REGION SPECIFIC CALIBRATION OF GEOTECHNICAL RESISTANCE FACTORS FOR AXIALLY LOADED DRIVEN STEEL PILES USING DYNAMIC MONITORING RESULTS

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ABSTRACT
Driven steel piles are the most common type of deep foundation used in Canada to support many structural elements of the oil and gas, and infrastructure projects. To calculate the capacity of a pile during preliminary design, semi-empirical equations are generally used. Although these equations are commonly used in practice, there are always uncertainties in calculating the geotechnical pile capacity, due to the uncertainty of the soil parameters (obtained from site characterization) and construction quality. To account for these uncertainties and ensure the foundation safety under working loads, a Geotechnical Resistance Factors (GRFs), recommended by the Canadian Foundation Engineering Manual (CFEM), is applied to the calculated pile capacity. To maximize this factor for driven piles within a specified region, it is required to calibrate pile capacity with respect to the region's soil conditions. This study aims to improve the recommended GRF value incorporated with Pile Driving Analyser (PDA) testing using probabilistic procedure thereby, increasing factored pile capacity and reducing costs. A case study at Winnipeg region is used in this research to demonstrate the methodology. The GRF value for axial capacity is calibrated using a Monte Carlo Simulation, site-specific geotechnical data, and PDA test results. Cost-Benefit Analysis is then carried out using the calibrated GRF values to obtain an accurate cost reduction estimate.

RÉSUMÉ

Les pieux tubulaires en acier sont le type de fondation profonde le plus couramment utilisé en Alberta pour soutenir la plupart des éléments structurels des projets pétroliers et gaziers et des projets d'infrastructure. Pour calculer la capacité d'un pieu lors de la conception préliminaire, des équations semi-empiriques sont généralement utilisées. Bien que ces équations soient couramment utilisées dans la pratique, il existe toujours des incertitudes dans le calcul de la capacité géotechnique des pieux, en raison de l'incertitude liée aux paramètres du sol (obtenus à partir de la caractérisation du site) et à la qualité de la construction. Pour tenir compte de ces incertitudes et garantir la sécurité de la fondation sous les charges appliquées, un facteur de résistivité géotechnique (FRG), recommandé par le Manuel Canadian des Fondations, est appliqué à la capacité de pieu calculée. Pour accroître l'utilisation de ce facteur en Alberta, il est nécessaire de le calibrer pour qu'il soit plus spécifique aux sols de locaux. La presente étude vise à améliorer la valeur recommandée de FRG combinées à des tests PDA en utilisant une procédure probabiliste, augmentant ainsi la capacité de pieux admissible et en réduisant les coûts des fondations profondes. Une étude de cas est faite dans cette recherche pour démontrer la méthodologie appliquée. La valeur FRG est mise à jour à l'aide de la simulation Monte Carlo, des données d'investigation géotechnique obtenues et des résultats des tests de pieux. Une analyse coûts-avantages est ensuite effectuée en utilisant les valeurs FRG mises à jour pour obtenir une réponse précise à la réduction des coûts.

1 INTRODUCTION

Driven pile foundations are frequently used in Canada to support heavy structures of oil & gas infrastructure, and bridge works (Bedair, 2013; Belbas, 2013). The pile capacity can be estimated using three types of analytical methods: 1) static analysis, 2) dynamic analysis, and 3) dynamic formulas. The static analysis (using α, and β method), developed empirically or semi-empirically using data from soil investigation data, are widely used and recommended by the Canadian Engineering Foundation Manual (CFEM, 2006). Prediction of pile capacity using static analysis method (empirical-based) provides results that are often different from the actual capacities achieved during pile driving. The reason for this disparity is due to the uncertainties that are encountered in the geotechnical realm. The highest level of uncertainty during construction is associated with soil properties, the driving equipment, and procedures used. There are wide number of factors which could impact the estimated pile capacity include the degree of disturbance of the soil during driving, soil - pile interaction, and changes in pile capacity over time due to setup or relaxation. These factors of uncertainty can lead to unacceptably high probabilities of service failure, which explains why Geotechnical Resistance Factors (GRFs) are being used in driven pile design (Jabo, 2014).

For a selected static analysis method, the pile capacity may be achieved using the Load and Resistance Factor Design (LRFD) methodology. There are several advantages of using the LRFD approach for designing deep foundations (Hamilton and Murff, 1992). The most important advantage of LRFD is the handling of uncertainties associated with design parameters by utilizing a rational framework of probability theory, leading to a constant degree of reliability. Consequently, the LRFD provides a consistent design approach for the entire structure (i.e., superstructure and substructure), which improves the overall design and construction perspective (Abdelsalam et. al., 2010).

The basic hypothesis of the LRFD is quantifying the uncertainties based on probabilistic approaches such as First Order Second Moment (FOSM), First Order Reliability Method (FORM), or Monte Carlo Simulation (MCS), 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 GRFs, where the factored loads should not exceed the factored resistances. Depending on the method used to estimate or measure pile capacity, the value of GRFs can be different. Low GRF values are assigned to an empirical-based method while higher GRF values are assigned to field testing methods. Reliable estimation of GRF values using field testing methods can help designers to potentially reduce the number and length of piles by using high values of GRF (Skirrow and Wang, 2008). To improve the design of foundation piles and their reliability, it is suggested by many researchers (Ng et. al., 2012; Zhang, 2004; Roling et. al., 2011) that GRF values listed in CFEM, 2006 or other building cods may be conservative, and it is recommended to be regionally calibrated to optimize results in the design capacity of pile foundations. This paper aims to calibrate GRFs used for pile design regionally using soil investigation data, obtained PDA results, and utilizing MCS and FOSM reliability analysis.

2 BACKGROUND

2.1 Design Method

The basic equation of the LRFD-based design can be expressed as follows (Dithinde, 2007):

$$\varphi\, R_n \;\geq\; \sum Y_i\, Q_{ni} \qquad [1]$$

Where:
$\varphi =$ Geotechnical resistance factor
$R_n =$ The ultimate geotechnical resistance
$\sum Y_i\, Q_{ni} =$ The summation of factored overall load effects for a given load combination condition
$Y_i =$ Load factor corresponding to a particular load
$Q_{ni} =$ Specified load component of the overall load effects (e.g. dead load due to weight of structure or live load due to wind, or both)
i =Represents various type of loads such as dead load, live load, wind load, etc.

The values for load factor (Y_i) and (φ) are various and can be obtained from applicable codes such as NBCC, CHBDC, AASHTO, etc. 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 (CFEM, 2006). A load factor, Y_i, of less than 1.0 for dead loads is used when the dead load component contributes to the resistance against overturning, uplift or sliding. Table 1 summarizes Geotechnical resistance factors for deep foundation based on CFEM, 2006 recommendations. The AASHTO Code (1997) specifies many more resistance factors than is provided by CFEM.

Table 1: Geotechnical resistance factors, φ, to axial load of deep foundation (CFEM, 2006)

Description	φ
Analysis using dynamic method (no field measurement)	0.4
Semi-empirical analysis using laboratory and in-situ test data	0.4
Analysis using dynamic monitoring results (PDA)	0.5
Analysis using static test results	0.6
Analysis of uplift resistance using semi-empirical method	0.3
Analysis of uplift resistance using static loading test results	0.4

The ultimate geotechnical resistance of pile foundation can be obtained from shaft resistance in friction piles as vertical distribution of load or from the pile point capacity in end bearing piles as direct application of load, or from both in the case where pile geometry and soil conditions allow. Therefore, the axial capacity of a single pile can be computed by using the following equation:

$$R_n = \sum_{z=0}^{L} C\, q_s \Delta z + A_t q_t \qquad [2]$$

Where:
$q_s =$ Unit shaft capacity of pile at any depth of z.
$q_t =$Unit base capacity of pile at toe elevation.
$C =$Circumference of pile.
$L =$ Embedded length is subdivided into segments of Δz.
$A_t =$Pile toe area

The two recommended methods α, β are suggested in the CFEM (2006), and by Bowles (1997) to estimate the skin and end bearing resistances. For coarse materials, it is suggested to use the β method which explained is below:

$$q_s = \beta\sigma_v^{'} \qquad [3]$$

$$q_t = N_t\sigma_l^{'} \qquad [4]$$

Where:
β =Combined shaft resistance factor, varies between 0.3 to 1.5 based on coarse material properties
$\sigma_v^{'}$ = Vertical effective stress close to the pile at depth z
N_t = Bearing capacity factor varies between 20 to 300 based on coarse material properties
σ_l' = Vertical effective stress at the pile toe

For fine materials, its suggested to use α method which is explained below:

$$q_s = \alpha s_u \quad [5]$$
$$q_t = N_t s_u \quad [6]$$

Where:
s_u= Minimum undrained shear strength at pile toe or z elevation
N_t=Bearing capacity coefficient that is a function of the pile diameter and varies between 6 and 9
α=Adhesion coefficient ranging from 0.5 to 1.0

2.2 Reliability Analysis

Assuming independent variables for load effects and resistance which is the case of static loading, and considering nominal values for load effect and resistivity effect as below:

$$Q_n = \frac{\bar{Q}}{\lambda_Q} \quad [7]$$
$$R_n = \frac{\bar{R}}{\lambda_R} \quad [8]$$

Where:
$\lambda_R =$ The ratio of mean value to nominal value for resistance;
$\lambda_Q =$ The ratio of mean value to specified value for load effects.

The λ_r is greater or equal than one and $\lambda_Q$value is less than one. These two factors are referred as bias factors in literature. The probability of failure, P_f, is typically represented by the reliability index term β, shown in the right-hand figure.

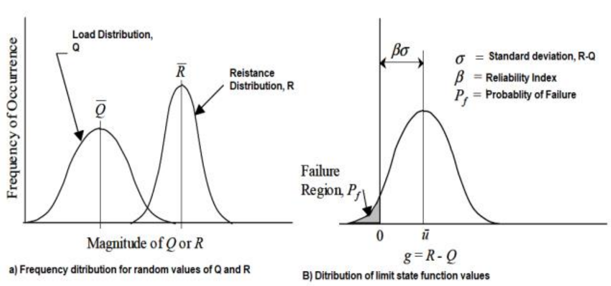

Figure 1: Probability of failure and reliability index (Withiam, et al.,1998).

The basic objective of reliability-based design is to ensure that the probability of failure does not exceed an acceptable threshold level (B_T). This objective can be defined as:

$$\beta = -\phi^{-1}(P_f) < B_T \quad [9]$$

Where:
B_T =Target reliability index
$\phi^{-1}(.)$=inverse of the standard normal or lognormal cumulative function

The selection of the target reliability index, β_T, is an important step in the calibration process. The selection of the target β_T to estimate the load and resistance factors depends on the desired threshold of P_f. In general, strength limit state resistance factors for structural design have been derived to produce a β_T value of 3.5 (Pf ≈ 1 in 5,000) for the structure components. In geotechnical design, β_T value of approximately 3.0 (P_f ≈ 1 in 1,000) can be used (Withiam, et al., 1998). Barker et al. (1991) indicates values of β_T a target value of $\beta_T = 2.33$ can be appropriate for driven piles. This value is used in this study.

For axial capacity of single piles design only the combination of Dead load (DL) and live load (LL) is considered. The probabilistic characteristics of the random variables DL and LL are assumed to be those used by AASHTO (Nowak, 1999) with the following load factors and lognormal distributions (bias and COV) for live and dead loads, respectively:

$$\gamma_{LL} = 1.75 \; \lambda_{LL} = 1.15 \; COV_{LL} = 0.2$$
$$\gamma_{DL} = 1.25 \; \lambda_{DL} = 1.05 \; COV_{DL} = 0.1$$

Once estimates for γ and λ have been made either a closed-form solution such as FOSM or MCS can be performed as reliability techniques. The FOSM is a first-order expansion of 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 and it was subsequently used by Barker et al. (1991) for NCHRP Report 343. Scott and Salagado (2003) indicated the lognormal distribution better represents and models the transient loads better and fully characterizes it by its first two moments. They added that, the magnitude of the transient loads and resistance found in geotechnical problems cannot be negative values, and the lognormal distribution can better represent their product even if the variables themselves are not log-normally distributed. Therefore, and in accordance with the 2007 AASHTO-LRFD specifications, the load and resistance Probability Density Function (PDFs) are assumed to follow lognormal distributions. By separation of the total loads into DL and LL, and by rearranging the formula according to the recommended AASHTO-LRFD (Paikowsky et al., 2004), the GRF (φ) can be calibrated as follows:

$$\varphi_R = (\lambda_R \left(\frac{\gamma_{DL} Q_{DL}}{Q_{LL}}\right) \sqrt{\frac{1+COV^2 Q_{DL}+COV^2 Q_{LL}}{1+COV_{QR}^2}}) \times (\left(\frac{\gamma_{DL} Q_{DL}}{Q_{LL}} + \lambda_{Q_{LL}}\right) \exp \left(B_T \sqrt{(1+COV_{QR}^2)(1+COV^2 Q_{DL}+COV^2 Q_{LL})}\right)^{-1} \quad [10]$$

Where:
γ_{DL}= Load factor for dead Loads
γ_{LL} = Load factor for Live Loads
$\lambda_{Q_{LL}}$ = Bias for live loads
$\lambda_{Q_{DL}}$ = Bias for dead loads
$\frac{Q_{DL}}{Q_{LL}}$ = Dead load to live load ratio which is assumed between 2.0 to 2.5 for pile foundation (Paikowsky et al., 2004)

While the FOSM, is a straightforward approach and easy to use, it underestimates the resistance factor by nearly 20%, 15%, and 12% for low, medium, and high site variability respectively (Jabo, 2014). On the other hand, the MSC simulation method provides a more feasible and accurate way to determine the probability of failure for the LRFD Strength Limit State Function (Allen et al. 2005). The MCS method requires the statistical distribution of selected input variables to be known (Jones, Kramer, & Arduino, 2002). The MCS method is simply a technique that utilizes a random number generator to extrapolate the "Culminative Density Function (CDF)". The CDF is characterized by the mean, standard deviation, and type of CDF function (e.g., normal, lognormal, etc.). This extrapolation of the CDF plots makes estimating β possible, since in most cases the quantity of measured data is inadequate to reliably estimate β. Devroye (1986) shows several ways to transform these random numbers into suitable numbers needed for a specific problem. The general procedure for implementing MCS is adapted from Hammersley and Handscomb (1964), and the various steps of the procedure are outlined by Phoon (2008).

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 is formed with the random values and evaluated for failure as below.

$$g\,(R,Q) = \left(\frac{\gamma_{DL} + \gamma_{LL}\frac{Q_{LL}}{Q_{DL}}}{\varphi}\right)\lambda_R - \left(\lambda_{DL} + \lambda_{L:}\frac{Q_{LL}}{Q_{DL}}\right) \quad [11]$$

Failure is defined when g ≤ 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 gives the probability of failure (P_f). The number of simulations (N) are determined using:

$$N = \frac{1-P_f}{COV_{pf}^2 \times (P_f)} \quad [12]$$

Where:
COV_{pf} is the desired coefficient of variation, for example, for a COV_{pf} of 10% a minimum number of N = 100,000 simulations is required for a reliability index of 3.00 corresponding to a probability of failure of 0.1%.

The limit state function (g) is evaluated N times and evaluated by the indicator function (J). The indicator function (J) is equal to 1 when g_i ≤ 0 (the failure region), and equal to 0 when g_i > 0 (the safe region). Through the simulation, a total of the failures is recorded and defined as Nf. The P_f is represented in the equations below (Paikowsky et al. 2010).

$$P_f = P(g \le 0) = \frac{1}{N}\sum_{i-1}^{N} J(g_i \le 0) \quad [13]$$

$$P_f = \frac{N_f}{N} \quad [14]$$

Where:
N is the number of simulations carried out

The reliability index (β) rearranged and calculated as:

$$\beta = \varphi^{-1}(1 - P_f) \quad [15]$$

2.3 Pile Dynamic Monitoring

High strain dynamic testing consists of estimating soil resistance and its distribution from force and velocity measurements obtained near the top of a foundation that is impacted by a hammer or drop weight. The impact produces a compressive wave that travels down the shaft of the foundation. A pair of strain transducers measure the signals necessary to compute force, while measurements from a pair of accelerometers are integrated to yield velocity. These sensors are connected to an instrument (such as a Pile Driving Analyzer ®), that records, processes and displays data and results. If the wave travels in one direction, force (F) and velocity (v) are proportional and related by the equation (Eqn.16).

$$F = zv \quad [16]$$

Where:
$z = EA/c$ is the pile impedance
E = modulus of elasticity of pile material
A = cross sectional area of the pile
c = material wave speed

The wave assumes an opposite direction (a reflection) when it encounters soil resistance forces along the shaft or at the toe. These reflections travel upward along the shaft and arrive at the pile top at times that are related to their location along the shaft. The sensors near the pile top take measurements that translate what is happening to the traveling waves and make it possible to estimate soil resistance and its distribution. The data obtained in this fashion permits the computation of total soil resistance, which includes both static and viscous components. The dynamic component is computed as the product of the pile velocity times the damping factor (a soil parameter related to energy dissipation within soil). The static component is the total soil resistance minus the dynamic component. Dynamic load testing takes a further step in analyzing the data and computing static capacity and resistance distribution. Dynamic pile monitoring takes advantage of the fact that, for driven piles, it is possible to compute the energy delivered to the pile, compression stresses at the pile top and toe and tension stresses along the shaft. Pile damage can also be evaluated using this method. The method has been successfully used to test most types of piles. In Canada, the method is typically used to verify the capacity of driven piles and to a lesser extend cast in place concrete piles.

3 SITE DESCRIPTION AND PILE SPECIFICATION

The site is located at the west of Winnipeg and dataset has been collected from R. Belbas, 2013. Based on this research, Eighteen HP310 x125 H-piles rolled from 350W steel were PDA tested at this site. The piles were driven using two different single-acting open-ended diesel hammers; a Delmag D19-32 set at fuel setting number 3 and 4 having rated energies of 47.1 and 57.6 kJ, respectively; and a Pileco D19-42 set at fuel setting numbers 3 and 4 with rated energies of 47.8 and 57.6 kJ, respectively. The piles were driven to practical refusal at 15 blows per 25 mm. The piles ranged in penetration depth from 19.9 to 31.6 m. Seven of the H-piles refused in the sand till and twelve were carried down to bedrock.

The soil stratigraphy consists of lacustrine clay and till covering limestone bedrock. The lacustrine clay varies significantly due to varying ground elevations and ranges in thickness from 14.2 to 21 m. The clay is generally stiff and becomes firm to stiff with depth. The till is highly irregular and continuously changes from a dense and moist to wet sand till to silt till that varies from compact and moist to very dense and dry. The entire till layer ranges in thickness from 7.2 to 9.6 m. The limestone bedrock is fractured at some of the site locations in the upper 1.5 m and turns massive below. In other locations the rock is entirely massive with no fractured zones.

Table 2 provides summary of soil investigation data in this area. Data of Sixteen (16) boreholes drilled in proximity of installed pile location were used to obtain related soil properties.

Table 2: Summary of Soil properties at above-mentioned site

Description	Approximate Depth (m)	SPT (N) (Avg.)
Clay / Clay Till	0.0 to 14	8 to 15
Clay/Silt/Sand	14 to 21	15 to 50
Sand/Silty Sand/ Silt / Bedrock (RQD:95 to 100%)	21 to 24	11 to 75
Bedrock (RQD:95 to 100%)	24.0 to 29	N/A

Table 3 presents the specifications of the piles that were installed at the site. The piles (length and size) were designed based on the SPT blow counts and recommended α, β methods as mentioned earlier. A GRF value of 0.5 was used to calculate the capacity of each pile. Figure 2 below shows the variation of capacity with depth for each pile. Each installed pile shaft was selected to conform to ASTM A252 grade 3 steel. Prior to installation of piles, initial WEAP analysis has been conducted for each group to find pile driving termination criteria. Pile monitoring was done for each pile during installation.

Table 3: Characteristics of installed driven steel pile

PILE TYPE	Unit	P1
Design load (Factored)	kN	900 to 1500
Pile size	mm	310 X 125
Pipe Type		H-Pile
Embedment	m	20 to 31
Termination Criteria	Blow counts	19 to 84
Pile number	Ea.	67

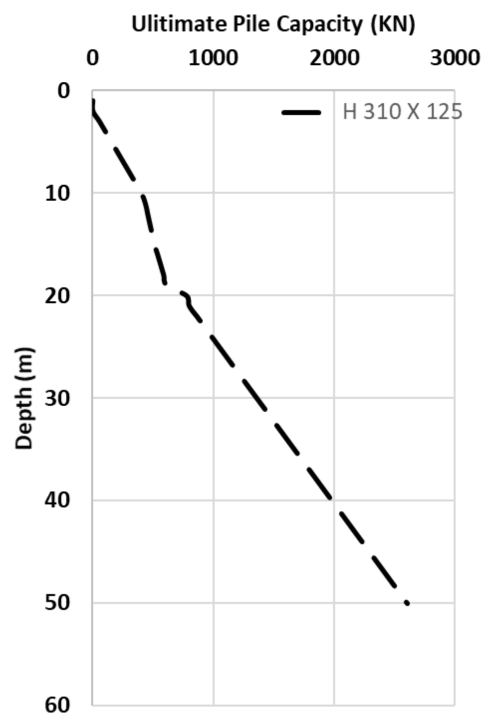

Figure 2: Estimated Pile Capacity at each depth for each type

4 PILE INSTALLATION AND PDA RESULTS

The piles were driven using two different single-acting open-ended diesel hammers; a Delmag D19-32 set at fuel setting number 3 and 4 having rated energies of 47.1 and 57.6 kJ, respectively; and a Pileco D19-42 set at fuel setting numbers 3 and 4 with rated energies of 47.8 and 57.6 kJ, respectively. Blow counts measurements were recorded every 0.25 m for each installed pile. The observed blow counts at embedment depth were generally higher than the blow counts anticipated during preliminary design calculation. The reason for higher than anticipated blow counts was due to the nature of soil and its variation at the site. The PDA testing was preformed according to ASTM D4945-12 test “Standard Test Method for High-Strain Dynamic Testing of Deep Foundations”. The capacity of piles at End of Drive (EOD) was measured by PDA equipment. Accelerometers and strain gauges were installed on the pile. The same hammer used during installation was also used to perform the testing on the piles. The test energy was determined using the wave equation analysis. The pile was tapped at the specified energy and the data was recorded in the 8G system. CAPWAP analysis was conducted on the collected data. Total of sixteen (16) PDA tests were conducted at this site. The results indicated the measured capacity of piles are generally higher than the estimated capacity calculated during the design stage using static (empirical-based) method. Figure 3 shows the comparison of measured capacities and the corresponding estimated value for each test piles.

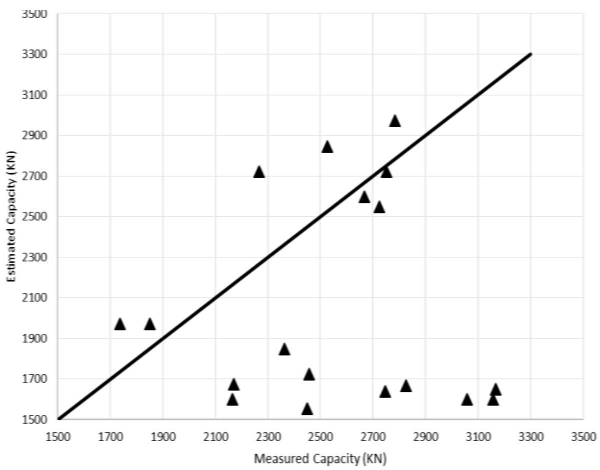

Figure 3: Summary of Soil properties at above-mentioned site

5 CALIBRATION ANALYSIS AND RESULTS

5.1 MCS Method:

To perform calibration using reliability analysis, MCS, the mean, standard deviation, and Coefficient of Variation (COV) as well as the type of distribution that best fits the data (normal or lognormal) must be determined for each random variable considered in the limit state function. The least squares method has been used to find the best value for each variable. The bias λ_r, defined previously as the ratio of the measured (PDA results) to predicted one (estimated at design stage) value, is used to generate the needed statistics. To characterize load and resistance data, a Cumulative Distribution Function (CDF) of the data has been developed. The CDF is a function that represents the probability that a bias value less than or equal to a given value will occur. This probability can be transformed to the standard normal variable (or variate), z, and plotted against the bias (X) values for each data point.

Figure 4 provides a CDF plotted using the standard normal variable as the vertical axis. As shown, if normally distributed data is used, CDF plotted in this manner is a straight line with a slope equal to $1/\sigma$, where σ is the standard deviation, and the horizontal (bias) axis intercept is equal to the mean, μ_s. As shown in this Figure, lognormally distributed data on the other hand is plotted as a curve and fits the data well.

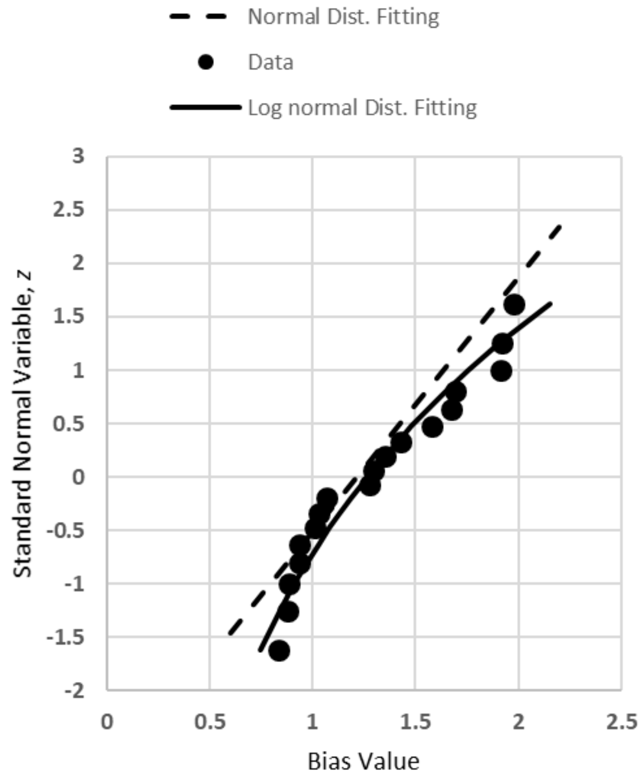

Figure 4: Fitted Log-Normal AND Normal CDF of Resistance Bias Factors

The MCS analysis begins with an assumed resistance (ϕ) factor to obtain value of P_f. The minimum number of simulations is calculated using Eqn. 14, the counts ranged from 100,000 for P_f = 0.001, and 10,000 for P_f = 0.01. Random variables are generated in using the statistical properties for resistance, live load, and dead load described earlier.

The random variables are then substituted into the performance function (Eqn.11). The number of failures (N_f) during the simulation, meaning when the performance function is less than zero (g(x) < 0), are counted then divided by the total number of simulations and the probability of failure (P_f) calculated using Eqn. 14. With known P_f the reliability index (β) is determined using Eqn. 15. The β_T of interest, 2.33 is then picked from the array of values with its corresponding φ factor.

5.2 FOSM Method:

Histograms of the bias values from each driven pile case are generated, and theoretical frequency distributions for lognormal distributions, are generated and compared to the observed values (Figure 5).

A normalized error is calculated for each bin range used in the histogram, and the sum of the errors can be used to show which distribution type best fits the data – this is known as the χ2 Test (Chi-Square test). Based on the desired significance level (95%) for fitting to a specific distribution and number bins, a maximum allowable error value (8.67) is determined, and this value is used to compare to the sum of errors between the observed and theoretical values generated in the histogram.

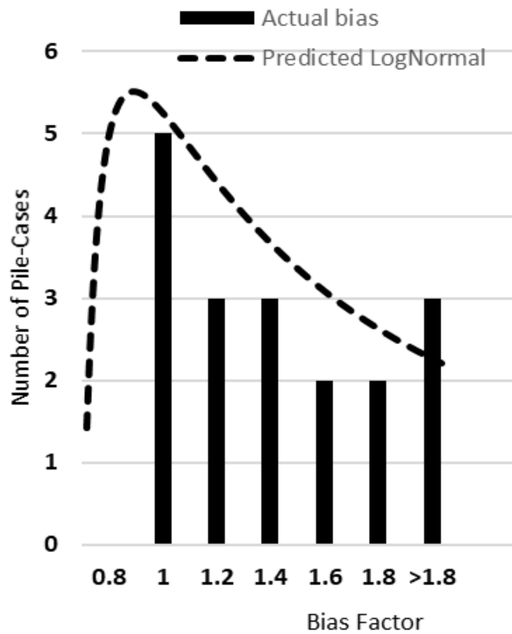

Figure 5: Histogram and Predicted Log-Normal PDF of Resistance Bias Factors for FOSM Method

6 DISCUSSION AND COST-BENEFIT ANALYSIS

The FOSM and MCS methods were used in this study to calibrate the geotechnical resistance factor regionally for using the LRFD design methodology. To be consistent with the LRFD calibration of driven piles as suggested in the CFEM, 2006, the values of the dead and live load factors and their corresponding statistical characteristics used for the FOSM and the MCS methods are as described in the previous section. For a target probability of failure of 0.1% and coefficient of variation of 10% in the MCS results, Eqn. 12 shows that 10,000 simulations would typically be sufficient for generating the resistance bias, λ_R, using the lognormal distribution; however, 100,000 simulations were used in this study to be conservative. The φ factors obtained from FOSM Simplified method were approximately 10 % percent lower than the φ factors obtained by MCS. Figure 6 shows resistance Factor versus Reliability Index based on FOSM and MCS. The 7% difference between MCS and FOSM is generally consistent with other research results in literature such as J.Jabo, 2014a. Using β_T=2.33, GRF is approximately determined to be 0.556, and 0.608 for FOSM and MCS, respectively which is about 12% to 20% more than what is recommended in the CFEM, 2006. Comparing the obtained GFR values with the other researches using similar methodology (such as J.Jabo (2014) for calibrated GFR values obtained at a site located in Louisiana, USA) is indicating that the range of 12% to 20 % improvement in the GRFs for axial pile capacity is reasonable.

The driven steel pile cost can usually be assumed to vary linearly with depth based on the unit price. The FHWA-NHI-16-009 (2016) report indicated that the cost is increasing linearly from $0 at 0 m penetration to about $9600 at 36 m. Applying the obtained GRF values from each method and redesigning the pile foundation, a cost reduction of approximately 10% and 15% for FOSM and MCS, respectively (between $1000 and $1800 per pile) can be achieved.

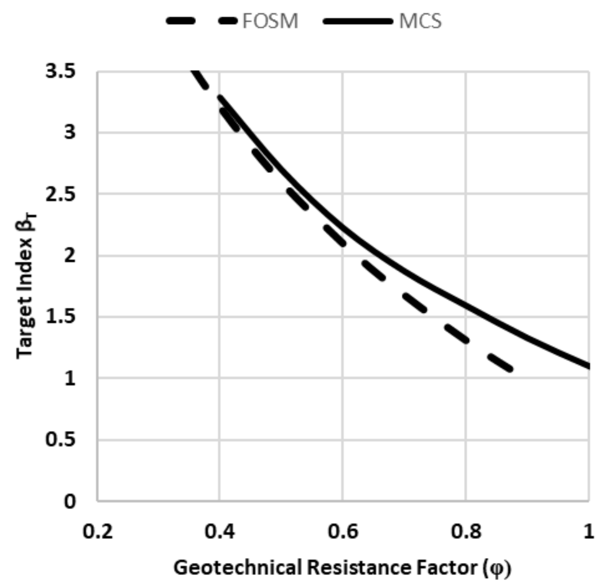

Figure 6: Resistance Factor versus Reliability Index for FOSM and MCS Pile Analysis Methods

7 CONCLUDING REMARKS

This study used a case study data at a site located at the west of Winnipeg to improve the estimated GRF values integrated with PDA testing using probabilistic procedures (FOSM and MSC) thereby, increasing factored pile capacity and reducing costs. Results from this study show that the GRF values obtained from MCS and FOSM analysis methods are improved by approximately 12% and 20%, respectively from the recommended values provided in the CFEM, 2006. Redesigning the pile foundation at this site with the estimated GRF values reduced the construction cost of the driven piles by 10% to 15% accordingly. Comparing obtained results with the other research showed that 12% and 20 % changes for GRFs due to local calibration are reasonable and acceptable.

8 ACKNOWLEDGMENT

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