Aligning the Design and Monitoring of Piles with the Load and Resistance Factor Design Methodology

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ABSTRACT
The 2005 National Building Code of Canada (NBCC 2005) mandated the use of limit states design methodology (specifically LRFD – load and resistance factor design) for foundations. The use of LRFD was designed to have a 'standardized' reliability of foundations. Despite greater than ten years of experience with the use of LRFD for foundations, misunderstanding of the methodology at both the design and construction stages of projects are leading to significant variability in the reliability of installed foundations. This paper highlights some of the key aspects of LRFD for both design and monitoring with comparison to the previously used working stress design. Common misapplications of LRFD at the design stage, common methods of monitoring pile installations (e.g., PDA, WEAP) along with the major limitations of each method, and how monitoring methods should be utilized in relation to LRFD are presented with discussion on the impact to the reliability of foundations.

Le 2005 Code National du Bâtiment du Canada (CNBC 2005) a mandaté l'utilisation de la méthodologie de conception des états limites (spécifiquement LRFD-conception de facteur de charge et de résistance) pour les fondations. L'utilisation de LRFD a été conçue pour avoir une fiabilité «normalisée» des fondations. En dépit de plus de dix années d'expérience avec l'utilisation LRFD pour les fondations, l'incompréhension de la méthodologie à la fois la conception et la construction des projets mènent à une variabilité significative de la fiabilité des fondations installées. Cet article met en évidence quelques-uns des aspects clés de LRFD pour la conception et la surveillance par rapport à la méthodologie du plan précédemment utilisé. Les applications incorrectes courantes des LRFD au stade de la conception, les méthodes communes de surveillance des installations de pieux (p. ex. PDA, WEAP) ainsi que les limitations principales de chaque méthode et la façon dont les méthodes de surveillance devraient être utilisées en relation avec les LRFD sont présentées avec discussion sur l'impact sur la fiabilité des fondations.

1 INTRODUCTION

Despite the mandate of the use of Load and Resistance Factor Design (LRFD) for geotechnical foundation design as originally directed by the National Building Code of Canada in the 2005 version (NBCC 2005) and continuing through to the present version, several aspects of the design and construction of foundations are still being undertaken using practices consistent with Working (or Allowable) Stress Design (WSD). The result is that constructed foundations may be overly conservative relative to the LRFD methodology. As shown in Becker (1996), the geotechnical resistance factors selected for use in NBCC 2005 were calibrated against WSD and typical factors of safety. The geotechnical resistance factors were also calibrated against reliability theory such that foundations would have a similar reliability to that used in structural design.

The switch to LRFD was intended to bring consistency between structural and geotechnical engineering, bring consistency across practicing geotechnical engineers, allow assessment of foundations independently for serviceability and ultimate limit states, and provide foundations with a desired target degree of reliability. Previously, it was difficult for structural engineers to know whether allowable loads provided by geotechnical engineers were based on serviceability or ultimate loading criteria. Additionally, there were discrepancies between how different geotechnical engineers estimated ultimate loads and applied factors of safety to establish allowable loads for foundations.

Due to the misapplication of LRFD in practice, both during design and construction monitoring, the reliability of foundations continues to vary significantly. It is important for practicing geotechnical engineers to understand that LRFD is based on selecting design limit states parameters that are cautious estimates of the mean values and not conservative lower bound values. Additionally, it must be understood that estimates of the resistances achieved during construction should be compared against design parameters that are cautious estimates of the mean. As such, measurements/inferences of the resistances achieved during construction should consist of a distribution around the near mean values indicated by design parameters.

While applicable to other foundation types, the points in this paper are primarily characterized around driven steel pile foundations loaded in compression with some discussion on helical (screw) piles.

2 LOAD AND RESISTANCE FACTOR DESIGN VERSUS WORKING STRESS DESIGN

The LRFD methodology developed for the NBCC was calibrated against WSD and typical factors of safety. When utilized properly, the LRFD methodology should provide similar foundation designs to those from the typical application of WSD methodology.

2.1 Load and Resistance Factor Design Methodology

2.1.1 Design

Load and resistance factor design is based on distributions of load effects and distributions of resistances being sufficiently separated so that an adequate target reliability of the structure is attained. Near, but cautious, estimates of the mean load effects (characteristic unfactored load) and mean resistance (characteristic unfactored resistance) are separated by load and resistance factors as shown in Figure 1. The general LRFD design equation is as follows:

$$\Phi R_n \geq \Sigma \alpha_i S_{ni} \quad [1]$$

where:
Φ is the geotechnical resistance factor;
R_n is the nominal (characteristic) resistance = $\bar{R}/\bar{k}_R$;
ΦR_n is the factored geotechnical resistance;
α is a load factor;
S_n is a nominal (specified) load = $\bar{S}/\bar{k}_S$;
$\Sigma \alpha_i S_{ni}$ is the sum of factored load effect;
$\bar{R}$ is the mean resistance;
$\bar{k}_R$ is the resistance bias factor;
$\bar{S}$ is the mean load effect; and
$\bar{k}_S$ is the load effects bias factor.

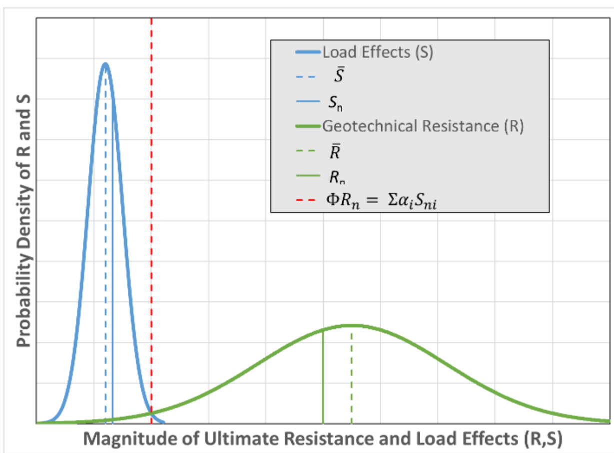

Figure 1. Distributions for Load and Resistance Factor Design

The values of characteristic ultimate resistance R_n shown in Figure 1 are based on a $\bar{k}_R$ of 1.1, which is consistent with the values developed for the NBCC (Becker 1996).

2.1.2 Construction Monitoring of Ultimate Resistance

When comparing design based on (characteristic) geotechnical resistances using LRFD, it should be expected that the measured resistances form a distribution around the inferred mean resistance. A hypothetical distribution of measured ultimate resistance values for the design condition shown on Figure 1 is presented on Figure 2 for pile foundations.

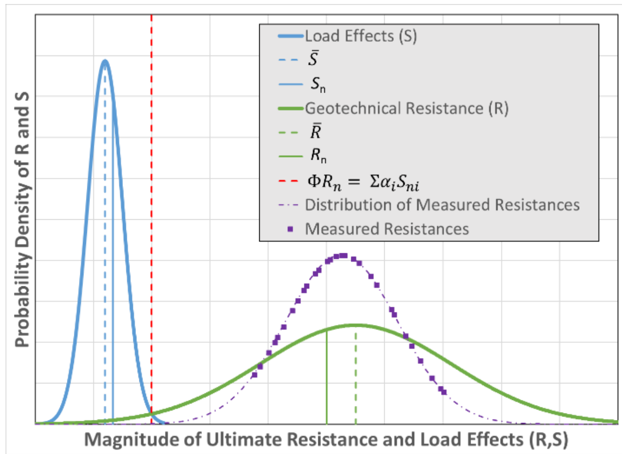

Figure 2. Measured Ultimate Resistance against Distributions for Load and Resistance Factor Design

For measured resistances below the characteristic resistance, the geotechnical and structural engineering teams would need to collectively assess which values are within tolerance. Measured resistances that are considered too much below the characteristic resistance would need remediation (e.g., longer piles)

However, it should be appreciated that the measured resistances may not account for the accuracy of the monitoring method. When incorporating the accuracy of the monitoring method, a wider distribution of the resistance may result.

2.2 Working Stress Design Methodology

2.2.1 Design

Working Stress Design (WSD) is based on selecting an ultimate resistance based on the soil conditions encountered and applying a factor of safety to determine the allowable load. The basic equation is as follows:

$$S_a = \frac{R_u}{FS} \quad [2]$$

where:
S_a is the allowable load
R_u is the ultimate resistance
FS is the factor of safety.

The application of WSD for the same distribution of loads and resistance indicated for LRFD design is shown graphically on Figure 3. Figure 3 shows an allowable load slightly higher than the unfactored load in Figure 1, a relatively conservative (just over one standard deviation from the mean) but not lower bound value for the ultimate resistance and an applied factor of safety of 2.5.

The allowable load is typically compared against the service (working) load. Provided that the structural engineer assigns a value to the working load as something slightly higher than the unfactored load, the situation shown in Figure 3 should result in approximately the same pile foundations as those derived from Figure 1.

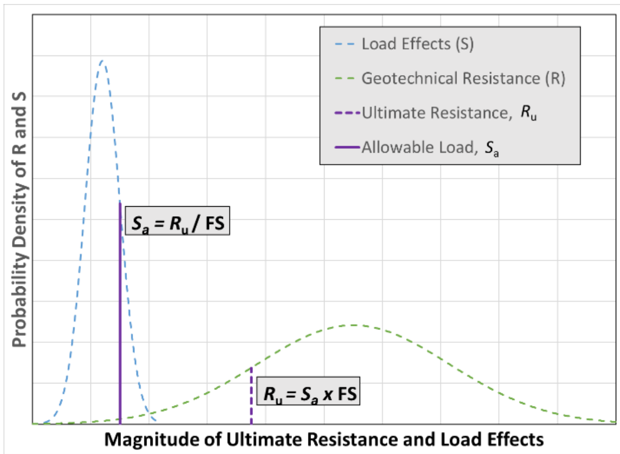

Figure 3. Illustration of Working Stress Design

It should be appreciated that in the case of WSD, the geotechnical engineer could easily select a characteristic value closer to the mean or closer to the lower bound and/or select a differing value for the factor of safety. As such, there could be significant differences in the probability of failure for different foundation designs based on the same data set interpreted by the geotechnical engineer. Similarly, the structural engineer could compare the allowable load against the unfactored load rather than service load. Between the variations in the design methods incorporated by both the geotechnical and structural engineers, the reliability of the final foundation design for WSD could be extremely variable.

2.2.2 Monitoring

In WSD, the field measurements of ultimate (nominal) resistances are expected to exceed the ultimate resistance value on which design is based. Values below the ultimate resistance values are considered to have insufficient resistance. The same distribution of measured values provided for the LRFD case shown on Figure 2 are shown for WSD on Figure 4.

The measured ultimate resistances in Figure 4 are shown to be equal or greater than the design ultimate resistance and therefore would be considered adequate. In WSD, where measured ultimate resistances are below the design ultimate resistance, remediation would be required. However, as mentioned above, when using LRFD, piles with measured ultimate resistance lower than the characteristic ultimate resistance value may not require remediation since the LRFD approach implicitly assumes a distribution of ultimate resistance.

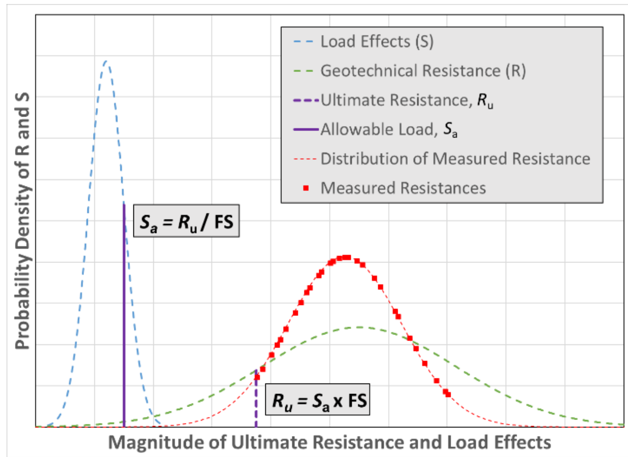

Figure 4. Measured Ultimate Resistances versus Ultimate Resistance used for Working Stress Design

2.3 Reasons for Switching to LRFD

The switch to LRFD was to bring consistency between geotechnical and structural design. Additionally, the use of LRFD was designed to have a 'standardized' reliability. This is achieved by the geotechnical engineer utilizing near-mean values and standardized (specified) resistance factors for the level of knowledge that the engineer has of the site. With improved knowledge of the resistance distribution, higher resistance factors may be used.

With WSD, there could be considerable variability in the assessment of the ultimate resistance based on the geotechnical engineer's interpretation of a conservative design value. Additionally, different factors of safety may be applied by different engineers. This could lead to highly variable foundation reliability. By utilizing a consistent set of resistance factors and consistent interpretation of the design resistance in LRFD, there should be less variability in the reliability of structures.

3 RELIABILITY AND GEOTECHNICAL RESISTANCE FACTORS FOR PILES

The reliability associated with the design of piles was calibrated using reliability indices. Resistance factors developed for the NBCC were calibrated to provide reliability indices, which relate to a probability of failure, between 3.2 and 3.5. An approximate relationship between the probability of failure and the reliability index is shown on Figure 5 (Paikowsky et al. 2004, after Baecher 2001).

In addition to the reliability index, the resistance factors were developed based on assumed geotechnical biases, coefficients of variation, and separation of the geotechnical resistance distribution from the structural load distribution. The First Order Second Moment Approximate (FOSMA) method (see FHWA 2001, for example) was used to develop the geotechnical resistance factors for the NBCC. The FOSMA relationship for geotechnical resistance factor is as follows:

$$\Phi = \bar{k}_R e^{-\theta \beta_T COV_R} \quad [3]$$

where θ is a separation factor, $\bar{k}_R$ is the resistance bias factor, β_T is the target reliability index, and COV_R is the coefficient of variation of the resistance distribution (the ratio of its standard deviation to its mean value). For all assessments the value of θ was taken as 0.75 (Becker 1996).

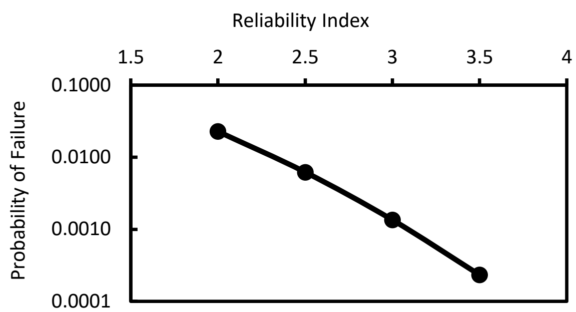

Figure 5. Reliability index and probability of failure (after Baecher 2001)

The developed resistance factors assumed a $\bar{k}_R$ of 1.1 and assumed a COV_R between 0.25 and 0.40. The higher COV_R values used to develop the NBCC were associated with the lower resistance factors. The COV_R values include assumptions about the variability of the ground, the variability within the method of measurement, and the variability in the methods of resistance calculations. As such, methods of direct measurement (such as Pile Driving Analyzer and static load tests) have less assumed variability than indirect methods (such as correlating resistance against the standard penetration test and laboratory test results).

For the geotechnical engineer applying LRFD, the $\bar{k}_R$ value used to derive the resistance factors is important to understand. It indicates that the characteristic resistance value, on which design is based, needs to be near the mean resistance to result in the target reliability being achieved (Becker 2017). Overly conservative estimates of the characteristic value have high values of $\bar{k}_R$ and applying the geotechnical resistance factors in NBCC results in overly conservative foundation design.

For piles designed for geotechnical resistance to axial compression load, the developed NBCC resistance factors are 0.4 for semi-empirical analysis using in-situ and laboratory test data, 0.5 for analysis using dynamic monitoring results, and 0.6 for analysis using static load test results.

4 RELIABILITY IMPLICATIONS FROM MISAPPLICATION OF THE LRFD METHODOLOGY

It is important that the LRFD methodology is properly understood by practicing geotechnical engineers. When improperly applied, there can be significant implications to the reliability of structures. Some misapplications could result in an overly conservative design while others could result in a less reliable design.

4.1 Conservative Design Resistance

Rather than select characteristic resistance parameters that are near the mean values when using LRFD, it is not uncommon for geotechnical engineers to select characteristic parameters that are relatively conservative, close to lower bound values as they would normally infer for WSD design methods.

Relative to Equation 3, this results in a higher bias, $\bar{k}_R$, with the COV_R and Φ remaining unchanged. The equation indicates that the reliability index would increase to obtain the same value of Φ. In other words, the resistance distribution will be further separated from the load effects distribution and the reliability will be increased. The result is foundations that are over designed and more costly than necessary.

Figure 6 shows graphically the distribution of the geotechnical resistance with a $\bar{k}_R$ of 1.3 instead of 1.1. While the graph appears similar to Figure 1, this overdesign of about 18 percent results in nearly an order of magnitude reduction in the probability of failure as the reliability index increases from 3.4 to 3.9.

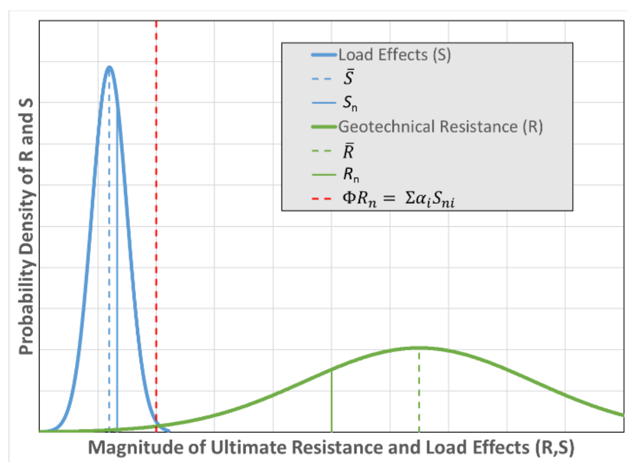

Figure 6. LRFD Distribution for $\bar{k}_R$ = 1.3 and Φ = 0.4

By only modifying the $\bar{k}_R$ values in Equation 3 between 1.3 and 1.5 for Φ values of 0.4, 0.5, and 0.6, the probability of failure reduces by between approximately 1 and 3 orders of magnitude with the larger reductions associated with larger biases. As can be seen, a small amount of unwarranted conservatism at the design stage can have a major increase in the reliability of a structure. This results in significant and unwarranted increases in the cost of the foundations.

4.2 Improper Selection of Higher Resistance Factors

For pile design, if a geotechnical engineer selects to use a resistance factor higher than 0.4 without adequate methods of direct measurement being conducted and design parameters based on semi-empirical analysis using in-situ and laboratory test data, there will be a significant impact to the reliability of the structure.

By applying a higher Φ in Equation 3, for the same values of bias, $\bar{k}_R$, and COV_R, the reliability index decreases. The result is foundations that are under-designed with an increased probability of failure.

Figure 7 shows graphically the distribution of using a geotechnical resistance factor of 0.5, a $\bar{k}_R$ of 1.1, and an unchanged COV_R. The graph shows a significant increase in the overlap between the load effects distribution and the geotechnical resistance distribution compared to Figure 1.

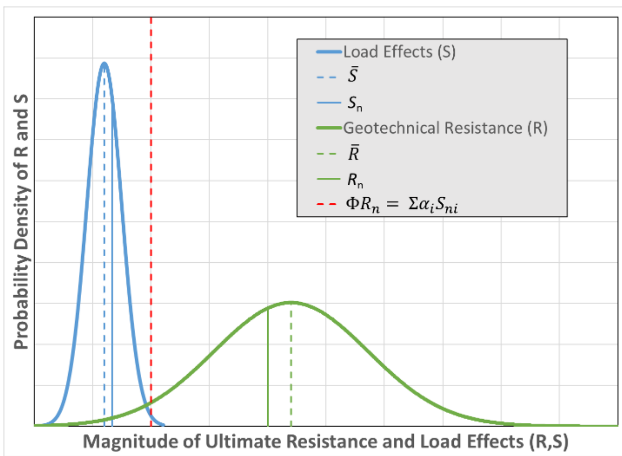

Figure 7. LRFD Distribution for Φ = 0.5 with semi-empirical analyses and $\bar{k}_R$ = 1.1

By modifying the Φ values in Equation 3 from 0.4 to 0.5 and leaving the $\bar{k}_R$ and COV_R values unchanged, the reliability index decreases from 3.4 to 2.6. This causes an approximately 25 times increase in the probability of failure. Without adequate direct measurement to warrant the increase in the resistance factor, there will be significant increased risk to the foundations.

4.3 Lack of Consideration of Monitoring Methods at Design Stage

Methods of direct ultimate resistance measurement reduce the uncertainty in the design (e.g., may confirm a lower COV_R) and thereby a higher resistance factor is warranted so that the reliability does not overly increase. Where adequate direct measurement of the ultimate resistance will be undertaken (e.g., PDA testing), it is inappropriate to utilize a smaller geotechnical resistance factor (such as 0.4) that presumes the absence of direct field measurement of ultimate resistance. Where the geotechnical engineer selects to use a resistance factor of 0.4 for axial compression, despite adequate methods of direct measurement being part of the monitoring plan, an unwarranted increase to the reliability of the foundations is being applied.

Figure 8 shows graphically how using a geotechnical resistance factor of 0.4, a $\bar{k}_R$ of 1.1, but a COV_R reduced to 0.3 (from 0.4) was considered appropriate for adequate dynamic monitoring results. The graph shows a significant decrease in the overlap between the load effects distribution and the geotechnical resistance distribution in comparison with Figure 1; consequently, the reliability index is higher than intended and the design may be overly conservative (and costly).

By leaving the Φ value in Equation 3 at 0.4 but using a COV_R of 0.3, a value applicable for dynamic monitoring results, and maintain a $\bar{k}_R$ of 1.1, the reliability index increases dramatically from 3.4 to 4.5. For a COV_R value of 0.25, applicable for static load testing, the reliability index further increases to 5.4 if a Φ value of 0.4 is still utilized. These results indicate foundations that would be overdesigned by 25 percent and 50 percent considering dynamic monitoring results and static load testing, respectively. These correspond to reductions in the probability of failure of about 1.8 and 4.0 orders of magnitude, respectively.

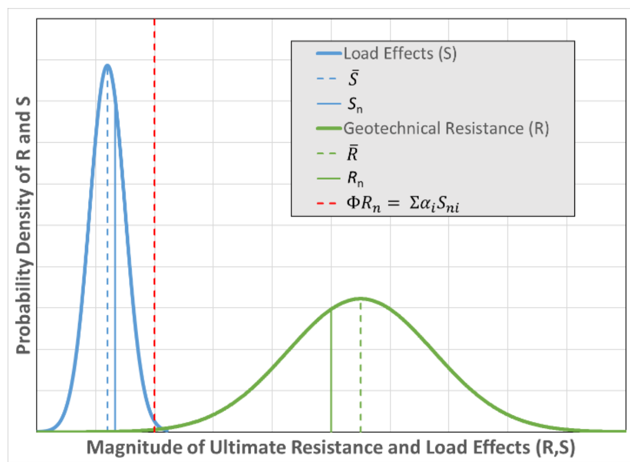

Figure 8. LRFD Distribution for Φ = 0.4 and $\bar{k}_R$ = 1.1 with sufficient dynamic monitoring results

It should be appreciated that selecting changes from the code-specified Φ values must be accompanied by sufficient quality dynamic monitoring results and/or static load testing to verify the use of a higher resistance factor in design (Thomson et al. 2016). It should be appreciated that where the direct measurements suggest that the characteristic values used during design were overestimated, a portion of the piles may need to be remediated. Remediation should be determined through discussions between the structural and geotechnical design engineers.

5 COMMON PILE MONITORING TECHNIQUES

There are many methods of pile monitoring that can be utilized to verify that piles are installed in accordance with design recommendations. However, not all methods provide an indication of the long-term pile ultimate resistance. This section discusses three pile monitoring techniques that are commonly used to indicate the pile resistance that is achieved. Static pile load testing is not presented, as this is more commonly completed at the design (not construction) stage.

5.1 Wave Equation Analyses of Piles (WEAP)

For driven piles, WEAP is commonly undertaken as part of a pile monitoring program. At a basic level, WEAP considers the pile hammer model, the driving helmet, the pile, and the soil stratigraphy to predict pile driving stresses and axial resistance. WEAP is suitable for selecting appropriate piling hammers and limiting the driving energy to reduce the potential for pile damage during installation.

For a specific combination of pile hammer model, driving helmet, pile geometry and the soil stratigraphy, WEAP predicts a relationship between the applied driving energy, pile driving stresses and axial resistances for given pile sets (blows per 0.25 m [or foot]). The values are generally considered inaccurate where the pile set is outside the range of 20 to 80 blows per 0.25 m (3 mm to 12 mm penetration per blow). At pile sets greater than 80 blows per 0.25 m, the ultimate resistance of the pile is not mobilized. At pile sets less than 20 blows per 0.25 m, inaccuracy in blow count measurements make the achieved resistance difficult to confidently estimate.

There are several limitations to the use of WEAP for estimating pile axial resistance. WEAP analyses is typically based on average pile hammer models and approximated soil conditions. To provide reasonable estimates of the actual axial resistance, WEAP must be calibrated against dynamic monitoring results for the specific hammer model and soil conditions. When driving piles into clay-based soils where significant set-up may be expected following initial driving, the WEAP analyses must either be estimated based on re-strike tests following set-up or a set-up needs to be assumed. Depending on the length of time allowed for set-up or the accuracy of the assumed set-up, the prediction of the pile axial resistance could vary significantly. Where the final pile set is outside the range of 20 to 80 blows per 0.25 m, the predicted pile resistances are inaccurate. Either adjustments to the pile driving energy or a different pile driving hammer should be utilized in that scenario.

Overall, WEAP is an additional tool that can be used to select pile driving hammers, limit driving stresses, and provide an assessment of whether the pile resistances achieved in the field are near the expected range. Where the WEAP is adequately calibrated against dynamic monitoring results, WEAP can be used to provide a reasonable prediction of pile axial resistance. WEAP on its own should not be considered to supersede other geotechnical information.

5.2 Pile Driving Analyzer (PDA) Testing

PDA is a form of dynamic monitoring where the force and acceleration applied to piles during driving are monitored (typically using strain gauges and accelerometers attached to the pile). The values obtained from PDA testing can be used to assess the functionality of the pile hammer, the forces imparted on the piles, the energy transferred to the piles and the soil response. It can also be used to assess damage to the piles caused by driving. The PDA can be used to quickly assess the pile resistance throughout driving; however, more thorough estimates of pile axial resistance can be determined using more rigorous (e.g., Case Pile Wave Analysis Program [CAPWAP]) analysis on a single hammer blow near the pile completion depth. The values obtained from CAPWAP analysis may be used to refine WEAP analyses. When an adequate amount of PDA testing is performed, the geotechnical resistance factors can be increased to 0.5 for compression loading.

Like WEAP, there are several limitations in the use of PDA for estimating pile axial resistance. When driving piles into clay-based soils where significant set-up may be expected following initial driving, the long-term axial resistance must either be estimated based on re-strike PDA tests following set-up or a set-up needs to be assumed. Depending on the length of time allowed for set-up and the accuracy of the assumed set-up, the prediction of the pile resistance could vary significantly. Similar to WEAP, and for the same reasons, CAPWAP analysis must be undertaken on a single hammer blow that moves the pile between 3 mm and 12 mm (corresponds to 20 to 80 blows per 0.25 m penetration) and preferably less than 8 mm penetration per blow. Where outside these ranges, either adjustments to the pile driving energy or a different pile driving hammer should be utilized. PDA testing and CAPWAP analyses are specialized methods of interpreting pile resistance. Only knowledgeable and suitably experience personnel who understand the limitations of the methodology should perform PDA testing and subsequent analyses. Experienced personnel can appropriately assess the quality of the data obtained, adequately resolve issues, and reliably interpret the PDA test results.

5.3 Torque Measurements

Torque measurements are generally limited to interpreting the geotechnical resistance of helical piles. As indicated in the 2006 Canadian Foundation Engineering Manual (CFEM 2006), the geotechnical resistance of a helical pile can be determined by multiplying the installation torque, T, by an empirical torque factor, K_T. In general, the K_T value decreases as the shaft diameter increases. However, this methodology has been shown to have poor correlation to static pile load tests (Tappenden et. al. 2006) with highly variable back-calculated K_T values for similar sized piles (Sakr, M. 2011, Sakr, M. 2012).

There are several published papers that attempt to refine the empirical torque factors for larger diameter shafts, compression loading versus tensile loading, and/or helical piles with multiple helices, such as those suggested by Perko (2009) and Sakr (2013). Additionally, there are many proprietary correlations used by various helical pile manufacturers to assess the relationship between installation torque and pile resistance. The required torque to install helical piles will depend on a wide variety of conditions including but not limited to soil stratigraphy, shaft diameter, helix diameter, number of helices, helix spacing, pitch of helix, thickness of the helix, crowd force used during installation, and pore water pressure generation. Given the number of potential variables, it is highly unlikely that a single number for a given helical pile geometry could reliably be utilized to estimate pile axial resistance. Additionally, the use of a torque correlations to estimate pile compressive and tensile resistances ignores the fundamental limitation of a rotational force being utilized to estimate an axial resistance.

Torque correlations should still be utilized for the sizing of helical pile installation equipment. Measurements of torque during pile installation are suitable for estimating the variability in soil conditions and the correlations can be utilized as a crude check on the pile resistance. However, the installation of helical piles should be governed by the design depth with adjustments for installation problems such as augering of the soil.

6 RELIABILITY IMPLICATIONS FROM MISAPPLICATION FROM PILE MONITORING

6.1 Field Test Measurements Improperly Superseding Design Values

It is expected and reasonable to utilize direct measurements of the pile axial resistance such as PDA testing and static load tests to supersede pile resistance estimates from semi-empirical analysis using laboratory and in-situ test data. However, it is not reasonable to have one method of semi-empirical analysis using laboratory and in-situ test data supersede another without sufficient justification.

Typically, initial design values are established based on a geotechnical engineer's interpretation of the in-situ and laboratory testing conducted over the course of a geotechnical investigation. The interpretation of design values will normally be based on local experience and standard practices. Where estimates of the geotechnical axial resistance of piles are based on WEAP for driven piles (absent of adequate calibration against dynamic monitoring results) or torque measurements for helical piles, these estimates are also being based on semi-empirical analysis using laboratory and in-situ test data. By relying on WEAP or torque correlations to establish minimum or target values during installation, the WEAP or torque correlations are superseding the design based on engineering analyses by inferring that the field measurements are more representative than the engineering analyses. Given the previously stated limitations of WEAP and torque correlations for estimating the geotechnical axial resistance of piles, it is not appropriate to supersede the design values without sufficient justification.

Rather than supersede the design, WEAP and torque correlations should be considered supplemental information to cross-check against design values. Where pile resistances estimated by WEAP and torque correlations provide significantly different results than design values (either higher or lower), further analyses would be recommended to determine the reason for the discrepancy. Only when further analyses support revising the design values should the WEAP or torque correlations be taken as more appropriate.

6.2 Design Characteristic Ultimate Resistance Taken as a Minimum Acceptable Ultimate Resistance

Further to the discussion in Section 2.1.2, it is inappropriate to use the design characteristic ultimate resistance as the basis of acceptable minimum resistances to be verified through pile monitoring. If all piles that are inferred based on pile monitoring to have ultimate resistance below the characteristic ultimate resistance are installed to deeper depth, an increased reliability will be achieved. As a holdover from WSD, piling contractors request termination criteria for driven pile installation. For driven piles, set criteria (number of blows per 0.25 m of penetration for a given driving energy) for the various pile types are commonly requested. Set criteria are normally established using WEAP (with or without calibration against dynamic monitoring results) and design resistances. For helical piles, minimum torques are requested that are normally estimated using torque correlations and design resistances. The termination criteria are usually combined with a minimum embedment depth on the design drawings.

By mistakenly using the design characteristic ultimate resistance as the minimum resistance to be achieved in the field, the mean resistance associated with deeper piles will tend to increase, the variation will tend to decrease, and the overall reliability of the installed foundations will tend to increase. Figure 9 shows the effect on the hypothetical distribution of monitoring results (shown on Figure 2) by increasing the axial resistance of all the piles by mistakenly assuming all field measurements must exceed the design characteristic value.

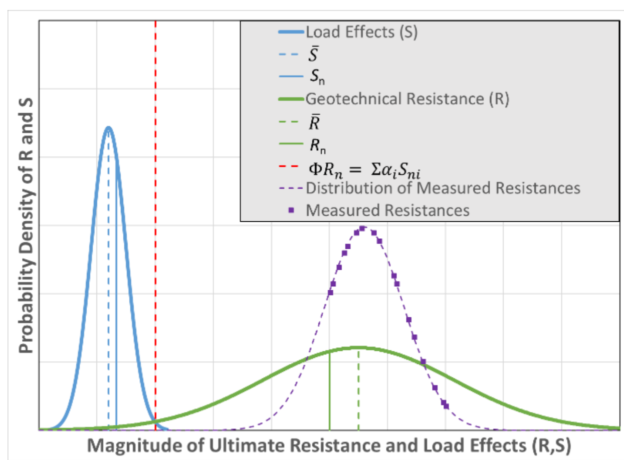

Figure 9. Effect on Measured Resistances when Design Characteristic Resistance is Mistakenly Taken as Required Minimum Resistance

By comparing the distribution of measured resistances on Figure 9 against those shown on Figure 2, the mean resistance increases about 5 percent and the COV_R is about 70 percent of the previous value. Relative to Equation 3, an increase in the $\bar{k}_R$ of 5 percent and a decrease in the COV_R to 70 percent of its original value results in the reliability index increasing dramatically from 3.4 to 5.0. This causes an approximately 3 orders of magnitude decrease in the probability of failure. Such unwarranted conservatism during construction results in a major, but unnecessary, increase in the reliability of a pile foundations and yields significant and unwarranted increases in the cost of the foundations.

6.3 Lack of Consideration of Monitoring Limitations

Further to the discussion in Section 5, each monitoring method has limitations on its ability to reliably assess the geotechnical resistance of a pile. The limitations can partially be based on the timing of the monitoring, such as monitoring driven piles at the end of initial drive compared to re-strike test following set-up. The set-up duration will also have an influence on the monitoring results (e.g., one week versus one month). Where set-up is not considered on driven piles that are solely monitored at the end of initial drive, the monitoring results will typically underestimate the ultimate resistance. Where a set-up is assumed, underestimation of the set-up will underestimate the ultimate resistance; similarly, overestimation of the set-up will overestimate the ultimate resistance. Where time is allowed for set-up and re-strike tests are performed, the degree of set-up will influence the assessment of ultimate axial resistance. On short duration piling projects, it is likely the set-up duration will be limited and therefore the re-strike tests may still underestimate the ultimate resistance, depending on soil type. Where significantly longer set-up durations (in the range of several weeks) are allowed, the re-strike tests will provide a better estimate of the ultimate resistance. The ability of the soils surrounding a pile to dissipate excess pore pressures will influence the duration required to achieve ultimate resistance following driving.

Other monitoring limitations, such as the pile set required to provide suitable resistance estimates in the cases of WEAP and PDA, need to be considered. If the ultimate resistance of the pile is not fully mobilized or excessive vibrations during pile driving make the achieved resistance difficult to predict, the ultimate resistance may be overestimated or underestimated.

Where the ultimate resistances are underestimated or overestimated, the reliability will increase or decrease, respectively.

6.4 Incorrectly Applying Resistance Factors

As previously discussed, the geotechnical resistance factors in the NBCC 2015 for piles in compression may be increased from 0.4 to 0.5 or 0.6 depending on the type and quantity of direct measurement of the pile resistance that is undertaken. The ability to increase the geotechnical resistance factors is dependent on a sufficient quantity of direct measurements being undertaken.

If a single PDA test is undertaken on a large quantity of piles, an increase of the resistance factor would not be warranted. Generally, industry practice is that PDA testing must be completed on 2 to 5 percent of the total number of piles but may need to be increased for a small number of piles or where higher subsurface variability is observed. Pile load tests may be completed on a fewer number of piles. However, the geotechnical engineer would be responsible for proving that the results of the pile load tests can appropriately be transferred to the production piles. Generally, the production piles should be the same type and of similar length and diameter as the loaded tested piles with adequate measurements to be able to infer the response of production piles. Where insufficient testing is undertaken during production or pile load testing, higher geotechnical resistance factors would not be applicable for use. When incorrectly increasing the geotechnical resistance factor to 0.5, the implications on the reliability would be similar to that described in Section 4.2.

Where an appropriate quantity of PDA testing or load testing is undertaken, the increase in the geotechnical resistance factors should be applied to all applicable production piles. If the increased resistance factors were only applied to the tested piles, a large increase in the reliability would occur. In the case of PDA testing, the implications on the reliability would be similar to that described in Section 4.3.

7 CONCLUSIONS

The use of LRFD for the design of foundations based on NBCC 2015 needs to be undertaken with a full understanding of the how the methodology relates to the reliability of foundations. The design and monitoring are to be based on near-mean characteristic values with the ability to utilize higher geotechnical resistance factors when adequate direct measurements of ultimate resistance are undertaken. Improper selection of characteristic values, use of resistance factors, and/or improper application of pile monitoring methods can have a major impact on the reliability of foundations and construction costs. Through proper understanding and application of LRFD, less variability in the reliability of foundations will occur within geotechnical practice.

8 ACKNOWLEDGEMENTS

The authors thank Dr. Peter Thomson, P.Eng. of Golder Associates Ltd. for his insightful review comments of this paper.

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