This resulted in using reliability theory with a target reliability index β of between 2.0 and 2.5. Barker et al. (1991) provided statistics for model error, systematic error in the measurement of the soil parameters, and error due to inherent spatial variability. Because the level of safety represented by the resistance factors determined by a calibration fitted to ASD safety factors may not be consistent with the level of safety represented by those determined using reliability theory, Barker et al. (1991) considered the results of previous studies to make the final selection of resistance factors. The α -Method was found the most reliable, followed by the λ -Method. The β -Method was found to be the least reliable of the five static analysis methods. The selected resistance factor for the α -Method heavily depended on the reliability of theory results and was higher than the resistance factor implied by fitting to ASD. Similarly, for the λ -method, the reliability theory calibration resulted in a resistance factor that was slightly lower than the resistance factor implied by past practice (Sabdelsalam, 2014).

Calibration Approach using FORM/FOSM

The LRFD calibration conducted by Paikowsky et al. (2004) in NCHRP report 507 was based on 338 pile case histories amassed in a more robust database of pile load tests. “There was no need to make any assumptions of spatial variability and systematic error as separate sources of error because the size of the database was large enough to adequately account for all sources of error. Although pile data was grouped in the database according to soil type, resistance prediction method, and pile type, the details of the characteristics of each subgroup were not provided. The study found that the resistance data followed a lognormal distribution. The LRFD calibrations were performed for the α -, the λ -, the β -, the SPT, the Fellenius, 2009, and the Nordlund/Thurman Methods using two reliability methods (Sabdelsalam, 2014). The two methods were the First Order Reliability Method (FORM) and First Order Second Moment Method (FOSM). However, the results relied more heavily on the FORM results. To select the target reliability indices, Paikowsky et al. referenced past work including NCHRP Report 343 by Barker et al. (1991) among others. They assigned a target reliability index of 3.0 for pile groups that contain fewer than five piles, and a target reliability index of 2.3 for pile groups that contained five or

more piles (Cadden, 2005). In general, the resistance factors obtained in NCHRP report 507 were more conservative than the values obtained in NCHRP report 343 for all methods of capacity prediction except for the Schmertmann CPT method. To consider the lack of redundancy in the foundation system, the study proposed a reduction of 20 percent of the resistance factors. The study also conducted a calibration of resistance factors for dynamic analysis methods. These methods involve pile tests that are carried out during pile driving and monitoring to verify the accomplishment of design pile capacity and to avoid pile damage during driving due to hammer impact. During calibration for dynamic tests, the pile database was divided into subgroups based on the method of analysis and driving conditions at the time of analysis such as end of driving (EOD) or beginning of restrike (BOR). The hammer used for the load testing was assumed to be the same as the hammer used for the production pile driving.

The number of load tests in each subgroup varied from 99 to 384. The study showed that the number of piles which should be dynamically tested on a project depended on the size of the project, the degree of variability in the subsurface conditions, amount of SLT information available on the project, as well as the reasons for performing dynamic tests. “Thus, they provided recommendations for the number of piles to be dynamically tested as well as the acceptance criterion for a driven pile project.” An average capacity was set to a minimum of 85 percent of the nominal capacity as an acceptance criterion for a set of driven piles.

Regionally LRFD Calibration (Case Histories)

Different states conducted a similar study to establish state-specific recalibration of resistance factors for driven piles. Those states faced similar challenges and converged to similar trends and conclusions as Paikowsky et al. (2004) which are summarized below (Sabdel-salam, 2014). Based on a database of 218 pile cases, McVay et al. (2000) reported the statistical parameters and the corresponding resistance factors for various dynamic methods used in LRFD procedures for driven piles in Florida. In total, eight dynamic methods were studied including the ENR, modified ENR, Florida DOT method, Gates driving formulas, CAPWAP, Case method for PDA, Paikowsky’s energy method, and Sakai’s energy method. It was demonstrated that the modern methods based on wave mechanics, such as signal matching, are about two times more cost-effective in reaching target reliability indices of 2.0 to 2.5 than the ENR and modified ENR driving formulas. Like previous studies, this study demonstrated that static analysis methods used in most design practices over-predicted the observed capacities as determined by static load testing; whereas most dynamic capacity evaluation methods used for quality control were found to under-predict

measured pile capacities. The study proposed an efficiency measurement index to objectively assess the performance of pile analysis methods to overcome the shortcomings of economy evaluation based on absolute values of the resistance factors (Sabdelsalam, 2014).

The Bridge Section of the Oregon Department of Transportation (ODOT) conducted a similar project of recalibration of resistance factor (Smith et al., 2011). However, it was solely limited to establishing a resistance factor for the GRLWEAP. In that study, existing national databases were assessed and used together with new cases from the literature to build a comprehensive database. It was reported that the information in these databases was not always correct. There were a number of anomalous load tests and many load tests which were missing critical data such as blow count, especially at the BOR condition. The FHWA static capacity software program, DRIVEN, was used for the capacity prediction and the GRLWEAP program was adopted for the creation of the bearing graph (Smith et al., 2011). These predictions generated a bias λ and COV statistics for a range of ODOT selected scenarios. A reliability index β of 2.33 associated with the case of redundant piles in groups was used to establish resistance factors and efficiency measures (ϕ/λ). The FOSM reliability method was used for the calibration of resistance factor at EOD and BOR. The results of the FOSM analysis served as the basis for further analysis with the Monte Carlo Simulations (MCS) method. The MCS was based on probabilistic procedures using random number generation and the resistance bias factor of (λ) found by lognormal tail fits of the data from EOD and BOR predictions. The study recommended the use of field measured hammer performance, CAPWAP based soil input parameters, and pile type into design (Sabdelsalam, 2014). D'Andrea et al. (2009) reported that the Louisiana Department of Transportation (LA DOTD), in conjunction with Louisiana Transportation Research Center (LTRC), conducted a similar calibration research project and completed the driven pile calibration in 2006. The pile load test database used for the calibration was created by conducting an extensive search in the projects library of the LA DOTD. Only pre-cast, pre-stressed concrete (PPC) piles that had been statically tested to failure and had adequate soil information were included in the study. A total of 42 PPC pile load tests met these criteria. In addition to the load test results, all other relevant information, including soil borings, pile driving logs, dynamic testing and analysis, and CPT data, were collected. It was reported that more than 90 percent of the pile load test data were from southern Louisiana, where the soil is weaker than that from northern Louisiana. Most of the soil profiles in the database consisted of clay, although approximately 25 percent of them were in mixed soils of sand and clay. Statistical analyses were used to evaluate the different pile design methods which included previously mentioned static design methods, direct CPT design methods, and dynamic measurement methods. The resistance factors from the calibration using FOSM were about 10 percent lower than those from the calibrations where more sophisticated methods such as FORM or Monte Carlo Simulation

were used. Due to limitations of information relative to soil type, the reported resistance factors for the static methods lumped all soil types together. The study proposed a separate calibration of resistance factors for northern Louisiana because that region presents stiffer soils than the southern region. In general, the LA DOTD calibration resulted in resistance factors approximately 25 percent to 60 percent greater than the resistance factors presented in the AASHTO (2004) calibration. However, the resistance factor obtained for dynamic analysis was much lower than the value provided by the AASHTO calibration. The potential reason postulated was that the dynamic analyses are less effective in clay soils. Wainaina et al. (2009) reported that North Carolina State University (NCSSU) used pile load test data from the North Carolina Department of Transportation (NCDOT) highway construction projects to perform a reliability-based calibration of resistance factors for axially loaded driven pile. A total of 140 piles with capacity predicted using signal matching via the PDA and 35 static load test data were compiled and grouped into different design categories and different geological regions. Resistance statistics were evaluated for each design category in terms of bias factors and coefficient of variation. Bayesian updating was employed to improve the statistics of the resistance bias factors, which were derived from a limited number of pile load tests. Load statistics presented in the AASHTO LRFD bridge design specifications were used in the reliability analysis during the calibration of the resistance factors. The calibration was performed for three methods of static pile capacity analysis commonly used by the NCDOT, namely the Vesic, the Nordlund, and the Meyerhof methods (Vesic 1975; Vesic 1973). FOSM and FORM techniques were utilized for conducting the reliability analysis. The findings demonstrated a substantial consistency between the resistance factors of AASHTO and NCDOT when utilizing the Nordlund method for pre-stressed concrete piles. Furthermore, it was noted that for all three static analysis methods concerning steel H-piles, the NCDOT resistance factors were considerably greater than those specified by the AASHTO. This observation led to the conclusion that there is a need to differentiate between high and low displacement piles because the current AASHTO resistance factors do not differentiate between pile types. Ng & Sritharan (2016) developed a systematic procedure for integrating pile setup into LRFD acceptance protocols. Using field data and reliability-based calibration aligned with execution methods (restrike, BOR CAPWAP), they found that explicitly accounting for setup can reduce the target driving resistance by 17% and slash the number of required retaps (from 37 to 15 in their case study), demonstrating clear construction cost savings. Yu et al. (2017) conducted a statewide Monte Carlo-based LRFD calibration of Φ for driven piles (under both tension and compression scenarios). By comparing older FHWA (1999) methods to newer ones (2010), they demonstrated that calibration can yield agency-specific Φ values that reflect prevalent construction practices and material behavior—sometimes significantly diverging from national AASHTO defaults but allowing more economical and

context-relevant design solutions. Ng et al. (2022) developed LRFD recommendations for driven piles in Intermediate Geomaterials (IGMs) across seven U.S. DOTs. They compiled a database of 223 full-scale load tests, proposed new static analysis equations for shaft and toe resistance, and performed reliability-based calibration. Their results show notably higher GRF values, with an average 48% increase in predicted shaft resistance at one day after End-of-Driving (EOD) in fine-grained IGMs, which translated to reduced steel usage during construction. Bian et al. (2021) introduced a reliability-based framework that explicitly incorporates pile setup into Φ calibration. Using a parametric analysis that includes the setup factor, variability (COV), reliability index β , and statistical correlation between initial capacity and setup, they quantified how setup affects both Φ and the Factor of Safety (FOS). The study demonstrates that including setup can adjust Φ notably—typically yielding more economical designs while preserving reliability. Zhang et al. (2024) applied a cutting-edge random finite-difference method (RFDM) combined with Bayesian updating to calibrate pile-group resistance factors. This approach lets engineers account for model bias and spatial variability more rigorously than classical FORM-only strategies. In scenarios where pile interactions are minimal, their method enables group efficiency (η) to approach 1.0 with justified Φ selection, optimizing design without compromising safety. Ng et al. (2022) conducted a comprehensive field test and geotechnical investigation program across seven state DOTs, resulting in a robust database of 223 pile load tests. They proposed new static analysis methods for calculating q_s , q_b resistance tailored for fine-grained IGMs and shales, and validated these against independent data. The study integrated dynamic parameter back-calculation into WEAP, yielding on average a 48% increase in predicted shaft resistance, higher LRFD resistance factors, and a means to significantly reduce construction inefficiencies via optimized steel use.

2.5 Summary and Conclusion

During the past 50 years, many efforts have been directed towards developing experimental methods and theoretical frameworks to investigate several ways to optimize GRF values in LRFD approach with pile foundation design. The focus of this chapter is mainly on summarizing the semi-empirical methods and LRFD approach for determining the pile capacity. The accuracy and limitations of these methods have been discussed and summarized in this chapter. The literature review explains also how the probabilistic methods such as FORM, FOSM, and Monte Carlo simulation work and can be used to calibrate the GRF value for pile foundation. Discussed are the reliability index values and corresponding geotechnical resistance factors for designing driven pile foundations as provided in the NBCC or CFEM. However, the use of the NBCC or CFEM resistance factors may

not be efficient and cost-effective for designing driven piles in Alberta. The suggested GRF factor by Canadian Engineering Codes has not been determined based on local experience, and it is one value across Canada. Currently, the design of the driven piles in Alberta is carried out using the suggested value by CEFM due to its widespread acceptability and the tendency of designers. The use of these factors in such circumstances would tend to produce a foundation size that is too conservative and expensive. Many studies in different regions except Alberta show that probabilistic methods can be used to improve GRF value significantly for that local zone. The objective of this study is to reduce the construction of driven pile foundation cost in Alberta by optimizing the GRF value. The main purpose of this research is to develop a framework for the calibration of GRF value for Alberta region and improve the current GRF value suggested by CEFM. The impact of the optimized GRF on the cost of construction of pile foundations is further investigated in this study. To increase the acceptability of the calibrated GRF value and encourage the engineers to use it, the calibration of resistance factors should be carried out using different reliability analyses and based on a comprehensive database. Details of the database and assumptions are summarized in the next chapter.

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Chapter 3

Database Collection and Methodology

3.1 Introduction

Due to the inherent complexity of soil deposition and soil-structure interaction behaviors, geotechnical engineering is fraught with uncertainties. These uncertainties, constrained by both time and budget, limit the extent of geotechnical sampling, testing, and bore-hole logging. According to Kulhawy and Phoon (2002), these uncertainties are primarily categorized into inherent variability, measurement uncertainties, and transformation uncertainties—the latter two often manifesting as data scatter due to limited information leading to statistical or sampling errors. Lazarte (2011) notes that transformation uncertainties arise during the conversion of field or laboratory measurements into design soil properties, often exacerbated by discrepancies in estimated versus actual properties due to variances in construction methodologies or quality control measures. This study endeavors to assess the bearing capacity of driven steel piles through a synthesis of laboratory and in-situ tests, and analytical methods, as delineated in the CEFM, 2023.

3.2 Database collection

This study compiled a database of six hundred and fifty-nine (659) PDA test results at End of Drive Condition (EOD) and three hundred and twenty-four (324) at Restrike from sixteen (16) diverse construction projects throughout Alberta, Canada, spanning both

infrastructure and the oil and gas sectors. The tests were carried out on areas with a wide range of geological and geotechnical properties. To cover different soil regions and geologic formations in the state, the geological map of Alberta developed by Hamilton et al. (1998) and AGS (2021) is divided into six zones as presented in Figure 1 and named as follows:

- Southwestern Alberta: SW
- Southeastern Alberta: SE
- Central Alberta: C
- Northwestern Alberta: NW
- Northeastern Alberta: NE
- Rocky Mountains and Foothills: RM & F

It should be noted that 5 projects have been carried out in southern Alberta and also, 6 projects and 5 projects have been performed in the north and center of Alberta, respectively. Due to the lack of appropriate data in Rocky Mountains and Foothills zone, this zone is ignored in this study.

To share data for public users and future studies, the following steps have been carried out:

- **Choose a Hosting Service:** Select a web hosting provider and purchase a plan. Secure a domain name through the hosting service or a domain registrar.
- **Set Up Your Website:** Install a Content Management System (CMS) like WordPress hosting service's control panel.
- **Implement User Authentication:** For a CMS: Use built-in features or plugins/extensions to create a user registration/login system.
- **For a custom website:** Develop a user authentication system that securely handles login credentials and session management, with a backend programming language Python.
- **Upload Data:** Once the website is set up, upload your data files to the server. This can typically be done through the CMS media library or via FTP for custom sites.

- **Restrict Access:** Configure the website to restrict access to the uploaded data.
- **For a CMS:** Use privacy settings or a plugin to limit data access to logged-in users or specific user roles.
- **Go Live and Monitor:** Launch website and monitor its performance and security regularly. Ensure that software and plugins are kept up to date.
- **Address for Data Access:** The full data has been uploaded into webpage at address below for accessibility. Limited access can be provided to review data as needed. <https://mpkardan.wixsite.com/website>

3.2.1 Alberta geological classifications

Alberta's regional geology exerts a direct and profound influence on the formation, distribution, and properties of surface soils across the province. The interactions between bedrock lithology, depositional environments, glacial history, and geomorphological processes have led to the development of distinct soil zones in correlation with the province's five major geological regions: Northeast (NE), Northwest (NW), Southeast (SE), Southwest (SW), and Central (C). In the Northeast, encompassing Fort McMurray and the Athabasca Basin, the bedrock is a combination of exposed Precambrian crystalline shield and overlying Cretaceous sedimentary formations, including the oil-rich McMurray Formation. These formations are blanketed by thick glacial tills and glaciolacustrine deposits. As a result, the dominant soil types in this region are Gleysols and Organic soils, often associated with wetlands, peat bogs, and poor drainage. These soils are typically acidic, poorly structured, and may have high water content, reflecting the low permeability of underlying fine sediments and flat topography. In disturbed zones, such as mined oil sands areas, Anthrosols (human-modified soils) are increasingly prevalent. In the Northwest region, including Peace River and High Level, the geology is composed of flat-lying Paleozoic and Mesozoic sedimentary strata, overlain by glacial and post-glacial deposits. The sediments include marine shales, sandstones, and evaporites, particularly in Devonian carbonate formations. The dominant soil types here are Luvisols and Brunisols, developed under forested boreal cover and reflecting moderate to high base saturation from calcareous parent material. Soils in this region tend to be well-drained, with moderate fertility, and are used for forestry and mixed agriculture. Areas with silty tills or lacustrine clays may also contain Vertisols with shrink-swell behavior. The Southeast region, encompassing Medicine Hat and Cypress Hills, features Cretaceous shales, coal seams, and sandstones, such as those of the Milk River and Belly River formations. Glacial retreat and meltwater action have led to the development of loess, sandy till, and alluvial parent materials. The prevailing soils here are Chernozems, which are dark, organic-rich soils typical of grassland ecosystems. These soils are generally fertile, well-drained, and calcareous, making the region one of Alberta's agricultural heartlands. In localized depressions and poorly drained flats, Solonchaks may occur, characterized by sodic horizons and hardpans. The Southwest region is geologically complex, formed by the Rocky Mountain Fold and Thrust Belt, which exposes a range of Paleozoic to Mesozoic sedimentary rocks, including limestones, shales, and coal-bearing units. Surface geology is further influenced by steep topography, alluvial fans, and glacial valleys. In this region, soils are primarily Brunisols and Regosols, reflecting young, developing soils under forest and rangeland. Brunisols dominate the montane and subalpine zones and are often well-drained, coarse-textured, and shallow due to bedrock proximity. In valley bottoms, Fluvisols occur in alluvial de-

posits, supporting riparian vegetation and agriculture. Soil development is limited by rapid drainage, erosion, and variable parent material. The Central region of Alberta, including Edmonton and Red Deer, is underlain by Cretaceous shales and sandstones such as the Paskapoo Formation, overlain by thick tills and glaciolacustrine deposits. These sediments provided a base for the formation of Dark Gray and Black Chernozems, particularly in the parkland belt, transitioning to Luvisols in more forested zones. The soils in this area are among the most agriculturally productive in Alberta, characterized by high organic matter, neutral to alkaline pH, and good moisture retention. In localized areas with fine-textured glaciolacustrine clays, Vertisols may occur, posing geotechnical challenges such as swelling and settlement. Figure below shows the geological map of Alberta and approximate each project location. Table below summarizes pile driving properties at each location.

Table 3.1: summarizes the pile driving properties of projects used in the database

Project Name	Location in Alberta	Diameter of piles (mm)	Type of piles	Depth of piles (m)	Number of installed piles	Number of PDA results at EOD	Number of PDA results at Restrike	Unfactored Load of piles (kN)	Measured Capacity (kN)	Match Factors
P1	NE	219	Pipe	15.5	100	3	3	350	428-520	1.34-1.92
P2	C	324	Pipe	14.4-15.2	498	6	3	1500	2105-3070	1.65-2.73
P3	SW	324-406	Pipe	6.9-11.4	168	8	6	1150-1950	1356-4261	1.8-2.12
P4	C	168-324	Pipe	8-10.5	378	18	17	840-1100	518-2093	2.02-2.79
P5	C	610-914	Pipe	11.2-37.2	708	82	41	1020-2560	1173-6800	1.82-3.23
P6	SE	219-324	Pipe	4-32.2	630	64	27	1700-2200	1614-4325	1.62-2.49
P7	NW	219-406	Pipe	4.6-18.2	2503	142	129	130-675	186-4497	2.49-3.10
P8	NW	324-406	Pipe	4.4-17.9	170	17	6	210-460	270-1480	1.77-2.36
P9	SW	324	Pipe	9.3-20.1	32	19	0	2200	2080-3803	1.81-3.11
P10	C	114-406	Pipe	8.4-21.1	71	10	8	215-710	360-2318	1.64-1.82
P11	NE	406-915	Pipe	6.2-40.2	523	176	5	1550-2140	916-10306	1.99-2.73
P12	NW	219-406	Pipe	10-16	68	5	5	570-1260	980-2828	1.31-1.73
P13	NE	406-914	Pipe	5.4-39.7	202	19	11	2430-3740	700-7000	2.12-2.61
P14	SW	273-508	Pipe	6-15.25	520	76	50	595-1340	985-3339	1.57-2.68
P15	SE	219-406	Pipe	4.9-17.7	299	21	12	450-720	562-1786	2.17-2.51
P16	C	219-324	Pipe	8-12	127	6	1	900-1200	826-3968	1.72-2.02

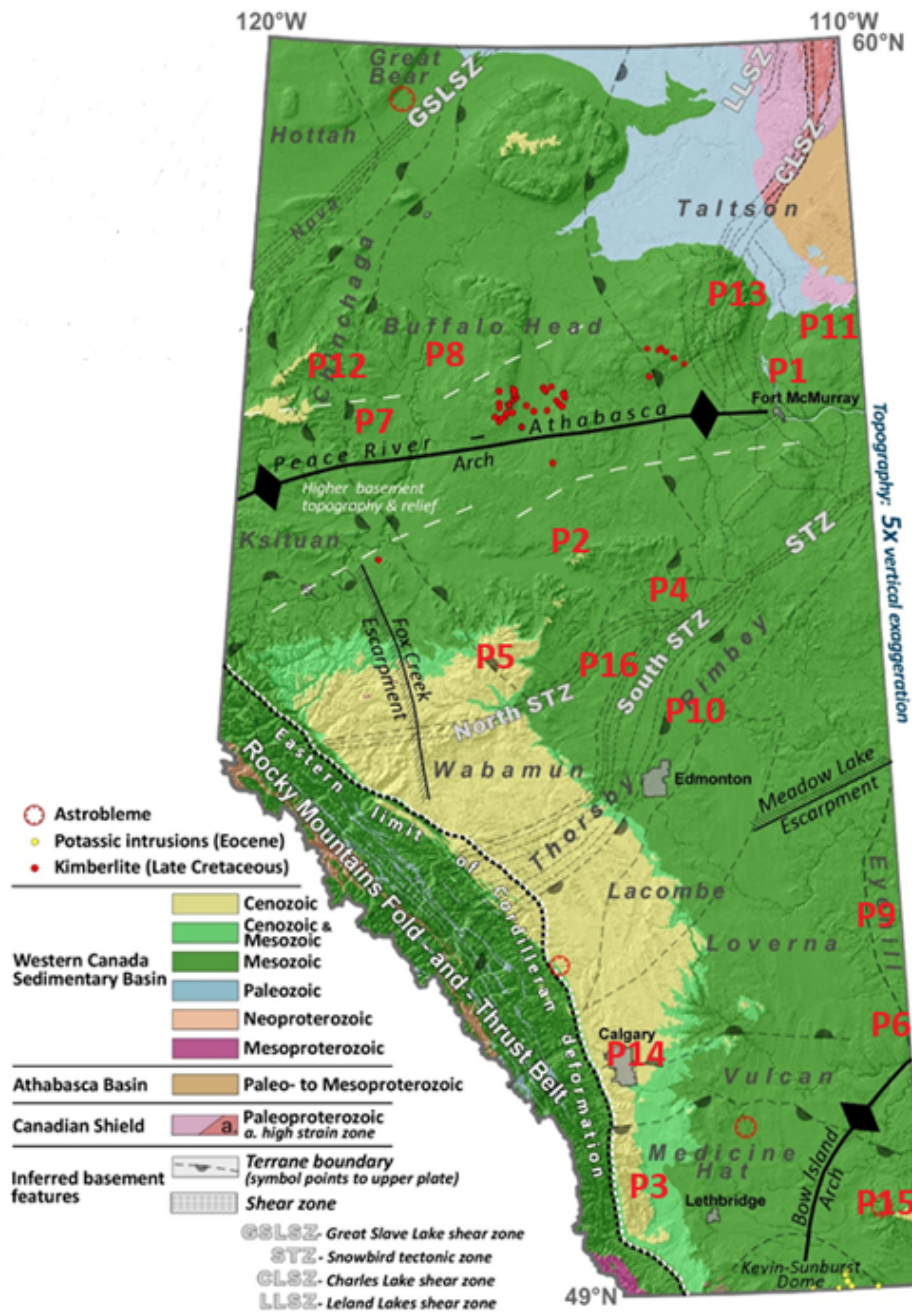


Figure 3.2: Alberta Geological map and approximate locations of each project

3.2.2 Geotechnical properties

The geotechnical properties of the site are a key parameter during the design and construction process of pile driving projects. Different soil types including coarse sand and gravel soils, fine silt and clay soils, glacial tills, etc. have been described in different regions of Alberta. The borehole studies, comprising CPT tests, have been carried out at each project to characterize soil properties. A summary of the geotechnical description of each project is presented below.

Project P1:

The soil stratigraphy consists of peat overlying a clay till layer.

Peat was encountered in some test holes drilled at this site at the surface. The peat was black, fibrous, and spongy, and its thickness was approximately 75 mm.

Clay Till Layer: Clay till was the major soil unit encountered in all test holes. The clay till was silty and sandy with trace gravel and trace oxides. The clay till was low to medium plastic and stiff to very stiff. Also, in some test holes, thin layers of sand and silt were observed. The sand layer was fine grained, contained some gravel, and the SPT-N blow counts in this layer varied from 8 to 10 indicating that the sand is loose to compact.

The SPT-N blow counts in clay till varied from 7 to 57 with an average of 15, and pocket penetrometer resistance varied from 1 kg/cm² to 4 kg/cm² with an average of 2.4 kg/cm², indicating that the clay till is stiff to very stiff.

The following figure shows the variation of SPT number vs depth in project P1.

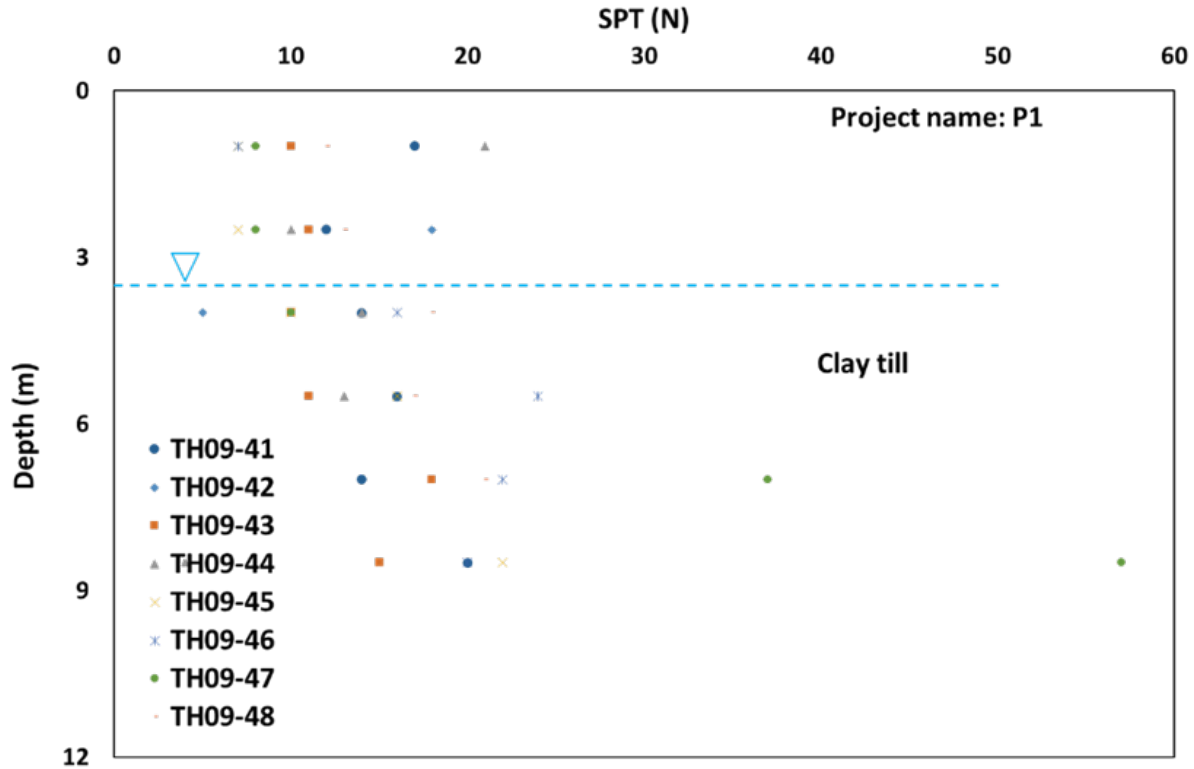


Figure 3.3: The variation of SPT number vs depth in P1

Project P2:

Fill Layer: The existing pavement section included 75 mm of asphalt concrete pavement (ACP) and layers of compact granular fill up to 425 mm thick. A 500 mm thick layer of firm clay fill was encountered below the granular fill in Borehole 1.

Silty Clay Layer: Lacustrine clay deposits were encountered below the fill in all boreholes. This material was present to a depth of 8.1 to 8.7 m below grade which corresponds to an elevation of about 868 to 869 m. This lacustrine deposit was a mixture of silt and clay with a trace of fine sand. The consistency of the soil was firm to stiff based on SPT N values of 4 to 11 and an average value of about 8. The plasticity of this deposit varied from low to high depending on the amount of clay, silt, and sand, which varied randomly with depth; however, the deposit was predominantly medium to high plastic in the lower portion, at depths below 7.0 m. The moisture content of this medium plastic layer ranged from 14 to 36, with an average of about 28.

Clay Till Layer: Clay till deposits were found below the lacustrine clay. The clay till deposit extended to a depth of 11.5 to 12.3 m in Boreholes BH1 and BH3 respectively, which correspond to an elevation of about 865 m. The clay till extended beyond the depths drilled in Borehole 2, 4 and 5. Auger refusal was encountered in Borehole 5 at a depth of 11.5 m. The local till is a mixture of sand, silt and clay with inclusions of gravel and coal fragments. The clay till was medium plastic and the consistency was stiff to hard based on SPT N values ranging from 14 to 72 with an average of about 34. The moisture content of this deposit ranged from 11 to 22 percent with an average of about 15 percent. Minor traces of seepage were encountered from fissures and thin sand lenses within the till.

The following figure shows the variation of SPT number vs depth in project P2.

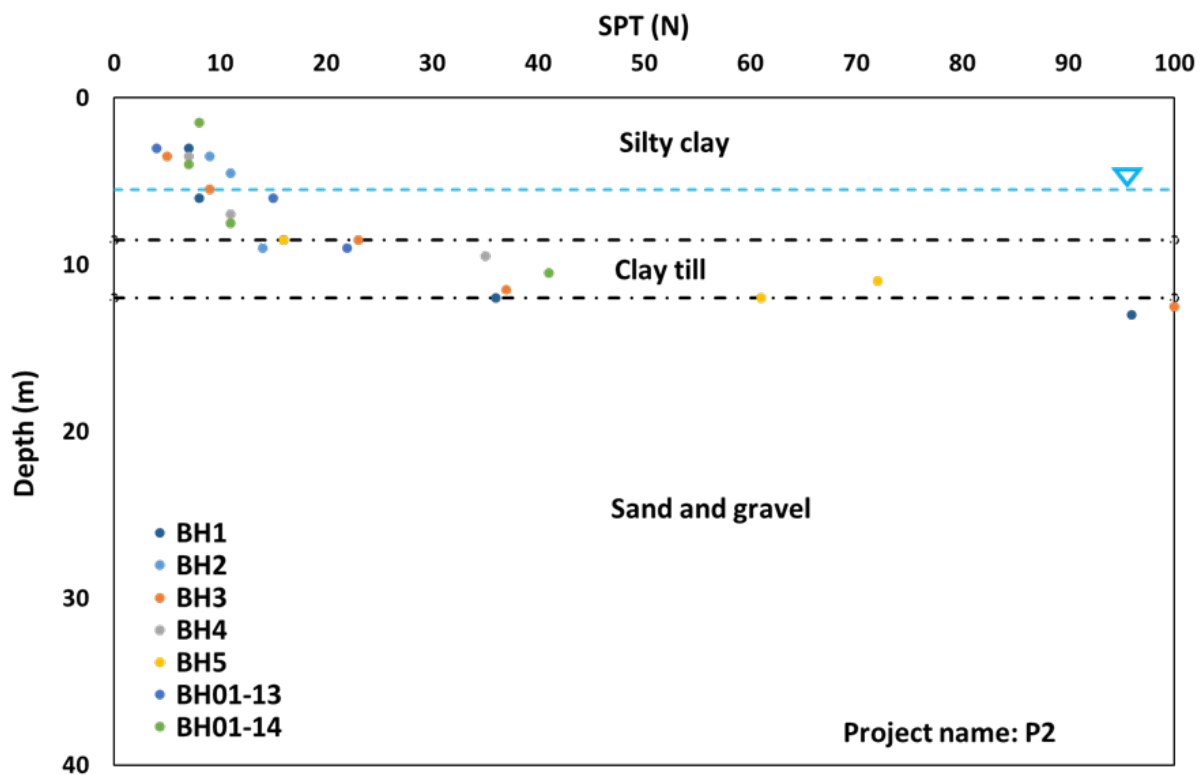


Figure 3.4: The variation of SPT number vs depth in P2

Project P3:

A layer of glacial clay till was encountered at the surface in all of the test holes. The layer of glacial clay till extended to depths of 3.7 to 6.3 m below grade and was underlain

by clay (Shale) that extended to the maximum depths explored (i.e. 11.1m to 12.6 m below grade).

Glacial Clay Till Layer: The glacial clay till contained a mixture of clay, silt, sand, and gravel. It was classified as moist, medium to high plasticity, and generally stiff to very stiff, becoming hard with increasing depth.

Occasional sand lenses (up to 10 mm in thickness) and shale inclusions (10 mm in diameter) were encountered throughout the depth of the glacial clay till. Although cobbles and boulders were not detected during test drilling, they are known to be present randomly throughout the glacial clay till deposit.

Unconfined compression strength tests were completed on five (5) samples of the glacial clay till and the results varied from 76 kPa to 139 kPa, with an average of about 100 kPa.

In addition to the strength testing, unit weight values were also obtained for the same five samples and were measured to vary from 19.0 kN/m³ to a high of 19.4 kN/m³, with an average of 19.2 kN/m³.

Clay (Shale) Layer: The clay shale was silty, high plastic, damp, grey, and contained occasional silt lenses. The shale was hard with SPT blow counts greater than 50 in most cases, except within the upper 3 m of the shale, where blow counts were in the order of 30 to 40. The following figure shows the variation of SPT number vs depth in project P3.

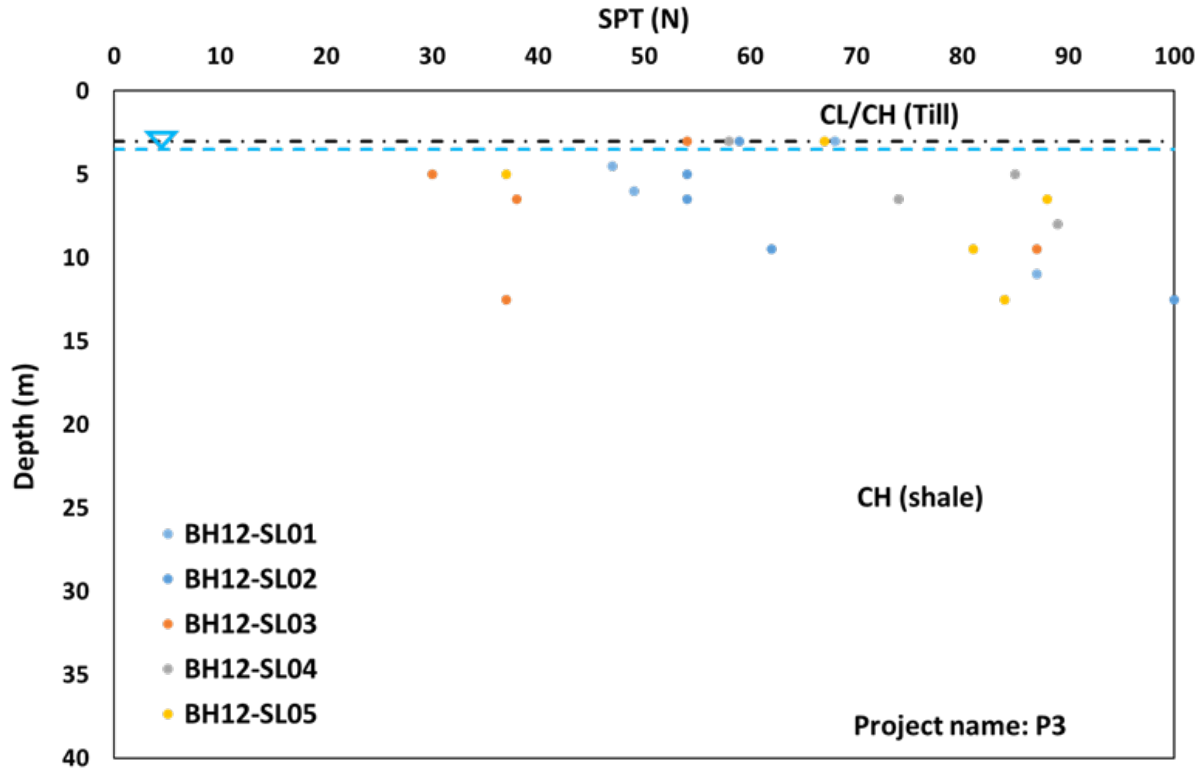


Figure 3.5: The variation of SPT number vs depth in P3

Project P4:

The subsurface soil conditions encountered were generally found to consist of a layer of topsoil (organics) ranging from 100 mm to 300 mm. Loose silty sand was encountered underlying the topsoil in BH4 through BH6 and extended to depths ranging from 0.3 , to 0.4 m below existing grade. The silty sand was described as brown with trace gravel, oxidation, and sulphates. Underlying the topsoil and the silty sand, glacial sandy clay till was encountered extending to beyond borehole termination depth in all boreholes.

Clay Till Layer: Clay till was encountered in all boreholes directly beneath topsoil and silty sand and extended beyond borehole termination depth. The sandy clay till was noted to contain trace gravel with occasional oxidation, sulphates, and coal. The clay till was

generally brown to grey in color and damp to moist. Sand layers were encountered in BH1, BH3, and BH5 at depths of 2.4 m to 3.5 m, 2.9 m to 3.2 m, and 3.3 m to 3.5 m, respectively.

The following figure shows the variation of SPT number vs depth in project P4.

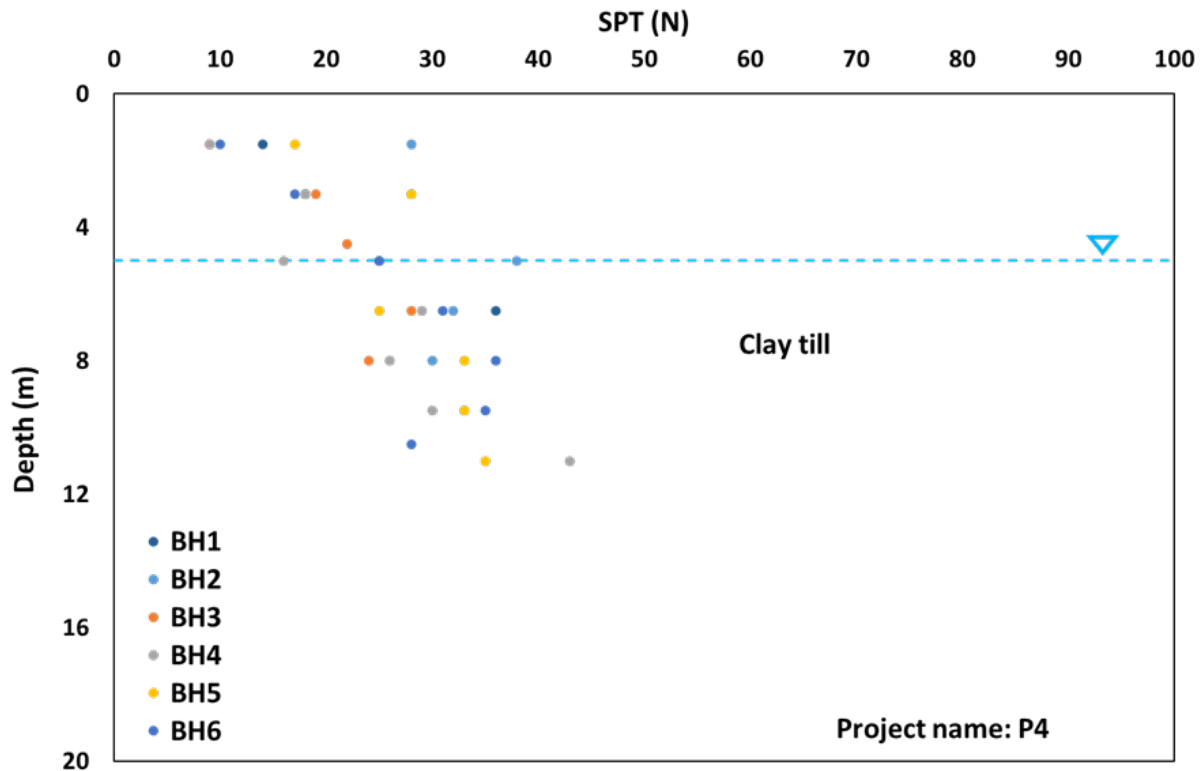


Figure 3.6: The variation of SPT number vs depth in P4

Project P5:

The soil conditions encountered in the upper 10 m (i.e. between elevations 630 and 620) appear to be generally similar in both site units and consist of predominantly stiff to very stiff clay till. However, below elevation 620 m, the soil conditions appear to be more variable, with interbedded sand, clay, clay shale, and clay till layers of varying thickness.

Clay Fill Layer: Clay fill was encountered in a few test holes, originating from the recent site grading operations across the site. Where encountered, the fill extended to depths ranging from about 1.5 to 1.8 m below ground surface. The fill consists primarily of medium plastic brown silty clay and clay till with moisture contents ranging between about 15 and 22 percent. It should be noted that the depth of fill may vary between the test hole locations. **Clay Layer:** A clay layer was encountered either at ground surface or below the clay fill in most of the test holes and extended to depths ranging from 0.5 to 2

m below existing ground surface. The clay was typically medium plastic, brown, silty, and contained some sand and trace gravel and organics. SPT N values in the clay ranged from about 4 to 9 blows per 300 mm penetration, indicating a firm to stiff consistency. The moisture content of the clay varied from 15 to 40 percent but was typically between about 15 and 20 percent.

Clay Till Layer: Clay till was the predominant soil type encountered in the test holes and extended to the termination depth of the test holes, of up to 30 m below existing ground surface. The clay till was typically silty, sandy and contained sand layers, lenses and occasional rafted clay shale bedrock. The upper clay till was brown in color, grading to grey below a depth of about 5 m to 8 m below ground level. Moisture contents ranged from about 10 to 30 percent but were more typically in the range of 15 to 20 percent. SPT 'N' values ranged from 5 to more than 50 blows per 300 mm penetration but were typically between about 10 and 30. Undrained shear strengths as obtained from laboratory compressive strength tests ranged from 100 to 370 kPa indicating that the clay till is very stiff to hard. Modulus of elasticity values (as obtained from cyclic loading of the confined compression tests) ranged from about 45 to 100 MPa (see Table C.1). Interpreted undrained shear strength as obtained from CPT tests and laboratory undrained shear strengths are plotted versus depth on Figure 5.2. Atterberg Limits tests undertaken on selected samples of the clay till indicated liquid limits typically ranging from about 26 to 38 percent and plastic limits ranging from 13 to 17 percent, respectively.

It should be noted that, although not encountered during the present field investigation, the clay till may contain cobbles and boulders. Sand Layer: Discontinuous sand layers with varying thickness (0.1 to 10 m thick) were encountered within the clay till in several test holes at depths ranging from about 6 m to 24 m below existing ground surface, but more frequently between about 10 m and 20 m depth. The sand was typically light brown grey and silty. SPT 'N' values ranged from 7 to more than 100 blows per 300 mm penetration, indicating that the sand is compact to very dense. The moisture contents in the sand varied from about 10 to 25 percent. The following figures show the variation of SPT number vs depth in two units of project P5.

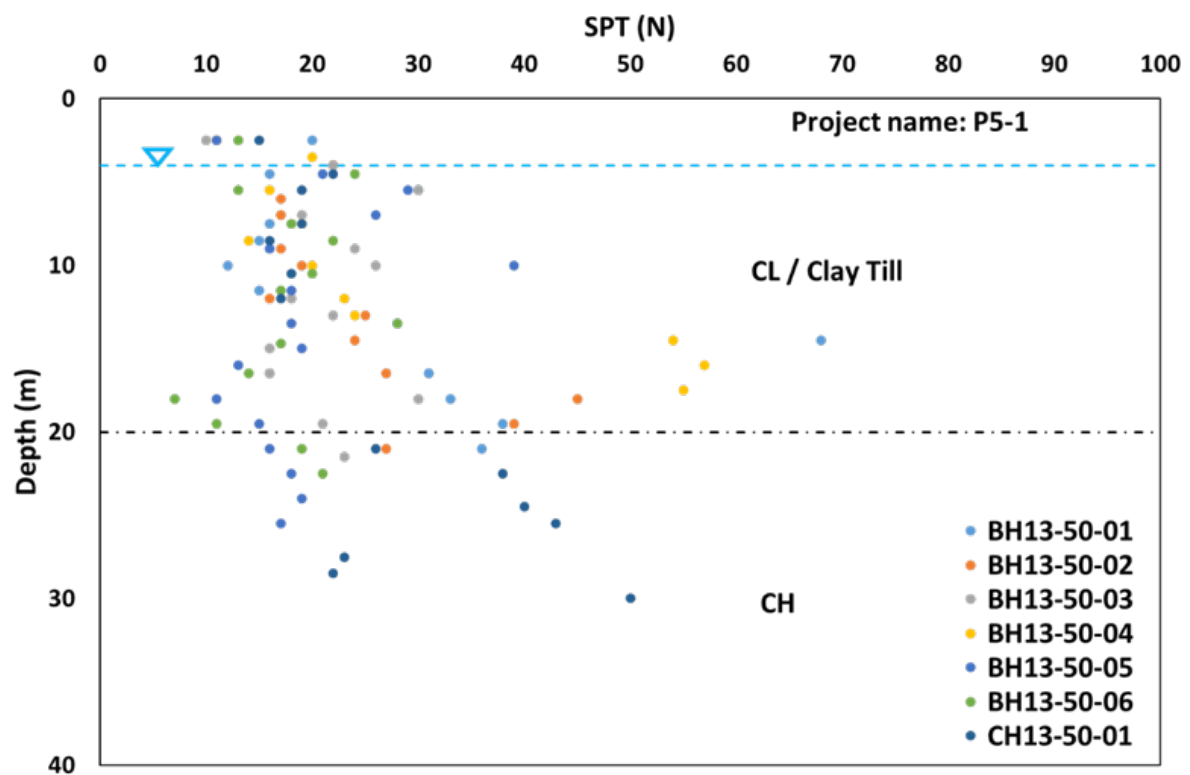


Figure 3.7: The variation of SPT number vs depth in P5-1

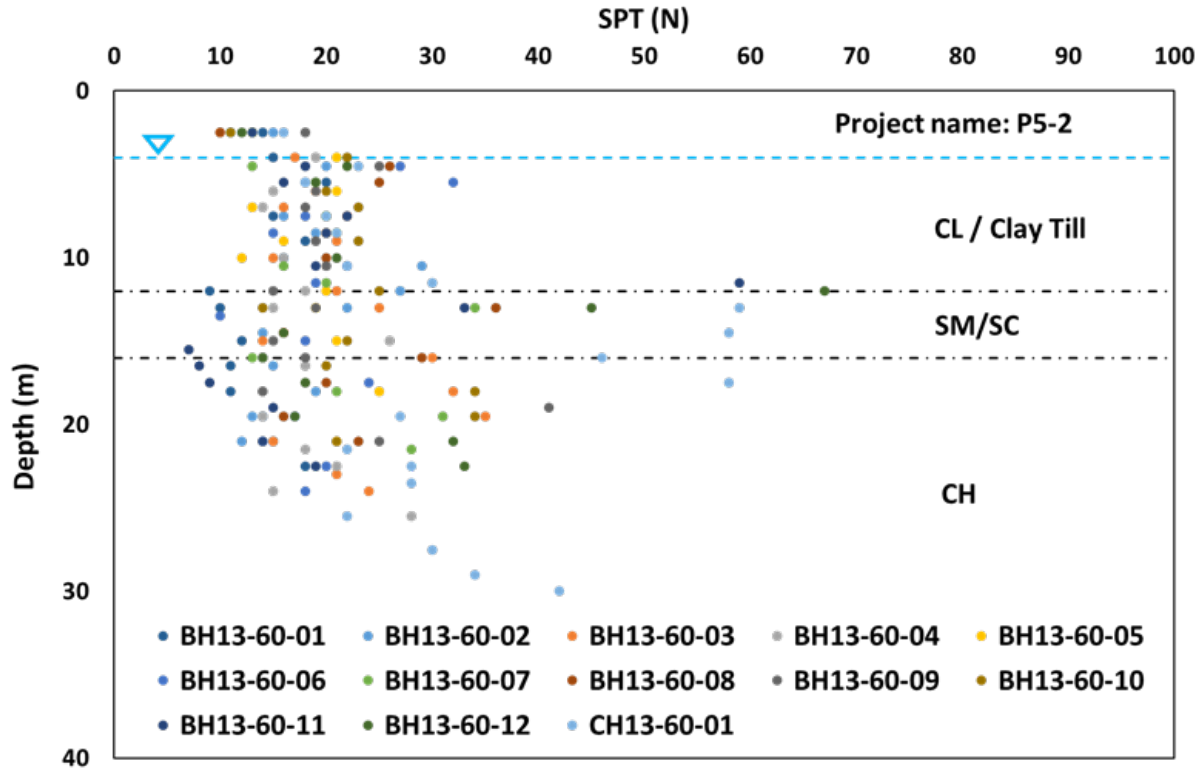


Figure 3.8: The variation of SPT number vs depth in P5-2

Project P6:

The soil stratigraphy consists of clay fill overlying lacustrine clay, silt till, and limestone bedrock. The lacustrine clay and silt till layers are 13 and 5 m thick, respectively. The clay is stiff, and the till is dense to very dense and dry. The condition of the bedrock is highly variable. It is intact and massive at some locations within the site and highly fractured in others.

Clay Till Layer: The lacustrine clay is generally mottled brown and grey, stiff, highly plastic, and weathered in the upper 1.5 to 4.6 m. Fractures and fissures are present in this portion of the clay layer. The clay generally becomes grey, firm to stiff, and intermediate to highly plastic below 1.5 to 4.6 m. The clay is comprised of varying amounts of silt. Traces of sand, gravel, silt, and gypsum inclusions are common in the clay, and cobbles and boulders can often be found near the till surface. The thickness of the clay layer ranges between about 0 and 21 m, with a typical thickness in the order of 9 to 12 m.

Silt (Till) Layer: Glacial Till is a heterogeneous mixture of clay, sand, gravel, cobbles and boulders within a predominately silt matrix. The clay and silt fractions are the main constituents of the till and can range between 40 to 60 percent of the soil deposit, with the silt fraction dominating. The till layer is typically about 3 to 6 meters thick, but can be as thick as 20 meters or more, or non-existent. The irregular till surface varies in depth from the ground surface; it may be exposed at the ground surface or encountered at a depth of more than 20 m.

Bedrock: The bedrock underlying the project site consists of four main Ordovician (second period of the Paleozoic era) sedimentary formations; the Stony Mountain, Winnipeg, Red River and Jurassic Amaranth formations. The bedrock is carbonate in nature and consists of primarily dolomite and dolomitic limestone. Shale and sandstone are found in some formation members but are not as prevalent. The surface of the bedrock is irregular and fractured. The presurface of the bedrock underwent periods of erosion and weathering, resulting in voids and fractures in the upper 0.3 to 0.9 m. The fractures generally contain infill consisting of clay to boulder particle sizes and till or mixtures of all. The upper 15 to 30 m of the bedrock contains discontinuities, with the frequency of these discontinuities usually decreasing with depth. Below the carbonate bedrock lies Precambrian basement.

rock consisting primarily of granite and gneiss. The following figure shows the variation of SPT number vs depth in project P6.

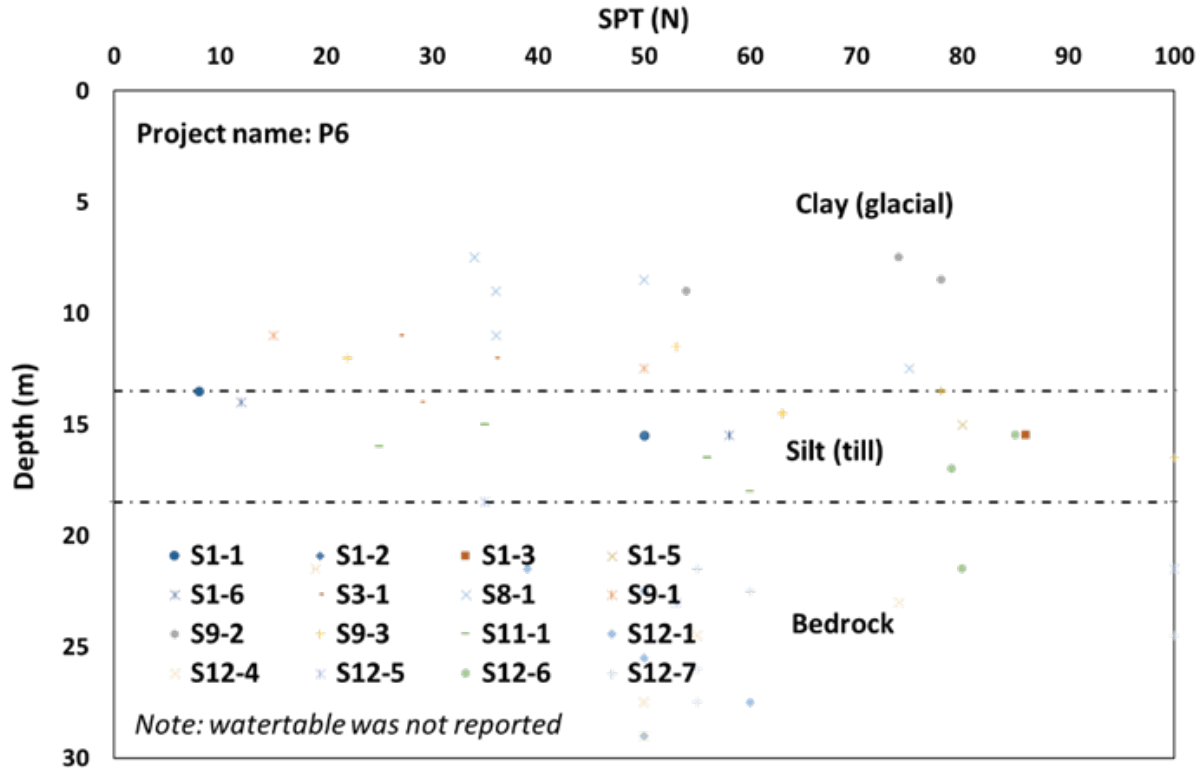


Figure 3.9: The variation of SPT number vs depth in P6

Project P7: Geotechnical investigation program of this project encompassed 54 boreholes totaling 606.5 m of proposed drilling depth and the installation of five (5) standard vibrating wire piezometers and 14 PVC wells (75 mm) for downhole vertical seismic profiling (VSP), within select boreholes. Boreholes that were advanced using the ODEX or hollow stem drilling method produced minimal cuttings.

A summary of the subsurface soil conditions encountered during this project's investigation is provided below. The subsurface soil stratigraphy at this project consisted of a thin layer of organic topsoil, overlying silts and clays, silty clay till and silty sand, underlain by weathered shale bedrock.

Silty Clay/Silt Layer: Brown to grey silty clay/silt was encountered at the surface or below topsoil within most of the boreholes, ranging in thickness from 0.2 to 2.1 m. The unit ranged from damp to moist, non- to medium plastic, and contained trace sand and trace gravel.

Silty Clay Till/Silt Till Layer: Brown to dark grey silty clay till/silt till was encountered in all boreholes, ranging in thickness from 2.1 to 13.7 m. The unit ranged from dry to moist, low to high plastic, typically containing trace to some gravel and trace sand to sandy. **Weathered Shale Layer:** Grey weathered shale was encountered in some boreholes ranging in thickness from 1.0 to 16.1, extending to the depth of drilling. The shale was typically damp, weak, highly to extremely weathered. In selected boreholes, the weathered shale exhibited medium to high plastic characteristics. The following figure shows the variation of SPT number vs depth in project P7.

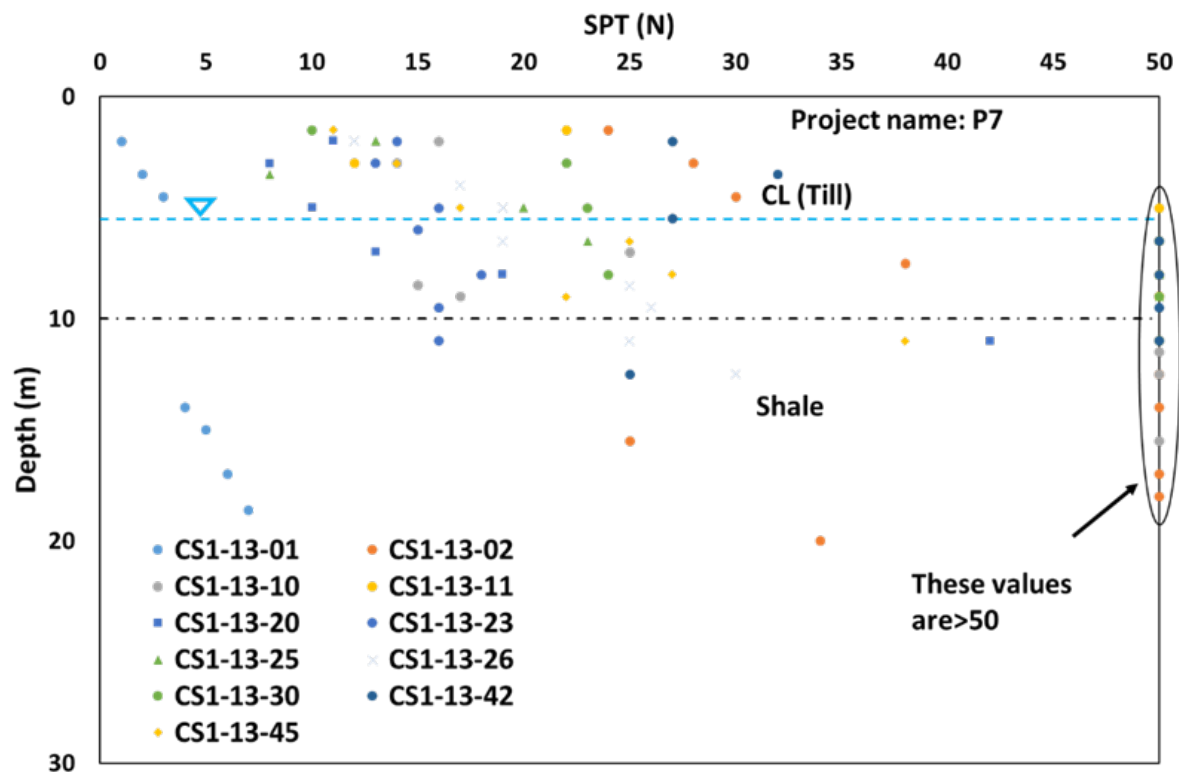


Figure 3.10: The variation of SPT number vs depth in P7

Project P8:

The subsurface soil conditions of project P8 is similar to project p7, as described above. The following figure shows the variation of SPT number vs depth in project P8.

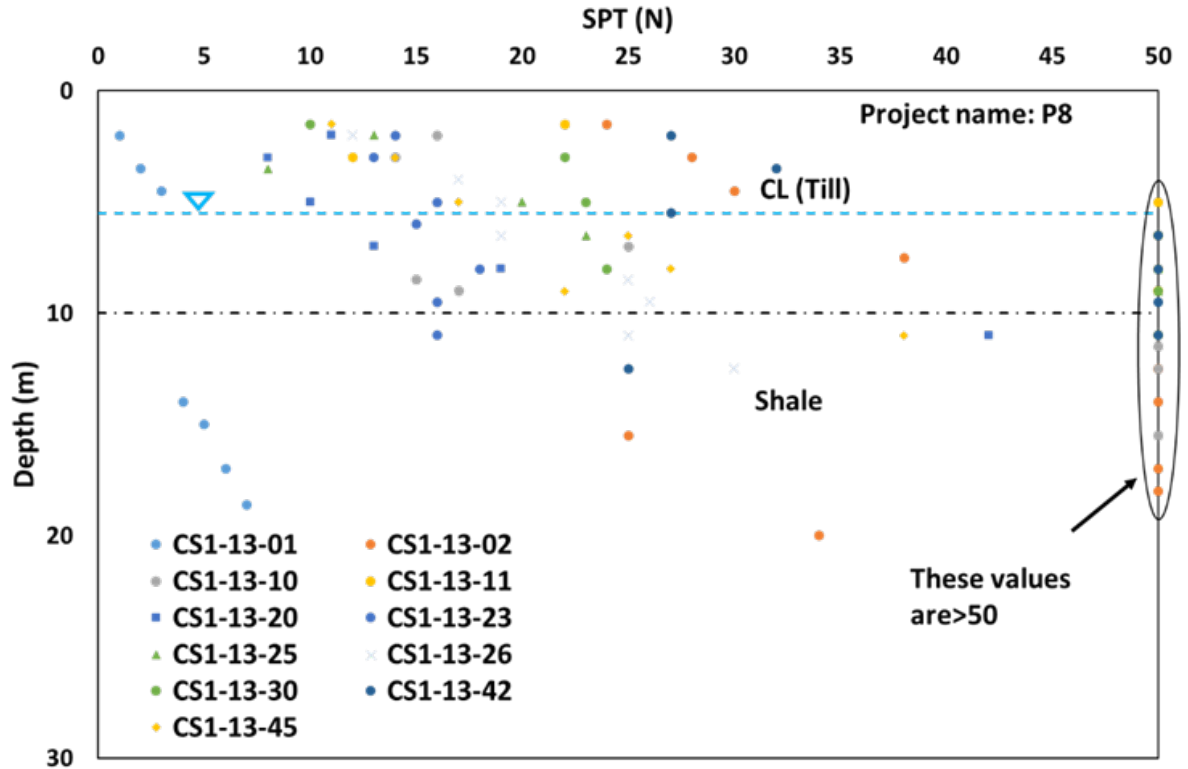


Figure 3.11: The variation of SPT number vs depth in P8

Project P9:

Clay Fill Layer: Clay fill material was encountered in each test hole drilled for the abutments and piers and ranged in depths from approximately 2.5 to 3.4 m below ground surface. The fill material generally consists of firm to very stiff, low to medium plastic clay containing variable amounts of sand and gravel.

Clay Till Layer: Soft to very hard, low to medium plastic native clay and clay till was encountered below the fill material in each of the test holes drilled in this project. The native clay and clay till generally contained some sand and trace gravel with occasional silt and sand layers encountered in the till. In some test holes such as TH17-1 and TH17-2, the clay and clay till layer extended to the clay and silt layer. Moisture contents in the clay and clay till layers ranged from 6% to 23%, but typically were between 10% and 20%.

Silt Layer: The clay and silt layer was encountered in test holes TH17-1, TH17-2, TH17-4 and TH18-14 at depths ranging from approximately 9 m to 12.3 m below existing

ground surface. Generally, the clay and silt were low to non-plastic and contained trace to some fine-grained sand and trace gravel. The layer was observed to be wet at some depths and ranged in consistency from firm to very hard. Bedrock Layer: Depth to bedrock was measured between 12.3 m (approximate El 1065 m) at TH17-03 and 16.8 m (approximate El 1065 m) at TH17-1. The bedrock is quite variable in consistency, typical for the Calgary area, and includes interbedded layers of extremely weak to strong claystone, siltstone, and sandstone.

Unconfined compressive strength tests performed on select samples of bedrock core yielded unconfined compressive strengths ranging from less than 1 MPa to 54 MPa (based on 9 tests). Note that the lower strengths are generally associated with the claystone. It is suspected that the lower unconfined strengths measured in some of the claystone samples reflect minor fissures in the samples and under-estimate the true in-situ strength of the claystone. Nonetheless, the claystone is significantly weaker than the siltstone and sandstone materials which are present as layers of varying thicknesses.

The following figure shows the variation of SPT number vs. depth in project P9.

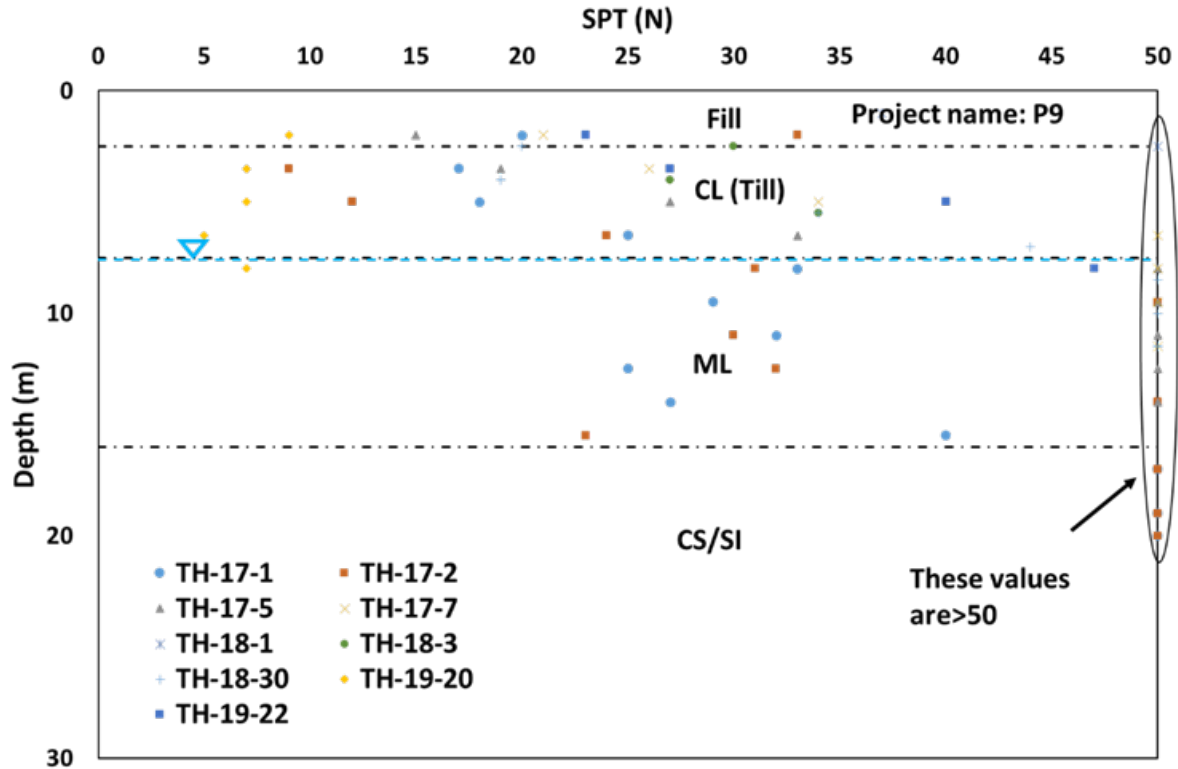


Figure 3.12: The variation of SPT number vs depth in P9

Project P10:

Conditions encountered in both parts of project P10 comprised an extensive deposit stiff clay till. Clay Till Layer: The results of 4 Atterberg limit tests on the clay till indicated liquid limit values of 42 to 46 percent, consistent with medium plastic material. In the main plant site,

a 2 m thick surficial layer of medium to high plastic, stiff clay was encountered above the glacial till in borehole SGP-2. In this project, N values typically ranged between 9 to 22, averaging 14 blows per 300 mm penetration. The following figure shows the variation of SPT number vs depth in project P10.

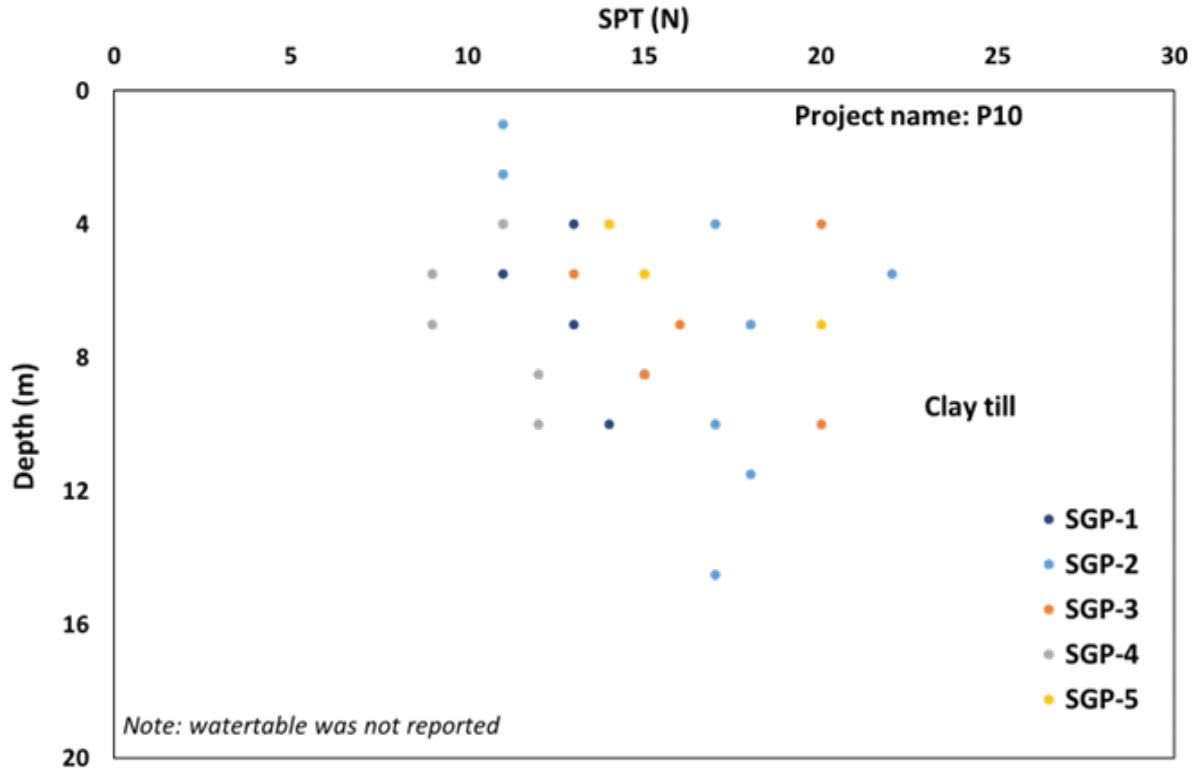


Figure 3.13: The variation of SPT number vs depth in P10

Project P11:

The subsurface layers of project P11 comprised three layers, including SP-SM (1st layer: silty sand and poorly graded sand), ML (2nd layer: low plasticity silt) and SM (3rd layer: silty sand till). Based on 43 boreholes, the average thicknesses of the 1st, 2nd and 3rd layers were 10, 5 and 25 m, respectively. The average watertable level was at a depth of 5 m.

Silty Sand Layer: The first subsurface layer of project P11 was SP-SM based on the Unified Soil Classification System (USCS). The average N-SPT value of the 1st layer was 35. The sandy deposits was fine to coarse, some silt to silty, trace to some clay, brown to dark brown.

Silt Layer: The 2nd layer was silt material with the average N-SPT value of 12. It also was sandy, trace clay, low plastic, firm and grey. **Sand Till Layer:** The third subsurface layer of project P11 was SP-SM based on the Unified Soil Classification System (USCS). The

average N-SPT value of the 2nd layer was 15. This sandy deposit was fine to coarse, silty, trace to some clay, trace gravel ($\leq 20\text{mm}$), loose, brown to dark brown, slight bituminous odour. The following figure shows the variation of SPT number vs depth in project P11.

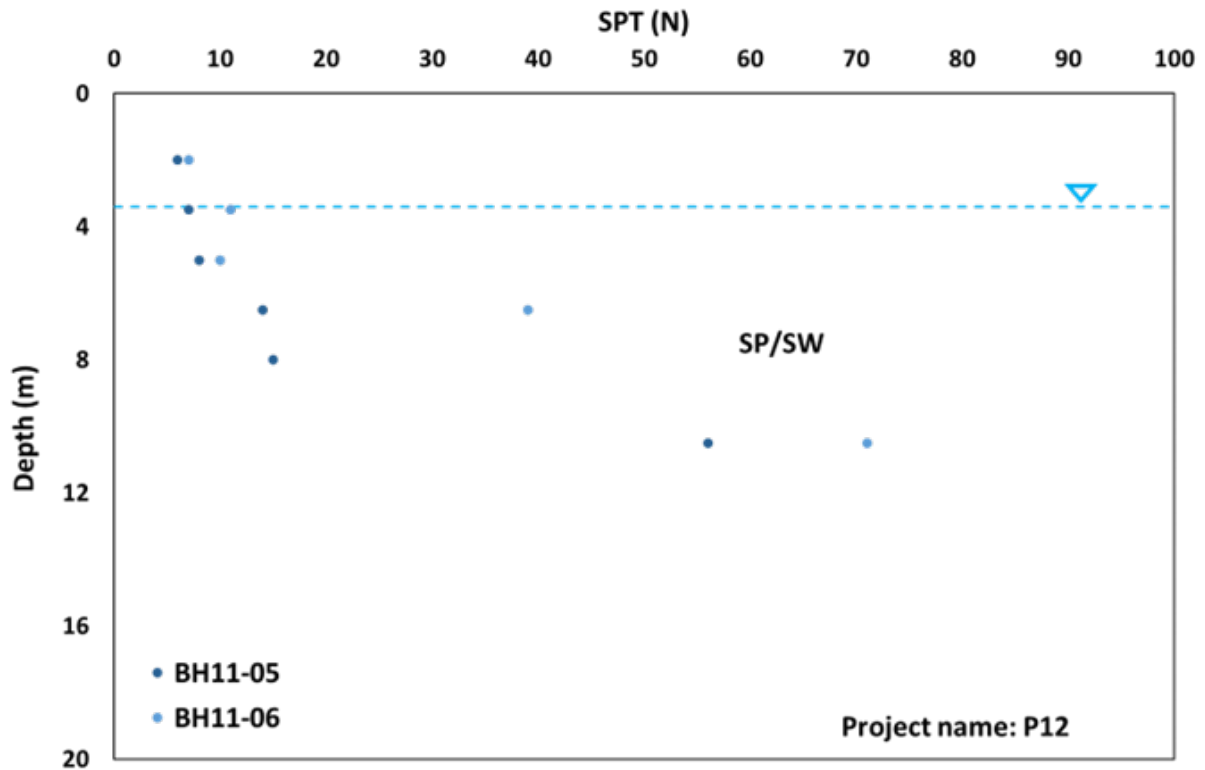


Figure 3.14: The variation of SPT number vs depth in P11

Project P12:

The subsurface layers of project P12 comprised granular materials, mostly sandy soil, with poorly graded and well-graded layers. Two main layers of SP/SW deposits were distinguished. The first layer, with a thickness of 8 m, had an average N-SPT value of 13, and the second, with a thickness of 3 m, had an average N-SPT of ≥ 50 .

The following figure shows the variation of SPT number vs. depth in project P12.

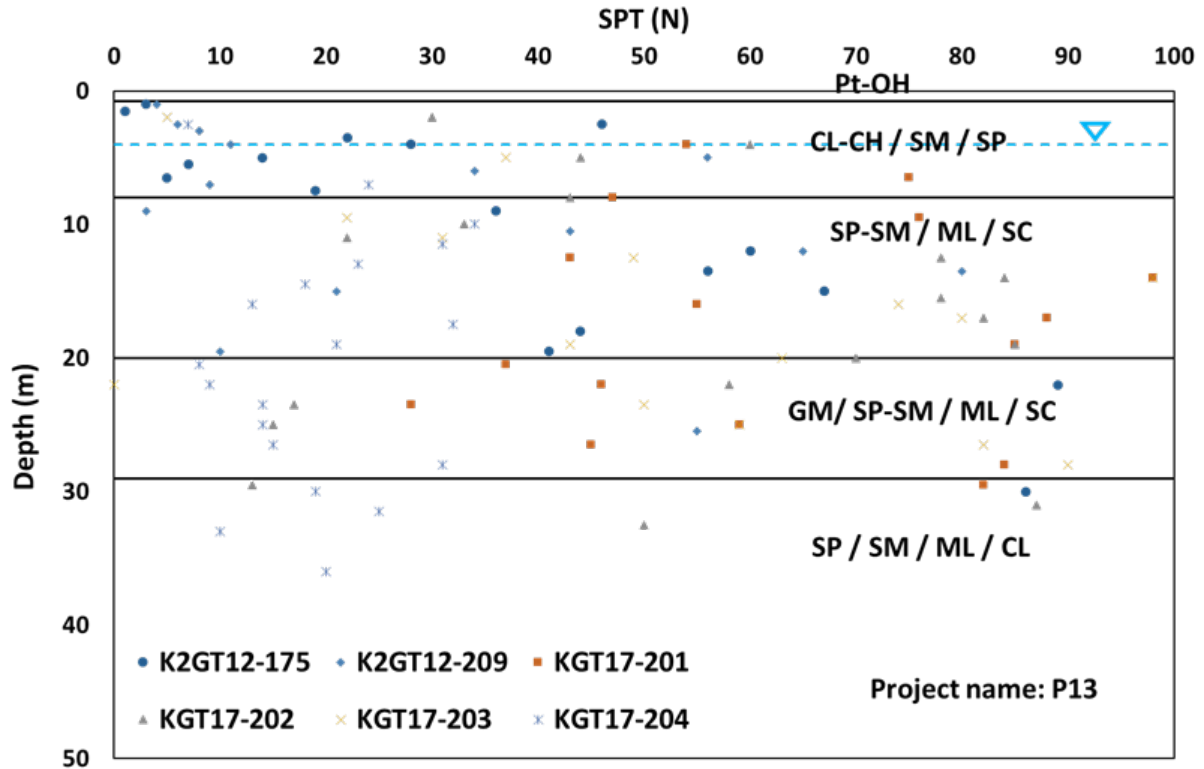


Figure 3.15: The variation of SPT number vs depth in P12

Project P13:

The interpreted site and subsurface soil conditions presented in this section are based on information obtained from the soil borings and the results of the laboratory tests obtained from geotechnical investigations performed in this project. A total of seven (7) soil borings were performed within the vicinity of the MSE Wall site as part of the geotechnical investigations.

Muskeg Layer: The first layer consists of Muskeg, soft clay, and organic peat from the existing ground surface to a depth of about 0.8 m below existing grade. The N-values from the Standard Penetration Test (SPT) performed within this stratum vary generally between 0 and 4.

Sand / Silty Sand / Clay Layer: Second layer consists of loose to very dense sand, silty sand, clayey sand, and silt from a depth of about 0.8 m to a depth of about 8 m below existing grade. The N-values from the Standard Penetration Test (SPT) performed within

this stratum vary between 5 and 56. Layers of stiff to very stiff clay were encountered within this stratum at the locations of Borings KGT17-204 and K2GT12-209 between depths of about 3 m and 9 m below existing grade.

Sand / Silty Sand / Silt / Clayey Sand Layer: Third layer generally consists of loose to very dense silt, sand and silty sand from below Stratum I to a maximum depth of about 20 m below existing grade. The SPT N-values within this stratum generally ranged between 3 and greater than 50 blows/100 mm. Layer of silt was encountered within this stratum at K2GT12-175 between depths of about 10 m and 15 m below existing grade.

Gravel / Silty Sand / Silt / Clayey Sand Layer: Fourth layer generally consists of medium dense to very dense silty sand, sand, silt, clay, and clayey sand from a depth of 20 m to a depth of 29 m below existing grade. The SPT N-values within this stratum generally ranged between 10 and greater than 50 blows/75 mm, indicating loose to very dense material.

Sand / Silty Sand / Sandy Silt / Silt / Clay Layer: Fifth layer generally consists of very dense oil sands (McMurray Formation) from below Stratum IV to a maximum depth that was explored by these borings. The SPT N-values within this stratum were greater than 50 blows, indicating very dense material. Layers of McMurray clay on top of Oil Sand were encountered within this stratum at K2GT12-175, 209, 210, and KGT17-201, 202, 20 between depths of about 25 m and 30 m below existing grade.

The following figure shows the variation of SPT number vs depth in project P13.

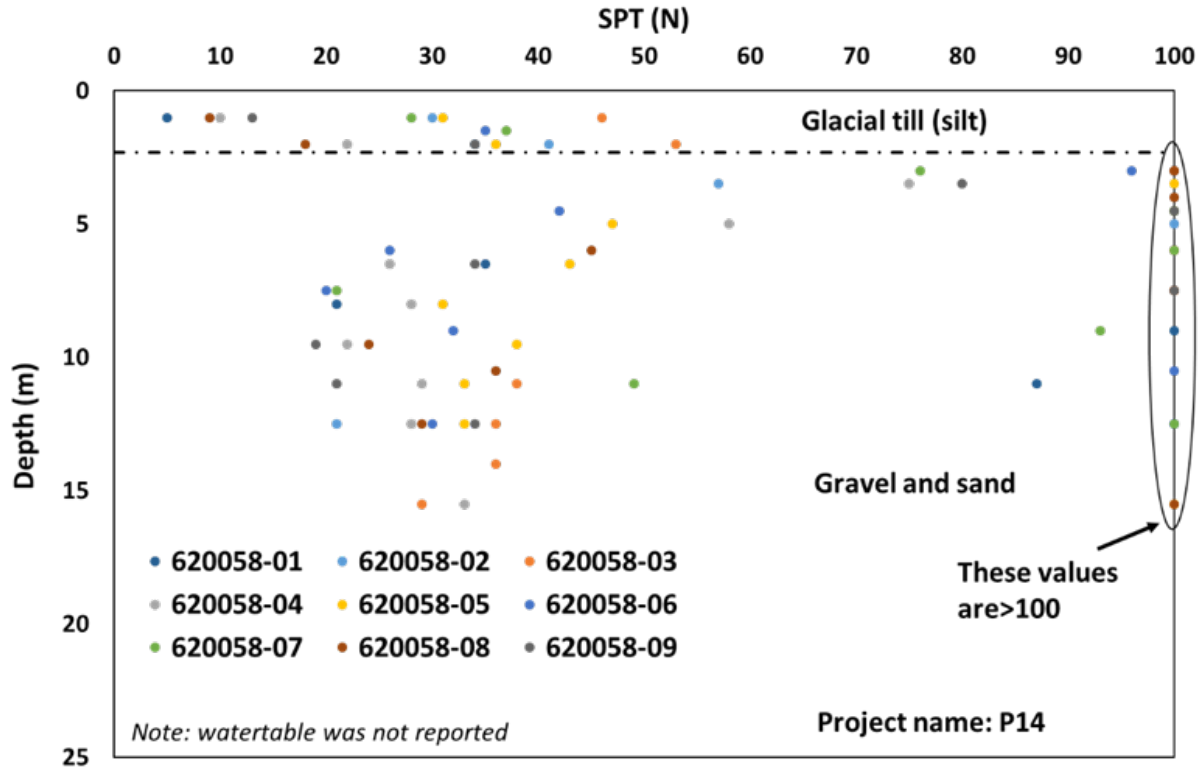


Figure 3.16: The variation of SPT number vs depth in P13

Project P14:

In this project, the geotechnical investigation included the field investigation program (borehole drilling), field and laboratory soil testing, groundwater level monitoring (piezometer installation), soil resistivity testing, and a report that provides geotechnical recommendations for the design and construction of the proposed development.

Based on the material encountered in the boreholes, the general soil profile consisted of topsoils over a variable thickness of glacial till (to depths of approximately 1.68 to 3.05 m) followed by varying sand/gravel deposits to the final depth of most boreholes. Inter/intra glacial till (sand/silt) deposits were encountered within the sand/gravel layers in most of the deep boreholes.

Gravel and Sand Layer: The till deposits were typically stiff to hard, low to medium plasticity, and dry to moist. The sand/gravel deposits were generally dense to very dense, fine to coarse, and dry, except for a few layers of compact sand/gravel at various depths

in boreholes 620058-02, 04, and 07. The inter/intra sand and silt till deposits varied in thickness between thin pocket/lens size to 0.8 m. Typically, these layers were very stiff (based on limited SPTs and drilling difficulty). The following figure shows the variation of SPT number vs depth in project P14.

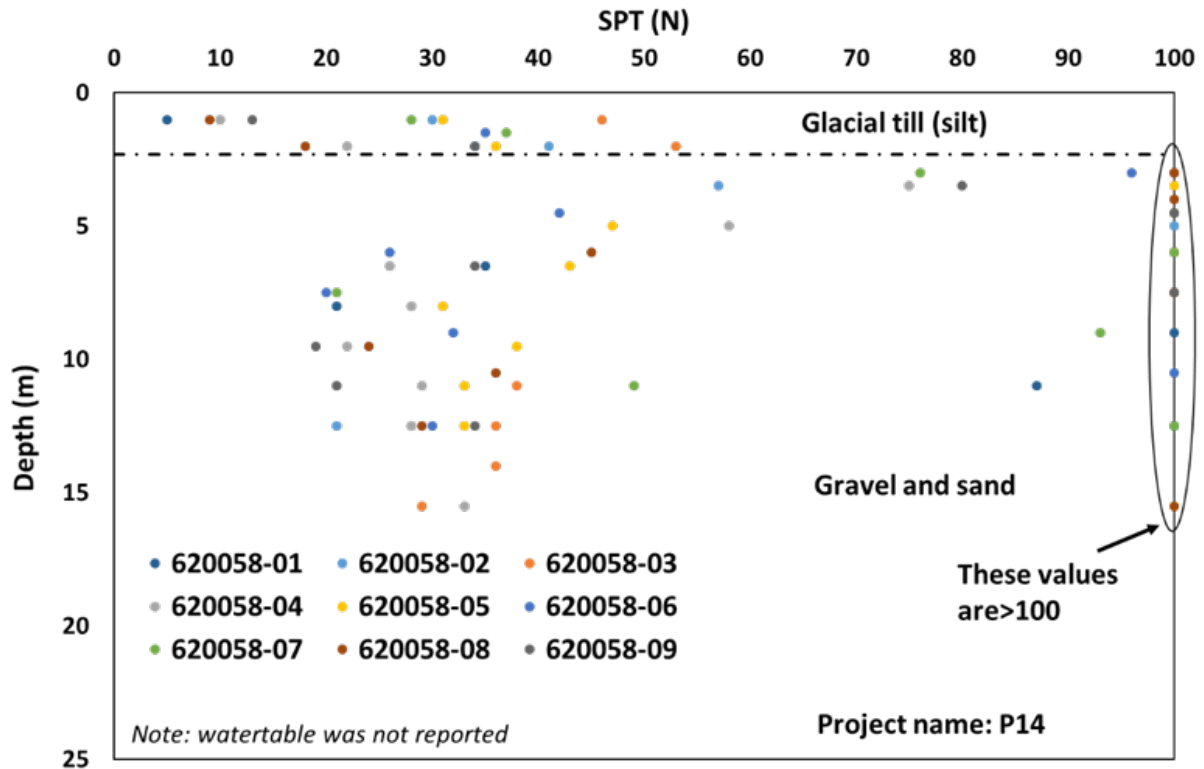


Figure 3.17: The variation of SPT number vs depth in P14

Project P15:

The subsurface soil stratigraphy at the Project P15 consisted of a succession of sands, silts, and clays.

Sand Layer: Brown to grey sand was encountered at the surface and at depth interbedded with silt and silty clay/clayey silt layer in all boreholes with an observed thickness ranging from 1.5 m to 9.1 m. The unit ranged from damp to wet, typically ranging from fine to medium grained and containing trace to some silt and trace clay. The unit ranged from loose to very dense, with uncorrected SPT 'N' blow counts ranging from 5 to 67 and an average SPT 'N' value of 25.

Silt Layer: Brown silt was encountered interbedding the sand within KMS-13-01 and -05, with an observed thickness ranging from 1.8m to 2.0 m. The unit was moist, typically containing some gravel, some fine-grained sand with trace to some clay. The unit ranged from very stiff to hard, with uncorrected SPT ‘N’ blow counts ranging from 17 to 35 and an average SPT ‘N’ value of 26.

Silty Clay/Clayey Silt Layer: Grey silty clay/clayey silt were encountered interbedding the sand within all boreholes except for KMS-13-05, with an observed thickness ranging from 0.5m to 10.5m. The unit ranged from damp to moist, non to medium plastic and contained trace to some sand. The unit ranged from stiff to hard, with an uncorrected SPT ‘N’ blow count ranging from 12 to 43 and an average SPT ‘N’ of 32.

The following figure shows the variation of SPT number vs. depth in project P15.

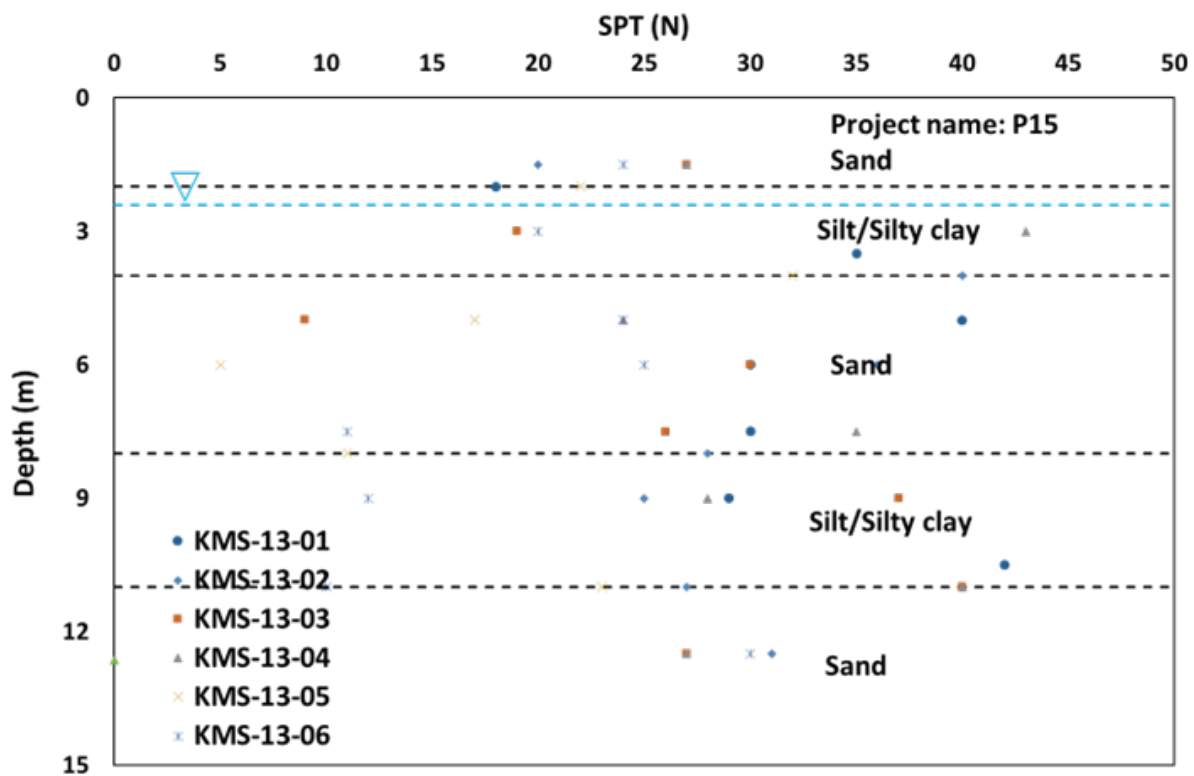


Figure 3.18: The variation of SPT number vs depth in P15

Project P16:

Beneath a thin surficial cover of organic topsoil, the descending stratigraphy observed in the test borings comprised clay till of variable thickness followed by an extensive deposit of sand. In some borings, localized silt or fine sand deposits up to 1 m thick were found above the till. In two boreholes located north of the ravine, (i.e. outside of the subject site) gravel was encountered within 4 m of the existing surface. Sand was not encountered within the 15 m drilling depth in BH 13-07 which was located on the north side of the crest of the ravine; and to a depth of 12 m in BH 13-19 which was located at the eastern extremity of the quarter section. Other than three test borings, groundwater was not encountered within the depths of exploration.

Clay Till Layer: Clay till was found beneath the organic topsoil in most borings. The clay till extended to depths varying widely from 1.5 to 8 m. The till was observed to be a silty, sandy clay matrix with plasticity ranging from low to medium. Contained within the till were discontinuous seams of fine-grained sand and silt. Moisture contents in the till varied typically between 10 and 12 percent. Sand Layer: Sand was encountered underlying the clay till and extends to some depth below the deepest termination depth of 21 m. Contained within the sand were thin, discontinuous clay lenses and seams. Moisture contents in the sand ranged from 2 to 10 percent with typical values of 5 percent or less, which were indicative of dry conditions. The following figure shows the variation of SPT number vs depth in project P16.

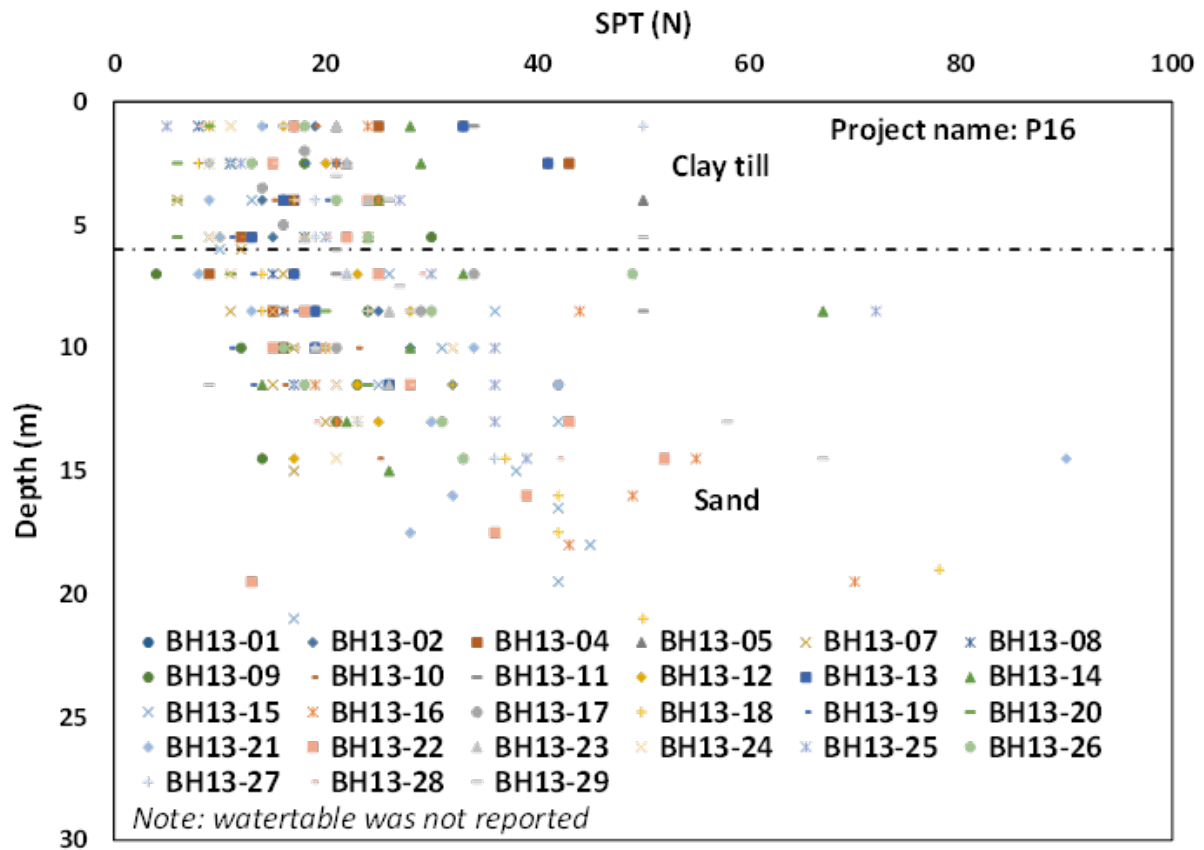


Figure 3.19: The variation of SPT number vs depth in P16

A summary of geotechnical properties of the projects obtained from drilled boreholes, laboratory and in-situ tests, and soil description in geotechnical reports is presented in the Table below.

Table 3.2: A summary of geotechnical properties of the projects

Project Name	Location in Alberta	Number of boreholes	Mx Depth of Borehole (m)	General soil layer	Pocket pen. range (kg/cm2)	UCS Range (kPa)	SPT range	CPT Range (qt) kPa	CPT Range (fs) kPa	Average W.T.L (m)
P1	NE	8	8.8	CL (till)	1 to 4	136-171	10-20	N.A.	N.A.	3.5
P2	C	6	13	CL (till)	N.A.	N.A.	7- 50	N.A.	N.A.	5.5
P3	SW	5	12.5	CH (SHALE)	N.A.	40-140	35- 50	N.A.	N.A.	3.5
P4	C	6	11	CL (till)	0.60 to 0.85	N.A.	20-32	N.A.	N.A.	5
P5	C	25	30	CL (till) / CH	0.75 to 0.145	101-402	11-50	845-30844	38-485	4
P6	SE	36	33.5	Clay (glacial) / Silt (till)	0.2 to 0.64	N.A.	30-50	N.A.	N.A.	2
P7	NW	54	24.2	CL (Till) / Shale	0.67 to 0.77	N.A.	8 - > 50	N.A.	N.A.	5.5
P8	NW	54	24.2	CL (Till) / Shale	0.67 to 0.77	N.A.	8 - > 50	N.A.	N.A.	5.5
P9	SW	21	26.7	CL (Till) / ML / CS/SI	> 216	N.A.	7 - > 50	N.A.	N.A.	7.6
P10	C	5	14.9	CL (Till)	N.A.	N.A.	10-17	N.A.	N.A.	> 15
P11	NE	43	61.4	SP-SM / ML	N.A.	N.A.	5 - > 50	N.A.	N.A.	5
P12	NW	2	10.6	SP/SW	N.A.	N.A.	6 - > 50	N.A.	N.A.	3.4
P13	NE	7	41.8	SP-SM/ CL/ ML/ CL/ CH	N.A.	N.A.	3- > 50	N.A.	N.A.	4
P14	SW	9	15.5	Gravel and Sand	N.A.	N.A.	5 - > 50	N.A.	N.A.	> 15.5
P15	SE	6	12.65	Silt/ Silty clay/ Sand	0.15 to 0.225	N.A.	11-43	N.A.	N.A.	2.4
P16	C	27	15	Clay till / Sand	N.A.	N.A.	9-38	N.A.	N.A.	> 15

3.2.3 PDA results

As stated earlier, a database of PDA test results is collected from sixteen oil and gas and infrastructure projects. This database includes six hundred and fifty-nine (659) PDA test results at End of Drive Condition (EOD) and three hundred and twenty-four (324) at Restrike of steel piles that have been driven in these projects.

All piles evaluated in this study were steel pipe-shaped piles with a minimum and maximum diameter of 114 mm and 915 mm, respectively. These piles are frequently utilized in Alberta because they are more cost-effective than other types and straightforward to install, especially given Alberta's geological conditions, which include numerous cobbles and stones in the underlying soil layers. The depth of installed piles varied from 4.5 m to 33 m. Different hammers, including diesel (represented by "D") and hydraulic (represented by "B" or "H") hammers, were used to drive the piles into the ground depending on the pile properties, including the diameter and length of piles, and ground properties, including the stiffness and type of soil. The hammers differed in ram weight, maximum drop height (maximum stroke) and striking efficiency.

It should be noted that among the 6845 piles that were installed in 16 projects evaluated in this study, 537 piles (approximately 8

The Case Pile Wave Analysis Program (CAPWAP) was used as a numerical modeling technique to analyze the PDA tests. The quality of results from a CAPWAP analysis depends on what is considered the "Best Match," defined by good agreement between the measured and the computed quantity. The Match quality (MQ) or match factor is an important parameter used to assess the CAPWAP analysis qualification (Fellenius, 1988). Although there is no definite upper or lower MQ limit for an acceptable CAPWAP result, analysis of data from driven piles with well-defined properties produces MQ typically between 2 and 4. Note that smaller values are uncommon, and also the value that would be acceptable in any given case will depend on the particulars of the situation. However, MQ values greater than 6 may indicate incomplete analysis, poor data, or a severe pile integrity problem, in which case the results could be unreliable (Rausche et al., 2010).

Table below presents a summary of pile properties installed in the projects and used for PDA tests.

Table 3.3: A summary of pile properties used for PDA tests

Project Name	Pile Type	Diameter of piles (mm)	Depth of PDA tests (m)	Type of hammer	Blow counts per last 25 cm	Avg. Max. Energy (EMX) (kN.m)
P1	Pipe	219	18	BSP-CX50	1-3	4-16.3
P2	Pipe	324	15-16	HHK-5A	3-25	49-62.3
P3	Pipe	324-406	7.5-12.5	HHK-5A	6.3-25	34-76.6
P4	Pipe	168-324	8.5-11	HHK-6A	1.2-13.9	14.2-73.9
P5	Pipe	610-914	12-37.5	D70-52, HHK-7S and HHK-12A	0.2-6	38-105
P6	Pipe	219-324	4.5-33	D19-32, D19-42 and HHK-5A	0.5-55	24.5-66.7
P7	Pipe	219-406	8-20.5	HHK-5S	10-28.6	10.2-70.8
P8	Pipe	324-406	8-20	HHK-5S	0.6-5.2	16.2-50.1
P9	Pipe	324	13-21	HHK-5S	1.9-41	50-70.6
P10	Pipe	114-406	10-21.5	HHK-5A	3-30	11.5-50.5
P11	Pipe	406-915	14.5-46	D50-32, D30-32, D100-42, HHK-5S and HHK-6S	3.6-21.7	14-218
P12	Pipe	219-406	10.5-16.5	HHK-6A	0.4-50	26.8-38.9
P13	Pipe	406-914	6-40.5	B6506 and BCG-180	0.5-33	50.4-145.7
P14	Pipe	273-508	6.5-16	HHK-5A	8.3-33.3	65.8-30.1
P15	Pipe	219-406	8-19.5	HHK-5S	13.7-25	24.3-67.6
P16	Pipe	219-324	8.5-12.5	HHK-5A	1-3	37.3-61.1

3.3 Data limitation and assumptions

Reliability based design focuses on the most important aspect of performance, which is the probability of failure. The probability of failure depends on both parametric and model uncertainties. The probability of failure is a more consistent and complete measure of safety since it is invariant to all mechanically equivalent definitions of safety, and incorporates additional uncertainty information. The main advantage of reliability analysis is the ability to carry out a much broader range of parametric studies without actually performing thousands of design checks with different inputs one at a time (Phoon, 2008). Calibration using reliability theory consists of limiting the probability of failure (pf) of structures to a certain acceptable extent. In order to do this technique some assumptions have been considered in this study as be-low:

3.3.1 Pile properties and loading condition

- **Pile type:** Alberta’s geological profile includes a broad spectrum of soil types—from soft alluvial and lacustrine clays in river valleys, to dense glacial tills, to sandy deposits and fill materials in urban areas. Open pipe piles are highly versatile and can be in-stalled effectively in both cohesive and granular soils, making them suitable for projects across diverse geological zones. Open pipe piles can often be driven with relatively high efficiency, especially when soil plug formation occurs naturally during driving, increasing resistance and reducing penetration depth. Their hollow cross-section reduces the required energy compared to closed-end piles of similar diameter. Moreover, open-ended piles can displace less soil, which minimizes ground heave—an important consideration in sensitive urban or environmental settings. As such in this study only open steel pipe has been considered.
- **Axial and lateral loading conditions:** Pile foundations are primarily used to transfer vertical (axial) loads from super-structures—such as buildings, bridges, or industrial equipment—into deeper, more competent soil or rock layers. Axial capacity ensures that the structure remains stable under service conditions, resists settlement, and meets bearing requirements—which are typically the most critical aspects for structural integrity. In many civil infrastructure projects (e.g., bridges, buildings, tanks), lateral loads—such as those from wind, seismic forces, or earth pressure—are relatively small compared to axial loads. As a result, lateral capacity rarely governs unless in special cases (e.g., offshore piles, retaining walls, or high seismic zones). Therefor in this study only Axial loading has been considered.

- **Pile group effect:** The pile group effect refers to the interaction between individual piles within a group, which often results in reduced efficiency compared to single isolated piles. However, there are well-established conditions under which this effect can be considered minimal or negligible, allowing for simplification in design. The pile group effect becomes minimal when the interaction between individual piles is sufficiently weak, such that the group capacity approaches the sum of individual pile capacities. In such cases, group efficiency approaches 1.0, and no significant reduction in capacity, stiffness, or settlement behavior occurs due to group interaction. This simplification can significantly ease analysis and design. As the spacing of piles in these projects are wide enough the group effect has been considered negligible.

3.3.2 Semi-Empirical calculation

- **Soil Properties:** In order to measure pile properties Cu results and pocket penetrometer results and CPT results have been used for fine grained soils. SPT results only used for course grained soil properties calculation.
- **Forst Impact:** Soil frost impact has not considered in this study. As primary of piles are lengthy, the impact of frost on pile capacity calculation can be considered negligible
- **Seasonal water table fluctuation** In this study the impact of fluctuation of water table in different seasons hasn't been considered
- **Overley / normal consolidated clay:** The clay materials have been considered as normal consolidated soil as such the negative pressure from overly consolidated clay impact excluded.

Figures. 3-20 to 35 show the measured vs. predicted results of piles in all projects. The details of each data, as mentioned earlier, can be found at the address below:

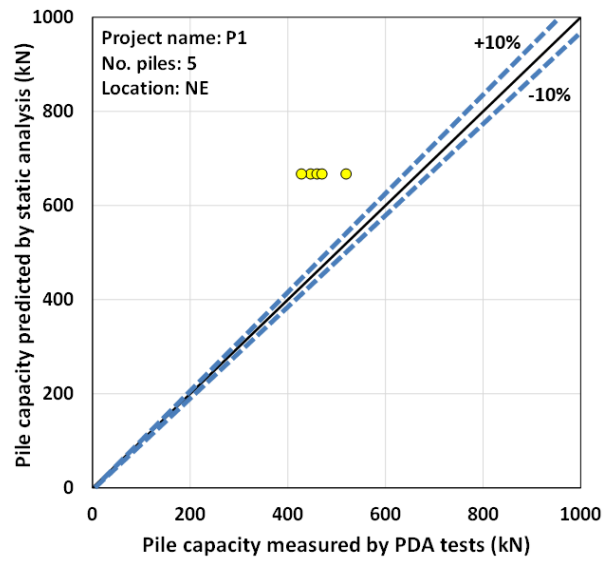


Figure 3.20: The measured vs predicted results for piles in P1

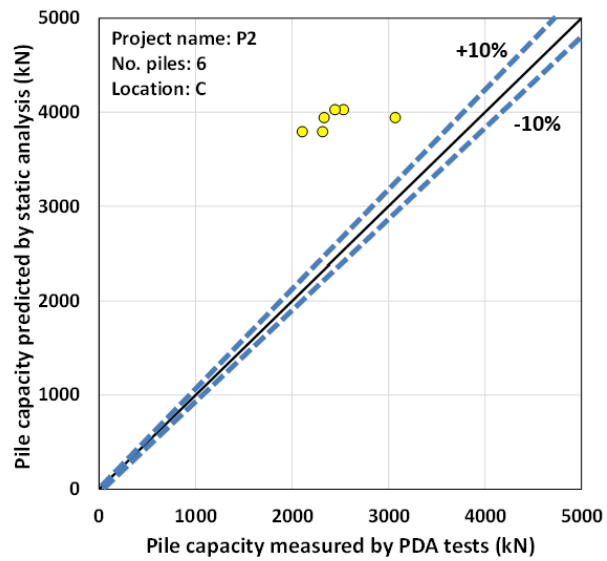


Figure 3.21: The measured vs predicted results for piles in P2

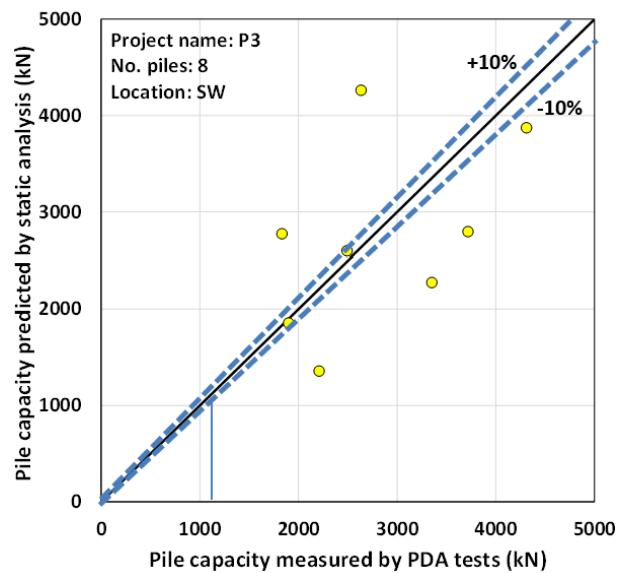


Figure 3.22: The measured vs predicted results for piles in P3

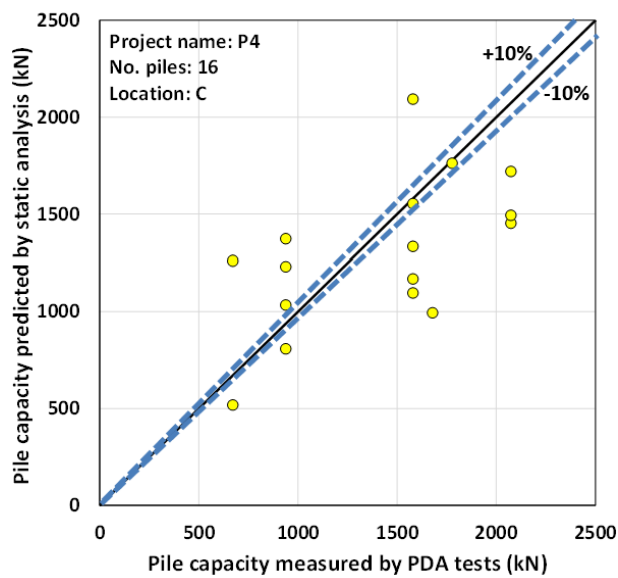


Figure 3.23: The measured vs predicted results for piles in P4

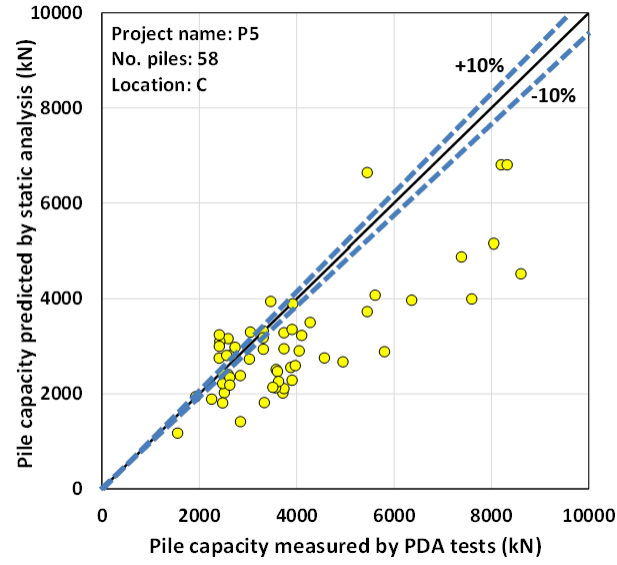


Figure 3.24: The measured vs predicted results for piles in P5

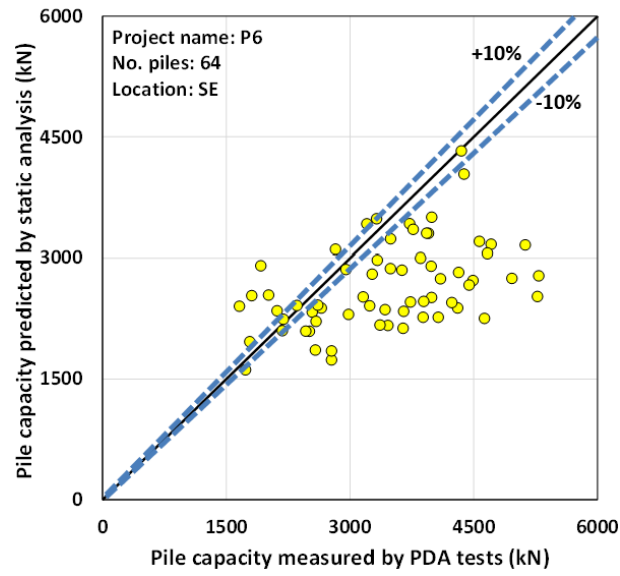


Figure 3.25: The measured vs predicted results for piles in P6

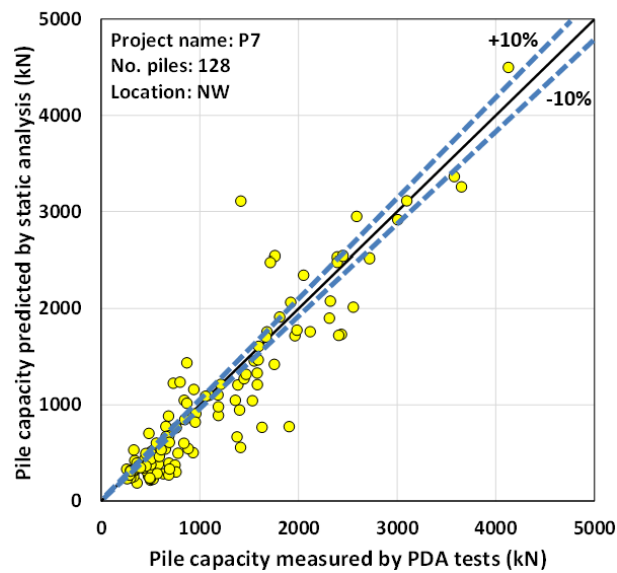


Figure 3.26: The measured vs predicted results for piles in P7

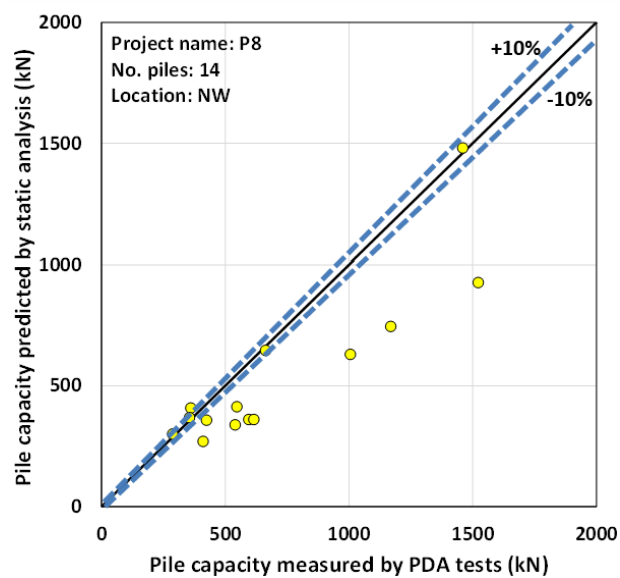


Figure 3.27: The measured vs predicted results for piles in P8

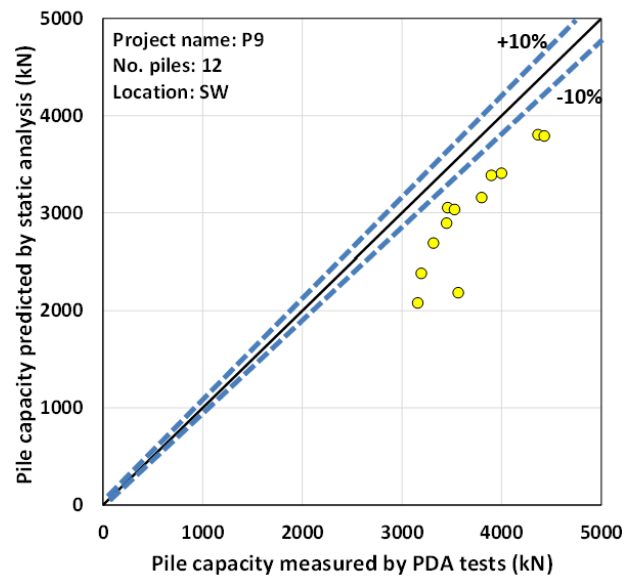


Figure 3.28: The measured vs predicted results for piles in P9

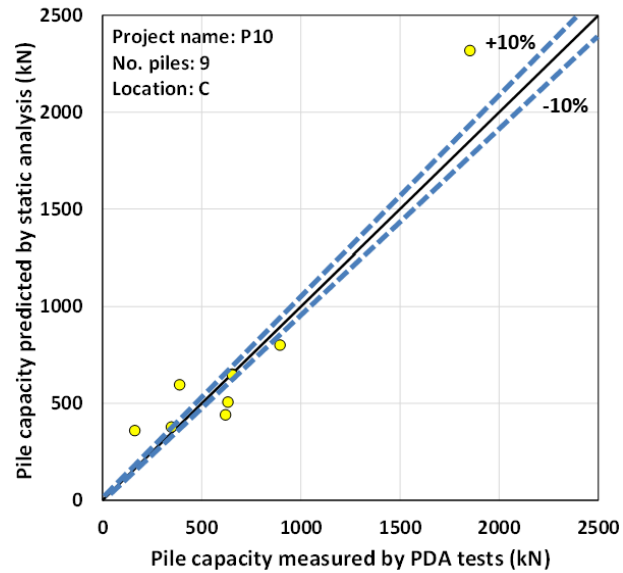


Figure 3.29: The measured vs predicted results for piles in P10

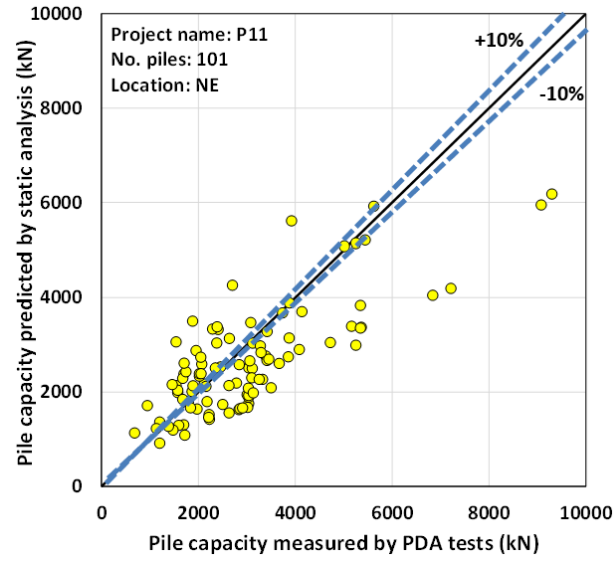


Figure 3.30: The measured vs predicted results for piles in P11

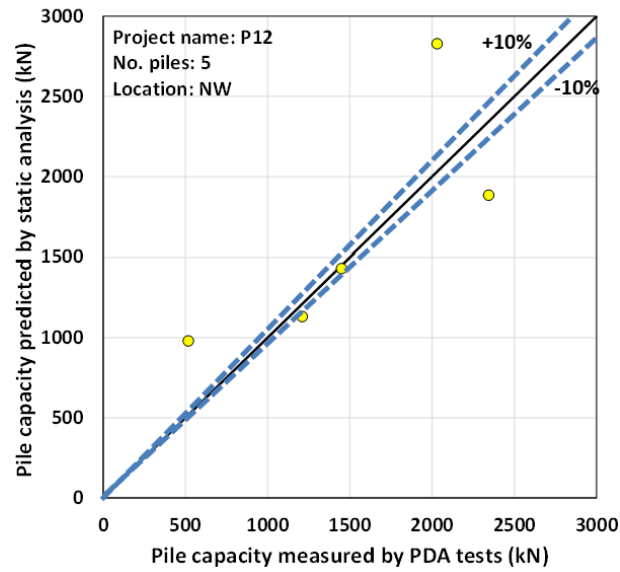


Figure 3.31: The measured vs predicted results for piles in P12

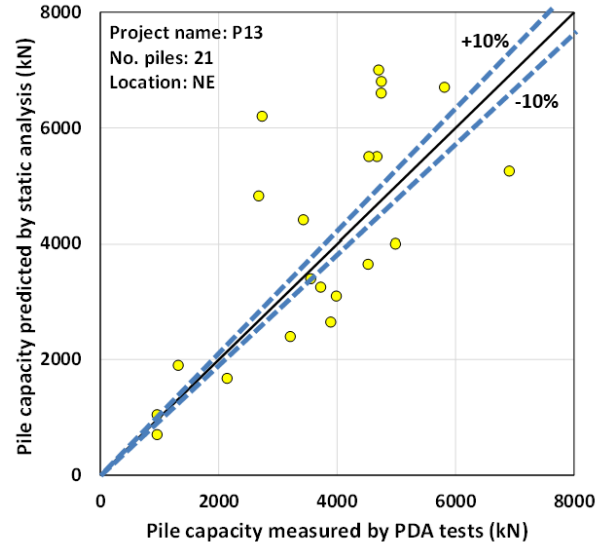


Figure 3.32: The measured vs predicted results for piles in P13

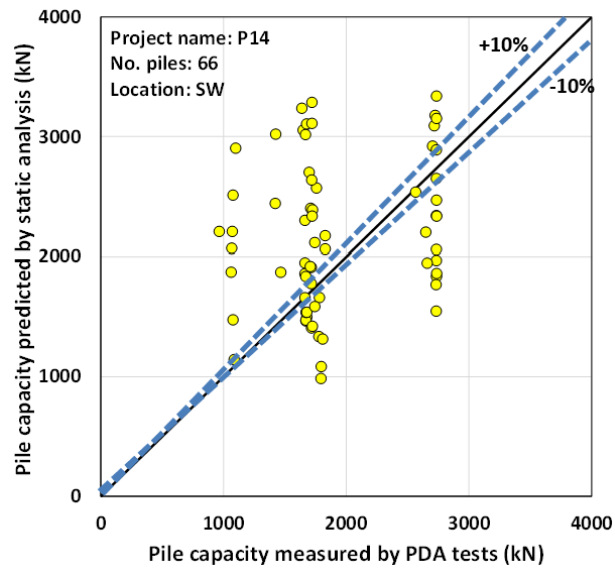


Figure 3.33: The measured vs predicted results for piles in P14

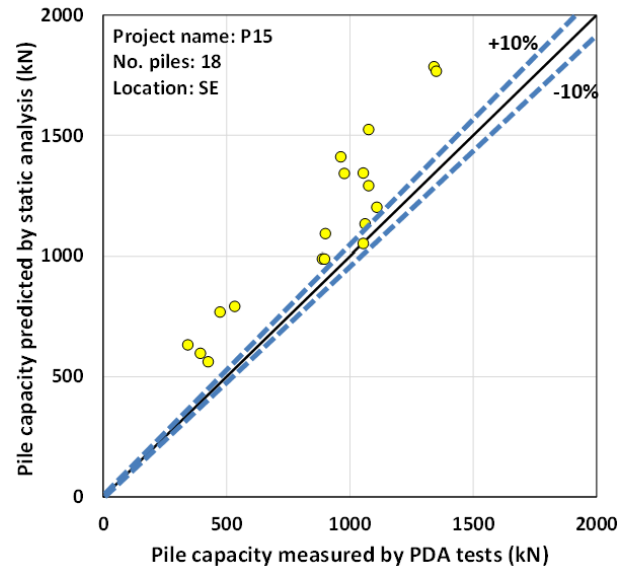


Figure 3.34: The measured vs predicted results for piles in P15

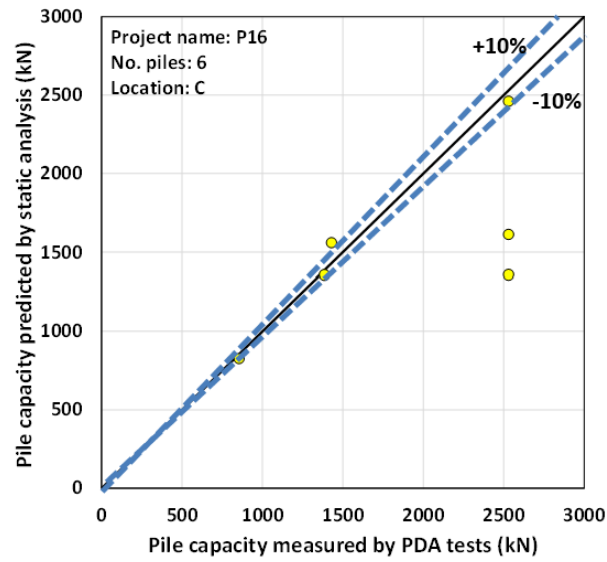


Figure 3.35: The measured vs predicted results for piles in P16

3.3.3 Static analyses of pile bearing capacity

The static methods which are widely proposed by well-known standards and manuals (e.g., CGS, 2006; API, 2000), were used to predict the pile bearing capacity in this study. To measure the empirical parameters used in prediction methods, including α, β and N_t (or N'_t), the results of in-situ and laboratory tests reported in geotechnical reports, correlation relations proposed by different researchers such as Tomlinson (1957) and Kulhawy and Jackson (1989) and the values suggested in Canadian Foundation Engineering Manual (CFEM) were used. It should be noted that the projects were classified into three main types using the “70% rule” as described by AbdelSalam et al. (2011), Jabo, (2014) and Bostwick (2014). Projects with more than 70% cohesive soils along the pile length are considered as “cohesive” soil type, projects with less than 30% cohesive soil layers along the pile length are considered as “cohesionless” soil type, and the others with cohesive soil between 30% and 70% are considered as the “mixed”.

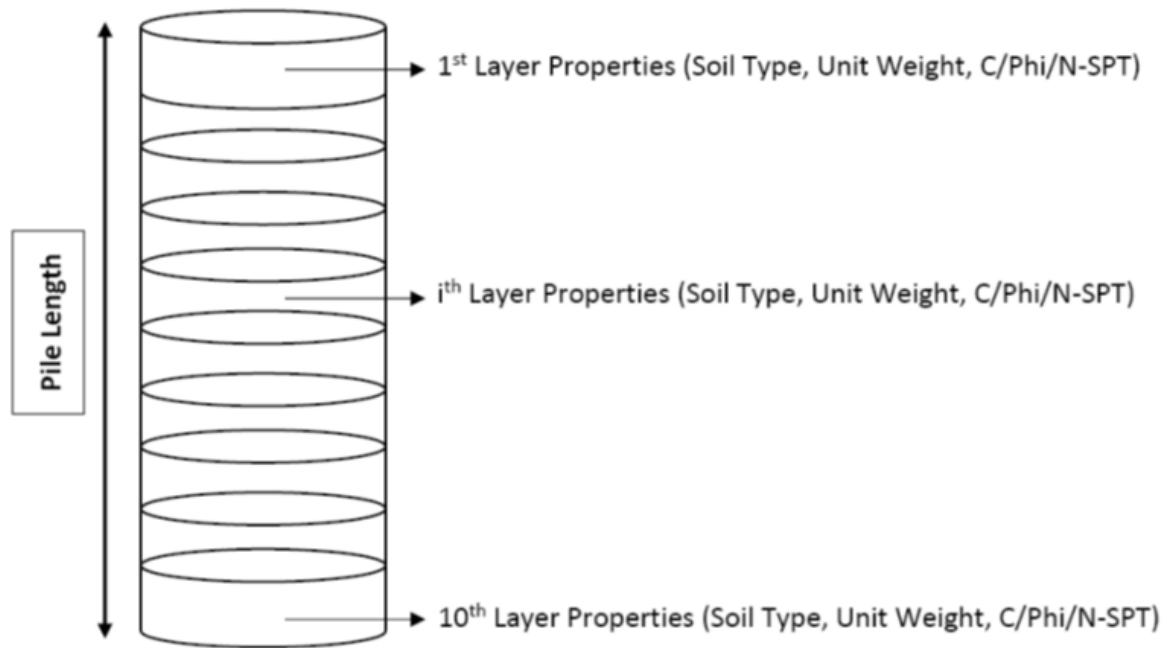


Figure 3.36: A schematic image of a pile divisions during calculation process

The results of static analysis were compared to the capacities measured by PDA tests, the results have been provided in next chapters.

3.3.4 Statistical Basis for Assuming Distribution in Calibration

In reliability-based design, pile resistances are treated as random variables due to inherent variability in soil properties, installation techniques, and measurement uncertainties. The calibration framework of Load and Resistance Factor Design (LRFD) necessitates the selection of an appropriate probability density function (PDF) to model the distribution of resistances. Commonly, either the normal or lognormal distribution is assumed. However, such assumptions require statistical validation through goodness-of-fit tests to ensure the selected model adequately represents the underlying data (SAbdelsalam, 2014). Among the candidate distributions, the lognormal distribution is often adopted in geotechnical engineering for several key reasons (Cao et al., 2021; Lin & Bathurst, 2019):

- **Non-Negativity** Geotechnical resistance parameters such as shaft resistance and end bearing capacity are strictly non-negative. The lognormal distribution, defined only over the positive real line, is thus well-suited for modeling such variables.
- **Right-Skewness** Empirical resistance data are frequently characterized by positive skewness, with a concentration of values near the mean and a long right tail. This behavior aligns with the inherent properties of the lognormal distribution.
- **Multiplicative Error Structure** Soil resistance values often emerge from the product of multiple random influencing factors (e.g., soil strength variability, pile installation effects). The central limit theorem for multiplicative processes supports the emergence of a lognormal distribution under such conditions.
- **Computational Advantages** The lognormal distribution offers analytical tractability in the estimation of reliability indices, resistance bias, and coefficients of variation (COV), particularly when applied within First Order Reliability Methods (FORM) used for LRFD calibration.

To substantiate the assumption of a lognormal distribution, various statistical hypothesis tests can be conducted. These tests evaluate whether the observed resistance data significantly deviate from the theoretical distribution under consideration. The null and alternative hypotheses are formulated as follows:

H_0 : The data follow the specified distribution (e.g., lognormal)

H_1 : The data do not follow the specified distribution

Commonly used goodness-of-fit tests in this context include:

- Chi-Square Test (χ^2)
- Kolmogorov–Smirnov Test (K–S)
- Anderson–Darling Test (A–D)
- ALilliefors Test (Monte Carlo-based for composite hypotheses)

Among these, the Chi-Square test is widely used due to its simplicity and compatibility with binned data (Ebeling, 1997). It compares the observed frequency counts in discrete intervals with expected counts derived from the cumulative distribution function (CDF) of the assumed PDF. The test statistic is defined as:

$$\chi^2 = \sum_{i=1}^K \frac{(O_i - E_i)^2}{E_i} \quad (3.1)$$

where O_i and E_i are the observed and expected frequencies respectively for bin i , and the latter frequency is given by $E_i = F(x_2) - F(x_1)$, where $F(x)$ is the CDF of the probability distribution being tested, and x_1, x_2 are the limits for bin i . The null and alternative hypotheses are H_0 for the data that follow the specified distribution; and H_1 for the data that do not follow the specified distribution. The hypothesis regarding the distributional form is rejected at the chosen significance level α if the test statistic is greater than the critical value defined $X^2_{(1-\alpha, k-1)}$ meaning the Chi-Squared inverse CDF with $k-1$ degrees of freedom and a significance level of α (SAbdelsalam, 2014). The Chi-Square test was applied to PDA data and estimated resistances for evaluating the appropriateness of the lognormal distribution. The resulting p-values were 0.50 and 0.58, respectively. Since both values are substantially greater than the conventional threshold of $\alpha = 0.5$, the null hypothesis cannot be rejected. These results suggest no statistically significant difference between the observed and expected frequencies under the lognormal assumption, thereby supporting the suitability of the lognormal distribution for modeling pile resistances in LRFD calibration.

3.3.5 Statistical Basis Parameters

There are several statistical methods that can be used for calibrating the LRFD resistance factors. The most robust techniques are the First Order Second Moment (FOSM), the

First there are some parameters that need to be assumed in the calibration techniques. The dead and live load bias factors are taken as 1.05 and 1.15, respectively. These values are consistent with findings in load modeling studies (Ellingwood et al., 1980; Nowak & Collins, 2000), which suggest that dead loads are typically estimated with higher accuracy because they are based on well-defined material densities and dimensions. In contrast, live loads are more variable and often underestimated in design, hence the higher bias factor.

The coefficients of variation (COV) are assumed to be 0.1 for dead loads and 0.2 for live loads. These values align with statistical load surveys (Ellingwood et al., 1980; Galambos et al., 1982) showing that dead loads exhibit relatively low variability, while live loads display significantly higher dispersion due to their dependence on occupancy, traffic, or environmental effects. These assumptions have also been widely adopted in Canadian and U.S. reliability-based calibration studies (Paikowsky et al., 2004; Allen, 2005). The coefficients of variation (COV) are assumed to be 0.1 for dead loads and 0.2 for live loads. These values align with statistical load surveys (Ellingwood et al., 1980; Galambos et al., 1982) showing that dead loads exhibit relatively low variability, while live loads display significantly higher dispersion due to their dependence on occupancy, traffic, or environmental effects. These assumptions have also been widely adopted in Canadian and U.S. reliability-based calibration studies (Paikowsky et al., 2004; Allen, 2005). Load factors of 1.25 for dead loads and 1.75 for live loads correspond to values recommended in the Canadian Foundation Engineering Manual (CFEM, 2006; 2023) and the Canadian Highway Bridge Design Code (CHBDC, 2014). Higher factors are applied to live loads because of their greater uncertainty, while smaller factors suffice for dead loads given their more predictable nature. The dead-to-live load ratio (QD/QL) is assumed as 3, which reflects typical conditions in building and infrastructure projects in North America where permanent loads from structural weight and finishes dominate over transient imposed loads (Nowak & Szeszen, 2003). Such a ratio is frequently employed in reliability calibration studies to provide representative conditions for foundation and structural reliability assessment (Paikowsky et al., 2004). Finally, for simulation-based approaches, the MCS method uses 100,000 simulations, which ensures adequate convergence and statistical stability in estimating small probability of failure values (Melchers & Beck, 2018). In contrast, FOSM and FORM rely on analytical approximations, eliminating the need for large-scale sampling. In summary, the chosen assumptions are well supported by established literature and design codes. They represent a balance between realistic representation of load variability and practical simplifications required for reliability-based design and calibration. The assumptions used in this study are presented in Table below.

Table 3.4: The assumptions used in reliability methods (Paikowsky et al., 2004, Allen, 2005, AbdelSalam et al., 2011)

Method	dead and live loads bias factors		coefficients of variation for the dead and live loads		dead and live load factors		dead to live load ratio	number of simulations
	λ_{QD}	λ_{QL}	COV_{QD}	COV_{QL}	γ_D	γ_L	QD/QL	N
FOSM	1.05	1.15	0.1	0.2	1.25	1.75	3	—
MCS	1.05	1.15	0.1	0.2	1.25	1.75	3	100000
FORM	1.05	1.15	0.1	0.2	1.25	1.75	3	—

3.4 Summary:

This chapter delves into the creation of a detailed database and the methodological approach utilized in this study. Dataset has compiled 659 PDA results at End of Drive (EOD) along with 324 Restrike outcomes from 16 projects in Alberta, encompassing both oil and gas, as well as infrastructure projects. These ventures spanned varied geological and geotechnical conditions. The geology of Alberta was subdivided into six regions (Southwest, Southeast, Central, Northwest, Northeast, and Rocky Mountains & Foothills), though the latter was omitted due to incomplete data. For future research facilitation, the database was uploaded to a secure-access website. The related geotechnical properties assessments comprised CPT, pocket penetrometer tests, unconfined compressive strength, and Atterberg limits for fine-grained soil and SPT for coarse-grained soil, provided for semi-empirical method estimations in each project. All recorded driven piles were open-ended steel pipe piles with diameters from 114 mm to 915 mm and depths ranging from 4.5 m to 33 m. The choice between diesel and hydraulic hammers depended on the soil and pile conditions. CAPWAP was used to assess the PDA results, with acceptable Match Quality (MQ) values ranging from 2 to 4. Assumptions for reliability calibration such as a single pile type, only axial loading condition, no pile group impact, no over-consolidation effects, etc. have been discussed. The best-fit model for statistical analysis was determined using Chi-square. Finally, a summary of statistical basis parameters used for calibration analysis is provided.

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Chapter 4

Recalibration of LRFD Resistance Factors for Driven Steel at End of Drive Conditions in Alberta, Canada

4.1 Background

The contents presented in this chapter are summarized from the manuscript below:

Roshani et al. (2024) Recalibration of LRFD Resistance Factors for Driven Steel Piles at End of Drive Conditions in Alberta, Canada. Manuscript published in an ASCE- Journal of Risk and Uncertainty Analysis.

4.2 Practical Applications

In the recent decades, many countries have moved from the allowable stress design (ASD) method to the load and resistance factor design (LRFD) method. The LRFD method employs the geotechnical resistance factor (GRF) to design geotechnical structures such as pile foundations. As recommended by different design codes, regional recalibration of GRF based on local in-situ pile tests can lead to being more reliable and removing unnecessary conservative assumptions during the pile design procedure. Using a comprehensive database, including 28 static load and 623 PDA tests carried out in Alberta, Canada, it is aimed to evaluate and calibrate the GRF of the LRFD method for the bearing capacity of driven steel piles based on site-specific geotechnical investigation reports and analytical

approaches considering different static analysis methods in this study. Based on the results, the recommended GRFs in cohesive, mixed, and cohesionless soils for $\beta = 2.33$ are 0.58, 0.62, and 0.64, respectively. However, for $\beta = 3.00$ the GRF values are 0.46, 0.49, and 0.49 for cohesive, mixed, and cohesionless, respectively. The regionally recalibrated GRFs are higher than those recommended by CFEM (2006), which may lead to reducing the number or length of piles required during the design process.

4.3 Introduction

The oil and gas industry has grown significantly in recent decades in Alberta, Canada. These industries contain heavy and sensitive infrastructures that generally need piles as the foundation. Driven steel piles, including H and pipe piles, are the most common type of deep foundations used in Canada (Belbas 2013; Balasingam et al. 2015; Ni 2018; Roshani et al. 2020). Estimating the pile bearing capacity is one of the most difficult problems in the geotechnical field due to the many uncertainties related to the pile foundations, including measurement noise/random error, systematic measurement errors and transformation uncertainty, as well as inherent soil uncertainties and model parameters (Briaud and Tucker 1988; Phoon and Kulhawy 1999a, 1999b; Dithinde et al. 2011). Although uncertainties in geotechnical problems are inevitable (e.g., there is a large uncertainty in site characterizations as input data during the design process of structures) (Aladejare and Wang 2017; Phoon 2018; Cami et al. 2020; Ching and Phoon 2020), many researchers have tried to assess the effect of different uncertainties in the design of geotechnical structures (Whitman 2000; Wang 2009; Shahnazari et al. 2016; Golafzani et al. 2019; Phoon 2023). In the recent decades, many countries, such as the United States, Canada, and European countries, have moved from the allowable stress design (ASD) method to the load and resistance factor design (LRFD) method (Foye et al. 2006). Separate load and resistance factors are employed in the LRFD method, considering the inherent uncertainties in estimating the allowable loads and resistance to create a consistent probability of failure in the design of structures such as pile foundations. To be more reliable and to remove unnecessary conservative assumptions during the pile design procedure, regionally calibrated LRFD resistance factors based on local in-situ pile tests can be effective. This approach is recommended by different design codes such as the Federal Highway Administration (FHWA) (AASHTO 2007). Reddy and Stuedlein (2013) stated that using a national database to predict pile capacity can lead to high inaccuracies and uncertainties. The geotechnical resistance factors (GRF) for different static analysis methods were evaluated based on a regional pile loading database, including 140 Pile Driving Analyzer (PDA) and 35 static load tests (SLTs) data in North Carolina (Kim 2003; Kim et al. 2005).

Similar efforts were made for the calibration of GRF in California based on a database of 127 driven piles, including 85 piles under compression load, 84 piles under uplift load, and 41 piles under both compression and uplift loads, and 79 drilled piles (Yu et al. 2017). Abdelsalam et al. (2011) used 80 static load tests on steel H-piles to establish regional LRFD design recommendations for pile foundations in Iowa State. In this study, the GRF values were evaluated for three major soil classifications, including sandy, clayey, and mixed soils. In addition, the effect of cohesive material of the soil deposit on the resistance factor was presented. The results showed that using a regional database resulted in resistance factors approximately 40%, 55%, and 42% higher than those recommended in AASHTO (2007) for sandy, clayey, and mixed soils, respectively. Similar results were reported by Jabo (2017) for pile resistance factors in Arkansas. Li et al. (2015) using 146 driven piles in China, also reported that greater resistance factors can be used for the design of pile foundations based on regional database analysis. Thomson et al. (2017) calculated the site-specific GRF of a large industrial project in Canada, which was 20% greater than that recommended by CFEM (2006) and proved the considerable effect of this study on a reduction of approximately 40% foundation construction costs. Bougataya and Stuedlein (2016) gathered a database of 189 piles in Washington and presented the benefit of using a high-quality, region-specific database for calibration of resistance factors. Asghari Pari et al. (2019) calibrated GRF values for onshore piles driven into sandy, clayey, and mixed soils in Iran based on 241 pile data. By comparing different static and dynamic methods for predicting the bearing capacity of piles, the importance of using a local database to increase the efficiency of the regional pile design procedure has been emphasized. **Table 4-1** presents the summary of the most important studies on recalibration of geotechnical resistance factor. As this approach has not yet been applied in Alberta. Therefore, there is a need to calibrate GRF recommended by CFEM for Alberta region. In this study, it is aimed to evaluate and calibrate the GRF of the LRFD method for the bearing capacity of driven piles in Alberta, Canada, based on site-specific geotechnical investigation reports and analytical approaches considering different static analysis methods recommended by the design codes such as Canadian Foundation Engineering Manual (CFEM). In this regard, a comprehensive database, including 28 static load (SL) and 623 PDA tests was collected.

Table 4.1: Literature review on the most important recalibration studies on GRF

Region / Country	LRFD Calibration Method	Data-base Piles No.	Field Test	Pile Type	Reference
North Carolina /U.S.A	FOSM, FORM	175	SLT, EOD, BOR	Pipe, H, PPC	Kim (2003)
World	FOSM, FORM	338	SLT, EOD, BOR	Pipe, H, PPC, T	Paikowsky et al. (2004)
World	MCS	338	SLT, EOD, BOR	Pipe, H, PPC, T	Allen (2005)
Iowa/ U.S.A	FOSM	90	SLT	H	AbdelSalam et al. (2011)
Rhode Island/ U.S.A	FOSM	6	SLT	Pipe	Bradshaw et al. (2014)
Arkansas /U.S.A	FORM, FOSM, MCS	138	SLT, EOD	Pipe, H, PPC	Bostwick (2014)
Shanghai/ China	FOSM	146	SLT	Square	Li et al. (2015)
Arkansas/ U.S.A	FORM, FOSM, MCS, DF	59	SLT, EOD, BOR	Pipe, H, PPC	Jabo (2014, 2017)
Canada	FOSM	25	SLT, EOD	Pipe	Thomson et al. (2017)
Washington/ U.S.A	DF	189	SLT, EOD, BOR	Pipe, H, PPC	Reddy and Stuedlein (2013), Bougataya and Stuedlein (2017)
California/ U.S.A	MCS	85	SLT	Pipe, H, PPC, CRP	Yu et al. (2017)
Inshore Regions /Iran	FORM, FOSM, MCS	241	SLT, EOD, BOR	Pipe, H	Asghari Pari et al. (2019)

4.4 Geotechnical and Geological Condition of Alberta

Acton et al. (2015) indicated that the geology of Canada can be divided into seven physiographic regions: Arctic Lands, Cordillera, Interior Plains, Hudson Bay Lowland, Canadian Shield Forest Lands, St Lawrence Lowlands and Appalachia. Physiographic regionalization is mainly referred to as the process by which regions with relatively homogeneous physical geography and landforms are determined. Alberta province is divided by three of Canada's seven physiographic regions, as described by Acton et al. (2015). These three regions are the Cordillera, Interior Plains and Canadian Shield, as illustrated in Figure 4-1. Although most of the province falls within the Interior Plains region.

The Interior Plains area of Canada encompasses the region between the Canadian Shield and the western Cordillera. The plains are distinguished by vast expanses of sedimentary bedrock consisting mainly of poorly consolidated shales, siltstones, and sandstones. Bedrock of the Interior Plains is the result of Quaternary glaciation (2.6 million to 10,000 years ago) and mainly consists of flat-lying limestones and shales of marine origin (Acton et al. 2015). Pawluk and Bayrock (1969) stated that Alberta is underlain by mixed sandstone-shale formation with a high proportion of sandy detritus.



Figure 4.1: Physiographic regions of Canada (Acton et al. 2015)

Non-marine sediments with young age, largely sands and gravels representing fluvial deposits flowing eastward from the newly developing mountains to the west, covered these marine sediments in the western part of the region. Erosion of the less resistant of these non-marine sediments, in association with uneven uplift that continued with the formation of mountains to the west, resulted in the carving of the western part of the region into a series of isolated uplands (Acton et al. 2015). To cover different soil regions and geologic formations in the province, the geological map of Alberta developed by Hamilton et al. (1998) and AGS (2021) is divided into six zones, including southwestern Alberta (SW), southeastern Alberta (SE), central Alberta (C), northwestern Alberta (NW), northeastern Alberta (NE) and Rocky Mountains and Foothills (RM&F) which are presented in **Figure 4-2**.



Figure 4.2: The geological zones of Alberta divided in this study (map adopted from Encyclopædia Britannica 1999)

4.5 Development of Database

Static load test (SLT) is the most well-known and reliable technique for measuring the actual pile nominal resistance and load-settlement relationships of piles. The static testing involves applying incremental static loads and measuring the resulting pile displacements. Another method of measuring pile resistance is Pile Driving Analyzer (PDA) as a dynamic

load test (DLT) which uses attached strain gauges and accelerometers to the pile during driving by hammer impacts. The PDA results were analyzed using the Case Pile Wave Analysis Program (CAPWAP) which is a post-processing signal matching method (Likins et al. 2000). Fellenius (2024) believed that PDA with CAPWAP can provide valuable information on load-transfer, especially for driven piles. PDA tests are typically performed under two driving conditions including End-of-Driving (EOD) and Beginning-of-Restrike (BOR) which are different at the time of analysis. EOD refers to the moment when the pile driving reaches its intended depth, and the driving process is completed. BOR is the phase where the pile is struck again after reaching the EOD condition. This is done to further examine the pile's behavior and assess its capacity under a repeated impact. It's important to note that the analysis of the PDA test results may differ for the EOD and BOR conditions. The data obtained during each phase is analyzed separately to understand the pile's behavior at different stages of the driving process. More details on the procedure of SLT and PDA can be found in ASTM D1143 and D4945. While SLT is indeed regarded as the most accurate method for measuring the piles' resistance if conducted properly, its expensive and time-consuming nature has constrained its application in various projects (Jabo 2014; Asghari Pari et al. 2019). Therefore, the PDA test is becoming increasingly popular as an alternative to measure the piles' resistance, especially in Canada.

For the purpose of the study, an extensive database of SLT and DLT results is collected through several field works (Interior Plains physiographic region). This database includes the results of 28 SLT and 623 PDA at EOD condition tests obtained from 16 construction projects in Alberta. Table 4-2 summarizes the main properties of the projects used in the database.

In each project, a geotechnical investigation program was implemented and comprised conventional boreholes, test pits, and field tests (e.g., CPT, SPT) with associated laboratory index such as undrained shear strength, element and stress deformation testing. Based on an overall review of geotechnical site investigation reports, it can be concluded that the soil layers at the Alberta (except Rocky Mountains and Foothills zone) consisted of sandy silty clay (glacial till) containing intermediate layers of sand and silty sand. The glacial till was cohesive and contained varying proportions of sand and gravel. The glacial till was generally intermediate in plasticity and varied in consistency from firm to hard, generally becoming stiffer with depth. Intermediate layers of fine sand and cohesionless silty sand were not encountered at all of the studied sites but were encountered at the majority of locations at varying elevations and thicknesses. The relative density of these interlayers varied from medium dense to very dense. The variability in the presence, depth, and thickness of cohesionless intermediate layers introduced uncertainty into the geotechnical strength estimates of the piles.

Table 4.2: Geotechnical and constructed piles properties of the projects

Projects	Location in Alberta	Date	Number of PDA Tests	Geotechnical properties			Pile properties	
				General soil type	Range of N_{spt}^1	Range of c_u^2 (kPa)	Type	Embed. length (m)
P1	NE	2013	6	Cohesive	10–20	150–300	Pipe	13.5
P2	C	2014	6	Cohesive	7–50	≥ 215	Pipe	14.5–15.2
P3	SW	2014	8	Cohesive	35–50	150–300	Pipe	6.9–12
P4	C	2015	18	Cohesive	20–32	60–85	Pipe	8–10.5
P5	C	2014–2015	67	Cohesive/Mixed	11–50	75–215	Pipe	11.2–37.2
P6	SE	2013	64	Cohesive	30–50	20–64	Pipe	4.3–32.2
P7	NW	2012	40	Cohesive	8–50	60–80	Pipe	4.6–18.2
P8	C	2015	45	Cohesive	8–50	60–80	Pipe	4.4–17.9
P9	SW	2019	20	Cohesive	7–50	≥ 215	Pipe	9.3–20.1
P10	C	2014	10	Cohesive	10–17	85–215	Pipe	8.4–21.1
P11	NE	2012–2013	147	Cohesionless	5–50	N/A	Pipe	6.2–40.2
P12	NW	2015	5	Cohesionless	6–50	N/A	Pipe	10–16
P13	NE	2018	27	Cohesionless	3–50	N/A	Pipe	5.4–39.7
P14	SW	2014–2015	67	Cohesionless	5–50	150–225	Pipe	6–15.25
P15	SE	2020	21	Cohesionless	11–43	N/A	Pipe	4.9–17.7
P16	C	2014	6	Mixed	9–38	150–450	Pipe	8–12

1: N_{spt} refers to the Standard Penetration Test values. 2: c_u refers to undrained shear strength. 1: N_{spt} refers to the Standard Penetration Test values. 2: c_u refers to undrained shear strength.

The projects used were spread over 5 zones of Alberta including NW, NE, C, SW, and SE as presented in **Fig. 4-2**. Due to the lack of pile load test data in the Rocky Mountains and Foothills region, this area was ignored in this study. All the piles were steel pipe piles with various diameters between 114 to 915 mm and embedded lengths between 4 to 40 m. Although there are many factors to classify pile foundations, including soil strength, pile length, pile length/diameter, pile construction method, etc., the use of soil type along the pile length is a simple and useful method for pile classification. As it can be seen in **Table 4.2**, projects are classified into three main types using the “70% rule” as described by AbdelSalam et al. (2011), Bostwick (2014), Jabo (2014), and Asghari

Pari et al. (2019). Projects with more than 70% cohesive soils along the pile length (i.e., $L_R = L_c/L_{tot} \geq 70\%$, L_c is the length of the pile embedded in cohesive soil and L_{tot} is the total embedded pile length) are considered as “cohesive” soil type, projects with less than 30% cohesive soil layers along the pile length (i.e., $L_R \leq 30\%$) are considered as “cohesionless” soil type, and the others with cohesive soil between 30% and 70% are considered as the “mixed” type. It should be noted that this classification method has some limitations in the case of end-bearing piles and the existence of poor soil layers (e.g., soft clay) without bearing capacity along the length of the piles.

4.6 Static Analysis Methods

As static analysis methods use various soil conditions and properties as well as correlation relationships, their uncertainty levels are different. In this study using the results of the geotechnical investigation programs, the pile bearing capacities were predicted by different well-known static analysis methods with different levels of reliability. Two α methods including α -CFEM (Tomlinson 1957; CFEM 2006) and α -API (Reese et al. 1998; API 2000) as well as β method (named β -AASHTO) (Burland 1973; AASHTO 2012) and λ method (Vijayvergiya and Focht 1972; US Army Corps of Engineers 1992; Kraft et al. 1981) were selected for piles driven into cohesive soil. For piles in cohesionless soil, the bearing capacity was calculated by the SPT-Meyerhof method (Meyerhof 1956, 1976) and two β methods including β -Bowles (Bowles 1996) and β -CFEM (CFEM 2006). As the β -AASHTO is based on effective stress conditions and λ method is a combined effective and total stress analysis, these methods were chosen for piles in mixed soil. In addition, AbdelSalam et al. (2011) showed the effectiveness of using a combination of methods (i.e., a method for cohesive soil and a method for cohesionless soil) for the prediction of pile bearing capacity in a mixed soil. Consequently, a combined method named α/β -CFEM, including α -CFEM for the cohesive part and β -CFEM for the cohesionless part of piles was also used for the LRFD calibration of mixed-soil sites in this study. It should be noted that the analysis methods selected are identified as commonly used methods in many countries and national codes (AbdelSalam et al. 2010, 2011). **Table 4-3** presents a summary of static analysis methods used in this study based on soil conditions.

Table 4.3: Summary of static analysis methods for steel open-ended driven piles used in this study.

Soil Description	Method	Shaft Resistance (q_s)	Toe Resistance (q_b)	Reference
Cohesive	α -CFEM	αc_u	$N_t c_u$	Tomlinson (1957); CFEM (2006)
	α -API	αc_u	$N_t c_u$	Reese et al. (1998); API (2000)
	β -AASHTO	$\beta \sigma'_v$	$N_t c_u$	AASHTO (2001)
	λ	$\lambda(\sigma'_v + 2c_u)$	$N_t c_u$	Vijayvergiya and Focht (1972); US Army Corps of Engineers (1992)
Cohesionless	β -CFEM	$\beta \sigma'_v$	$N_t \sigma'_t$	Burland (1973); CFEM (2006)
	β -Bowles	$\beta \sigma'_v$	$N_q \sigma'_t + 0.5 \gamma' B$	Bowles (1996)
	SPT-Meyerhof	kN	$0.4DN'/B$	Meyerhof (1956, 1976)
Mixed	β -AASHTO	$\beta \sigma'_v$	$N_t c_u$ or $N_q \sigma'_t$	AASHTO (2001); CFEM (2006)
	λ	$\lambda(\sigma'_v + 2c_u)$	$N_t c_u$ or $N_q \sigma'_t$	Vijayvergiya and Focht (1972); US Army Corps of Engineers (1992); CFEM (2006)
	α/β	Cohesive part: αc_u Cohesionless part: $\beta \sigma'_v$	Cohesive part: $N_t c_u$ Cohesionless part: $N_t \sigma'_t$	Tomlinson (1957); Burland (1973); CFEM (2006)

Notes: α : adhesion factor B : pile diameter β : shaft resistance factor c_u : undrained shear strength D : pile embedment depth γ' : buoyancy unit weight k : coefficient λ : dimensionless coefficient N : average SPT blow counts along the pile length N' : average SPT blow count of the bearing stratum N_t & N_q : bearing capacity factor σ'_t : vertical stress at the pile toe σ'_v : vertical effective stress at depth z

4.7 LRFD Design

Considerable efforts have been made in recent years on the application of LRFD in geotechnical engineering (e.g., Barker et al., 1991; O'Neill and Reese, 1999; Paikowsky et al., 2004; Foye et al., 2009; Yang and Liang, 2009; McVay et al., 2010; Tang and Phoon, 2019; Bian et al., 2021). In the LRFD method, load factors typically greater than unity are applied to the nominal loads to obtain factored loads. On the other hand, resistance factors which are less than unity, and also called strength reduction factors, are applied to the ultimate capacity (Coduto, 2001). A simple definition of failure is when factored loads exceed factored capacities. The basic equation of LRFD-based design can be expressed as follows (Dithinde, 2007):

$$R_r = \phi R_n \geq \sum \alpha_i Q_i \quad (4.1)$$

where ϕ is the geotechnical resistance factor, R_n is the ultimate (nominal) geotechnical resistance determined by engineering analysis using field parameters or performance data and represents the best estimate of the normal strength and takes account of all factors influencing the resistance, R_r is the factored resistance, α_i is the load factor corresponding to a particular load, Q_i is the specified load component of the overall load effects, and i represents various types of loads such as dead load, live load, etc.

The load factors are calculated according to the uncertainties of the loads and their probability of occurrence. The GRFs are estimated based on the variabilities of the geotechnical parameters and the analysis of uncertainties during the calculation of the ultimate (nominal) resistances (CFEM, 2006). The values of load and resistance factors can be obtained from various codes such as AASHTO (2012), National Building Code of Canada (NBCC, 2005), and Canadian Highway Bridge Design Code (CHBDC, 2014). Due to differences in codes in various countries, load factors are generally between 0.85 and 1.3 for dead loads and 1.5 to 2.0 for live and environmental loads. A load factor of less than unity for dead loads is used when the dead load component contributes to the resistance to overturning, uplift, and sliding (CFEM, 2006).

The basic theory of the LRFD method is defined based on the use of probability density functions (PDF) representing the load (Q) and resistance (R) and their relation to the limit state function (g) which is presented in Eq. (4-2):

$$g = R - Q \quad (4.2)$$

The main aim of reliability-based design is to ensure that the probability of failure does not exceed an acceptable threshold level. The probability of failure is defined as follows:

$$p_f = \text{Prob}(g < 0) \leq p_T \quad (4.3)$$

where $\text{Prob}(\cdot)$ is the probability of an event, p_f is the probability of failure, and p_T is the acceptable target probability of failure. Finally, the reliability index (β) can be defined by a proper equation such as follows:

$$\beta = -\phi^{-1}(p_f) \quad (4.4)$$

where $\phi^{-1}(\cdot)$ is the inverse of the standard normal cumulative function, ϕ .

4.8 Calibration Approach

Several probabilistic approaches are used in reliability-based design and in the calibration of the LRFD resistance factor for driven pile foundations (Jabo, 2014). The most robust techniques are the First Order Second Moment (FOSM), the First Order Reliability Method (FORM), and the Monte Carlo Simulation (MCS). The FOSM calibration technique is a probabilistic reliability method that produces a resistance factor (ϕ) for a given reliability index (β) which was originally proposed by Cornell (1969). This method, as a closed-form solution, is a simple method which has led to conservative results (Allen, 2005). FORM, developed by Hasofer and Lind (1974), is an improvement over FOSM and avoids its lack of invariance problem. Monte Carlo Simulation uses randomly generated numbers for the component variables to determine the probability distribution function (PDF) of the uncertain variable (i.e., the resistance factor in the current research). It was reported in previous studies that the GRF values calculated by MCS and FORM are similar and higher than those obtained by FOSM (Paikowsky et al., 2004; Allen, 2005; Abu-Farsakh et al., 2012; Jabo, 2014). In this study, the MCS reliability method was considered for the calibration of geotechnical resistance factors of pile foundations in Alberta. More details and applications about the MCS technique in civil engineering can be found in Phoon (2008, 2016), Mordechai (2011), Paikowsky et al. (2010), Shahnazari et al. (2016), and Pan et al. (2017).

For the design of pile foundations, the MCS technique is used to generate random values of load and resistance functions based on their statistical properties. As previously described, the limit state function (g) is formed with the random values and evaluated for failure as presented by Eq. (4-5):

$$g(R, Q) = \frac{\left(\gamma_D + \gamma_L \frac{Q_L}{Q_D}\right)}{\phi} \lambda_R - \left(\lambda_D + \lambda_L \frac{Q_L}{Q_D}\right) \quad (4.5)$$

where:

γ_D, γ_L = Dead and live load factors

λ_D, λ_L = Dead and live load bias factors

λ_R = Mean bias factor (i.e., R_m/R_p)

Q_D/Q_L = Dead to live load ratio

The load statistics and factors are shown in Table 4-4.

Table 4.4: Statistical Characteristics and Load Factors (CHBDC 2014)

Load Type	Bias (λ)	COV (COV)	Load Factor (γ)
Dead Load	$\lambda_D = 0.90$	$\text{COV}_D = 0.10$	$\gamma_D = 1.20$
Live Load	$\lambda_L = 1.05$	$\text{COV}_L = 0.27$	$\gamma_L = 1.70$

In addition to the values presented in **Table 4-4** the ratio of dead load to live load (Q_D/Q_L) should be defined. Paikowsky et al. (2004) presented the very low sensitivity of resistance factors to Q_D/Q_L . They used Q_D/Q_L values between 2.0 to 2.5 as a reasonable range, while Allen (2005) and Abu-Farsakh et al. (2013) used a conservative ratio of 3.0. Also, Paikowsky et al. (2004) and AbdelSalam et al. (2011) showed that a higher Q_D/Q_L resulted in somewhat smaller resistance factors.

In this study, a Q_D/Q_L equal to 3.0 is considered. **Figure 4-3** illustrates the effect of Q_D/Q_L on the reliability index based on the results of this study. Note: the values of mean and COV presented in **Figure 4-3** are described in the following sections.

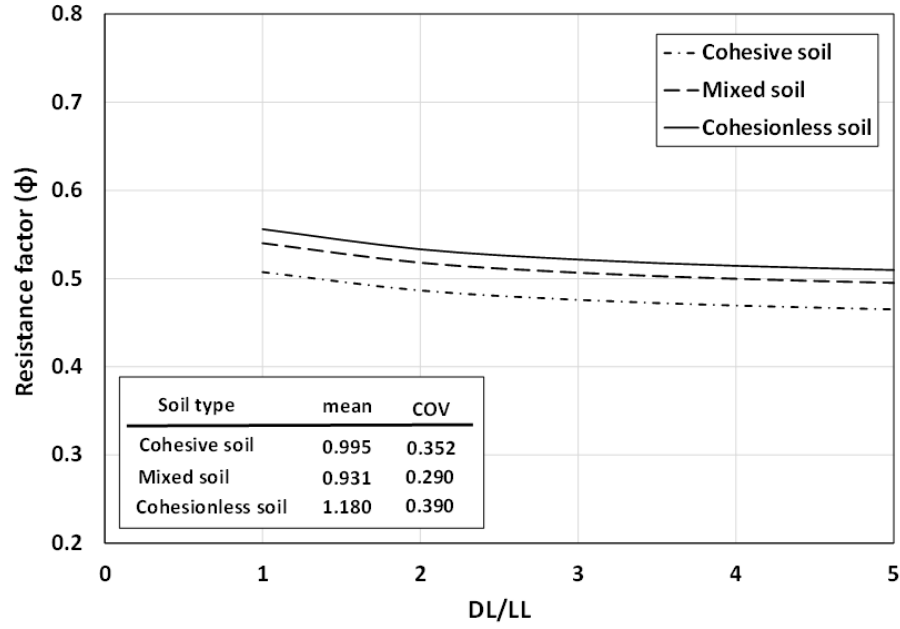


Figure 4.3: Change in Resistance Factors with Respect to $\frac{Q_D}{Q_L}$ Ratio

4.9 Evaluation of Resistance Factors in Alberta

4.9.1 SLT vs DLT Relationship

As the SLT has the most reliable result, the measured DLT capacity at EOD (hereafter DLT-EOD) was adjusted to represent the SLT capacity using average bias between SLT and DLT-EOD. The transformed sample bias $(\lambda_R)_T$ and coefficient of variation $(COV_R)_T$ between the static analysis method and SLT capacity of individual piles was estimated as follows:

$$(\lambda_R)_T = \lambda_R \cdot \bar{\lambda}_{\text{SLT-DLT}} \quad (4.6)$$

$$(COV_R)_T = \sqrt{COV_R^2 + COV_{\text{SLT-DLT}}^2} \quad (4.7)$$

where λ_R and COV_R are the mean and COV of the sample bias between the static analysis method formula and the DLT-determined capacity, and $\bar{\lambda}_{\text{SLT-DLT}}$ and $COV_{\text{SLT-DLT}}$

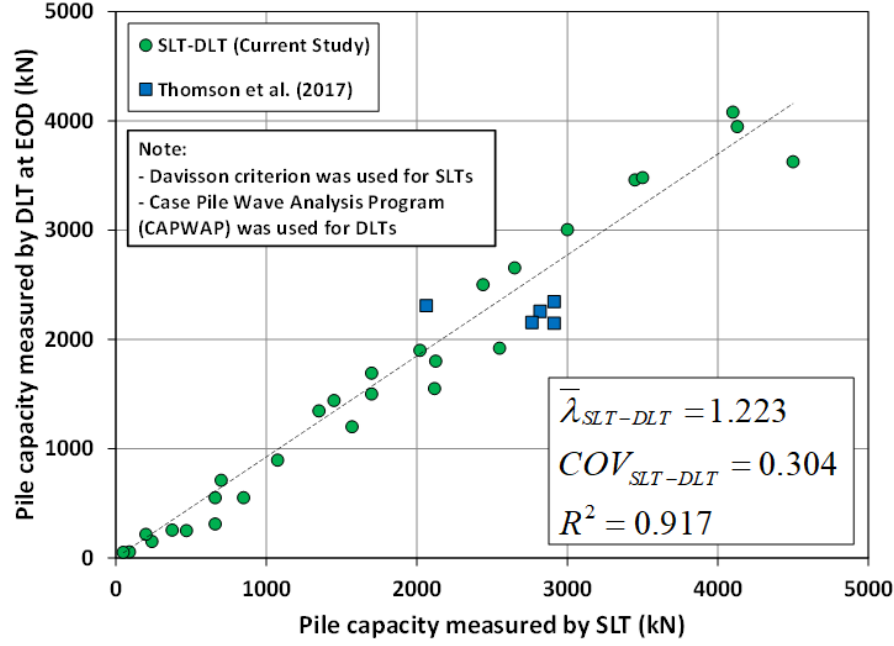


Figure 4.4: Correlation between DLT-EOD-predicted pile capacity and SLT Pile Capacity

are the average and COV of the bias between DLT and SLT capacities. In this study, 28 SLT results were gathered at different locations of Alberta. In addition, 5 SLTs carried out by Thomson et al. (2017) in the Interior Plains physiographic region of Canada have also been added. In all the gathered data, the Davisson offset limit criterion (Davisson 1972) for SLTs and CAPWAP method (Likins et al. 2000) at EOD condition for DLT analyses were used. It should be noted that each SLT and associated PDA were carried out on the same sites in order to have a more accurate correlation between DLT and SLT.

The comparison of SLT and DLT-EOD results is presented in **Fig. 4-4**. This figure indicates a very good correlation between the two in-situ test results with a relatively high regression coefficient (i.e., $R^2 = 0.917$). It can be inferred that by applying the bias factor $\bar{\lambda}_{SLT-DLT}$, the DLT-EOD results can be considered as the measured ultimate load of the driven piles. The $\bar{\lambda}_{SLT-DLT}$ bias factor measured for this study is 1.223. Similar compatibility reports between DLT and SLT have been reported by Fellenius and Altaee (2002), Reddy and Stuedlein (2013), Thomson et al. (2017), and Asghari Pari et al. (2019).

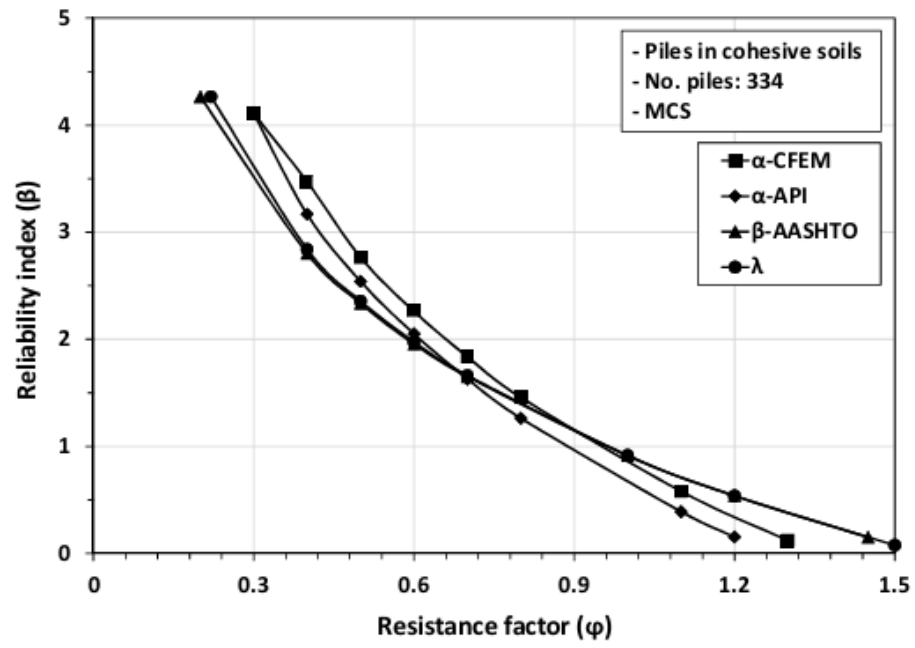
4.9.2 Calibration of GRF Based on Soil Type

Based on the “70% rule” described earlier, 334, 267, and 22 piles were in cohesive, cohesionless, and mixed soils, respectively. The predicted bearing capacity for each pile (R_p) was calculated using appropriate static analysis methods (such as α , β , λ methods) based on soil types. The soil properties such as undrained shear strength, SPT, and CPT results have been used to estimate parameters needed for these static analysis methods.

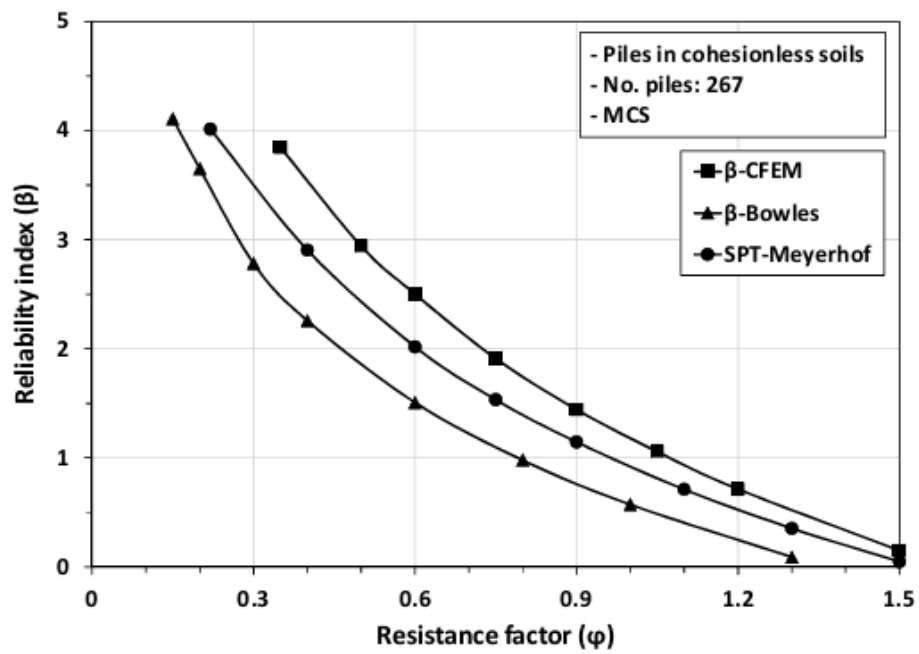
To obtain the statistical properties of the resistance bias factor used in calibration approaches, λ_R values were calculated for each pile. The Anderson and Darling (AD) (1954) test was used to examine the goodness of fit for the assumed distribution function of λ_R values. As can be seen in Table 4-5, the best distribution function for all static analysis methods was the log-normal distribution because the AD values of this type of distribution were lower than those of the normal distribution. The statistical properties, including transformed mean $(\lambda_R)_T$ and coefficient of variation $(COV_R)_T$ of the bias factor for different static analysis methods, are presented in Table 4-5.

The variation of the reliability indices and the geotechnical resistance factors was obtained based on MCS using statistical properties (Fig. 5). Generally, the increase in β (i.e., the decrease in the probability of failure) resulted in a non-linear decrease in the resistance factor.

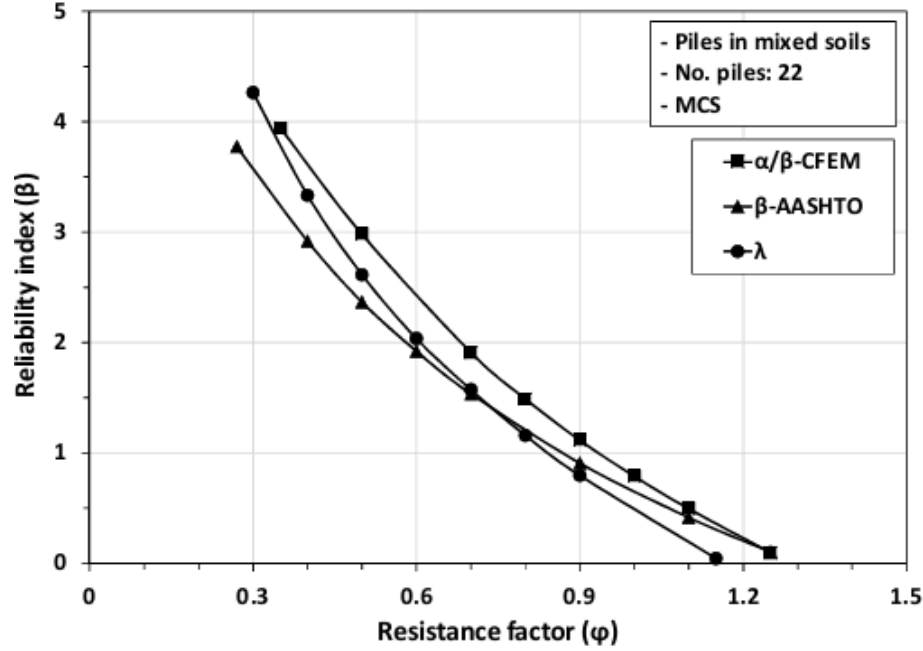
An appropriate β can be selected depending on the reliability of soil properties, degree of quality control, pile redundancy, construction condition, etc. (AbdelSalam et al. 2011). Based on the recommendation of Paikowsky et al. (2004), the β values for the evaluation of the bearing capacity of single piles are defined as $\beta = 2.33$ (i.e., $p_f = 1\%$) for redundant piles (defined as 5 or more piles per pile cap) and $\beta = 3.00$ (i.e., $p_f = 0.1\%$) for non-redundant piles (defined as 4 or fewer piles per pile cap). A summary of calculated GRFs for the static analysis methods used in this study based on three main soil types is presented in Table 4-5.



(a)



(b)



(c)

Figure 4.5: Relationship between the reliability index and resistance factor in (a) cohesive soil, (b) cohesionless soil and (c) mixed soil

Of the four static analysis methods used to predict the pile bearing capacity in cohesive soil, the α -CFEM method gave the highest ϕ of 0.585, followed by the α -API, λ and β -AASHTO methods with ϕ values of 0.540, 0.506, and 0.501, respectively for $\beta = 2.33$. The highest and lowest GRFs for piles in cohesionless soils belonged to the β -CFEM and β -Bowles with ϕ values of 0.642 and 0.383, respectively for $\beta = 2.33$. The combined CFEM method (α/β -CFEM) led to the highest GRF values of 0.618 for mixed soil, followed by the λ and β -AASHTO methods with ϕ values of 0.546 and 0.507, respectively for $\beta = 2.33$. Similar trends were observed for redundant piles (i.e., $\beta = 3.00$) using the presented calibration method. It should be noted that higher GRF values do not necessarily indicate an efficient design method. The efficiency factor as stated by McVay et al. (2002) is the ratio of the resistance factor to the mean bias factor of the corresponding design method (i.e., $\frac{\phi}{(\lambda_R)_T}$) and was calculated to effectively compare the efficiency of the static analysis methods used in this study. The higher $\frac{\phi}{(\lambda_R)_T}$ indicates a more economic and cost-effective design procedure. As can be seen in Table 5, the highest efficiency factors were obtained

by the static analysis method recommended by CFEM (2006) in all soil types.

Based on the static analysis method recommended by CFEM (2006), the highest GRF values were obtained in piles driven in cohesionless soils, followed by mixed and cohesive soils, respectively. On the other hand, the α/β -CFEM method which was used for mixed soils showed the highest efficiency in pile design. In addition, the difference between resistance factors for redundant (i.e., $\beta = 2.33$) and non-redundant (i.e., $\beta = 3.00$) piles is approximately 20, 23 and 19 percent for driven piles in cohesive, cohesionless and mixed soils, respectively. AASHTO (2012) recommended that a 20% reduction in the resistance factor for a redundant foundation can be used to obtain the resistance factor for a non-redundant pile foundation, which is consistent with the results of this study.

The result also showed that the mean value of the bias factor $(\lambda_R)_T$ for piles installed in cohesionless soils was greater than 1, which means the bearing capacity predicted by the static analysis was lower than that measured by the PDA tests. On the other hand, the values of $(\lambda_R)_T$ for piles in cohesive and mixed soils were lower than unity, except for the β method used in cohesive soil. In addition, the highest standard deviation belonged to piles in cohesionless soils. Therefore, it can be concluded that the prediction of pile bearing capacities in cohesionless soils by static analysis leads to unreliable underestimated values. Similar results were reported for the regional GRF values in Arkansas (Bostwick 2014; Jabo 2014), Louisiana, and Mississippi (Fortier 2016).

In **Fig. 4-6**, the GRF values for the reliability index of 2.33 and 3.00 as a function of cohesive fine (clay) content along the pile length, calibrated for Alberta, are illustrated. As the fine content along pile length increased, the GRF values decreased. When the cohesive soil content along a pile is near the borders of the “70% rule” classification method (i.e., 30% or 70%), the appropriate GRFs can be selected based on clay content along pile length obtained from the geotechnical report of the project.

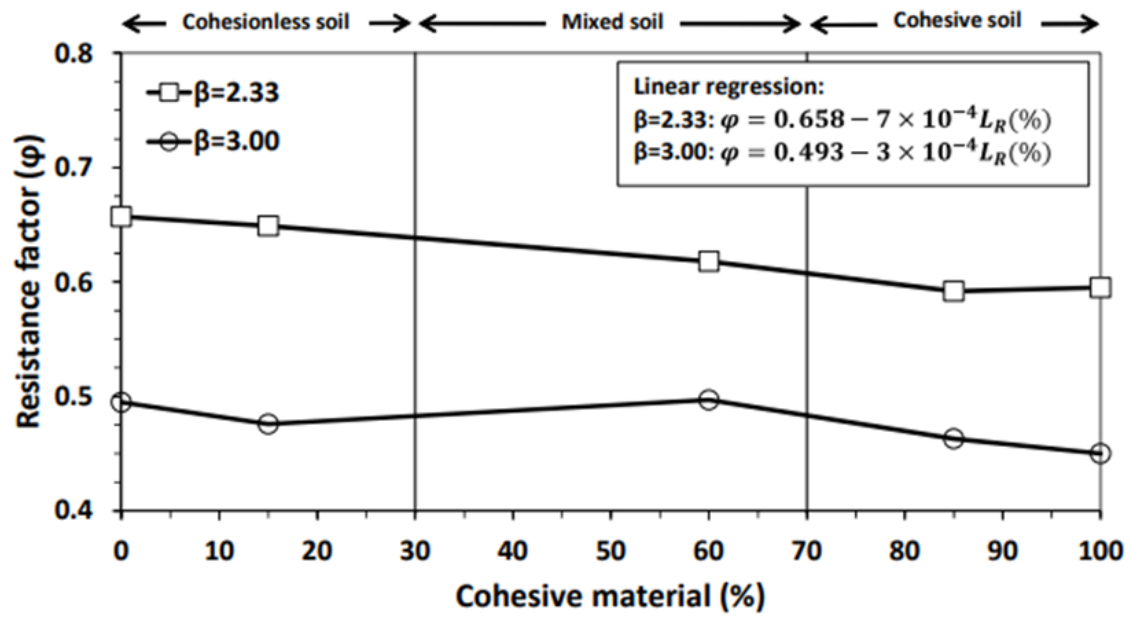


Figure 4.6: GRF values according to the cohesive fine content along the pile length for (a) $\beta = 2.33$ and (b) $\beta = 3.00$.

Table 4.5: Summary of statistical properties and GRF values for different static analysis and reliability methods based on soil type.

Soil type	Analysis method	No. of piles	Mean	COV	AD test for normality		Resistance factor		Efficiency factor	
					Nor.	Lognor.	$\beta = 2.33$	$\beta = 3.00$	$\beta = 2.33$	$\beta = 3.00$
Cohesive	α – CFEM	334	0.995	0.352	2.350	0.436	0.585	0.465	0.458	0.457
	α – API	334	0.933	0.359	3.326	0.402	0.540	0.435	0.417	0.405
	β – AASHTO	334	1.205	0.500	6.267	0.600	0.501	0.370	0.416	0.394
	λ	334	0.942	0.352	1.703	0.652	0.506	0.378	0.517	0.501
Cohesionless	β – CFEM	267	1.180	0.390	5.818	0.332	0.462	0.454	0.414	0.394
	β – Bowles	267	1.083	0.579	5.377	0.325	0.451	0.448	0.395	0.376
	SPT-Meyerhof	267	1.171	0.475	13.461	0.123	0.352	0.447	0.324	0.309
Mixed	α/β – CFEM	22	0.931	0.305	0.694	0.214	0.618	0.458	0.397	0.384
	β – AASHTO	22	0.970	0.404	0.762	0.113	0.507	0.387	0.523	0.399
	λ	22	0.843	0.309	0.667	0.159	0.546	0.443	0.648	0.425

4.10 Calibration of GRF Based on Geological Regions of Alberta

As stated earlier, the entire map of Alberta was divided into six regions based on regional soil properties and geological formations in the state. The MCS calibration method was implemented to calibrate the GRFs for each region. In Table 4-6, the statistical properties for different geological regions of Alberta are presented.

The table employs the α -CFEM, β -CFEM, and α/β -CFEM methodologies along with the 70% rule for soil classification to estimate soil capacity. These approaches are shown to provide more precise estimates, as evidenced in Table 4-6.

Table 4.6: Statistical properties and GRF values of different geological regions of Alberta.

Geological regions	No. of piles	Mean	STD	COV	Efficiency factor	
					$\beta = 2.3$	$\beta = 3.0$
All regions	623	1.069	0.420	0.393	0.520	0.385
NE	180	1.121	0.468	0.417	0.511	0.376
NW	157	1.016	0.370	0.364	0.574	0.435
C	107	1.083	0.408	0.377	0.546	0.406
SE	85	1.035	0.448	0.432	0.495	0.362
SW	94	1.156	0.547	0.473	0.453	0.324

4.11 Recommended GRF in Alberta

Deep foundations are generally used in groups in infrastructures and oil and gas projects, which have led to a high redundancy in piles. As a result, a reliability index of 2.33, corresponding to a probability of failure of 0.01, can be used for these structures. Based on the CFEM (2006), a GRF of 0.40 is recommended for the design of pile foundations using laboratory and in-situ test data. However, considering the findings presented in this study (refer to Table 4-7 – encompassing all regions), and upon undertaking calibration across Alberta's entirety as a unified entity, it is evident that enhancements in GRF values are attainable. Specifically, revised values of 0.58, 0.62, and 0.64 are proposed for piles driven into cohesive, mixed, and cohesionless soils, respectively. As can be seen, these values are about 45%, 55%, and 60% greater than those recommended in CFEM.

Table 4-7 also delineates the suggested GRF values for various regions within Alberta, categorizing them based on different soil conditions: cohesive, cohesionless, and mixed. It is important to highlight that in regions where data is insufficient for analyzing the regional GRF of specific soil types (cohesive, cohesionless, mixed), it has been presumed that the trends and ratios of GRF values observed across Alberta for these soil types apply. From the analysis, it is discernible that region-specific calibration of GRF values yields higher figures compared to the calibration for Alberta as a whole, thereby offering greater precision. Specifically, the calibrated GRF values in the Northwest (NW) and Northeast (NE) surpass the aggregate calibrated GRF value for all regions by approximately 3-4%, whereas the calibrated values for the Southeast (SE) and Southwest (SW) are marginally lower, by about 4-5%. This discrepancy, particularly the reduced GRF values for the SE and SW regions, is attributed to the scant data available for these areas.

Table 4.7: Comparison between recommended GRF values by different design specifications and the results of this study.

Reliability index (β)	Regions	All soil types	Cohesive soil	Cohesionless soil	Mixed soil
2.33	All regions	0.55	0.58	0.64	0.62
	NE	0.57	0.60	0.66	0.64
	NW	0.58	0.61	0.67	0.65
	C	0.54	0.57	0.63	0.61
	SE	0.51	0.54	0.59	0.57
	SW	0.52	0.55	0.61	0.59
3.00	All regions	0.41	0.46	0.49	0.49
	NE	0.42	0.47	0.50	0.50
	NW	0.44	0.49	0.53	0.53
	C	0.41	0.46	0.49	0.49
	SE	0.37	0.42	0.44	0.44
	SW	0.37	0.42	0.44	0.44

4.12 Conclusions

Predicting the bearing capacity of pile foundations is a major concern in geotechnical engineering. There are several methods for predicting bearing capacity, including static analysis methods (using soil properties), static and dynamic load tests. The reliability of

these methods varies depending on their uncertainties. Different geotechnical resistance factors used in the LRFD method are recommended in codes and guidelines based on the prediction method used. An extensive database of in-situ pile load tests, including 28 SLTs and 623 PDA tests from various infrastructures and oil and gas projects in Alberta, was collected to regionally calibrate the GRF value for different static analysis methods using the MCS calibration approach. The following conclusions can be drawn from this study:

- The design methods recommended by CFEM (2006) resulted in the highest resistance and efficiency factors among the static analysis methods used in this study. Therefore, their application for practical projects is validated.
- The $\frac{R_m}{R_p}$ value of the derived pile in cohesionless soils is greater than 1. However, the ratio in cohesive and mixed soils is less than 1, indicating that the static analysis method is more conservative in predicting pile bearing capacity in cohesionless soils.
- The recommended GRFs for Alberta as a whole, based on the 70% rule classification, are:
 - For $\beta = 2.33$: 0.58, 0.62, and 0.64 for cohesive, mixed, and cohesionless soils, respectively.
 - For $\beta = 3.00$: 0.46, 0.49, and 0.49 for cohesive, mixed, and cohesionless soils, respectively.
- As the fine content along the pile length increases, the GRF values decrease. Two relationships between GRF and LR were developed for practical applications when the cohesive soil content along a pile is near the borders of the “70% rule” classification method.
- According to the geological regions of Alberta, higher GRF values can be used in northern regions. The recommended GRFs for a reliability index of $\beta = 2.33$ are:
 - 0.57, 0.58, 0.54, 0.51, and 0.52 for the northeast, northwest, central, southeast, and southwest regions of Alberta, respectively.
 - For $\beta = 3.00$: 0.42, 0.44, 0.41, 0.37, and 0.37 for the NE, NW, C, SE, and SW regions of Alberta, respectively.
- The findings suggest that more localized calibration of GRF values can lead to further optimization, resulting in more efficient design outcomes. This efficiency manifests in the reduction of pile length/size, significantly lowering installation costs.

4.13 References

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Chapter 5

Calibration of Resistance Factors of Driven Steel Piles Considering Separate Factors for Toe and Shaft Resistances

5.1 Background

The contents presented in this chapter are summarized from the manuscript below: Roshani et. al. (2024). Calibration of resistance factors for reliability-based design of driven steel piles considering separate factors for toe and shaft resistances Manuscript published in an International Geotechnical Journal.

5.2 Introduction

Driven steel piles can be installed with relative ease and efficiency, making them an attractive option as a type of deep foundation in Alberta oil and gas projects (Skirrow and Wang, 2008). There are many uncertainties (in both load and capacity) in the design process of deep foundations which can be attributed to the inherent soil variability, measurement noise/random error, systematic measurement errors, and transformation uncertainty (Pishgah Gilani and Jamshidi Chenari, 2013; Jamshidi Chenari and Kamyab Farahbakhsh, 2015; Jamshidi Chenari and Alaie, 2015; Jamshidi Chenari et al., 2018; Heidarie Golafzani

et al., 2019). The Load and Resistance Factor Design (LRFD) method can be employed to separate uncertainties in loading from uncertainties in resistance. Probability theories are used to ensure a prescribed margin of safety in this method (Whitman, 2000; Paikowsky et al., 2004; Wang, 2009; Roshani et al., 2020). In LRFD, partial safety factors are applied to the load and resistance separately. In this regard, the strength (e.g., the bearing capacity of a pile) is reduced and the load is increased by multiplying the corresponding nominal values by resistance and load factors, respectively (Paikowsky et al., 2004; Heidarie Golafzani et al., 2020). In the design process of geotechnical structures such as pile foundations, the resistance factor is also called the geotechnical resistance factor (GRF). Based on the project condition at the time of design, three levels of understanding, including high level, typical level, and low level, can be considered based on knowledge of foundation conditions, experience, typical structure conditions, quality, and quantity of materials testing, etc. (Fenton et al., 2016; Bathurst and Jamshidi Chenari, 2023). If project knowledge is greater than typical understanding of project conditions (lower amount of uncertainties), the higher value of the GRF can be considered.

Predicting the bearing capacity of piles is one of the most difficult and interesting questions for engineers and researchers. (Haque et al., 2017; Moshfeghi and Eslami, 2018; Song et al., 2019). The ultimate bearing capacity of driven steel piles (R_{ult}) can be divided into two components, including toe (or base) resistance ($R_{ult,b}$) and shaft resistance ($R_{ult,s}$). Typically, similar GRF is used for both $R_{ult,b}$ and $R_{ult,s}$, and all uncertainties related to these resistances were combined into a single GRF. However, given the varying degrees of uncertainty in predicting $R_{ult,b}$ and $R_{ult,s}$, it is more accurate to use separate GRF values for the components of the pile bearing capacity (Kim et al., 2011).

Studies that evaluate the separate GRFs for the $R_{ult,b}$ and $R_{ult,s}$ of pile foundations are limited and most research has been performed to calculate a single GRF for R_{ult} of driven piles (Paikowsky et al., 2004; Roberts et al., 2008; AbdelSalam et al., 2011; Jabo, 2017; Asghari Pari et al., 2019). Bach et al. (2013) assessed the $R_{ult,b}$ and $R_{ult,s}$ GRFs for drilled piles based on 26 pile load tests in Los Angeles. They found that using separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ can result in a more cost-effective design. Bian et al. (2018) also noted that various parameters impact the GRFs for $R_{ult,b}$ and $R_{ult,s}$, with varying degrees of influence. Moreover, using similar GRFs for $R_{ult,b}$ and $R_{ult,s}$ tends to yield conservative results. Asghari Pari and Fakharian (2023) conducted a study on driven piles in cohesive soils in Iran, focusing on determining GRFs for the $R_{ult,b}$ and $R_{ult,s}$. Their findings revealed that employing separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ can either increase or decrease the total GRF, depending on the specific conditions and characteristics of the pile foundations being analyzed. Based on the results reported by Abu-Farsakh et al. (2012, 2013) a lower GRF should be used for $R_{ult,s}$ compared to $R_{ult,b}$ of piles drilled in mixed soils based on pile load

tests in Louisiana. However, opposite results were obtained for drilled piles in weathered rock in Korea (Kim et al., 2019).

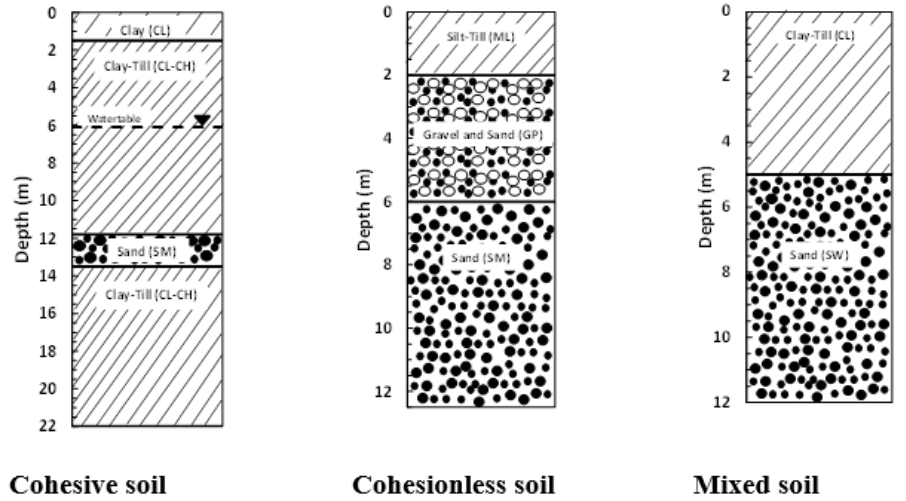
Previous studies have primarily focused on calibrating separate GRFs for the $R_{ult,b}$ and $R_{ult,s}$ in drilled piles, with limited attention given to driven piles. To address this gap, a comprehensive study is needed to determine separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ of driven steel piles in Alberta. This study will utilize semi-empirical methods recommended by CFEM (2006) and incorporate field pile tests like Pile Driving Analyzer (PDA) and static load tests (STLs). It also considers in-situ soil properties for different soil types, including fine-grained, coarse-grained, and mixed soils. Two different criteria have been used to classify the soils (70% rule and R Ratio). The objective is to propose distinct GRFs for $R_{ult,b}$ and $R_{ult,s}$ of driven steel piles, ultimately improving the design and construction practices for such piles in Alberta. This calibration is conducted for various soil types using probabilistic methods such as First Order Second Moment (FOSM), First Order Reliability Method (FORM), and Monte Carlo Simulation (MCS), with the goal of enhancing the cost-effectiveness and reliability of pile foundation projects.

5.3 Development of Database

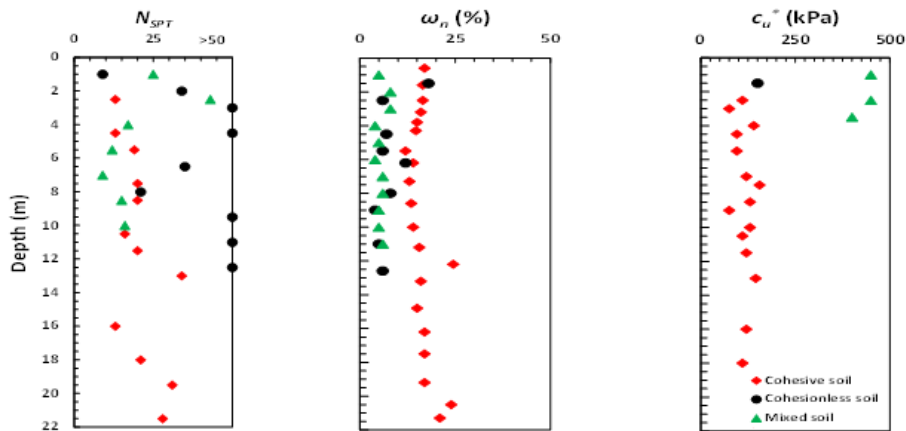
There are two well-known in-situ tests used to measure the bearing capacity of pile foundations: the PDA as a dynamic load test (DLT) and the static load test (SLT). Although the SLT is the most accurate method for estimating R_{ult} , its high cost and time-consuming nature limit its use in projects (Jabo, 2014; Asghari Pari et al., 2019). Additionally, the SLT is destructive, making it impractical for production piles that are not constructed for testing. As a result, the PDA test is gaining popularity as an alternative for measuring R_{ult} , especially in North America. For evaluating the GRF values for $R_{ult,b}$ and $R_{ult,s}$ separately, the field test results of six hundred twenty-three (623) driven steel pipe piles were assessed. These piles were installed in different regions of Alberta, each featuring cohesive, cohesionless, and mixed soil compositions, with diameters ranging from 114 to 915 mm, and embedment lengths from 4 to 40 m.

To gather relevant data, extensive geotechnical investigations were conducted in different regions. These investigations included borehole investigation, conducting in-situ tests like standard penetration test and pocket penetrometer test, and performing laboratory tests such as moisture content, particle size distribution, unconfined compression tests, and direct shear tests. **Figure 5-1** shows a 22 m deep borehole drilled in cohesive soil, primarily clayey, with a water table at approximately 6 m below the ground surface. **Figure 5-1** display borehole logs in cohesionless and mixed (cohesive/cohesionless soil) soils, respec-

tively. These boreholes reached a depth of about 12 m, and no water table was observed during drilling in cohesionless and mixed soils.



(a)



**Based on pocket penetrometer test*

(b)

Figure 5.1: A summary of laboratory and in-situ test results in a borehole in (a) soil profile and (b) soil properties (NSPT: standard penetration value, ω_n : natural moisture content and c_u : un-drained shear strength)

5.4 LRFD Design and Calibration Approach

As previously described, in the LRFD approach the resistances and loads are multiplied by resistance and load factors, as shown below:

$$R_r = \phi R_n \geq \gamma_D Q_D + \gamma_L Q_L \quad (5.1)$$

where ϕ is the resistance or strength reduction factor (GRF in pile design process), R_r and R_n are the factored resistance and ultimate (or nominal) resistance, respectively, γ_D and γ_L are the factors of dead load (Q_D) and live load (Q_L), respectively. The GRFs are estimated based on the variability of the geotechnical parameters and the analysis of uncertainties during the calculation of R_n (CFEM, 2006).

The basic theory of the LRFD method is defined on the basis of the use of probability density functions (PDF) representing the load (Q) and resistance (R) and their relation to the limit state function (g) as below (Dithinde, 2007):

$$g = R - Q \quad (5.2)$$

The main aim of reliability-based design is to ensure that the probability of failure does not exceed an acceptable threshold level. The probability of failure is defined as follows:

$$p_f = \text{Prob}(g < 0) \leq p_T \quad (5.3)$$

where $\text{Prob}(\cdot)$ is the probability of an event, p_f is the probability of failure, and p_T is the acceptable target probability of failure. Due to some difficulties in calculating p_f , another parameter called the reliability index (β) is defined as follows:

$$\beta = -\phi^{-1}(p_f) \quad (5.4)$$

where $\phi^{-1}(\cdot)$ is the inverse of the standard normal cumulative function, ϕ .

5.4.1 Defining Separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ (ϕ_b and ϕ_s)

There are several probabilistic approaches used in reliability-based design and in the calibration of GRFs for pile foundations, including FOSM, FORM, and MCS (Jabo, 2014). FOSM, initially proposed by Cornell (1969), and FORM are the most commonly used methods (Kim, 2003). Linearizing the performance function at the mean values of the random variables in FOSM introduces error for non-linear limit state functions (Heidarie Golafzani et al., 2019). Paikowsky et al. (2004) conducted extensive analyses using FOSM

and FORM with a vast database of deep foundations, finding that the difference in resulting resistance factors is less than 10%, and FOSM tends to yield slightly lower (or more conservative) resistance factors. The MCS, on the other hand, is a stochastic problem-solving technique that approximates the probability of certain outcomes by running multiple trial runs (simulations) using random variables. More details and applications of the MCS technique in civil engineering can be found in Phoon (2008, 2016), Paikowsky et al. (2010), Mordechai (2011), and Shahnazari et al. (2016).

Prior studies have primarily focused on adjusting a single GRF for R_{ult} to achieve a specific reliability index. There is a lack of well-developed techniques for determining separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ (Asghari Pari and Fakharian, 2023) using various reliability analyses.

Calibration of Separate GRFs Based on FOSM

The basic equation of LRFD-based design and the limit state function can be rewritten as follows:

$$(\phi_b R_{ult,b} + \phi_s R_{ult,s}) - (\gamma_D Q_D + \gamma_L Q_L) \geq 0 \quad (5.5)$$

where ϕ_b and ϕ_s are the toe and shaft GRFs, respectively, and $R_{ult,b}$ and $R_{ult,s}$ are ultimate (or nominal) toe and shaft resistances, respectively.

At the critical state condition, $R_{ult,b}$ and $R_{ult,s}$ can be derived as follows:

$$R_{ult,b} = \frac{\gamma_D Q_D + \gamma_L Q_L - \phi_s R_{ult,s}}{\phi_b} \quad (5.6)$$

$$R_{ult,s} = \frac{\gamma_D Q_D + \gamma_L Q_L - \phi_b R_{ult,b}}{\phi_s} \quad (5.7)$$

Based on Bian et al. (2016), the reliability index (β) using the FOSM method can be calculated as:

$$\beta = \frac{\ln \left[\frac{(\lambda_{Rb} R_{ult,b} + \lambda_{Rs} R_{ult,s})}{(\lambda_{QD} Q_D + \lambda_{QL} Q_L)} \sqrt{\frac{1 + COV_{QD}^2 + COV_{QL}^2}{1 + COV_{R_{ult,b}}^2 + COV_{R_{ult,s}}^2}} \right]}{\sqrt{\ln \left[(1 + COV_{R_{ult,b}}^2 + COV_{R_{ult,s}}^2)(1 + COV_{QD}^2 + COV_{QL}^2) \right]}} \quad (5.8)$$

where λ_{Rb} and λ_{Rs} are the bias factors for $R_{ult,b}$ and $R_{ult,s}$, respectively; $COV_{R_{ult,b}}$ and $COV_{R_{ult,s}}$ are the coefficients of variation for λ_{Rb} and λ_{Rs} , respectively; λ_{QD} and λ_{QL} are the bias factors for the dead and live loads, respectively; and COV_{QD} and COV_{QL} are the

coefficients of variation for λ_{QD} and λ_{QL} , respectively. Substituting Eqs. 5-6 and 5-7 into Eq. 5-8 and replacing β with the target reliability index (β_T), gives the following equations:

$$\phi_b = \frac{\lambda_{Rb} \left[\frac{\gamma_D \rho + \gamma_L}{\rho + 1} - \phi_s S \right]}{\frac{\lambda_{QD} \rho + \lambda_{QL}}{\rho + 1} \cdot \exp \left(\beta_T \sqrt{\ln \left[(1 + COV_{R_{ult,b}}^2 + COV_{R_{ult,s}}^2)(1 + COV_{QD}^2 + COV_{QL}^2) \right]} \right)} \cdot \sqrt{\frac{1 + COV_{QD}^2 + COV_{QL}^2}{1 + COV_{R_{ult,b}}^2 + COV_{R_{ult,s}}^2}} - \lambda_{Rs} S \quad (5.9)$$

$$\phi_s = \frac{\lambda_{Rs} \left[\frac{\gamma_D \rho + \gamma_L}{\rho + 1} - \phi_b B \right]}{\frac{\lambda_{QD} \rho + \lambda_{QL}}{\rho + 1} \cdot \exp \left(\beta_T \sqrt{\ln \left[(1 + COV_{R_{ult,b}}^2 + COV_{R_{ult,s}}^2)(1 + COV_{QD}^2 + COV_{QL}^2) \right]} \right)} \cdot \sqrt{\frac{1 + COV_{QD}^2 + COV_{QL}^2}{1 + COV_{R_{ult,b}}^2 + COV_{R_{ult,s}}^2}} - \lambda_{Rb} B \quad (5.10)$$

where $\rho = Q_D/Q_L$, $B = R_{ult,b}/(Q_D + Q_L)$ and $S = R_{ult,s}/(Q_D + Q_L)$. Based on the LRFD criterion, the reliability index of a pile in a design procedure should be greater than or equal to the target reliability index.

Eqs. 5-9 and 5-10 illustrate that resistance factors of toe and shaft are mainly dependent on ρ , β_T , B , S , and other parameters. The calculation procedure based on Eqs. 9 and 10 is that, first, ϕ_b computed using Eq. 9 will be submitted into Eq. 10 to compute ϕ_s , and the calculated ϕ_s will be resubmitted into Eq. 9 to compute ϕ_b , again. This iteration process should be continued up to gaining an acceptable calculation accuracy. CFEM (2006) recommended that the load statistics and factors can be obtained by the applicable codes such as AASHTO. The load statistical properties were adopted in this study based on AASHTO (2012) as $\lambda_{QD} = 1.05$, $\lambda_{QL} = 1.0$, $COV_{QD} = 0.1$, $COV_{QL} = 0.2$, $\gamma_D = 1.25$, and $\gamma_L = 1.75$. In addition, the ratio of dead load to live load was defined as 3. Previous studies have shown that the GRFs are not sensitive to the Q_D/Q_L ratio, especially for ratios greater than 2.5 (Paikowsky et al., 2004; Allen, 2005; Abu-Farsakh et al., 2013; Heidarie Golafzani et al., 2019).

5.4.2 Calibration of Separate GRFs Based on FORM

For calculation of toe and shaft GRFs based on FORM, the framework presented by Kim et al. (2011) is considered in this study. The following steps are performed for calculation of separate toe and shaft resistance factors:

1. Select a reasonable equation representing the Ultimate Limit State (ULS) of driven piles;
2. Select proper load factors for effective load types (e.g., γ_D and γ_L for dead and live loads, respectively) from the design specifications;
3. Calculate the uncertainties of $R_{ult,b}$ and $R_{ult,s}$ using a high-quality database including the results of pile load tests and site investigations;
4. Determine an appropriate β_T considering the importance of the structure;
5. Perform reliability analysis based on the uncertainties of the loads and resistances;
6. Calculate the separate GRFs of the toe and shaft resistances.

When a driven pile is axially loaded to the ULS, many possible combinations of $R_{ult,b}$, $R_{ult,s}$, Q_D , and Q_L can satisfy the ULS equation. The surface consisting of these combinations is defined as the failure surface (Kim et al., 2011). In the procedure of calculating the separate GRFs, Foye (2005) introduced $R_{b,LS}$, $R_{s,LS}$, $Q_{D,LS}$ and $Q_{L,LS}$ as the most probable ULS (or the most probable failure) values of $R_{ult,b}$, $R_{ult,s}$, Q_D and Q_L , respectively. As stated by Kim et al. (2011), the combination of $R_{b,LS}$, $R_{s,LS}$, $Q_{D,LS}$ and $Q_{L,LS}$ is unique and can be obtained by reliability analyses using FORM. The optimum toe and shaft GRFs and optimum dead and live load factors are defined as the ratios of the most probable ULS to the nominal values as follows:

$$\phi_b^* = \frac{R_{b,LS}}{R_{ult,b}} \quad (5.11)$$

$$\phi_s^* = \frac{R_{s,LS}}{R_{ult,s}} \quad (5.12)$$

$$\gamma_D^* = \frac{Q_{D,LS}}{Q_D} \quad (5.13)$$

$$\gamma_L^* = \frac{Q_{L,LS}}{Q_L} \quad (5.14)$$

Then, the toe and shaft GRFs can be obtained using the following equations proposed by Foye (2005):

$$\phi_b = \min \left(\frac{\gamma_D}{\gamma_D^*}, \frac{\gamma_L}{\gamma_L^*} \right) \phi_b^* \quad (5.15)$$

$$\phi_s = \min \left(\frac{\gamma_D}{\gamma_D^*}, \frac{\gamma_L}{\gamma_L^*} \right) \phi_s^* \quad (5.16)$$

More details about the methodology can be found in Foye (2005) and Kim et al. (2011).

5.5 Calibration of Separate GRFs Based on MCS

By combining Eqs. 6 and 7, the following equation can be used for calculating the separate GRFs for toe and shaft resistances:

$$g = \left[\frac{\gamma_D Q_D / Q_L + \gamma_L}{\phi_s + \phi_b R_{ult,b} / R_{ult,s}} (\lambda_{Rs} + \lambda_{Rb} R_{ult,b} / R_{ult,s}) - \lambda_D Q_D / Q_L - \lambda_L \right] Q_L \quad (5.17)$$

Figure 5-2 illustrates the relation between the toe and shaft resistance factors (i.e., ϕ_b and ϕ_s), for two target reliability indices of 2.33 and 3.0 of piles driven in cohesive soils. It can be seen that there are various compositions of ϕ_b and ϕ_s values that satisfy specified target reliability indices. The toe and shaft resistance factors have an inverse relation; that is, ϕ_s increases by decrement of ϕ_b , and vice versa.

Bach et al. (2013) and Bach (2014) proposed a ratio called “Acceptance Correlation Ratio” (ACR) to accept a pair of ϕ_b and ϕ_s . Bach (2014) stated that the resistance factor and the ratio of the coefficient of variation to the mean of the resistance bias factor, COV_R / λ_R , have a better correlation than the one between the resistance factor and other statistical parameters such as λ_R , σ_R , or COV_R . The good correlation between the resistance factor and the ratio COV_R / λ_R is an important basis for calibrating separate shaft and base resistance factors later. The ACR is defined as follows:

$$\frac{\phi_s}{\phi_b} \approx \text{ACR} = \frac{COV_{RB} / \lambda_{RB}}{COV_{RS} / \lambda_{RS}} = \frac{COV_{RB} \lambda_{RS}}{COV_{RS} \lambda_{RB}} \quad (5-18)$$

As an instance, the accepted pair of ϕ_b and ϕ_s for cohesive soils of the current study is demonstrated in **Figure 5-2** using dashed lines.

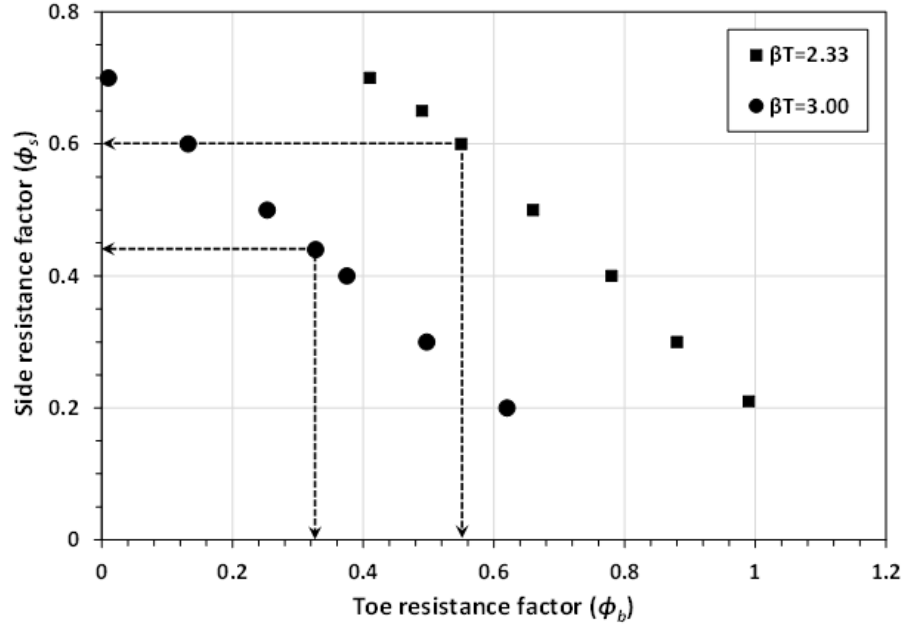


Figure 5.2: Relation between ϕ_b and ϕ_s for cohesive soils and the accepted pairs based on ACR

5.6 Measured and Predicted Resistances of Pile Foundations

The results of PDA tests using Case Pile Wave Analysis Program (CAPWAP) proposed by Likins et al. (2000), a well-known in-situ testing method, were considered as the measured pile bearing capacities in this study. The PDA tests were carried out on all steel-driven piles in End-of-Driving (EOD) condition using ASTM D1143 and D4945 to measure the bearing capacity of pile foundations, and CAPWAP has been used to analyze data. As a high-quality database is crucial in calibrating the resistance factor based on the reliability method, twenty-eight (28) SLTs were conducted to validate the construction and analysis procedure of the PDA tests. Figure 5-3 shows the comparison between pile bearing capacity measured by SLTs using the Davisson offset limit criterion (Davisson, 1972) and PDA tests using the CAPWAP method (Likins et al. 2000) at EOD condition. As the SLT has the most reliable result, the measured DLT capacity at EOD was adjusted to represent the SLT capacity using the average bias between SLT and DLT. The transformed sample bias,

$(\lambda_R)_T$, and coefficient of variation, $((\text{COV}_R)_T)$, between the static analysis method and SLT capacity of individual piles can be estimated as follows:

$$(\lambda_R)_T = \lambda_R \cdot \lambda_{\text{SLT-DLT}} \quad (5.18)$$

$$((\text{COV}_R)_T) = \sqrt{((\text{COV}_R)^2 + (\text{COV}_{\text{SLT-DLT}})^2)} \quad (5.19)$$

where λ_R and (COV_R) are the mean and COV of the sample bias between the static analysis method formula and the DLT-determined capacity and $\lambda_{\text{SLT-DLT}}$ and $(\text{COV}_{\text{SLT-DLT}})$ are the average and COV of the bias between DLT and SLT capacities. Based on the results, a very good correlation can be observed between the two in-situ test results with a relatively high coefficient of determination (i.e., $R^2 = 0.917$). It can be inferred that by applying the bias factor $\lambda_{\text{SLT-DLT}}$, the PDA results can be considered as the measured ultimate capacity of the steel driven piles. The measured $\lambda_{\text{SLT-DLT}}$ for this study is 1.223. Similar compatibility between PDA and SLT has been reported by Fellenius and Altaee (2002), Beim and Luna (2012), Reddy and Stuedlein (2013), Thomson et al. (2017) and Asghari Pari et al. (2019).

Several predictive methods with various assumptions and model uncertainties have been developed to estimate the ultimate pile bearing capacity. As the main objective of this study is the regional calibration of the geotechnical resistance factor in Alberta and considering available soil properties, three (3) static analysis methods recommended in CFEM (2006) were selected. For the steel piles driven into cohesive soil, the α -method, which is based on total stress analysis, was used. In the case of drained condition (i.e., for non-cohesive soils), an effective stress analysis method, called the β -method, proposed by Burland (1973) was used to predict the bearing capacity of piles in cohesionless soils. A combined method, called the α/β -method, was chosen for piles in mixed soils. In the combined method suggested by AbdelSalam et al. (2011) an analysis method to calculate the bearing capacity of a pile in cohesive soil (e.g., α -method) and an analysis method to calculate the bearing capacity of a pile in cohesionless soil (e.g., β -method) are used. A summary of the static analysis methods and their relationships used in this study is presented in Table 5-1. The following classification criteria have been used to categorize soils:

70% Rule: The "70% rule" is used for classifying piles based on the percentage of cohesive soil along their length. According to this rule, if 70% or more of the pile's length is driven into cohesive soils (i.e., $L_c/L_{\text{tot}} \geq 70\%$), static analysis methods developed for cohesive soils (e.g., α -method) are chosen. Conversely, when the cohesive soil along the pile length is equal to or less than 30% (i.e., $L_c/L_{\text{tot}} \leq 30\%$), the soil is classified as cohesionless,

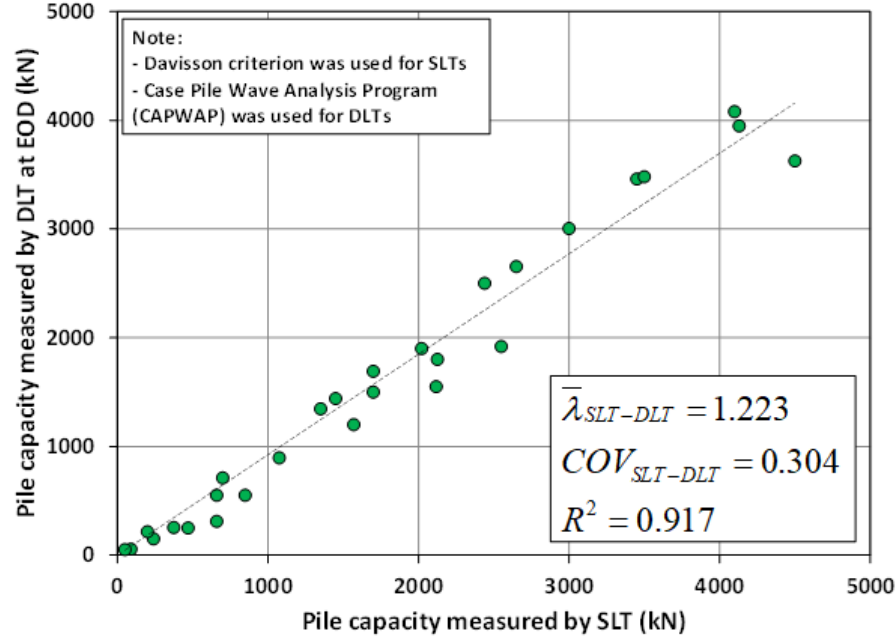


Figure 5.3: Correlation between pile capacity measured by DLT tests and SLT

and appropriate static analysis methods such as the β -method can be used. In the third scenario, when the cohesive soil along the pile length falls between 30% and 70%, the soil is considered mixed, and a combined method (e.g., α/β -method) can be applied. Based on this rule, 334 piles were driven in cohesive soil, 267 piles in cohesionless soil, and 22 piles driven in mixed soil.

R Ratio: Although the "70% rule" is a simple method for pile classification based on the soil layers along the embedded pile length, it can be misleading under certain conditions. For instance, if a pile rests on very dense sand, very stiff clay, or bedrock, the toe resistance accounted for a larger portion of the total resistance, regardless of the amount of soil type along the pile length. Another pile classification criterion called "R ratio" is introduced in this study based on the ratio of shaft resistance to total resistance (i.e., $R_{ult,s}/R_{ult}$). On this basis, if $R_{ult,s}/R_{ult} \leq 30\%$, the pile is classified as "toe-bearing pile", if $R_{ult,s}/R_{ult} \geq 70\%$, the pile is classified as "friction pile" and in other conditions, the pile is "toe-friction". Based on this classification method, 128, 282, and 213 piles were classified as "toe-bearing", "toe-friction" and "friction pile", respectively. It can be inferred that using the "R ratio" method has resulted in better classification of piles so that all categories have a sufficient number of piles for a reasonable reliability analysis.

The log-normal distribution was selected as the most suitable distribution function for λ_R values in this study. To verify this assumption, the database was tested for goodness of fit to the specified distribution by the method proposed by Anderson and Darling (1954) called the Anderson-Darling (AD) method. The results showed that the log-normal distribution was the best distribution function for the database used in this study. The log-normal distribution has been recommended for the resistance bias factor by many previous studies (e.g., Briaud and Tucker, 1988; AbdelSalam et al., 2011; Heidarie Golafzani et al., 2019).

Based on the recommendation of Paikowsky et al. (2004), the β values for the evaluation of the bearing capacity of single piles are defined as $\beta = 2.33$ (i.e., $p_f = 1\%$) for redundant piles (defined as 5 or more piles per pile cap) and $\beta = 3.00$ (i.e., $p_f = 0.1\%$) for nonredundant piles (defined as 4 or fewer piles per pile cap). Table 5-1 and Figure 5-4 indicate the summary of static analysis methods used in this study and the comparison of measured and predicted pile capacities using two classification criteria (i.e., 70% rule and R ratio), respectively.

Table 5.1: Summary of static analysis methods for steel open-ended driven piles used in this study

Soil Description	Method	Shaft Resistance (q_s)	Toe Resistance (q_b)	Reference
Cohesive	α	αc_u	$N_c c_u$	Tomlinson (1957); CFEM (2006)
Cohesionless	β	$\beta \sigma'_v$	$N_t \sigma'_t$	Burland (1973); CFEM (2006)
Mixed	α/β	Cohesive part: αc_u	Cohesive part: $N_c c_u$	—
		Cohesionless part: $\beta \sigma'_v$	Cohesionless part: $N_t \sigma'_t$	

Note: α : adhesion factor, D : pile embedment depth, B : pile diameter, N_c : bearing capacity factor, β : shaft resistance factor, σ'_v : vertical stress at the pile toe, c_u : undrained shear strength, σ'_t : vertical effective stress at depth z .

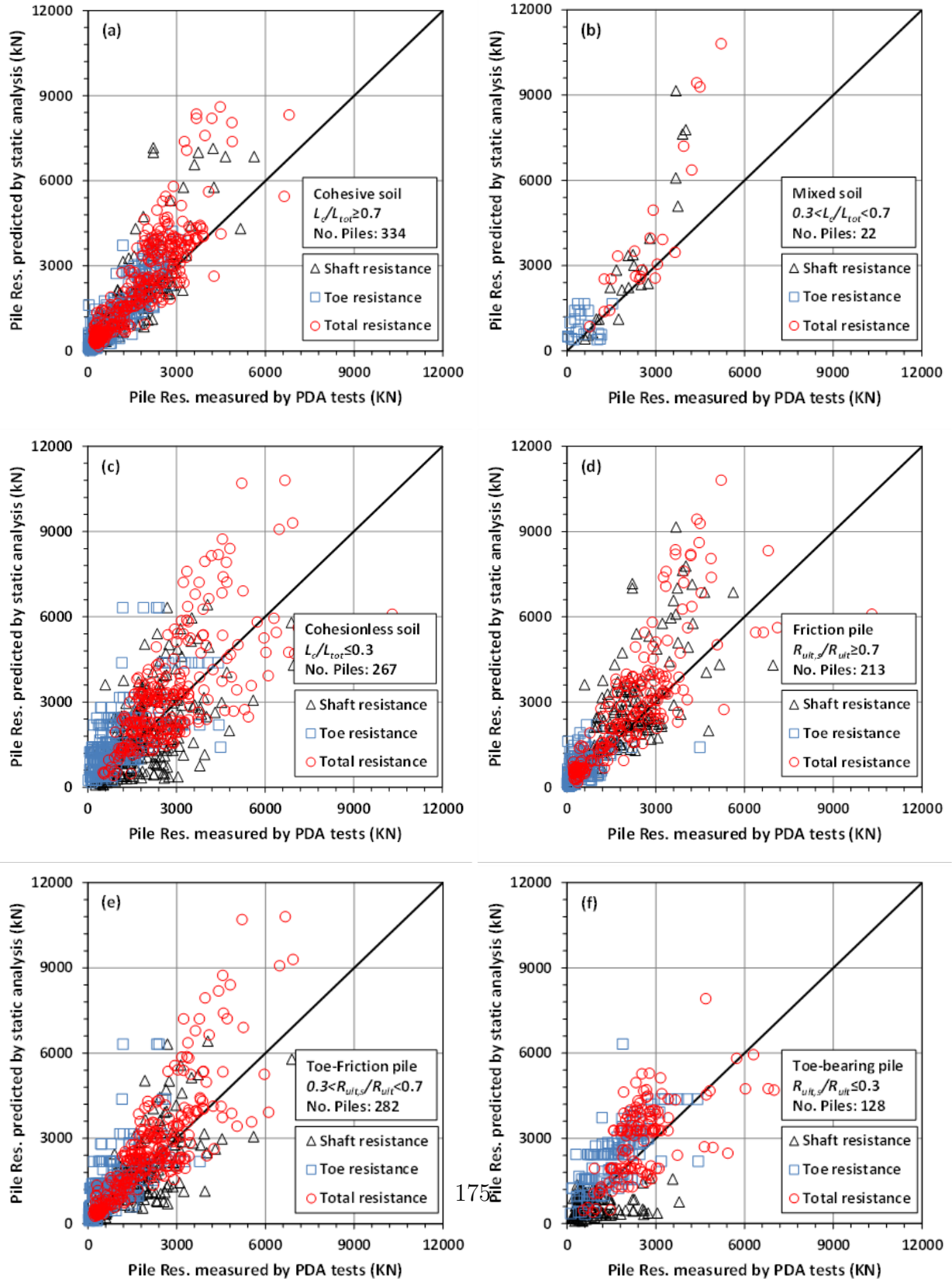


Figure 5.4: Comparison between measured and predicted pile capacities in (a) cohesive soil, (b) mixed soil, (c) cohesionless soil, and (d) friction, (e) toe-friction and (f) toe-bearing piles

Results and Discussions

5.6.1 Calibration of single GRF as total resistance factor (ϕ_t)

Figure 5-5 illustrates the variation of β and ϕ_t for piles driven in various soil types considering the 70% rule and R ratio Classifications using FOSM, MCS and FORM. Although FOSM is a simple method that can be used in reliability-based method, it has some shortcomings that reduce the accuracy (Haldar and Mahadevan, 2000; Heidarie Golafzani et al., 2019). As reported in previous studies (e.g., Paikowsky et al. 2004; Allen, 2005; Bach and Van Gelder, 2016; Heidarie Golafzani et al., 2019) the GRF values calculated through MCS and FORM are comparable and higher than those obtained by FOSM.

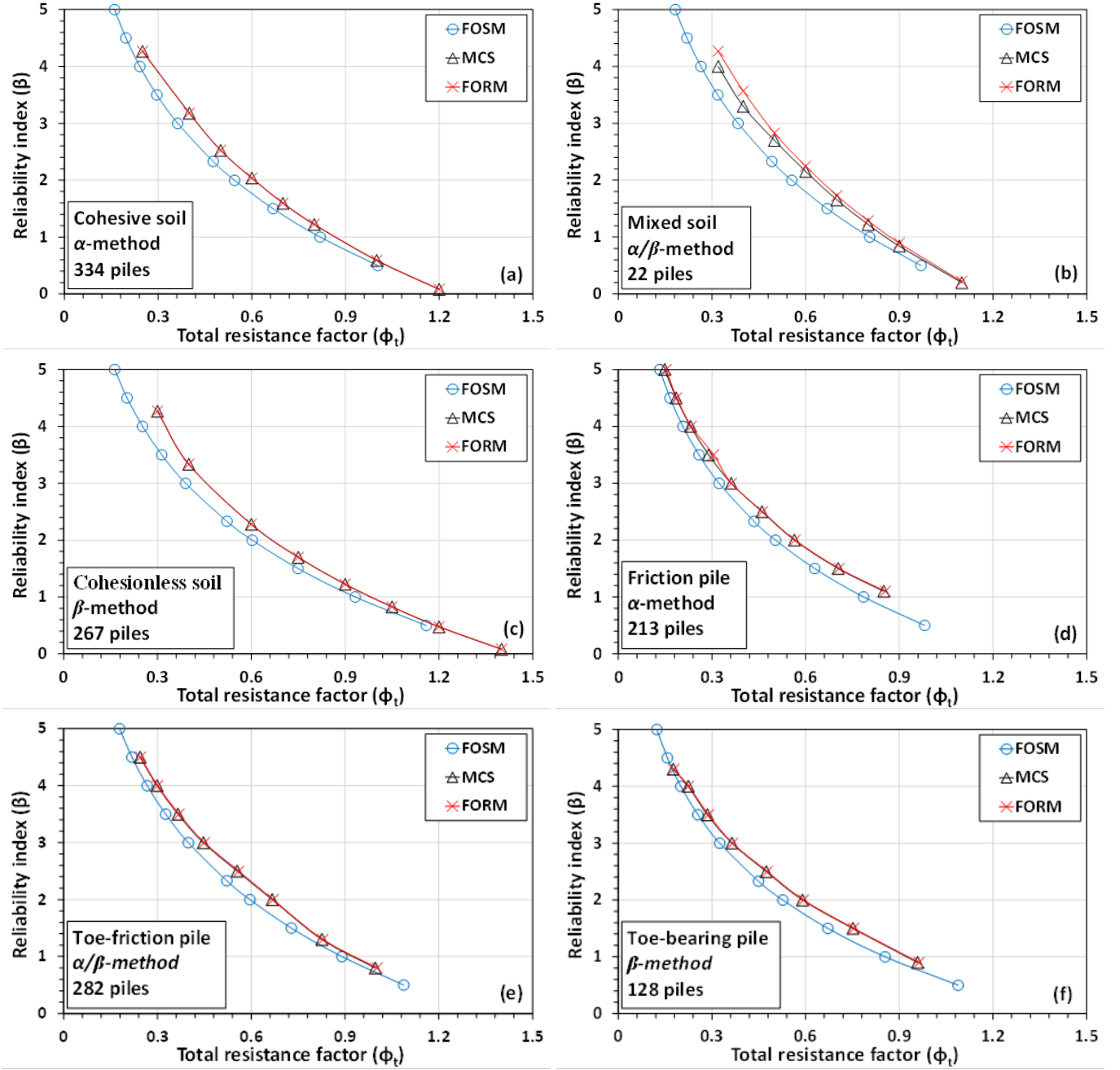


Figure 5.5: Relationship between the reliability index and total GRF for piles (a) in cohesive soil, (b) in mixed soil, (c) in cohesionless soil, and (d) friction, (e) toe-friction and (f) toe-bearing piles

Based on the results, the GRF values for piles driven in cohesionless soils are greater than those for piles in cohesive soils. As the higher GRF values do not necessarily indicate an efficient design method (Heidarie Golafzani et al., 2019, 2020), the efficiency factor (i.e., ϕ/λ_R) proposed by McVay et al. (2002) was calculated for MCS approach to effectively compare the efficiency of the static analysis methods used in this study. The higher efficiency factor indicates a more economic and cost-effective design procedure. Table 5-2 presents a summary of the calibrated total GRFs for $\beta = 2.33$ and 3.00, and the efficiency factors.

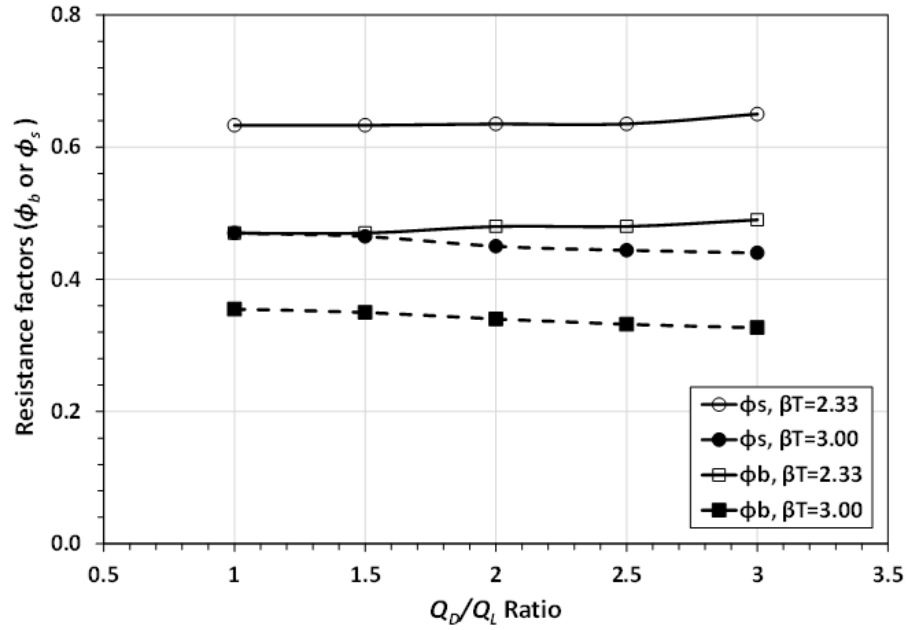


Figure 5.6: Effect of Q_D/Q_L Ratio on Variation of ϕ_b and ϕ_s for Cohesive Soils Using MCS

Table 5.2: Summary of statistical properties and GRF values for different static analysis and reliability methods based on soil/pile type

Soil /pile type	Analysis method	No. of piles	Mean	COV	ϕ ($\beta = 2.33$)			ϕ ($\beta = 3.00$)			ϕ/λ_R^*	
					FOSM	MCS	FORM	FOSM	MCS	FORM	$\beta = 2.33$	$\beta = 3.00$
C	α	334	0.995	0.352	0.476	0.537	0.540	0.363	0.426	0.429	0.540	0.428
Ch	β	267	1.180	0.390	0.522	0.589	0.592	0.390	0.456	0.460	0.499	0.386
M	α/β	22	0.931	0.290	0.506	0.571	0.575	0.398	0.450	0.454	0.614	0.484
F	α	213	1.008	0.401	0.435	0.489	0.492	0.322	0.362	0.364	0.485	0.359
TB	β	128	1.160	0.451	0.449	0.505	0.507	0.325	0.364	0.367	0.435	0.314
TF	α/β	282	1.073	0.345	0.522	0.584	0.585	0.390	0.447	0.449	0.544	0.417

Note: C: Cohesive; Ch: Cohesionless; M: Mixed; F: Friction; TB: Toe-bearing; TF: Toe-friction

*: Calculated based on MCS

5.6.2 Calibration of Separate GRFs as Toe and Shaft Resistance Factors (ϕ_b and ϕ_s)

It is necessary to analyze initially the effect of Q_D/Q_L ratio on the separate GRFs for toe and shaft resistances. As **Figure 5-6** shows Variation of Q_D/Q_L ratio has not a considerable effect on ϕ_b and ϕ_s values. Similar findings were reported by Asghari Pari and Fakharian (2023).

Figure 5-7 illustrates the comparison of toe and shaft GRFs for different soil and pile types considering the R ratio and 70% rule. The value of ϕ_s is found to be greater than that of ϕ_b . Like the findings for the total GRF, the separate GRFs based on FOSM are lower compared to those calculated based on MCS and FORM.

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The comparison of predicted $R_{ult,b}$ and $R_{ult,s}$ obtained by different reliability methods demonstrates consistent trends with those seen in the total bearing capacity and for further analysis, the MCS method is chosen as the reliability method, while the α -method, β -method, and α/β -method are selected as the static analysis methods for cohesive, cohesionless, and mixed soils, respectively.

Table 5-3 shows the comparison of total and separate GRFs for different soil and pile types considering the R ratio and 70% rule. Upon comparison, it is evident that the individual shaft or toe GRFs are lower than the total GRF considering the 70% rule. This observation aligns with findings from prior studies such as Yang et al. (2010) and Abu-Farsakh et al. (2012), which also reported similar results. Using the R ratio classification, friction piles have the highest shaft GRF among the shaft GRFs of three pile types, and toe-bearing piles have the highest toe GRF among the defined pile types. In this classification criterion, total GRF is between toe and shaft GRFs.

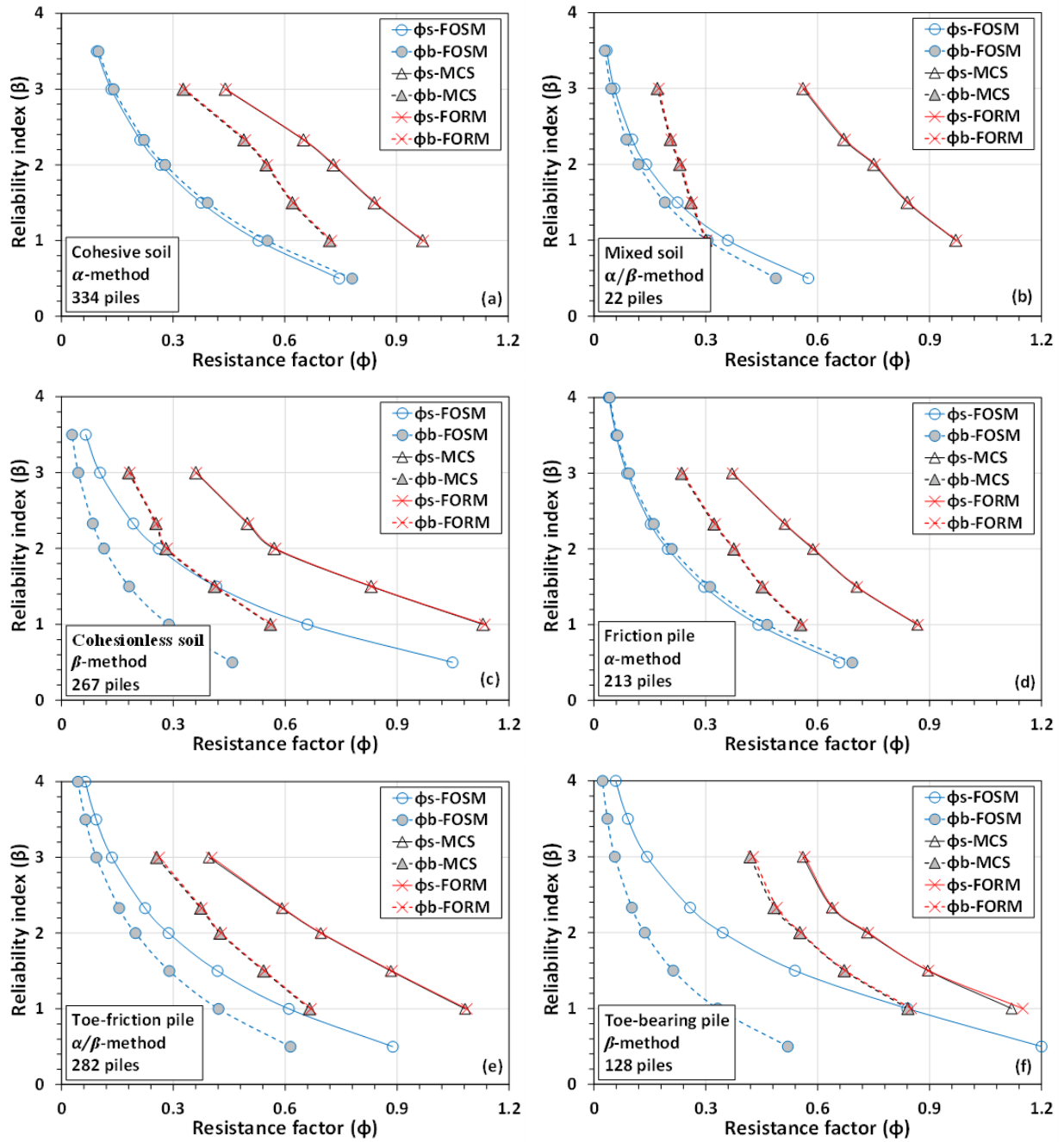


Figure 5.7: Relationship between the reliability index and separate GRFs for piles (a) in cohe-sive soil, (b) in mixed soil, (c) in cohesionless soil, and (d) friction, (e) toe-friction and (f) toe-bearing piles

Table 5.3: Summary of statistical properties and GRFs for shaft, toe and total resistances based on “70% rule” and “R ratio” using MCS analysis

Soil/pile type	Resistance	Statistical properties of bias factors		Resistance factor	
		Mean	COV	$\beta = 2.33$	$\beta = 3.00$
Cohesive ($L_c/L_{tot} \geq 0.7$)	Shaft	0.992	0.423	0.492	0.345
	Toe	1.038	0.591	0.355	0.263
	Total	0.995	0.352	0.537	0.426
Cohesionless ($L_c/L_{tot} \leq 0.3$)	Shaft	1.909	0.841	0.508	0.302
	Toe	0.833	0.738	0.250	0.151
	Total	1.180	0.390	0.589	0.456
Mixed ($0.3 < L_c/L_{tot} < 0.7$)	Shaft	0.893	0.285	0.521	0.409
	Toe	1.039	1.077	0.158	0.132
	Total	0.931	0.305	0.565	0.450
Friction pile ($R \geq 0.7$)	Shaft	0.822	0.465	0.509	0.371
	Toe	0.866	0.768	0.318	0.235
	Total	1.008	0.401	0.487	0.361
Toe-bearing pile ($R \leq 0.3$)	Shaft	1.848	0.928	0.638	0.561
	Toe	0.729	0.489	0.480	0.419
	Total	1.160	0.451	0.503	0.364
Toe-friction pile ($0.3 < R < 0.7$)	Shaft	1.047	0.547	0.592	0.395
	Toe	0.723	0.617	0.371	0.255
	Total	1.073	0.345	0.584	0.446

Figure 5-8 shows the variations of separate GRFs considering 70% rule and R ratio classifications. However, the results of using the 70% rule do not have a clear logical trend, but the results based on R ratio present that increasing “R ratio” results in a decrease of ϕ_b and ϕ_s for both $\beta = 2.33$ and $\beta = 3.00$.

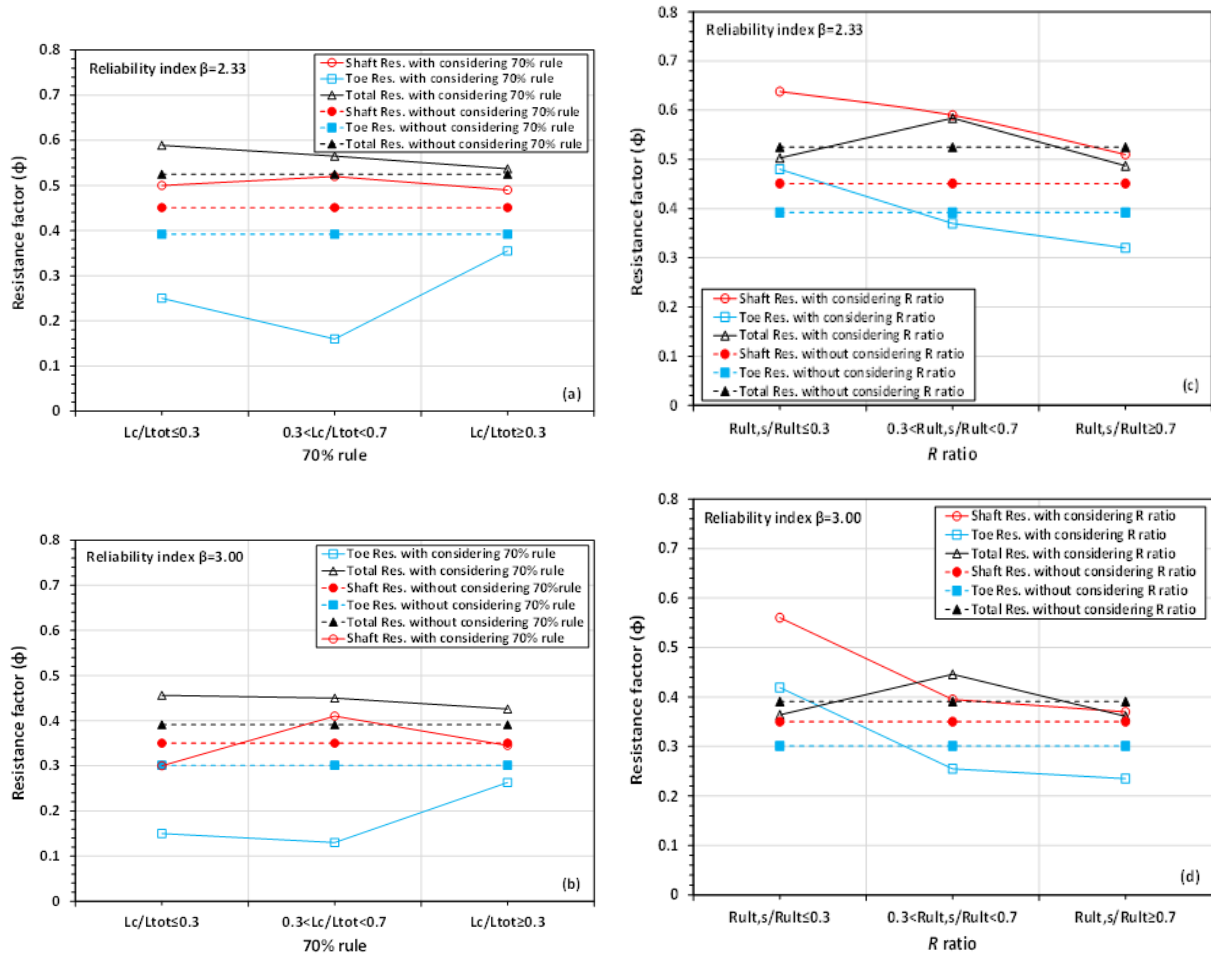


Figure 5.8: Shaft and toe resistance factors using MCS analysis based on (a) “70% rule”, $\beta = 2.33$, (b) “70% rule”, $\beta = 3.00$, (c) “R ratio”, $\beta = 2.33$ and (d) “R ratio”, $\beta = 3.00$.

Discussion

To evaluate the effectiveness of using separate GRFs during the pile design process, the factored total resistances calculated by total GRF (i.e., $\phi_t R_{ult}$) and separate GRFs (i.e., $\phi_s R_{ult,s} + \phi_b R_{ult,b}$) were compared in **Figure 5-9** for $\beta = 2.33$. Based on the results, using separate GRFs resulted in a lower factored pile resistance. As the resistance mechanisms of a pile in the shaft and toe parts are different, the two parts of the uncertainty must be considered separately in order to calculate the resistance factor (Asghari Pari and

Fakharian, 2022). Based on the results of this study, the uncertainty of the shaft and toe resistances is quite different (refer to the mean and COV values of the bias factors in Table 5-3) based on both the “70% rule” and “R ratio” classification methods. Therefore, using the same uncertainty for the shaft and toe resistances (such as that used in the total GRF) can give misleading results. Consequently, it can be concluded that the GRF separation approach leads to more reliable factored resistances.

As previously stated, in the case of redundant piles, that is a common condition in infrastructures and oil and gas projects, $\beta = 2.33$ can be selected in the design procedure. CFEM (2006) recommends a resistance factor of 0.40 ($\phi_{\text{CFEM}} = 0.40$) for R_{ult} calculated by static analysis methods including α - and β -methods. In other words, it considers similar GRFs for toe, shaft, and R_{ult} . Based on the reliability analysis of steel piles driven in Alberta, at $R \leq 0.3$, both resistance factors for toe and shaft capacities are higher than those recommended by CFEM (2006), where ϕ_b and ϕ_s are 1.20 and 1.60 times higher than ϕ_{CFEM} . When $0.3 < R < 0.7$, the ϕ_b calculated by MCS is lower than ϕ_{CFEM} . However, ϕ_s is 1.48 times greater than 0.40. At $R \geq 0.7$, ϕ_b and ϕ_s are 0.80 and 1.27 times the ϕ_{CFEM} . Figure 5-10 illustrates the comparison of factored total resistance calculated by total GRF recommended by CFEM (2006) (i.e., $\phi_{\text{CFEM}}R_{\text{ult}}$) and separate GRFs using MCS results for piles driven in Alberta (i.e., $\phi_s R_{\text{ult},s} + \phi_b R_{\text{ult},b}$). The results showed that the averaged factored total resistance calculated by separate GRFs is approximately 27%, 24%, and 19% higher than those recommended by CFEM (2006) in $R \leq 0.3$, $0.3 < R < 0.7$, and $R \geq 0.7$ conditions, respectively.

Table 5-4 presents an overview of resistance factors proposed for driven steel piles in the technical literature and also the results of the current study. It should be noted that the main reasons for the difference in results are the differences in reliability methods, soil types, and static analysis methods, which prove the importance of regionally calibrating GRF values. As a limitation, the results presented for mixed soils are obtained based on a limited number of data, and the accuracy should be verified during future studies.

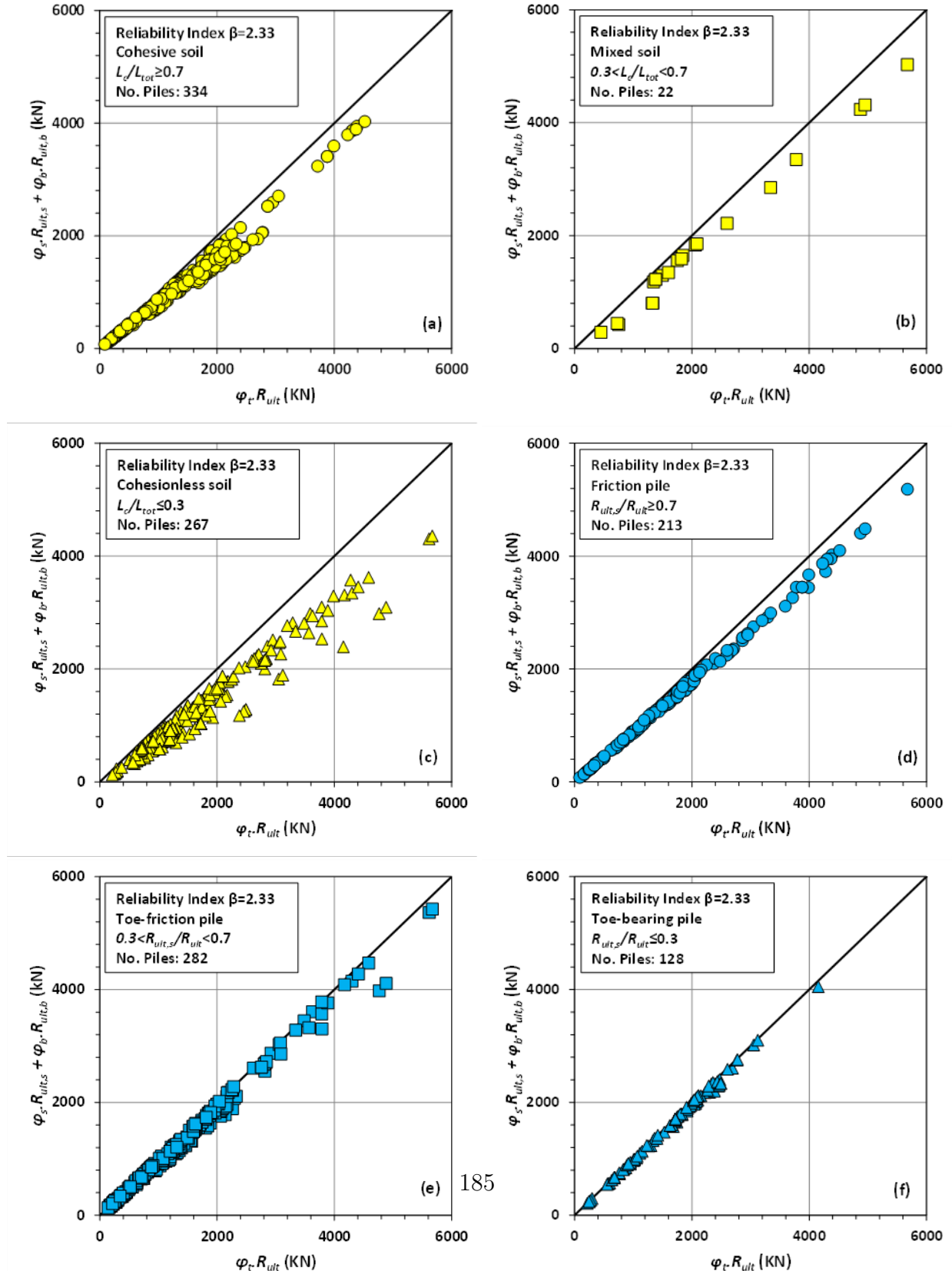


Figure 5.9: Comparison of factored resistance of pile based on total and separate GRFs based on MCS analysis for piles (a) in cohesive soil, (b) in mixed soil, (c) in cohesionless soil, and (d) friction, (e) toe-friction and (f) toe-bearing piles

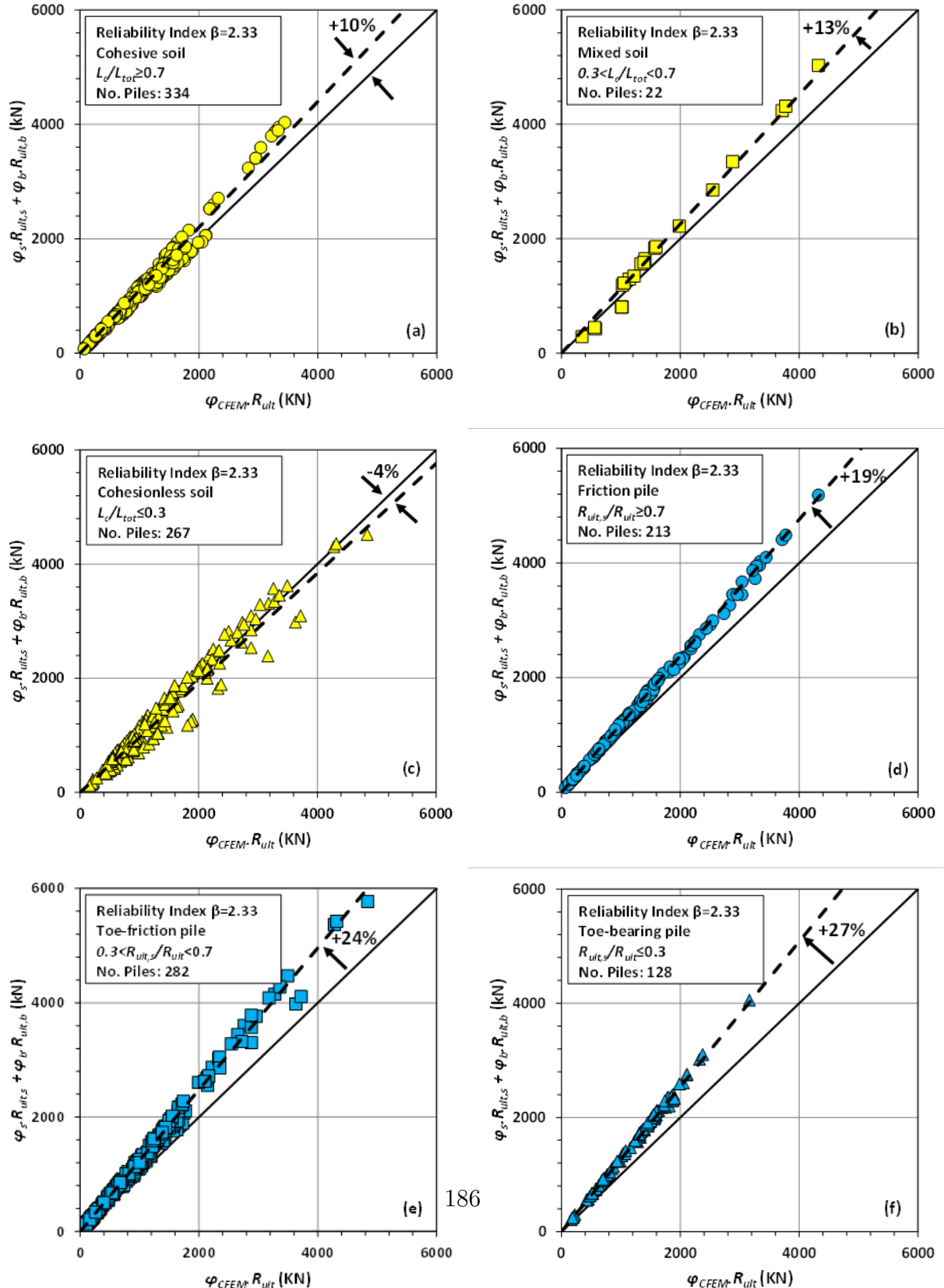


Figure 5.10: Comparison of factored resistance of pile using separate GRFs of Alberta and GRF recommended by CFEM (2006) for piles (a) in cohesive soil, (b) in mixed soil, (c) in cohesionless soil, and (d) friction, (e) toe-friction and (f) toe-bearing piles

Table 5.4: A summary of previous efforts on developing separate GRFs for toe and shaft

Ref.	Soil type	Location	Static Analysis Method	Reliability Method	ϕ_s	ϕ_b
CFEM (2006)*	All soils	Canada	α -CFEM / β -CFEM	N/A	0.40	0.40
Paikowsky et al. (2004)	Cohesive	USA	α -Tomlinson	FORM	0.25	0.25
	Cohesive		α -API	FORM	0.29	0.29
	Mixed		α -Tomlinson / Nordlund/Thurman	FORM	0.24	0.24
	Mixed		β -Method/Thurman	FORM	0.22	0.22
	Cohesionless		β -Method	FORM	0.36	0.36
AASHTO (2012)	Cohesive / Mixed	N/A	α -Tomlinson	N/A	0.25	0.25
	Mixed		β -Method	N/A	0.35	0.35
	Cohesionless		Nordlund/Thurman**	N/A	0.45	0.45
Bian et al. (2018)	Cohesive	N/A	FOSM	FOSM	0.60	0.39
Asghari Pari and Fakharian (2023)	Cohesive	Iran	α -API	MCS	1.60	0.35
	Cohesionless		β -API	MCS	0.29	0.48
Current study	Cohesive	Alberta (Canada)	α -CFEM	MCS	0.49	0.36
	Mixed		α & β -CFEM	MCS	0.52	0.16
	Cohesionless		β -CFEM	MCS	0.51	0.25

Note: * Based on recommendations of NBCC (2005) and CHBDC (2000). ** Hannigan et al. (2005).

Conclusions

To evaluate the effect of important parameters on the R_{ult} and the GRF values used in the LRFD design method, a comparative study on effect of using a single GRF for R_{ult} and separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ was carried out in this study. In this regard, a local database of steel pipe pile foundations in Alberta, Canada, including 623 PDA tests and 28 SLTs, was developed to calibrate the GRF for driven pile foundations using the FOSM, FORM, and MCS. The following conclusions can be obtained from the results of this study:

- Based on the “70% rule”, the single GRF for R_{ult} was greater than the separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ for all cohesive, cohesionless and mixed soils.
- Although the “70% rule” is a simple method for pile classification based on the soil layers along the embedded pile length, it can be misleading in some conditions. In this regard a new classification criterion called “R ratio” was defined as the ratio of shaft resistance to total resistance. On the basis of this method a pile can be categorized into “toe-bearing pile”, “friction pile” or “toe-friction pile”.
- Based on MCS, using the “R ratio” criterion, for $\beta = 2.33$ the regionally calibrated ϕ_t values were obtained 0.503, 0.584 and 0.487 for “toe-bearing”, “toe-friction” and “friction piles”, respectively. The ϕ_s and ϕ_b were 0.638 and 0.480 for “toe-bearing piles”, 0.592 and 0.371 for “toe-friction piles” and 0.509 and 0.318 for “friction piles”, respectively.
- Considering $\beta = 3.00$, the single GRF values were 0.364, 0.446 and 0.361 for driven steel piles categorized as “toe-bearing”, “toe-friction” and “friction piles”, respectively, based on the “R ratio” criterion. The separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ were 0.561 and 0.419 for “toe-bearing piles”, 0.395 and 0.255 for “toe-friction piles” and 0.371 and 0.235 for “friction piles”, respectively. Based on the “R ratio”, the single GRF for R_{ult} was between the separate toe and shaft GRFs among the defined pile types. The results based on R ratio presents that increasing “R ratio” resulted in decreasing of ϕ_b and ϕ_s for both $\beta = 2.33$ and $\beta = 3.00$.
- Comparison of factored R_{ult} calculated by total GRF (i.e., $\phi_t R_{ult}$) and separate GRFs (i.e., $\phi_s R_{ult,s} + \phi_b R_{ult,b}$) showed that using separate GRFs resulted in a lower factored pile resistance and it can be concluded that the GRF separation approach leads to more reliable factored resistances.

- The results showed that the averaged factored total GRF calculated by separate GRFs are approximately 27%, 24% and 19% higher than those recommended by CFEM (2006) in $R \leq 0.3$, $0.3 < R < 0.7$ and $R \geq 0.7$ conditions, respectively in Alberta. This may lead to reduction in pile construction cost due to less number/length of piles in a specific construction project.
- Comparison of the calculated GRFs based on “R ratio” criteria showed that the ϕ_b and ϕ_b decreased by increasing “R ratio”. In other words, the steel piles with higher $R_{ult,b}$ (i.e., a lower “R ratio”) have higher separate GRF.

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Chapter 6

The Effect of Setup Time in Geotechnical Resistance Factor of Driven Steel Piles in Alberta: A Case Study

6.1 Background

The contents presented in this chapter are summarized from the manuscript below: Roshani, et al. (2024). The effect of setup time in geotechnical resistance factor of driven steel piles in Alberta: a case study. Manuscript published in a Deep Foundation Institute Journal.

6.2 Introduction

In the traditional framework of Allowable Stress Design (ASD), the factor of safety is utilized to encompass all uncertainties associated with both load and resistance. Conversely, the Load and Resistance Factor Design (LRFD) method adopts separate load and resistance factors, recognizing the inherent uncertainties in estimating allowable loads and resistance. This methodology seeks to establish a uniform probability of failure in the design of geo-structures. (Whitman, 2000; Wang, 2009; Shahnazari et al., 2016; Sekhavatian and Janalizadeh Choobbasti, 2018; Golafzani et al., 2019; Doan, 2023).

In the absence of a comprehensive pile load database, the geotechnical resistance factor (GRF) for pile foundations, as recommended in the initial AASHTO LRFD specifications, was derived through an adjustment to the allowable stress design. (Abu-Farsakh et al., 2012). The accumulation of a valuable pile load database has been facilitated through the comprehensive undertaking of in-situ pile load tests, encompassing both static and dynamic load tests. This initiative has been carried out across various locations worldwide in recent decades. Notably, established design codes like the Federal Highway Administration (FHWA) and AASHTO (2007) have advocated for regionally recalibrated resistance factors. This recalibration is grounded in local in-situ pile tests, aiming to eliminate overly conservative assumptions and enhance the reliability of design outcomes. Hence, numerous calibration studies conducted in diverse global locations have sought to enhance design efficiency and minimize foundation sizes. These investigations aim to formulate GRFs that align with local soil conditions and construction practices, thereby optimizing the overall design process. (Kim, 2003; Kim et al., 2005; Abdelsalam et al., 2011; Bougataya and Stuedlein, 2016; Ng et al., 2017; Yu et al., 2017; Asghari Pari et al., 2019; Roshani et al., 2020).

It is noteworthy to mention that a significant factor influencing pile capacity is excess pore water pressure. The presence of water content in the soil surrounding a pile induces excess pore water pressure, leading to a reduction in effective stress during the installation of driven piles. Following the completion of pile installation or the end of drive (EOD) condition, the dissipation of pore pressure continues over time, resulting in a gradual increase in effective stress. This increase ultimately contributes to the enhancement of pile ultimate resistance (Konrad and Roy, 1987; Bian et al., 2021). This increase is commonly referred to as the pile setup effect in the literature (Yang and Liang, 2009; Hosseini and Rayhani, 2017). Haque and Abu-Farsakh (2018) stated that the increase in pile resistance due to the setup effect occurs in all soils, but it is more significant in cohesive soils. Analyzing the outcomes of 11 cases involving piles driven into sand, Chow et al. (1998) observed an approximately 50% increase in pile capacities per logarithmic cycle of time starting from 1 day after pile driving. Similarly, Axelsson (2002) reported a setup effect ranging from 15% to 65% per logarithmic cycle of time. Salgado (2008) noted that the rise in pile capacity over time, attributed to pore water dissipation and soil remolding, is commonly observed in driven piles.

Previous investigations on piles driven in fine-grained clayey deposits suggest that the primary cause of the pile setup effect is the increase in radial effective stresses resulting from the dissipation of excess pore water pressure induced by pile driving effects (Rosti, 2016). The application of the anisotropic modified Cam-Clay model in Finite Element Method (FEM) modeling appears to be a viable approach for studying the dissipation of

pore water pressure in driven piles (Rosti, 2016).

Komurka et al. (2003) explained the increase in pile ultimate resistance in three phases. In phase 1, the rate of the pile setup effect aligns with the rate of excess pore water dissipation, displaying a non-linear trend in logarithmic scale. At time t_0 , the non-linear behavior concludes (end of phase 1), and further pore water dissipation follows a linear trend in logarithmic scale (phase 2). During phase 2, the disturbed soil around the pile undergoes an increase in effective vertical and horizontal stresses, ultimately leading to an enhancement of soil shear strength according to consolidation theory. The duration of phase 2 depends on soil parameters (type, permeability, and sensitivity) and pile properties (type and size). The logarithmically linear rate of pore pressure dissipation may extend if the soil is less permeable, and a larger volume of soil is displaced by the pile (Hosseini, 2015). The final phase of pile setup is termed aging. After the completion of phase 2, there is a period where the dissipation rate is very slow, and the pile setup effect is independent of effective stress during this phase (phase 3). In this phase, soil aging occurs, potentially related to thixotropy, secondary compression, particle interference, and clay dispersion (Camp et al., 1993; Long et al., 1999).

Due to uncertainties in interpreting phases 1 and 3, most previous studies have focused on phase 2. Various research efforts have been undertaken to estimate the actual increase in pile resistance due to the pile setup effect, including studies by Skov and Denver (1988), Svinkin et al. (1994), Long et al. (1999), Svinkin and Skov (2000), Khan and Decapite (2011), and Abedlsalam et al. (2012). Among these studies, the relationship proposed by Skov and Denver (1988) is widely recognized and practical, serving as the basis for analyses in this study. Eq. 1 presents the logarithmic empirical relationship developed by Skov and Denver (1988):

$$R_t = R_0 \left(1 + A \log \frac{t}{t_0} \right) \quad (6.1)$$

where A and t_0 are empirical parameters which depend on soil type, R_t is the predicted axial capacity at time t after driving, R_0 is the measured axial capacity at time t_0 , and t is time after pile installation in days.

Table 6-1 presents a summary of proposed A and t_0 values based on Skov and Denver's relation reported in previous studies. Due to the substantial uncertainties associated with predicting the setup process, the beneficial impact of the long-term setup of driven piles has not been extensively utilized in piling projects. Bian et al. (2021) believed that the resistance factor of driven piles in sand and clay decreases with the increase of M_{setup} factor which is equal to $A \log \frac{t}{t_0}$ in Skov and Denver's relation.

Table 6.1: The proposed A and t_0 values based on Skov and Denver's relation in the literature.

Reference	Soil Type	A	t_0 (days)	Pile Type	Location
Skov and Denver (1988)	Clay	0.6	1	Concrete piles	Denmark
	Sand	0.2	0.5	Concrete piles	Denmark
Svinkin et al. (1994)	Clay	0.36-1.07	1 to 2	Pre-stressed concrete piles & H piles	Ohio
Axelsson (1998)	Sand	0.2-0.8	1	Concrete piles	-
Camp and Parmar (1999)	Sandy Clay /Sandy Silt	0.37-1.31	2	Concrete piles	South Carolina
Yang and Liang (2006)	Clay	0.1-1	1	Steel pipe, HP, concrete, timber	North Florida
Bullock et al. (2005)	Dense fine sand & soft to medium stiff silty clay	0.2	1	Concrete piles	North Florida

6.3 Database

There are two widely recognized in-situ tests employed for assessing the bearing capacity of pile foundations: the Pile Driving Analyzer (PDA), utilized as a Dynamic Load Test (DLT), and the Static Load Test (SLT). While the SLT is considered the most accurate method for estimating pile bearing capacity, its extensive cost and time-consuming procedure often impose limitations on its application in projects (Jabo, 2014; Asghari Pari et al., 2019). Moreover, SLT is considered a destructive test and is impractical for application on production piles, further limiting its usability. Consequently, the popularity of the PDA test is on the rise as an alternative method to predict the bearing capacity of piles, particularly in Canada. PDA tests are commonly conducted under two driving conditions: End-of-Driving (EOD) and Beginning-of-Restrike (BOR), each with distinct characteristics during the analysis. EOD corresponds to the moment when the pile driving reaches its intended depth, signifying the completion of the driving process. On the other hand, BOR is the phase where the pile is struck again after reaching the EOD condition.

In this study R_0 is taken as R_{EOD} which defined the measured pile capacity from Case Pile Wave Analysis Program (CAPWAP) analysis at day of t_0 after EOD, and R_{SET} defined

as setup capacity gained up to t days since EOD. Therefore, the Eq. 1 can be rewritten as the following:

$$R_{\text{BOR}} = R_{\text{EOD}} + R_{\text{SET}} = R_{\text{EOD}} \left(1 + A \log \frac{t}{t_0} \right) \quad (6.2)$$

To assess the regional impact of time on the GRF values for driven steel pile foundations in Alberta, an evaluation was conducted based on field results from PDA tests on three hundred forty-six (346) driven steel pipe piles. Each of these piles underwent at least two separate PDA tests at different times, one in the EOD condition and another in the BOR condition, to examine the setup time effect. The piles varied in diameter from 114 to 915 mm and had embedment lengths ranging approximately from 4 to 40 m. For the purpose of calibration, extensive geotechnical investigations were carried out in the vicinity of the pile tests. These investigations involved drilling boreholes, conducting in-situ tests such as standard penetration tests, and performing laboratory tests such as moisture content analysis, particle size distribution analysis, unconfined compression tests, and direct shear tests. This comprehensive approach facilitated a detailed understanding of the geological and geotechnical conditions influencing the behavior of the piles in the region.

Figure 6-1(a) presents a 22 m deep borehole that was drilled in cohesive deposit which consisted mostly of clayey soil. The watertable was reported at the depth of about 6 m below the ground surface. In **Figure 6-1(b)** and **Figure 6-1(c)** logs of drilled boreholes in cohesionless and mixed (i.e., cohesive/cohesionless soil) soils are illustrated, respectively. The depth of these boreholes was about 12 m and in addition, the watertable has not been observed at the time of drilling in cohesionless and mixed soils. According to a pile classification method proposed by AbdelSalam et al. (2011), which categorizes piles based on soil type along their length, a pile is deemed cohesive when 70% or more of its length is driven into cohesive soils (e.g., CL soil). In such cases, static analysis methods associated with cohesive soils, such as the α -method, can be employed to predict the bearing capacity. Conversely, if the cohesive soil along the pile length is less than 30%, the pile is classified as cohesionless, and appropriate static analysis methods like the β -method can be applied. In the third scenario, where the cohesive soil along the pile length falls between 30% and 70%, the pile is considered mixed, and a combined method (α/β -method) can be used. This classification method, known as the "70% rule," has been suggested in previous studies for assessing the bearing capacity of piles (AbdelSalam et al., 2011; Bostwick, 2014; Jabo, 2014; Asghari Pari et al., 2019). A summary of the database properties used in this study is presented in **Table 6-2**.

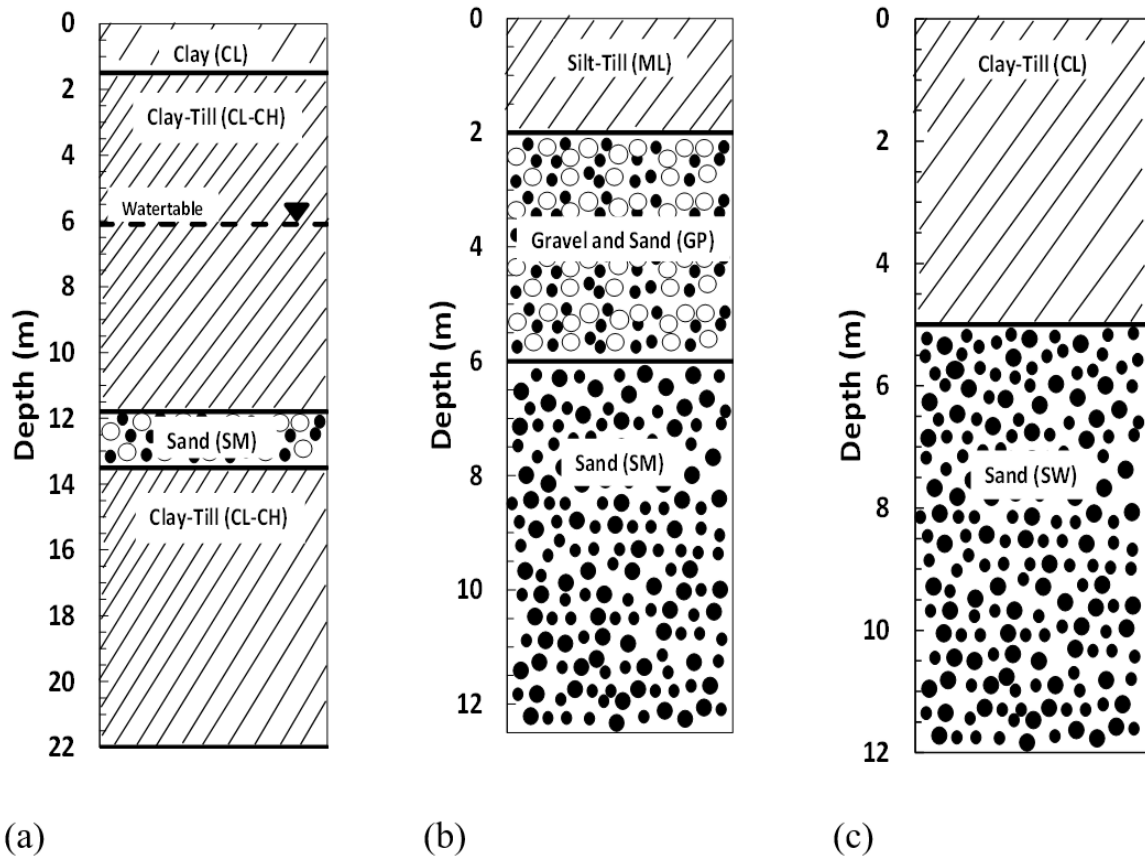


Figure 6.1: A summary of soil layers in 3 different boreholes, (a) cohesive soil, (b) cohesionless soil and (c) mixed soil (Note: The abbreviations are presented based on Unified Soil Category System (ASTM D2487, 2020))

Table 6.2: Summary of database properties used in this study

General soil type	Date tested	Number of steel piles	Range of c_u	Range of N_{SPT}	Pile type	Diameter (mm)	Embed. length (m)
Cohesive	2013-2020	256	20-300	7- 50	Pipe	114 - 914	4.2-32.2
Cohesionless	2012-2020	83	N/A	3-50	Pipe	219 - 914	4.9-39.7
Mixed	2014-2015	7	90-450	9-50	Pipe	324 - 914	8.0-37.2

6.4 Calibration approaches for single and separate resistance factors

The LRFD methodology entails the application of load factors, typically greater than one, to nominal loads in order to derive factored loads. Conversely, resistance factors, also referred to as GRF, are applied to the ultimate capacity, and these factors are less than one. In simpler terms, failure is considered to occur when factored loads exceed factored capacities. The fundamental equation of LRFD-based design can be succinctly expressed as follows (Dithinde, 2007):

$$R_r = \phi_t R_n \geq \sum \alpha_i Q_i \quad \text{or} \quad \phi_t R_n - (\gamma_D Q_D + \gamma_L Q_L) \geq 0 \quad (6.3)$$

where ϕ is the geotechnical resistance factor for total bearing capacity, R_n is the ultimate (nominal) geotechnical resistance determined by engineering analysis using field parameters or performance data and represents the best estimate of the axial strength and takes account for all factors influencing the resistance, R_r is the factored resistance, α_i is the load factor corresponding to a particular load, Q_i is the specified load component of the overall load effects and i represents various types of loads such as dead load (Q_D), live load (Q_L), etc. and γ_D and γ_L are dead and live load factors, respectively. The minimum R_n required to satisfy the limit state design equation can be obtained by the following equation:

$$R_n = \frac{\gamma_D Q_D + \gamma_L Q_L}{\phi_t} \quad (6.4)$$

The load factors are calculated according to the uncertainties of the loads and their probability of occurrence. The GRFs are estimated on the basis of the variabilities of the geotechnical parameters and the analysis of uncertainties during the calculation of the ultimate (nominal) resistances (CFEM, 2006). The values of load and resistance factors can be obtained from various codes such as AASHTO (2012), National Building Code of Canada (NBCC, 2005), and Canadian Highway Bridge Design Code (CHBDC, 2000).

The basic theory of the LRFD method is defined on the basis of the use of probability density functions (PDF) representing the load (Q) and resistance (R) and their relation to the limit state function (g) which is presented in Eq. (5).

$$g = R_m - Q_m = \lambda_R R_n - (\lambda_D Q_D + \lambda_L Q_L) \quad (6.5)$$

where R_m and Q_m are random variables representing actual resistance and loads, respectively, λ_R is the resistance bias factor and λ_D and λ_L are the dead and live load bias factors,

respectively. The main aim of reliability-based design is to ensure that the probability of failure does not exceed an acceptable threshold level. The probability of failure is defined as follows:

$$p_f = \text{Prob}(g < 0) \leq p_T \quad (6.6)$$

where $\text{Prob}(\cdot)$ is the probability of an event, p_f is the probability of failure, p_T is the acceptable target probability of failure. Due to some difficulties in calculating p_f , another parameter called the reliability index (β) is defined as follows:

$$\beta = -\phi^{-1}(p_f) \quad (6.7)$$

where $\phi^{-1}(\cdot)$ is the inverse of the standard normal cumulative function, ϕ . Various probabilistic approaches are employed in reliability-based design and in the calibration of the LRFD resistance factor for driven pile foundations. (Jabo, 2014). The most robust techniques are the First Order Second Moment (FOSM), the First Order Reliability Method (FORM), and the Monte Carlo Simulation (MCS). In this paper, MCS, as a powerful stochastic problem-solving technique, was used for the calibration of regional GRF, considering the setup time effect on the bearing capacity of driven steel piles.

Monte Carlo Simulation uses randomly generated numbers for the component variables to determine the probability distribution function (PDF) of the uncertain variable (i.e., the resistance factor in the current research). For the design of pile foundations, the MCS technique is used to generate random values of load and resistance functions based on their statistical properties. As previously described, the limit state function (g) is formed with the random values and evaluated for failure as presented by Eq. 8.

$$g(R, Q) = \left(\frac{\gamma_D \frac{Q_D}{Q_L} + \gamma_L}{\phi_t} \right) \lambda_R - \left(\lambda_D \frac{Q_D}{Q_L} + \lambda_L \right) \quad (6.8)$$

where Q_D/Q_L = dead to live load ratio. CFEM (2006) recommended that the load statistics and factors can be obtained by the applicable codes such as AASHTO. In this study, the load statistical properties were adopted based on AASHTO (2012) as $\lambda_D = 1.05$, $\lambda_L = 1.15$, $\text{COV}_D = 0.1$, $\text{COV}_L = 0.2$, $\gamma_D = 1.25$, and $\gamma_L = 1.75$. In addition, the ratio of dead load to live load was defined as 3.00. Previous studies have shown that the GRFs are not sensitive to the Q_D/Q_L ratio, especially for ratio values greater than 2.5 (Paikowsky et al., 2004; Allen, 2005; Abu-Farsakh et al., 2013). In this study, the bias factor is defined as R_m/R_p , where R_m is the measured bearing capacity (based on PDA results) and R_p is the predicted bearing capacity (based on the static analysis method). Considering Eq. 7, the failure state is defined when $g \leq 0$. A predetermined number of simulations is executed with the random values, and a count of the total times failure occurred is divided by the

number of simulations to give the probability of failure (p_f) (Paikowsky et al., 2010). The minimum number of simulations (N) is determined using the following equation:

$$N = \frac{1 - p_f}{\text{COV}_{p_f}^2 \cdot p_f} \quad (6.9)$$

For a COV_{p_f} of 10% a minimum number of $N = 100,000$ simulations are required for $\beta = 3.00$ corresponding to $p_f = 0.1\%$. The objective of MCS is the numerical integration of the expression for failure probability, as given below:

$$p_f = \sum_{i=1}^N I(g_i \leq 0) \quad (6.10)$$

$$P_f = \frac{N_f}{N} \quad (6.11)$$

where I is an indicator function which is equal to 1 for $g_i \leq 0$ (the failure region), and equal to 0 for $g_i > 0$ (the safe region). Considering the number of cases in the failure region, N_f , the failure probability can be directly obtained by the N_f/N ratio. The above procedure caused us to obtain a single resistance factor for axial bearing capacity of pile foundations. Considering the effect of pile setup, the total ultimate resistance of piles at BOR condition can be separated in initial ultimate resistance (i.e., at EOD condition) and setup resistance as previously described by Eq. (2). Based on the LRFD design concept and similar to Eq. (4), the following equations can be written:

$$R_{EOD} = \frac{\gamma_D Q_D + \gamma_L Q_L - \phi_{SET} R_{SET}}{\phi_{EOD}} \quad (6.12)$$

$$R_{SET} = \frac{\gamma_D Q_D + \gamma_L Q_L - \phi_{EOD} R_{EOD}}{\phi_{SET}} \quad (6.13)$$

Using the concept of the limit state function, the following equation can be used for calculating the separate GRF values for the pile resistances at EOD condition and setup time effect:

$$g(R, Q) = \frac{(\gamma_D \frac{Q_D}{Q_L} + \gamma_L) (\frac{R_{EOD}}{R_{SET}} \lambda_{EOD} + \lambda_{SET})}{\frac{R_{EOD}}{R_{SET}} \phi_{EOD} + \phi_{SET}} - (\lambda_D \frac{Q_D}{Q_L} + \lambda_L) \quad (6.14)$$

where λ_{SET} and λ_{EOD} are mean bias factors for setup effect and pile capacity at EOD condition, respectively. It can be seen that there are various compositions of ϕ_{EOD} and

ϕ_{SET} values that satisfy specified target reliability indices. The ϕ_{EOD} and ϕ_{SET} have an inverse relation, that is, ϕ_{SET} increases by decrement of ϕ_{EOD} , and vice versa. For solving this issue, Bach et al. (2014) proposed a ratio called “Acceptance Correlation Ratio” (ACR) to accept a pair of ϕ_{EOD} and ϕ_{SET} as follows:

$$\frac{\phi_{SET}}{\phi_{EOD}} \approx \text{ACR} = \frac{\text{COV}_{R_{EOD}} \lambda_{R_{SET}}}{\text{COV}_{R_{SET}} \lambda_{R_{EOD}}}. \quad (6.15)$$

More details about the procedure can be found in Bach et al. (2014) and Asghari Pari and Fakharian (2023).

6.5 Calibrating GRF Incorporating Setup Effect

Measured and Predicted Pile Capacity

The CAPWAP was used as a numerical modeling technique to measure pile capacity using the PDA tests in EOD and BOR conditions. The Match Quality (MQ) or match factor of performed tests were between 2 to 3. MQ is an important parameter used to assess the CAPWAP analysis qualification (Fellenius and Altaee, 2002). While there isn't a fixed upper or lower limit for an acceptable CAPWAP result, an examination of data from driven piles with well-defined properties typically yields Modal Quality (MQ) values between 2 and 4. It's important to note that smaller values are infrequent, and the acceptability of a specific value depends on the circumstances of the situation. However, MQ values exceeding 6 may suggest incomplete analysis, poor data quality, or a significant issue with pile integrity. In such cases, the results may be deemed unreliable (Rausche et al., 1972).

The variation of pile bearing capacity (i.e., R_{BOR}/R_{EOD}) with time is shown in Figure 6-2 based on PDA tests. Although pile resistance increased at BOR condition in all soils, the effect of pile setup was more significant in piles driven in cohesive soil.

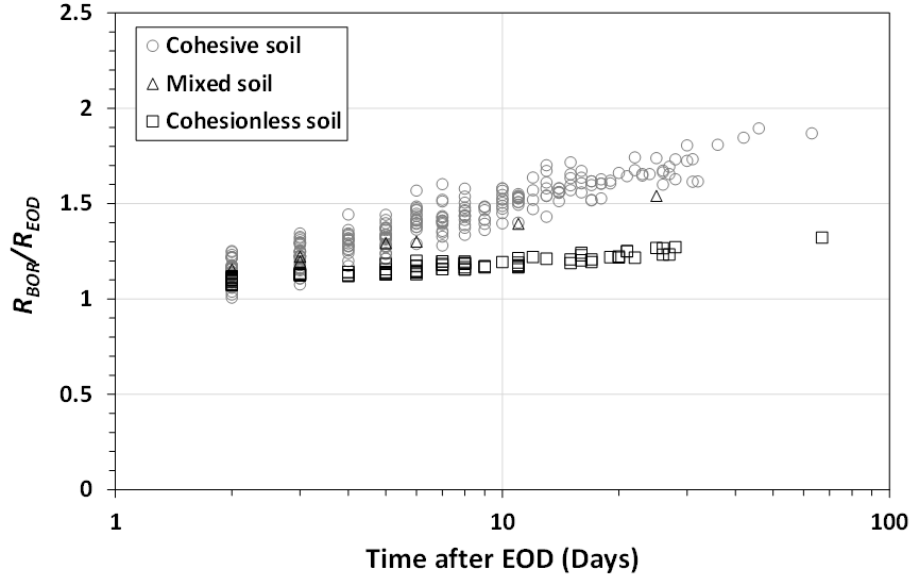


Figure 6.2: Variation of pile bearing capacity ratio with time based on PDA test results

As previously described, the most reliable method for measuring pile bearing capacity is SLT. Based on the gathered database, twenty-eight (28) SLTs were conducted to validate the PDA test procedures and CAPWAP analysis. Figure 6-3 shows the comparison between pile bearing capacities measured by SLTs using the Davisson offset limit criterion (Davisson, 1972) and PDA tests using the CAPWAP method (Likins et al., 2000).

To improve the reliability level of this study, it is necessary to transform the results of PDA into SLT considering the relation between PDA and SLT results. In this way, the transformed sample bias, $(\lambda_R)_T$, and coefficient of variation, $((COV_R)_T)$, between the static analysis method and SLT capacity of individual piles can be estimated as follows:

$$(\lambda_R)_T = \lambda_R \cdot \bar{\lambda}_{SLT-PDA}, \quad (6.16)$$

$$((COV_R)_T) = \sqrt{COV_R^2 + COV_{SLT-PDA}^2} \quad (6.17)$$

where λ_R and COV_R are the mean and COV of the sample bias of pile resistance calculated by the static analysis method and PDA tests, and $\bar{\lambda}_{SLT-PDA}$ and $COV_{SLT-PDA}$ are the mean and COV of the bias of PDA and SLT capacities. Based on the results, a very good correlation can be observed between the two in-situ test results with a relatively high regression coefficient (i.e., $R^2 = 0.917$). It can be inferred that by applying the bias

factor $\bar{\lambda}_{SLT-PDA}$, the PDA results can be considered as the measured ultimate bearing capacity of the driven piles. $\bar{\lambda}_{SLT-PDA}$ bias factor measured for this study is 1.223. Similar compatibility reports between PDA and SLT have been reported by Fellenius and Altaee (2002), Reddy and Stuedlein (2013), Thomson et al. (2017), and Asghari Pari et al. (2019).

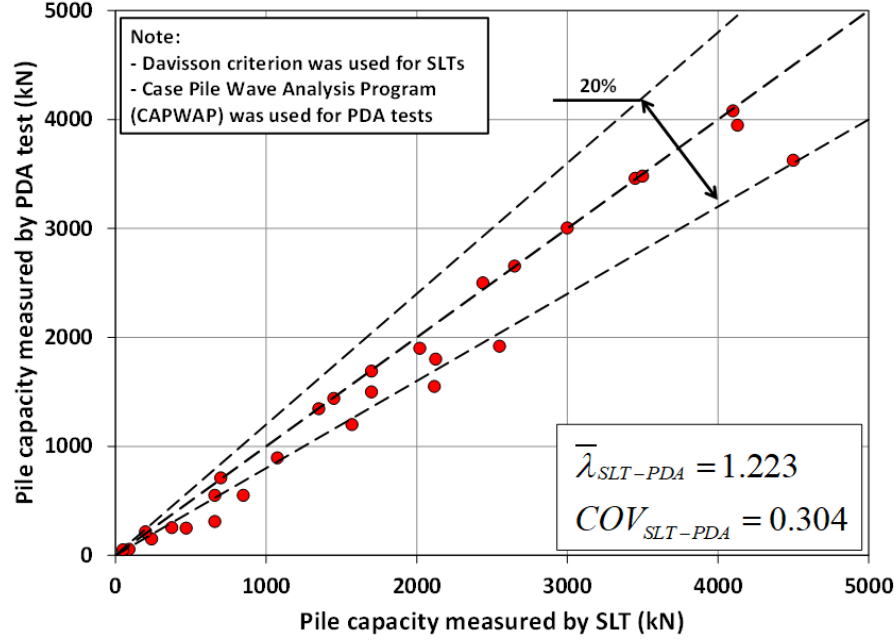


Figure 6.3: Correlation between pile resistances measured by PDA and SL tests

The α -method, β -method, and α/β -method described in CFEM (2006) were used to calculate pile capacities in cohesive, cohesionless, and mixed soil types, respectively, as the predicted pile capacity at EOD condition. To obtain pile capacity at BOR condition, the relationship recommended by Skov and Denver (1988) (Eq. (1)) was used. In this regard, the parameter A was taken to be 0.6, 0.2, and 0.4 for cohesive, cohesionless, and mixed soil types, respectively, and t_0 was assumed to be 1, 0.5, and 0.75 days for piles driven in cohesive, cohesionless, and mixed soils, respectively (Skov and Denver, 1988). A comparison between the bearing capacity of piles measured by PDA tests and predicted by the static analysis at EOD and BOR conditions is presented in **Figure 6-4**.

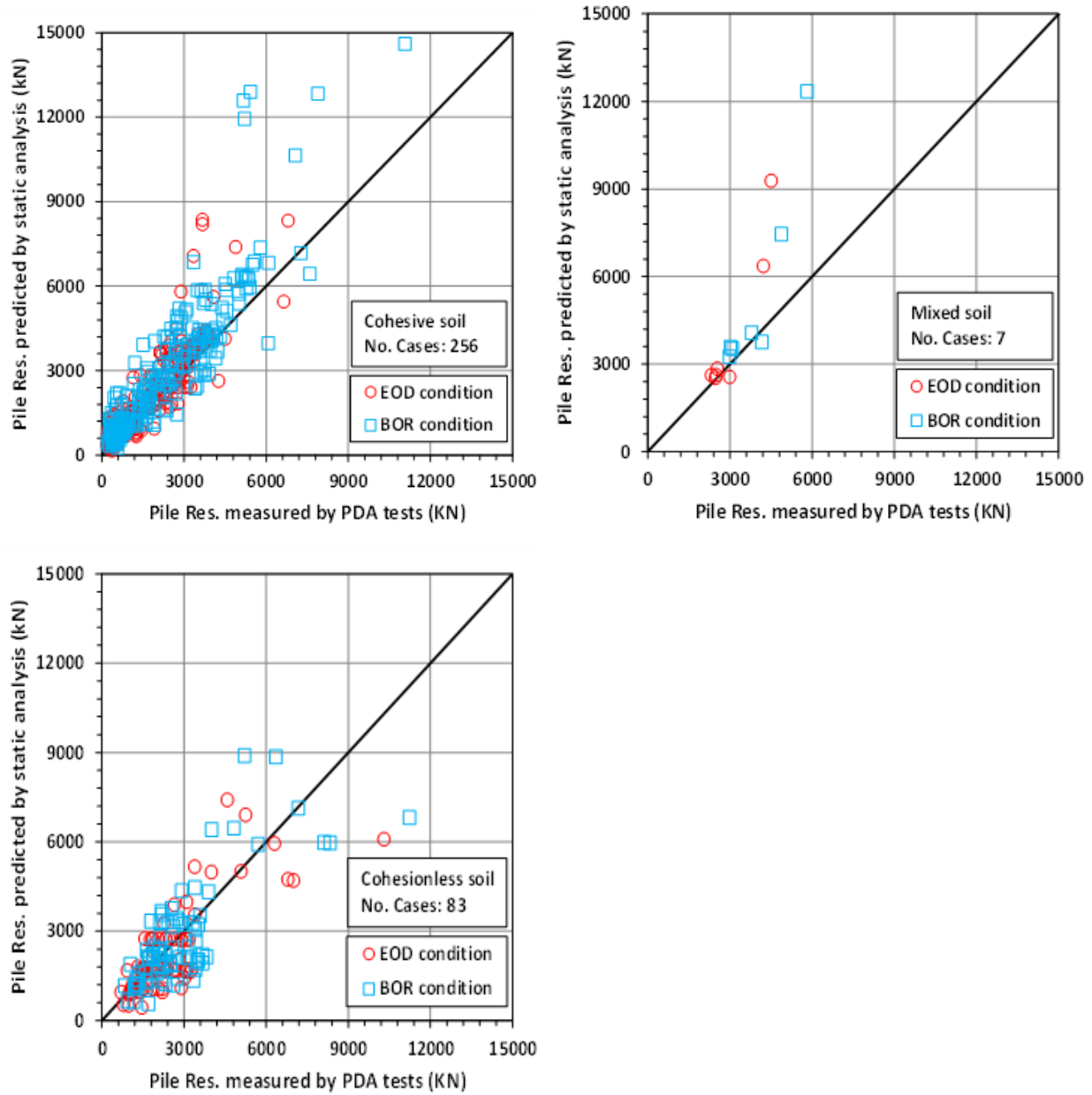
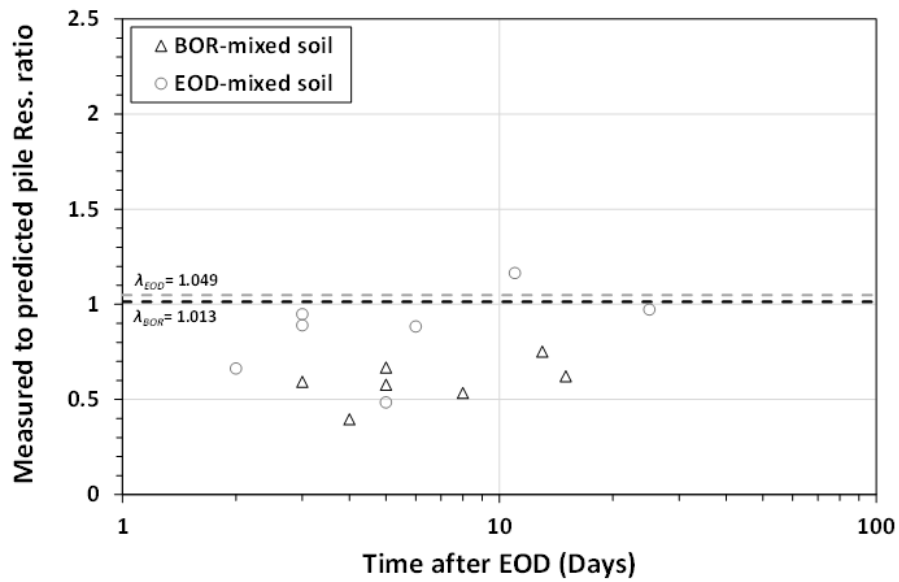
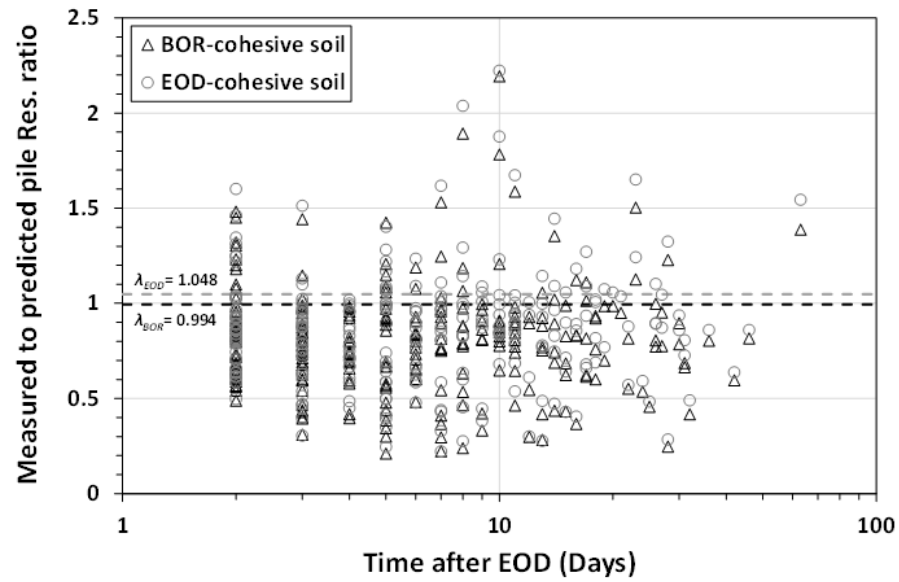


Figure 6.4: Comparison between pile capacities measured by PDA tests and predicted by static analysis at EOD and BOR conditions

The variation of pile resistances with time for piles driven in cohesive, mixed, and

cohesionless soils is illustrated in Figure 6-5. It can be concluded that the mean of bias factors was lower for pile resistance in the BOR condition. Similar results were reported by Jabo (2014) and Asghari Pari et al. (2019).



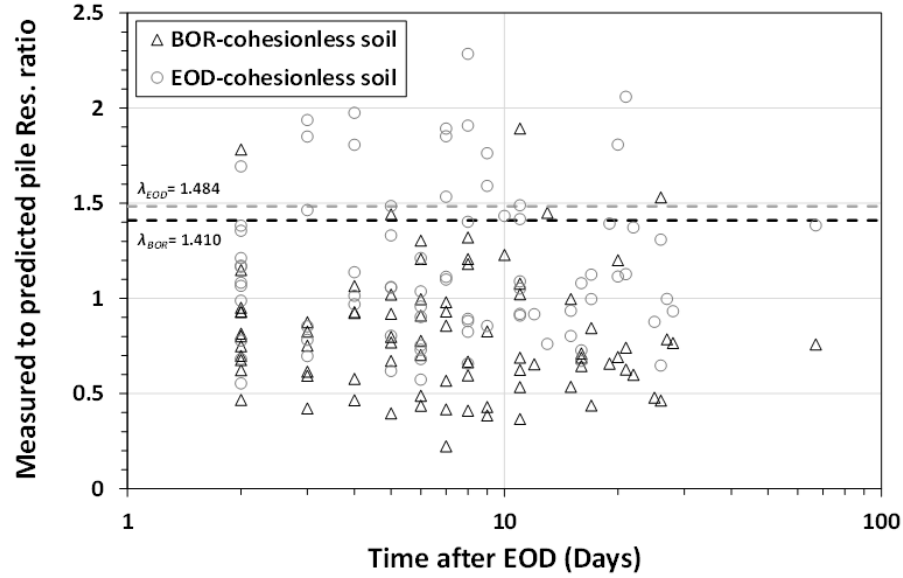


Figure 6.5: Pile resistance variations with time in (a) cohesive soil, (b) mixed soil and (c) cohesion-less soil

6.5.1 Single GRF Value

The resistance bias factors, including λ_{EOD} and λ_{BOR} , were calculated for all piles based on measured and predicted pile resistances using the results of PDA tests and static analysis, respectively. The Anderson and Darling (AD) test (Anderson and Darling, 1954) was used to examine the goodness of fit for the assumed distribution function of the bias factors. The best distribution function for resistance bias factors was the log-normal distribution because the AD values of this type of distribution were lower than those of the normal distribution.

Figure 6-6 illustrates the variation of the reliability index and the GRF for piles at BOR condition. Regardless of soil types, the GRF values of driven piles at BOR condition decreased with increasing β values (i.e., decreasing the probability of failure). When the reliability index increased from 1 to 4, the GRF values decreased by approximately 70%, 64%, and 74% for piles driven in cohesive, mixed, and cohesionless soils, respectively. It can be concluded that the resistance factor of steel driven piles is very sensitive to the desired reliability index.

Based on the recommendation of Paikowsky et al. (2004), the β values for the evaluation of the bearing capacity of single piles can be considered as $\beta = 2.33$ (i.e., $p_f = 1\%$) for

redundant piles and $\beta = 3.00$ (i.e., $p_f = 0.1\%$) for nonredundant piles. The statistical properties of the resistance bias factors and a summary of the calculated single GRFs for $\beta = 2.33$ and 3.00 at BOR condition are presented in **Figure 6-6**. The GRF values for piles driven in cohesionless soil were greater than those for piles in cohesive soil at BOR condition.

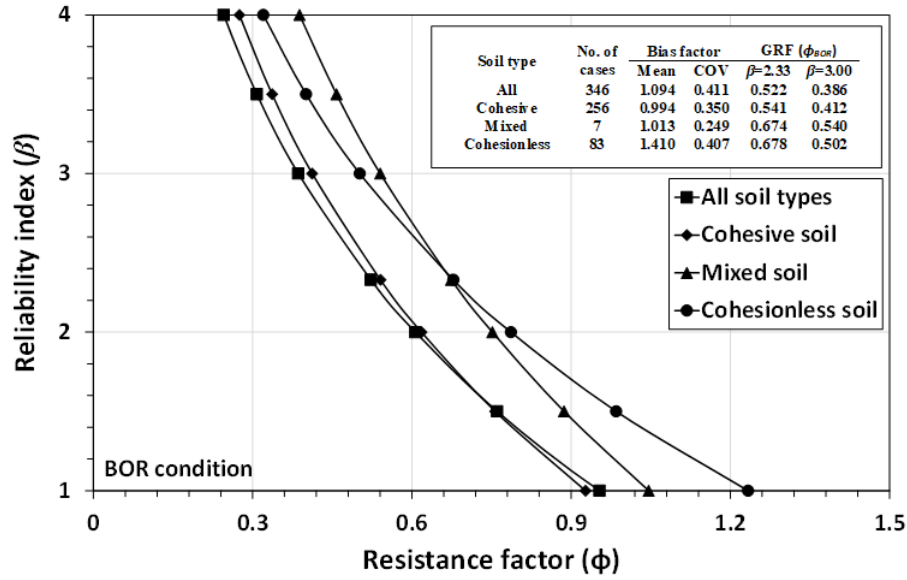


Figure 6.6: Relationship between the reliability index and resistance factor based on single GRF for piles in (a) all soil types, (b) cohesive soil, (c) mixed soil, and (d) cohesionless soil.

6.5.2 Separate GRF Values

Considering the varying degrees of uncertainty in pile bearing capacity in the End-of-Drive (EOD) condition and setup effect, as highlighted by Yang and Liang (2009) and Bian et al. (2021), it is logical to utilize distinct GRF values in the Load and Resistance Factor Design method. As previously mentioned, the Acceptance Criteria Ratio (ACR) developed by Bach et al. (2014) should be applied to determine an acceptable pair of ϕ_{EOD} and ϕ_{SET} . An example of the accepted pair for cohesive soils is illustrated in **Figure 6-7**.

The relationship between resistance factors (ϕ_{EOD} and ϕ_{SET}) and the reliability index (β) for piles driven in various soil types is depicted in **Figure 6-8**. Similar to findings with single GRFs, an increase in β results in a decrease in both ϕ_{EOD} and ϕ_{SET} .

For piles in clayey soils, the separate GRFs for pile resistance at EOD condition and setup effect were 0.683 and 0.461 (for $\beta = 2.33$) and 0.591 and 0.403 (for $\beta = 3.00$), respectively. In mixed soils, the ϕ_{EOD} and ϕ_{SET} values were 0.760 and 0.695 for $\beta = 2.33$ and 0.671 and 0.612 for $\beta = 3.00$. For piles in cohesionless soils, the ϕ_{EOD} and ϕ_{SET} were 0.851 and 0.628 for $\beta = 2.33$ and 0.667 and 0.502 for $\beta = 3.00$, respectively.

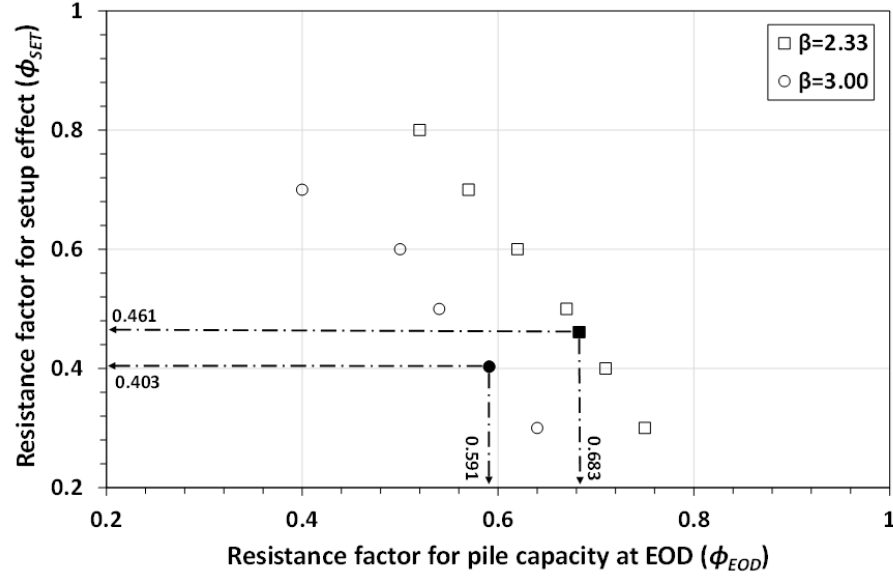


Figure 6.7: Relation between ϕ_{SET} and ϕ_{EOD} for piles driven in cohesive soils and the accepted pairs based on ACR.

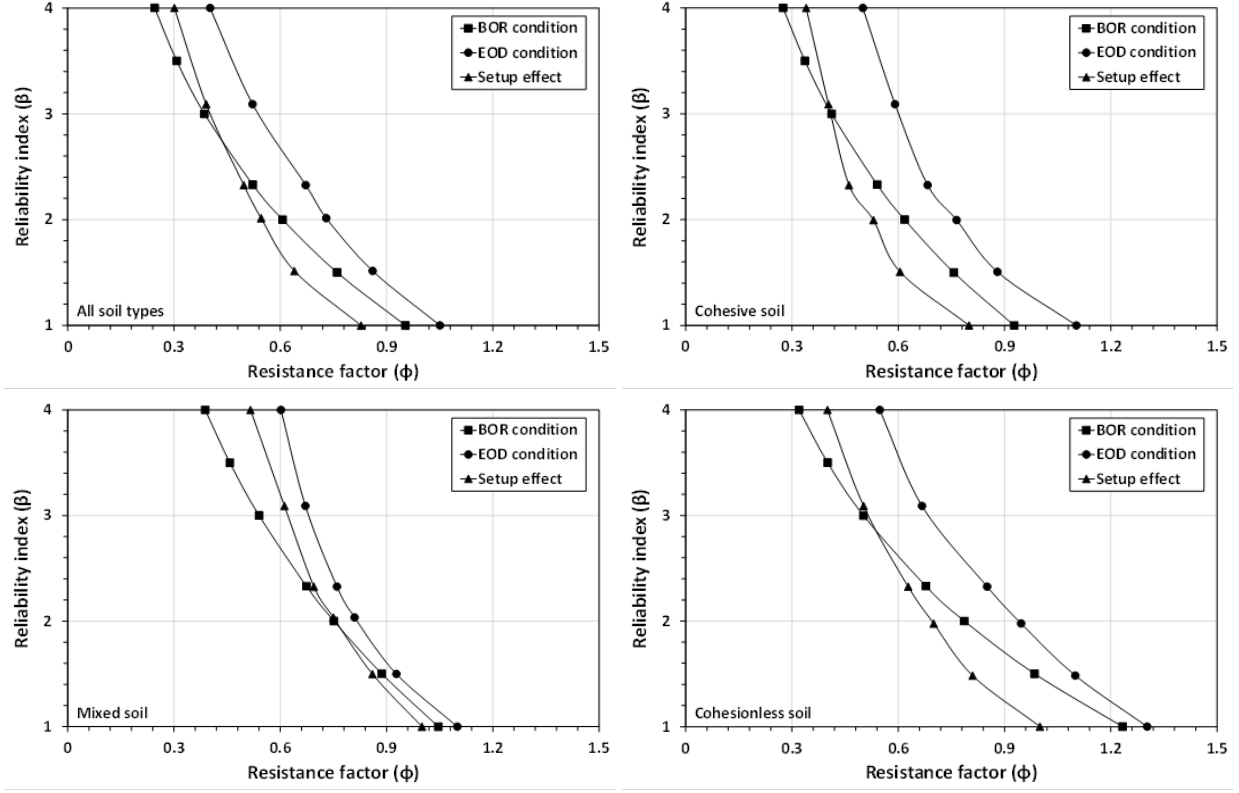


Figure 6.8: Relationship between the reliability index and resistance factor based on single GGRF at BOR and separate GFRs for EOD plus setup method for piles in (a) all soil types, (b) cohesive soil, (c) mixed soil and (d) cohesionless soil

6.6 Discussion

The variation of GRF values, including ϕ_{BOR} , ϕ_{EOD} , and ϕ_{SET} , based on soil conditions is presented in **Figure 6-9**. As can be seen, the GRF of EOD is higher than the GRF of BOR, which is mainly due to the decrement of the bias factor from EOD to BOR condition as previously presented in **Figure 6-5**.

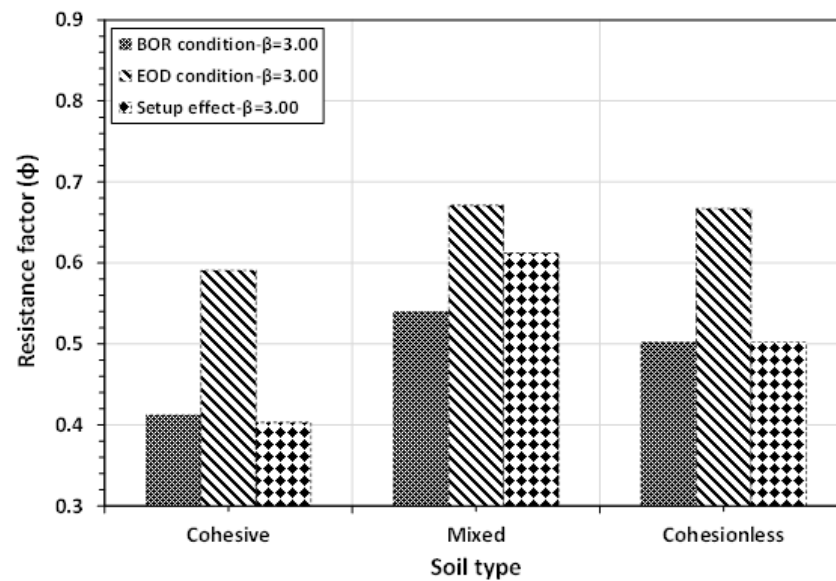
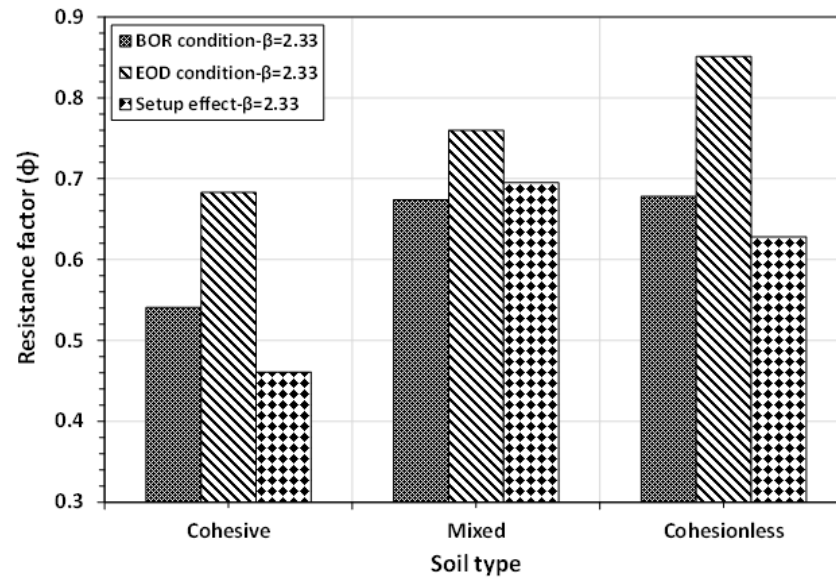


Figure 6.9: Comparison of GRFs based on bearing capacity type and soil condition.

The GRF values of EOD and setup effect are calculated based on the separate GRFs method developed by Bach et al. (2014). The main reason is that the calculated GRFs by

the separate GRFs method are coupled together, and their effectiveness on ASD should be measured by comparing the ASD based on GRF of BOR and GRFs of EOD and setup effect. In addition, **Figure 6-9** reveals that the GRFs of piles driven in cohesive materials are lower than those of piles in cohesionless soils. The main reason is the uncertain effect of EPWP (excess pore water pressure), which is commonly developed during the construction procedure of driven piles, and the setup effect (including nonlinear and linear EPWP dissipation phases) has some uncertainties. Therefore, the behavior of piles after the installation process is less reliable in cohesive soils. Yang and Liang (2009) calculated the GRFs of EOD and setup effect as 0.65 and 0.5 for $\beta = 2.33$ and 0.5 and 0.4 for $\beta = 3.00$ using the FORM calibration technique. Jesswein et al. (2021) calibrated the GRFs of EOD and setup effect based on a database of 52 PDA tests on driven steel piles constructed in cohesive deposits of Alberta. Similar to the current study, they demonstrated higher GRF values for EOD in comparison to GRF of setup effect for reliability indices of 2.33, 3, and 3.5. Ng and Sritharan (2016) calculated GRFs of EOD and setup effect as 0.58 and 0.27 for $\beta = 2.33$ and 0.45 and 0.2 for $\beta = 3.00$ using the FOSM calibration technique. Their results based on FORM techniques were approximately 10% higher. To compare the effectiveness of using separate GRF values for pile resistance in EOD condition and setup effect with using a single GRF for pile capacity during the pile design process incorporating setup effect, the factored total resistances (also known as ASD) calculated by the single GRF (i.e., $\phi_{BOR} \cdot R_{BOR}$) and separate GRFs (i.e., $\phi_{EOD} \cdot R_{EOD} + \phi_{SET} \cdot R_{SET}$) were compared in **Figure 6-10**. Based on the results, using separate GRF values resulted in a higher factored pile resistance in all soils, leading to a more cost-effective design process. Higher improvement in factored resistance was observed in higher reliability indices. For example, when the reliability index increased from $\beta = 2.33$ to 3.00, the factored total resistance calculated by separate GRFs improved from 13% to 20% for piles driven in cohesionless soil. In addition, the separation method showed a higher effect on the factored resistance of piles in cohesive soil. For instance, in $\beta = 2.33$, the factored total resistance improved by approximately 18% in cohesive soil, while this parameter was increased by approximately 13% for piles driven in cohesionless soil. It means that the setup effect is more significant in piles driven in cohesive soils. The lowest improvement in factored total resistance was observed for piles driven in mixed soil (8% in $\beta = 2.33$ and 14% in $\beta = 3.00$). As the number of cases for the calibration process of piles in mixed soil was low (i.e., 7 cases), the GRF values should be used with caution in this condition. Similar results were reported by Ng et al. (2010) based on valuable research on setup effects on the bearing capacity of piles constructed in clayey, mixed, and sandy soils of Iowa. Ng et al. (2010) stated that the setup time effects cause a considerable increase in the ultimate bearing capacity of piles, and a greater influence of setup was observed for piles embedded in clay and mixed soils than for piles embedded in sand.

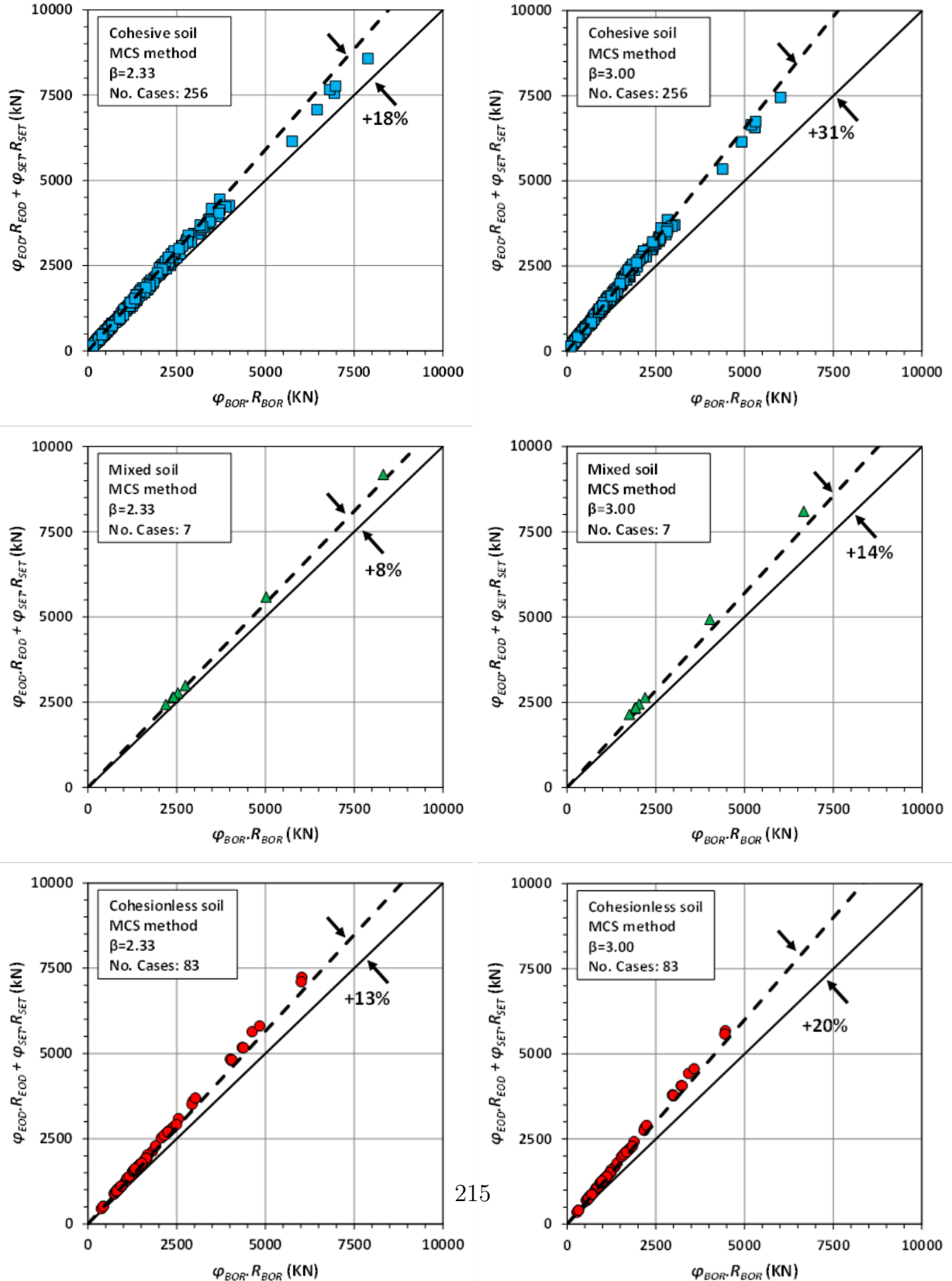


Figure 6.10: Comparison of factored total resistances calculated by separate GRFs for piles driven in (a) cohesive soil, (b) mixed soil and (c) cohesionless soil

6.7 Conclusions

The paper focuses on LRFD-based resistance factor calibrations for steel driven piles, with an emphasis on incorporating pile setup in the design process to enhance and realize the pile bearing capacity estimation. To this end, a database of 346 driven steel pipe piles constructed in Alberta was used. Based on previous studies, a logarithmic relationship proposed by Skov and Denver (1988) was adopted to calculate the load capacity gained due to the setup effect. A novel approach was used to calibrate the separate resistance factors for bearing capacity at the EOD condition and the capacity gained due to setup time effects. The following conclusions can be obtained from the results of this study:

- When compared to the LRFD-based calibrated results for the EOD condition plus setup time effect with the recommendation proposed by CFEM (2006), the newly developed separate resistance factors of EOD and setup effects have improved the efficiency of the steel driven pile foundation design in cohesive, mixed, and cohesionless soils.
- By comparing the LRFD calibrations between the BOR and the EOD plus setup, the resistance factor is improved for EOD plus setup due to the efficiency of the newly developed calibration method which resulted in separate GRFs for EOD and setup.
- The results of this study are based on static analysis methods proposed by CFEM (2006). It should be noted that the calibrated GRFs will vary by the selected design or static analysis method, which likely have different model uncertainties.
- Considering the single GRF approach for BOR, at $\beta = 2.33$, the GRF values of the ultimate capacity were obtained as 0.541, 0.674, and 0.678 for piles driven in cohesive, mixed, and cohesionless soils, respectively. For $\beta = 3.00$, the GRF values were 0.412, 0.540, and 0.502 for piles driven in cohesive, cohesionless, and mixed soils, respectively.
- Considering the separate GRFs approach incorporating setup effect, at $\beta = 2.33$, the GRF values of EOD were 0.683, 0.760, and 0.851, and GRFs of setup were 0.461, 0.695, and 0.628 for piles driven in cohesive, cohesionless, and mixed soils, respectively. Similarly, for $\beta = 3.00$, the values of ϕ_{EOD} were 0.591, 0.671, and 0.667, and ϕ_{SET} values were 0.403, 0.612, and 0.502 for piles driven in cohesive, cohesionless, and mixed soils, respectively.
- When compared to the LRFD calibrated results for the EOD plus setup with the recommendation proposed by CFEM (2006) or other regulations, the separate GRF

method using the gathered database of constructed piles of Alberta, has improved the efficiency of the steel pipe pile foundation design.

- Using separate GRF values resulted in a higher factored pile resistance in all soils, leading to a more cost-effective design process. In addition, setup effect was more significant in piles driven in cohesive soils. At $\beta = 2.33$, the factored total resistance improved by approximately 18% in cohesive soil, while this parameter was increased by approximately 13% for piles driven in cohesionless soil.

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Chapter 7

Cost-Benefit-Analysis

7.1 Introduction

Driven steel pile foundations support most bridge structures in Alberta. Most bridge pile foundations consist of driven pipe or H-Pile or timber piles. During preliminary design, the ultimate axial load capacity of piles is usually estimated using semi-empirical equations based on static analysis methods and soil mechanics principles (Tschebotarioff, 1973). The allowable or work stress on each pile is then determined by applying a factor of safety to the ultimate pile capacity. Soil parameters used in these calculations are estimated using engineering judgment, laboratory testing, and correlations with in-situ test results (usually SPT or CPT-based).

In the case of driven piles, the prediction of pile drivability done prior to construction is made using engineering judgment, empirical energy equations, pilot piles, and wave equation analysis (WEAP programs). In the case of driven piles, the load capacity is estimated using driving resistance formulae often supplemented with set and compression values provided from the pile monitoring program. On rare occasions, Alberta Transportation has used full-scale load tests to directly measure pile load capacity. High strain dynamic testing, such as the Pile Dynamic Analysis (PDA), is now used routinely on department projects and has provided a cost-effective means of estimating pile capacity and near real-time monitoring of construction QA/QC. Other pile capacity ‘measurement’ techniques such as the Osterberg Cell and Statnamic testing are now accepted in Alberta industry and are being introduced for use on large-scale Alberta Transportation projects.

In concurrence with the introduction of the PDA method, major agencies are moving away from allowable stress design (ASD) or work stress design (WSD) methods for

substructure design and away from load factor design (LFD) methods for superstructure design. The trend in design methodology is toward the use of Limit States Design (LSD) methods (FHWA, 2001). During this current transition period, many agencies continue to use the WSD methodology and determine allowable loads by application of a Factor of Safety. The LSD method is based on Load and Resistance Factor Design (LRFD) which uses geotechnical resistance factors (GRF). In the absence of what might be termed codified GRFs that are specific to Alberta soils and bedrocks, many practitioners in essence use a GRF that is the inverse of a familiar Factor of Safety. It is expected that local or regional GRF values will be developed as the LSD method matures in Canada.

In relation to pile designs based on semi-empirical equations, there are clear incentives to base designs on more reliable and defensible design methods. Accurate estimation of the increase in pile resistance with time can be incorporated into a rational design by means of reducing the number of piles, shortening pile lengths, reducing pile cross-sectional area (with use of smaller-diameter piles), or reducing the size of driving equipment (use of smaller hammers or cranes). Incorporating any or a combination of these benefits can result in cost reductions and savings in pile design.

Sellountou and Roberts (2007) investigated the cost-effectiveness of dynamic pile installation using a case study. The results showed that dynamic testing such as PDA can be used in combination with static load testing to reduce the number of static load tests through establishing correlations between the two. Significant cost savings can be obtained particularly for large project sites with variable soil conditions requiring multiple static load tests. The inclusion of restrrike driving tests, CAPWAP analysis of the collected PDA data, and the correlation of dynamic PDA testing results with static compression load tests can significantly improve the value of dynamic testing and the acceptance of dynamic testing results. A well-planned pile load test program that includes both static and dynamic load tests, completed prior to the final pile design and start of production pile driving, can reduce the uncertainties associated with pile driving and can result in significant cost savings by improving the efficiency of the final pile design compared to pile designs based on static capacity calculations alone.

Skirrow and Wang (2008) provided a study on cost-benefit analysis for pile projects. In this study, 5 different methods during pile design have been considered as follows:

- Method 1 - Conventional Monitoring, e.g., Gates, with an assigned resistance factor 0.35.
- Method 2 - Conventional Monitoring w/Field Measurements, with an assigned resistance factor 0.40.

- Method 3 - Dynamic Test, e.g., PDA, with an assigned resistance factor 0.50.
- Method 4 - Dyn/Static Test, e.g., Statnamic, with an assigned resistance factor 0.55.
- Method 5 - Static Test, e.g., classic load test or O-Cell, with an assigned resistance factor 0.60.

Based on the results for typical small piling projects of 10 or fewer piles, there may be no justification for advanced pile capacity monitoring programs beyond field monitoring and conventional good construction practices. As more piles are involved at a site, the optimization process suggests that more elaborate pile capacity methods provide better value. The decision of which test method to be used during pile monitoring should be considered during the design phase of the project, since it is easier to adjust designs at that stage than during construction. Significant piling project cost savings can be achieved using a well-planned and statistically defensible testing program.

Abu-Farsakh et al. (2017) evaluated the effect of using setup effect on the bearing capacity of pile foundations. A cost-benefit analysis demonstrated that the successful incorporation of long-term pile setup resistance in the design of pile foundations at the Bayou Boeuf Bridge site could have resulted in an average shortening of 18 ft (5.5 m) per pile with a total of 10,188 ft (3,106 m) for all the piles in the project. This reduction in pile length could have led to net cost savings of \$612,638.

7.1.1 Cost Saving Analysis

As different analyses have been performed during this research, it is important to evaluate their effectiveness on the cost of construction projects. In this research, the database of 16 projects has been gathered. These projects are located in different zones of Alberta. The cost analysis has been conducted based on the results that were achieved at Chapters 4, 5, and 6. The cost saving calculation has been considered based on the length of pile reductions in each scenario. The installation cost per linear meter of steel driven pile including materials, labor, and equipment has been considered approximately \$1000/m (Skirrow and Wang, 2008). **Figures 7-1 to 7-3** indicate summary of cost savings based on these scenarios at each condition.

In cost analysis, 6 conditions were considered as follows:

- **Condition 1:** Performing “Static Analysis” without any PDA or Static Load Test (SLT) results. In this condition, the GRF value is equal to 0.4 based on CFEM.

- **Condition 2:** Performing “Static Analysis” with PDA results on 10% of constructed piles. In this condition, the GRF value is equal to 0.5 based on CFEM.
- **Condition 3:** Performing “Static Analysis” with SLT results on 1% of constructed piles. In this condition, the GRF value is equal to 0.6 based on CFEM.
- **Condition 4:** Performing “Static Analysis” using the means and methods, and conclusions of Chapter 4. In this condition, the GRF value is a calibrated single GRF at EOD condition which is presented in above parts of this report.
- **Condition 5:** Performing “Static Analysis” using the means and methods, and conclusions of Chapter 5. In this condition, the GRF values are two new separate GRFs at EOD condition which are presented in above parts of this report.
- **Condition 6:** Performing “Static Analysis” using the means and methods, and conclusions of Chapter 6. In this condition, the GRF value is a new single GRF based on soil type at BOR condition which is presented in above parts of this report.

The GRFs values for each condition have been calibrated based on the following scenarios:

1. Soil types: Cohesive, Mixed, Cohesionless
2. Regionally: NE, NW, SW, SE, C of Alberta
3. Locally: Based on project site (12 sites)

For cost analyses, the prices of different steps of construction were obtained from a study performed by Skirrow and Wang (2008) and the average Canada inflation rate history from 2008 to 2024 was considered as 2% to make the prices real. Table 7.1 presents the prices considered for “Setup Cost,” “Pile Supply,” and “Driving Cost.” In the cost analysis part of this study, it is assumed that the number of required piles is constant for all the C1 to C6 conditions, and it is tried to calculate the total and average pile lengths considering the total applied loads due to superstructures. Figures 7.1 to 7.3 present the variation of construction cost saving versus initial average pile length.

Table 7.1: Prices of construction steps based on Skirrow and Wang (2008) with 2% annual inflation (2008-2024).

Setup Cost (\$/Pile)	Pile Supply (\$/m)	Driving Cost (\$/m)
4855.9	254.4	103.6

Cost savings for various conditions in cohesive, mixed, and cohesionless soils are shown in Figures 7.1 to 7.3. The results indicate that using GRFs based on BOR conditions provides the highest cost savings due to reductions in total and average pile lengths. Setup effects were found to be more significant in cohesive soils, as previously reported by Haque and Abu-Farsakh (2018), Chow et al. (1998), and Axelsson (2002).

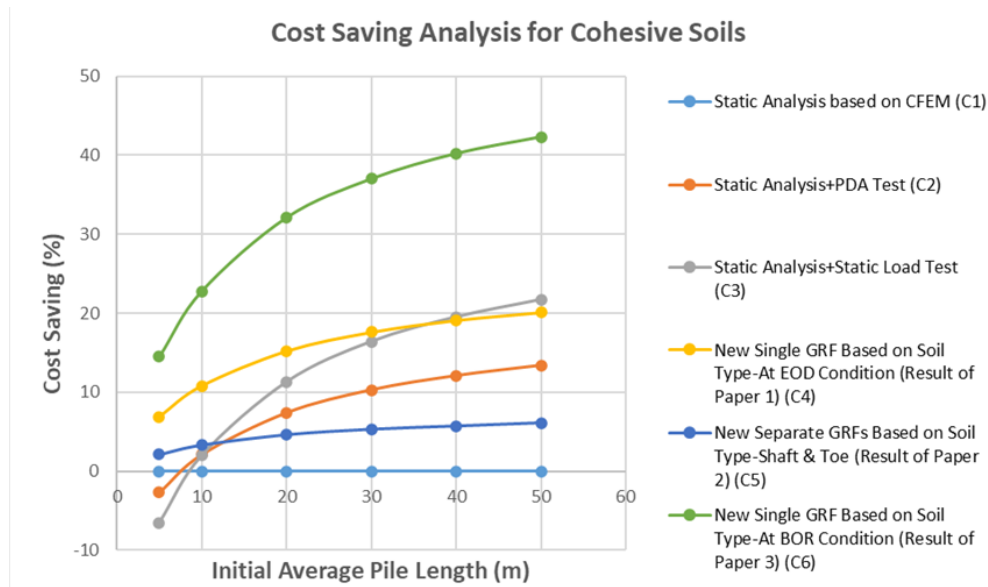


Figure 7.1: Cost Saving percentage of different conditions in comparison to static analysis based on CFEM (condition 1) for piles with different lengths in Cohesive soils

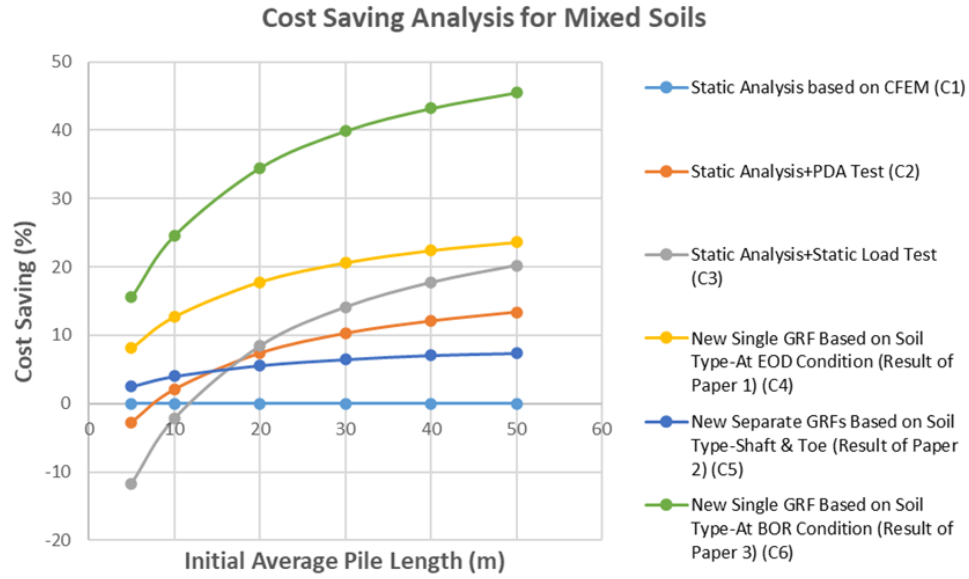


Figure 7.2: Cost Saving percentage of different conditions in comparison to static analysis based on CFEM (condition 1) for piles with different lengths in Mixed soils

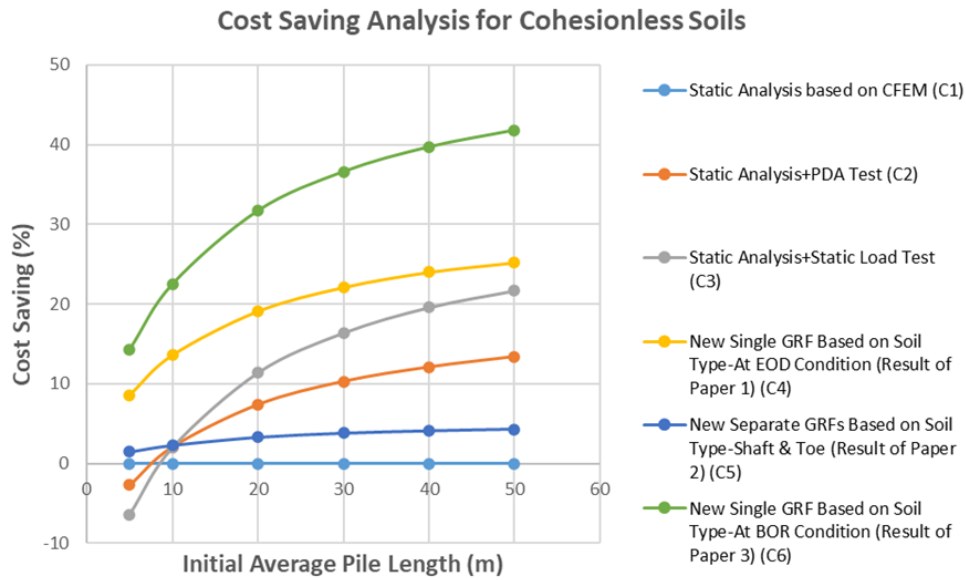


Figure 7.3: Cost Saving percentage of different conditions in comparison to static analysis based on CFEM (condition 1) for piles with different lengths in Cohesionless soils

In these graphs, the number of required piles was assumed constant, and the total length of the piles was calculated based on different conditions as described above. As can be seen, using the GRF based on BOR condition causes a considerable cost saving due to the reduction of total and average pile lengths. The increase of pile resistance due to pile setup was reported by previous studies. Haque and Abu-Farsakh (2018) stated that the increase in pile resistance due to the setup effect occurs in all soils, but it is more significant in cohesive soils. Analyzing the outcomes of 11 cases involving piles driven into sand, Chow et al. (1998) observed an approximately 50% increase in pile capacities per logarithmic cycle of time starting from 1 day after pile driving. Similarly, Axelsson (2002) reported a setup effect ranging from 15% to 65% per logarithmic cycle of time. Previous studies indicate that numerous factors, including geological and geotechnical conditions, the pile installation process, the type and timing of in-situ loading tests, analysis methods, etc., influence the setup effect (Hosseini, 2015; Abu-Farsakh and Haque, 2018; Haque and Abu-Farsakh, 2019).

This underscores the significant importance of employing a local database for calibrating GRF values. Based on the results, using separate GRF values resulted in a higher factored pile resistance in all soils, leading to a more cost-effective design process. Higher improvement in factored resistance was observed in a higher reliability index. Therefore, it can be concluded that using GRF at BOR condition has the 1st rank among the studied conditions and has the highest cost saving rate. After the GRF at BOR condition, using the GRF at EOD condition has the 2nd rank among the studied methods, and it can considerably reduce the construction cost of driven piles.

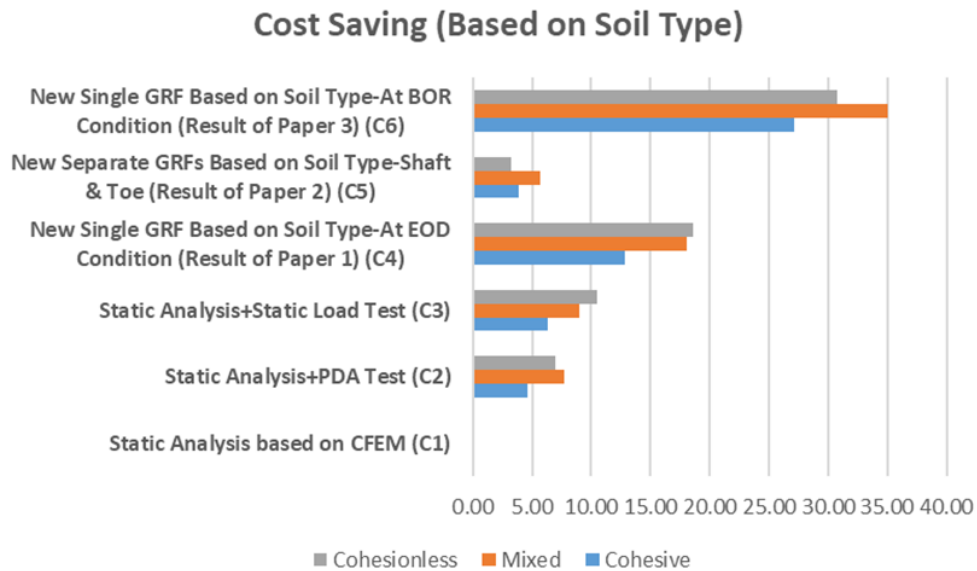


Figure 7.4: Cost Saving percentage of different conditions in comparison to static analysis based on CFEM (condition 1) for different soil types.

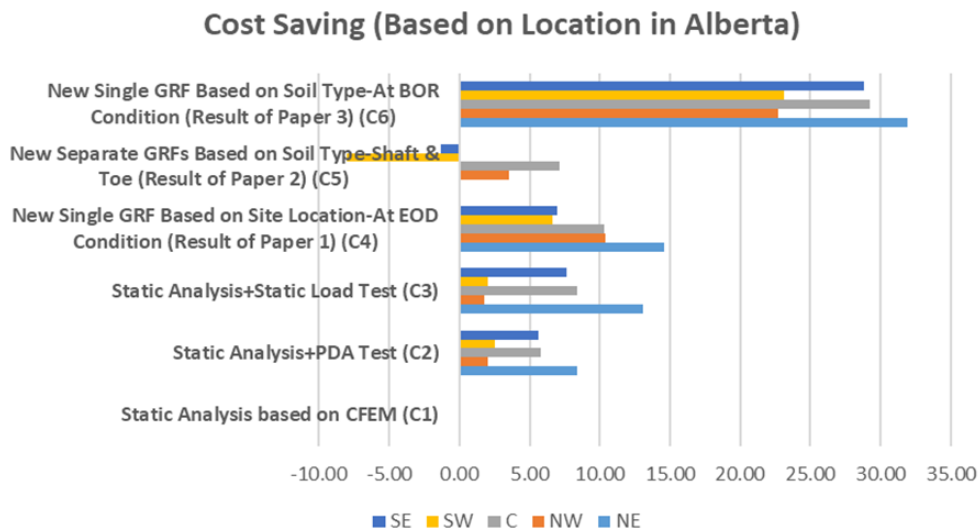


Figure 7.5: Cost Saving percentage of different conditions in comparison to static analysis based on CFEM (condition 1) for different locations in Alberta.

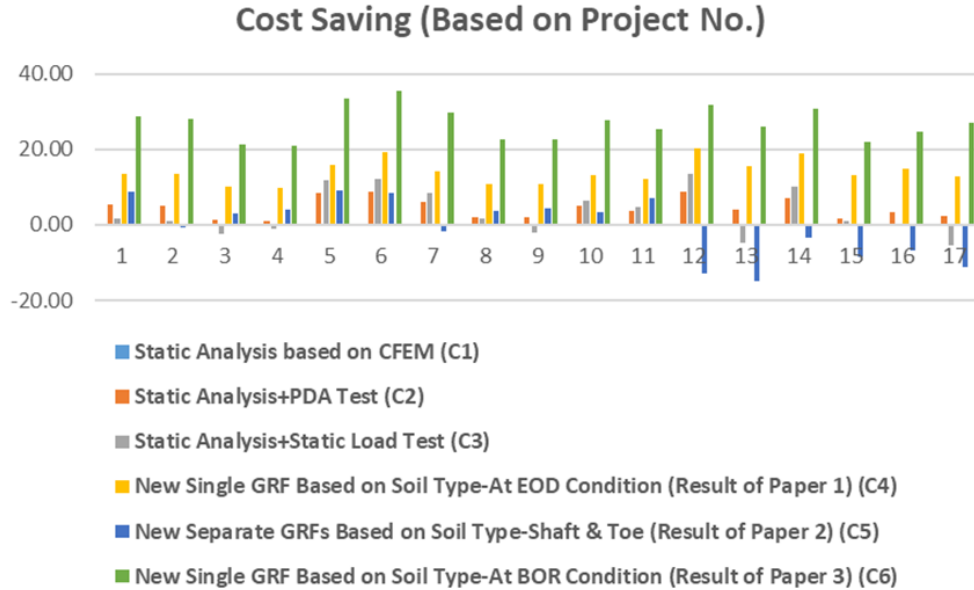


Figure 7.6: Cost Saving percentage of different conditions in comparison to static analysis based on CFEM (condition 1) for different projects (Locally).

As stated by Skirrow and Wang (2008), using higher GRF values associated with PDA tests or higher-order pile capacity measuring methods can result in significant project cost savings. Performing the cost analysis, the following conclusions are obtained:

- Based on above results, it is obvious that the results obtained based on the GRF at BOR condition, present the best condition for cost reduction. Using the GRF at BOR condition can reduce the construction cost from 23% up to 35%. As demonstrated earlier, the capacity increase due to setup is significant and worth including during the design stage of the construction projects using the driven steel piles. It can be concluded that using the regionally calibrated GRF at the BOR condition in Alberta can approximately reduce the construction cost by about 30% compared to the use of $GRF = 0.4$ for static analysis methods (such as α -method, β -method, etc. that are presented in CFEM (2008)). It should be considered that due to lack of sufficient data about the effect of setup time on variation of steel driven piles bearing capacity, this efficiency may increase further, and it is necessary to gather a more comprehensive database for further studies on this topic.
- Based on above results, it is obvious that the results obtained based on the GRF at the EOD condition, has the 2nd rank in cost saving conditions. Using the GRF

at EOD condition can reduce the construction cost from 13% up to 18%. It can be concluded that using the regionally calibrated GRF at EOD condition in Alberta can reduce construction costs by approximately 15% compared to using $\text{GRF} = 0.4$ for static analysis methods (such as α -method, β -method, etc. which are presented in CFEM (2008)).

- Based on above results, it is obvious that the results obtained based on the $\text{GRF} = 0.6$ in condition of using the static load test (SLT), has the 3rd rank in cost saving conditions. Due to the high cost of the SLT, this saving is achievable in projects with a considerable number of piles.
- The results showed that the use of separate GRFs for the shaft and toe of the driven steel piles shows a cost saving for 3 different soil types of about 4%. Evaluation of the effects of separate GRFs on 5 sites in Alberta and on 16 projects showed that the use of these GRFs under certain conditions can reduce the cost of construction and under other conditions it can increase the cost. Therefore, the effects of the separate GRFs and the toe and shaft parts on the actual bearing capacity of steel driven piles need to be further investigated.
- The results also showed that using the GRF values locally will generally provide more cost reduction (30% to 40%) compared with GRF values that calculated regionally (22% to 32%) and GRF values that calculated based on soil types (26% to 35%).

7.2 References

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Chapter 8

Summary and Conclusion

8.1 Summary and Conclusion

The following points provide summaries of this research of calibration results for axially loaded steel driven piles in Alberta, Canada:

1. Dataset Collection:

- The dataset consists of 659 PDA test results at End of Drive (EOD) and 324 Re-strike results collected from 16 projects across Alberta, including both oil and gas as well as infrastructure works, covering a wide range of geological and geotechnical conditions. Alberta's geology was classified into six regions—Southwest, Southeast, Central, Northwest, Northeast, and the Rocky Mountains & Foot-hills—although the latter was excluded due to insufficient data.
- Geotechnical characterization included Cone Penetration Tests (CPT), pocket penetrometer measurements, unconfined compressive strength tests, and Atterberg limits for fine-grained soils, along with Standard Penetration Tests (SPT) for coarse-grained soils. These parameters were used in semi-empirical capacity evaluations for each project. All piles in the database were open-ended steel pipe piles with diameters ranging from 114 mm to 915 mm and lengths between 4.5 m and 33 m. Depending on site conditions, either diesel or hydraulic hammers were employed. PDA results were interpreted using CAPWAP, with Match Quality (MQ) values between 2 and 4 considered acceptable.

- Several assumptions were introduced for reliability calibration, including the consideration of a single pile type, axial loading only, negligible pile group effects, and the absence of over-consolidation influences. The Chi-square test was used to identify the best-fit model for statistical analysis, and the chapter concludes with a summary of the statistical parameters applied in the calibration framework.

2. Recalibration of LRFD resistance factors for driven steel piles at end of drive conditions in Alberta, Canada:

- The design methods recommended by CFEM (2023) yielded the highest resistance and efficiency factors among all the static analysis methods evaluated in this study. Therefore, their application in practical projects is recommended.
- For piles in cohesionless soils, the ratio of measured to predicted value was found to be greater than 1. In contrast, for piles in cohesive and mixed soils, the ratio was less than 1. This indicates that the static analysis method tends to be more conservative when estimating pile bearing capacity in cohesionless soils.
- The recommended GRFs for Alberta, based on the 70% rule classification, are: Cohesive soils: 0.58 (for $\beta = 2.33$), 0.46 ($\beta = 3.00$) Mixed soils: 0.62 ($\beta = 2.33$), 0.49 ($\beta = 3.00$) Cohesionless soils: 0.64 ($\beta = 2.33$), 0.49 ($\beta = 3.00$)
- Results show that as the proportion of fine content along the pile length increases, the GRF values decrease. To support practical applications, two relationships between GRF and load ratio (LR) were developed, especially for cases where the cohesive soil content is close to the boundary defined by the “70% rule.”
- By dividing Alberta into five regions (NE, NW, SE, SW, C), it can observe that higher GRFs can be applied in northern regions.

3. Calibration of Resistance Factor Using Separate Toe and Shaft Resistance

- According to the “70% rule”, the single GRF for R_{ult} was consistently greater than the separate GRFs for $R_{ult,b}$ and $R_{ult,s}$ across cohesive, cohesionless, and mixed soil conditions
- While the “70% rule” provides a straightforward method for classifying piles based on the soil layers along the embedded pile length, it can be misleading under certain conditions. To address this limitation, a new classification criterion—referred to as the “R-ratio”—was introduced. The R-ratio is defined as the ratio of shaft resistance R_S to total resistance R_{ult} . Using this method, piles can be classified as toe-bearing piles, friction piles, or toe-friction piles.

- The analysis indicates that as the proportion of fines along the pile length increases, the GRF decreases. To enhance practical design applications, two new relationships between GRF and load ratio (LR) were developed, particularly useful when the cohesive soil content is near the boundaries defined by the “70% rule.”
- Overall, the findings demonstrate that using separate GRFs for toe and shaft resistances provides more reliable factored resistance estimates compared to applying a single total GRF. The regionally calibrated GRFs were approximately 27%, 24%, and 19% higher than the CFEM (2023) value considering typical confidence from site

4. GRF Calibration at EOD vs. BOR:

- A comparison of LRFD calibrations between Beginning-of-Driving (BOR) and EOD plus setup conditions shows that the resistance factors are higher for the EOD plus setup case. This improvement highlights the effectiveness of the newly developed calibration method, which provides separate GRFs for EOD and setup.
- When using the separate GRF approach with setup effects, the calibrated resistance factors consistently produced higher factored pile resistances compared to CFEM (2023) recommendations and other existing regulations.
- Incorporating setup effects was especially significant for cohesive soils, where the factored total resistance increased by approximately 18% at $\beta = 2.33$, compared to about 13% in cohesionless soils.

5. Cost Benefit Analysis:

- **Cost Reduction at BOR Condition:** Based on the results, GRFs calibrated at the BOR condition yielded the greatest potential for cost reduction. Applying these GRFs can decrease construction costs by approximately 23–35%. As demonstrated earlier, the setup effect provides a significant increase in capacity, making its inclusion in the design stage of driven steel pile foundations highly beneficial. Overall, using the regionally calibrated BOR GRFs in Alberta can reduce construction costs by around 30%, compared to the recommended CFEM (2023) value of $\Phi = 0.40$ for static analysis methods (e.g., α method, β method) considering average confidence of site condition. However, due to the limited availability of data on the influence of setup time on the variation of steel driven pile capacity, further research is required to refine these efficiency estimates.

- **EOD Condition and Cost Savings:** The calibrated GRFs based on the End-of-Driving (EOD) condition represent the second most favorable approach for cost savings. Using these factors can reduce construction costs by approximately 13–18% compared to applying the CFEM (2023) recommended value of $\phi = 0.40$ considering average confidence from site condition. The improved efficiency observed for the EOD plus setup calibration highlights the benefits of explicitly incorporating setup effects in design.
- **Static Load Test (SLT) Approach:** The use of a single GRF of 0.60 for pile design based on static load testing (SLT) was found to be the third most effective cost-saving approach. While this method can achieve cost reductions, the high cost of conducting SLTs means it is most advantageous in projects involving a large number of piles.
- **Evaluation of Separate GRFs for Shaft and Toe Resistance:** The analysis indicates that adopting separate GRFs for shaft resistance (φ_s) and toe resistance (Φ_b) results in an average cost saving of approximately 4% across three soil types. However, evaluations of 16 projects across five sites in Alberta revealed mixed outcomes: in some cases, the use of separate GRFs led to cost reductions, while in others it increased costs. This finding emphasizes the need for further investigation into the effects of separate GRFs and the relative contributions of shaft and toe resistances on the ultimate capacity of driven steel piles.

8.2 Future Studies

The recommendations and suggestions offered for future research studies are summarized in this section:

- The techniques and models proposed in this thesis are based on limited results from 16 projects in Alberta. To optimize the results for calibrated GFR values, more projects or data are required.
- This research has not considered the effect of unsaturated conditions on the estimation of pile capacity and calibrated GRF. Further study on this impact would be valuable, as accounting for the effect of unsaturated conditions could improve the GRF value.

- The uncertainty related to the obtained soil properties warrants further investigation. Improving the accuracy of soil property determination can enhance the calculated GRF values.
- The number of boreholes for soil investigations prior to pile design can also impact the cost of pile construction. Since boreholes help define soil properties, the relationship between GRF values and borehole investigations can be further explored.
- This research did not consider the influence of pile diameter or pile length on GFR factor; further investigation is required to evaluate these effects.
- The impact of over-consolidated clay conditions was not examined in this study, and additional research is needed to assess how negative effective stresses influence the GRF values.