COMPARISON OF LRFD RESISTANCE FACTORS TO TENSION PILE LOAD TESTS

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Through reliability-based calibration of larger pile load test databases, recent developments of the Load Resistance Factor Design (LRFD) methods, which include geotechnical load and resistance factors for deep foundations design, have advanced the state of practice for geotechnical engineering. These load test databases upon which the current geotechnical load and resistance factors have been developed are limited, however, and do not provide sufficient data to support the development of resistance factors for uplift (tension) pile capacity. As such, some recommended resistance factors for uplift capacity estimation methods are simply derived by reducing the recommended resistance factors for piles in compression by a factor of 0.10.

The results of three (3) tension loading tests performed to failure on 508 mm (20-in) square precast, prestressed concrete piles, driven though granular soil in southeastern Massachusetts, are presented and compared to static pile capacity calculations and dynamic analyses performed using the Pile Driving Analyzer (PDA) during installation. These results are presented along with current recommended LRFD resistance factors for driven piles under uplift (tension) loading conditions. These comparisons offer an assessment of the applicability and limitations of current resistance factors methods for piles under uplift loading and may be useful additions to the existing database for future calibrations of LRFD resistance factors for driven prestressed concrete piles in sand.

Introduction

In 2010, two (2) natural draft cooling towers were constructed by Kiewit (Kiewit) Construction Company of Woodcliff Lake, New Jersey at the Brayton Point Power Station in Somerset, Massachusetts. These massive 500 foot tall, 400 foot diameter cooling towers were designed to convert the existing “open loop” system into a “closed loop system” thereby dramatically reducing the thermal impact to Mt. Hope Bay.

Based on design guidance from the the German VBG Design Code and the Massachusetts State Building Code (MSBC), tension (i.e. uplift) and lateral loads due to wind loads and design seismic events represented the critical loading conditions.

Weighing approximately 190,000 kips upon completion, the cooling tower loads are transferred to 44 foundation nodes (i.e. pile caps) evenly spaced along ring beams which run the circular perimeter of each tower base. Each pile cap node was designed to resist a 2,000 ton compression load, 200 tons in tension (i.e. uplift), and 600 tons in the lateral direction.

Driven 508 mm (20-in) square precast, prestressed concrete (PPC) piles were selected to support the large loads imposed by the tower structures. The original design consisted of a total of 1056 ring piles and 595 basin piles for both towers. To demonstrate that the driven 508 mm (20-in) PPC piles could provide the sufficient resistance to support the controlling loading conditions a comprehensive loading test program was devised by GZA GeoEnvironmental Inc. (GZA) of Norwood, Massachusetts, on behalf of Kiewit. The multiphase testing program included, dynamic pile testing, compression loading tests, tension loading tests to failure, and lateral loading tests.

One (1) compression loading test, five (5) tension loading tests, three (3) lateral loading tests, and twelve (12) dynamic tests were performed on the driven PPC indicator piles.

Site Conditions

A geotechnical study for the project detailed the site history which included the historic placement of 20 to 30 feet (6-9 meters) of cao ash over miscellaneous fill. The miscellaneous fill generally consisted of silty sand and occasional layers of widely graded sand and gravel. The Standard Penetration Test (SPT) N-values varied from 0-100 blows per foot (bpf) with the majority ranging from 10-50 bpf. In some locations, below the fill, a layer of organics was encountered which consisted of organic clay and peat with fine sand containing root fibers and shell fragments. The SPT N-values for the organics varied from 5-30 bpf with the majority ranging from 2-8 bpf. Underlying the organics was a 5-50-ft thick layer of silty sand consisting silty fine to medium sand with some coarser sand and gravel. The SPT N-values ranged from 4-149 bpf, with the majority ranging from 5-55 bpf. The higher SPT N-values were a result of the presence of numerous boulders and obstructions in the stratum. Glacial till was encountered below the silt and sand at depths ranging from 27.5-67.5-ft which consisted of a very dense gray silty sand and gravel. Most SPT N-values ranged from 20-100 bpf. The glacial till stratum is underlain by bedrock consisting of shale, siltstone, and sandstone from the Rhode Island Formation.

PILE LOAD TEST PROGRAM

Embedment Strain Gages

Prior to pile driving, GZA installed six (6) vibrating wire embedment strain gages in each indicator pile. With assistance from the Vynorius Company, strain gages were installed to the reinforcing strands at casting yard in Salisbury, Massachusetts prior to concrete placement. Pairs of strain gages were installed in diametrical locations positioned at 0.5 feet, 5.5 feet, and 15 feet above toe of pile. The Strain gages were cast into the pile to provide load transfer (i.e. micro strain) measurements along the pile shaft during static compression and tension loading tests.

Dynamic Pile Testing

A total of twelve (12) indicator piles were dynamically tested using the Pile Driving Analyzer (PDA) in general accordance with the project specifications and the ASTM Method Designation D4945-89, “Standard Test Method for High-Strain Testing of Piles.”

Dynamic pile testing was conducted using a PAK Model PDA to measure the driving stresses, estimate static capacities, and to evaluate hammer performance during pile installation. In addition, the dynamic pile testing was used to develop a preliminary driving criteria based on the predicted “Case Method” pile capacity at the end of drive (EOD) and the beginning of restrike (BOR).

The preliminary driving criteria was developed based on the pre-driving wave equation analysis of piles (WEAP) using the GRLWEAP Software to size the appropriate hammer system to drive the PPC piles. Prior to installation, the indicator pile locations were pre-augured approximately 30 feet through the existing fill material with a 16-inch auger to clear any obstructions and improve energy transfer to the pile toe. The indicator piles were driven using a Delmag D46-32 (ram weight 10,140 lbs.) open-ended diesel impact hammer.

The PDA was used to make real-time dynamic force and acceleration measurements of the indicator piles during impact driving. These measurements were evaluated in the field to estimate pile capacity and monitor piles stresses and hammer performance. The computer program CAPWAP was used to conduct the post-driving signal matching. The signal matching process was accomplished by performing numerous iterations of changing the soil model variables for each pile element in contact with the soil until the best match of the measured and calculated forces signals were obtained.

Dynamic Testing Results

The average shaft resistance calculated using CAPWAP signal matching was 310 kips with a corresponding standard deviation (STDEV) of 10 and coefficient of variation (COV) of 0.03 for the three (3) indicator piles which were scheduled to be load tested in tension. Table 1 below provides a summary of the estimated shaft resistances calculated using CAPWAP for the three Tension Load Test piles.

Table 1 CAPWAP Estimated Shaft Resistance	
Indicator Pile	**Shaft Resistance (kips)**
IP2A	300
IP5	320
IP8	310
Average	310.0
STDEV	10.0
COV	0.03

Table 2 Tension Loading Test Results	
Indicator Pile	**Measured Capacity (kips)**
IP2A	184
IP5	216
IP8	280
Average	226.7
STDEV	48.9
COV	0.2

Tension Loading Tests

A total of five (5) tension loading tests were conducted on the 508 mm (20-in) square PPC piles in accordance with the Massachusetts Building Code and ASTM Method D3689-90, "Standard Method for Individual Piles Under Static Axial Tensile Load." Three (3) out of the five (5) tests were conducted to failure and will be considered in this report.

The initial design tensile capacity of the piles was 80 kips. The loading sequence for each loading test followed the specified quick test method with load being applied in increments of 10% of the design load, each held for five (5) minutes. The loading schedule was maintained until either; the applied tension load required continuous jacking to maintain the applied load; average pile head deflections exceeded one inch; or the applied tension load reached the structural limits of the reaction frame. The loading was then removed in four equal decrements of 25% of the maximum applied load, each held for ten minutes, until a zero loading condition was reached. Table 2 summarizes the tension loading test results at 0.5 inch pile head movement and Figure 1 provides plots of the load-movement relationship observed for each test.

STATIC PILE CAPACITY ANALYSIS

Calculated Uplift Pile Capacities

As a comparison to the ultimate uplift capacities determined from the tension load tests to failure (IP2A, IP5 and IP8), the Nordlund Method and Beta Method were used to calculate the available shaft resistance along the pile as specified in Chapter 9 of the Design and Construction of Driven Pile Foundations FHWA Report No. FHWA-NHI-05-042 (Hannigan, P.,Goble, G., Likins, G., Rausche, F., 2006).

A separate analysis was conducted for each of the three (3) indicator piles considering the soil strata and properties found at each location. Table 4 summarizes the results of the unfactored uplift pile capacity computations based on the CAPWAP analysis, as well as the Nordlund and Beta Methods.

The calculated mean bias (ratio of measured capacity to calculated capacity) based on the three (3) indicator piles using CAPWAP, the Nordlund Method, and Beta Method were calculated as 0.73, 0.21, and 0.52 respectively. This indicates that the estimated shaft resistances based on CAPWAP, Nordlund Method and Beta Methods over predicted the capacity by a factor of 1.37, 4.81, and 1.94 respectively.

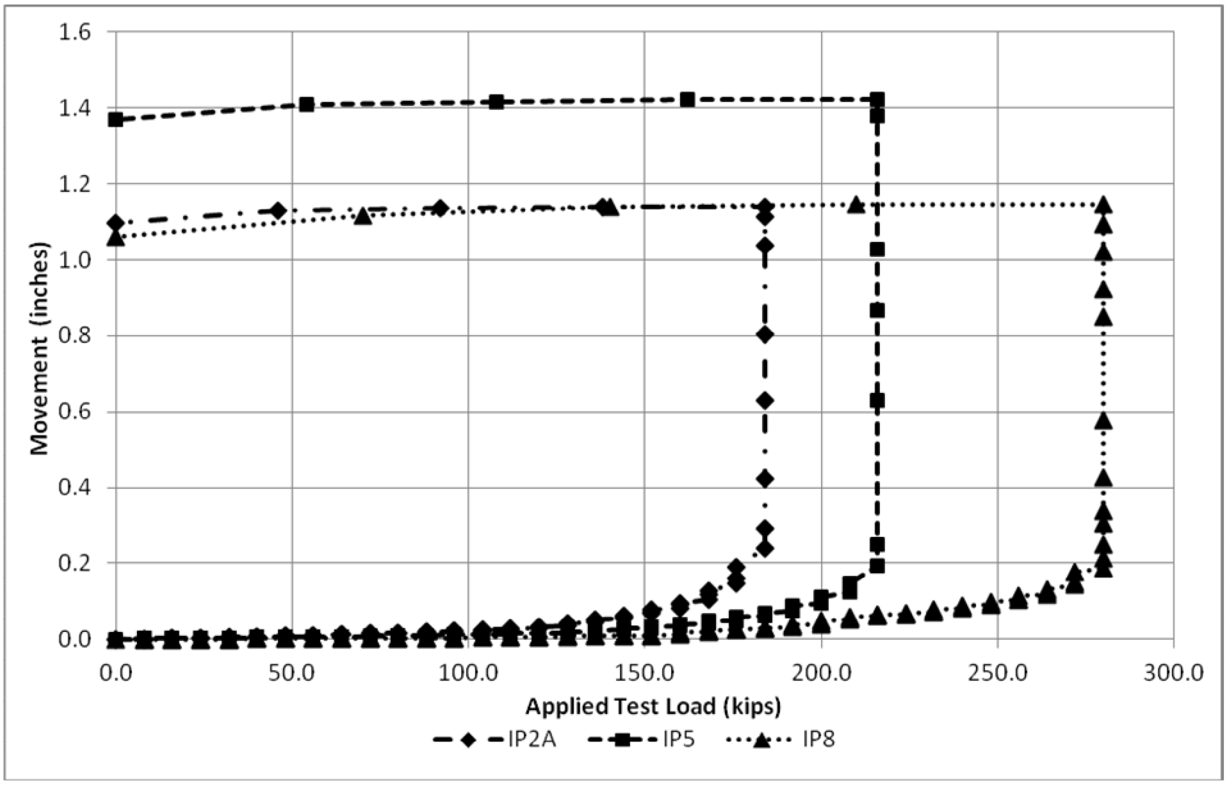

Figure 1: Load-Movement Curve for Tension Loading Tests to Failure

LRFD IN GEOTECHNICAL ENGINEERING

History – Allowable Stress Design

Prior to the implementation of Load and Resistance Factor Design (LRFD) in geotechnical engineering, Allowable Stress Design (ASD) was used successfully since the early 1800's. ASD compares the design load (Q) to the resistance (R_u) with the use of a designated Factors of Safety (FS). The assumed allowable load (Q_{all}) is taken as the ratio of the available resistance (R_u) to the Factor of Safety (FS) as shown in Equation 1.

Equation 1

$$Q \leq Q_{all} = \frac{R_U}{FS}$$

The recommended FS was typically selected from a table with a range of values determined on the level of construction controls (Paikowsky et al., 2004). Table 3 illustrates the recommended FS for the Standard Specification for Highway Bridges (AASHTO, 1997).

In both past and current ASD practice, the FS was primarily selected based on engineering judgment and site-specific experience. Historically, as failures occurred using the existing FS values, an increase of the FS would be determined and implemented going forward based on engineering judgment. In the absence of failure however, the FS was not generally reduced, resulting in the continued application of a possible over conservative FS (Allen et al., Feb. 2005).

The primary advantage of using ASD is that it is relatively fast and simple and has successfully worked since it was first implemented. For this reason, a basic question had risen throughout the geotechnical engineering community - "Why change it if it works?".

To answer this question, it is necessary to highlight the limitations of the ASD FS selections. Table 3 combines all uncertainty into one FS and does not provide a direct evaluation

Table 3: Recommended ASD Factors of Safety (AASHTO, 1997)					
Basis for Design and Type of Construction Control	**Increasing Design/ Construction Control**				
Subsurface Exploration	**X**	**X**	**X**	**X**	**X**
Static Calculation	**X**	**X**	**X**	**X**	**X**
Dynamic Formula	**X**				
Wave Equation		**X**	**X**	**X**	**X**
CAPWAP Analysis			**X**		**X**
Static Load Test				**X**	**X**
Factor of Safety	**3.50**	**2.75**	**2.25**	**2.00***	**1.90**

** For any combination of construction control that includes a static load test, FS=2.0*

of the methods that are being used. Moreover, it is generic as it does not provide details of the type or basis of design and/or construction controls. In addition, there is no differentiation of the type of subsurface investigation undertaken; the static analysis method employed; the type and/or total number of dynamic measurements collected (End of Drive or Restrike); and also the total number of static load tests performed.

One of the major reasons for the development of LRFD in geotechnical engineering was to address the bias and overall uncertainty caused by the limitations listed above.

Calibration Approach by Fitting to ASD

The recommended resistance factors provided in the Standard Specifications for Highway Bridges (AASHTO, 2004) were primarily derived from research conducted by Barker et al. (1991) as part of NCHRP Report 343. Two main approaches were used to develop the resistance factors; calibrate by fitting to ASD and using reliability based theory. Calibration by fitting to ASD is conducted by adjusting the new LRFD resistance factors in order to obtain similar results based on the old ASD specifications. Equation 2 is the typical equation used for calibration by fitting to ASD,

Equation 2

$$\phi = \frac{\gamma_{DL}\frac{DL}{LL} + \gamma_{LL}}{(\frac{DL}{LL} + 1)FS}$$

where $\gamma_{DL} = 1.25$ and $\gamma_{LL} = 1.75$ are the load factors for dead load and live load, respectively, DD/LL is the dead load to live load ratio, FS is the ASD factor of safety, and φ is the calculated LRFD resistance factor.

Calibration by fitting is usually conducted where statistical data is unavailable. The major limitation of calibration by fitting is similar to that of the FS used in ASD. In both cases, there is no consideration of the bias or variability of the load and resistance as well as the probability of failure. In most cases, the recommended resistance factors in the NCHRP Report 343 are derived from calibration by fitting to ASD. (Allen et al., Feb 2005). In conclusion, since calibration by fitting is only a mathematical based design equivalency to calculate the same values based on ASD, it should not be used as a valid calibration for φ (Smith, 2011).

Calibration Approach by Reliability Theory

Factor calibration using reliability-based theory is used to determine the likelihood that failure will occur. In general, failure can be excepted when the loads applied to a given structure are greater than the available resistance. The basic equation that is used for reliability based design is:

Equation 3

$$\Sigma\gamma_i Q_{ni} \le \phi R_n$$

where γ_i is the load factor applicable to a specific load component, Q_{n_i} is the nominal (ultimate) load, φ is the resistance factor, and R_n is the nominal (ultimate) resistance available.

Several different levels of probabilistic designs can be considered when conducting reliability-based calibration. However, regardless of the level of probabilistic design, the common steps to be performed are: 1. Develop a limit state equations and the random variables to be considered, including all parameters that describe the failure mechanism. 2. Statistically characterize the data based on the calibration approach method used. The key parameters to be considered are the mean, standard deviation, and coefficient of variation (COV), and the type of distribution that fits the data (normal / lognormal). 3. Assign the target reliability index (β). The reliability index (β) is defined as the number of standard deviations of the derived probability density function (PDF) for the load effect and resistance (R-Q) separating the mean safety margin from the nominal failure value of zero (Paikowsky et al., 2004). 4. Determine the load and resistance using the same reliability theory that was used to determine the target reliability (Allen et al., Sept. 2005). The calibration process flow chart is shown in Figure 2.

Calibration by Paikowsky et al. (2004)

Paikowsky et al. (2004) developed a robust database of pile load test results for driven piles and drilled shafts and their research and recommendations are presented in the NCHRP Report 507. A rigorous calibration using reliability-based theory was used in the development of the resistance factors, and the analysis generally followed the flow chart shown in Figure 2. The data was statistically categorized using First-Order-Reliability-Method (FORM) in order to be consistent with the load factors found in the current structural code.

The majority of the database case histories were based on SPT and CPT field testing data. The database was first separated into groups based on the soil conditions and then further subdivided based on the pile types (H-Pile, concrete pile, pipe pile) and static analysis method used to calculate the available resistance. Based on the size of the database, it was assumed that the full scale field data adequately accounts and addresses all sources of error.

Target reliabilities indices (β) of 2.3 and 3.0 and probability of failures of 0.1% and 1.0% were chosen for redundant and non-redundant pile configurations, respectively. A redundant pile configuration is defined as five (5) or more pile per pile cap, and a non-redundant pile configuration is defined as four (4) or fewer piles per pile cap. With the division of the database into the subgroups, resistance factors for very specific soil conditions, pile types, and static analysis methods were recommended. Refer to NCHRP Report 507 for additional information.

Calibration of Allen et al. (2005)

The Federal Highway Administration (FHWA) Publication No. FHWA-NHI-05-052 (Allen et al., Feb 2005) provides a summary of the historical development of the resistance factors needed for geotechnical design and also recommends modifications to the same resistance factors based on recent developments. It should be recognized that the focus of the FHWA report is the state-of-practice and the implementation of the already completed research to be incorporated into the AASHTO LRFD specifications. Similar to the NCHRP Report 507, the FHWA report follows the reliability analysis presented in the flow chart in Figure 2.

The data, however, was categorized using the Monte Carlo method (Allen et al., Feb 2005). In the FHWA report, the results presented by Paikowsky et al. (2004) and Barker et al. (1991) were evaluated and new resistance factors were recommended. In addition, the FHWA report determined that there was no clear pattern in the calculated capacities for different pile types.

The database was split into subdivisions based only on soil conditions and static pile analysis methods and subdivisions based on pile type were not considered. It is stated in the FHWA report that “intuitively, there should be at least a minor difference between the various pile types regarding the skin friction. Therefore, it is recommended that designers be made aware of the more detailed data and resistance factor recommendations in NCHRP Report 507 (Paikowsky et al. 2004), to assess on a project specific basis whether or not the selected resistance factor should be differentiated based on pile type” (Allen et al., Feb 2005). The FHWA report also does not take into account the redundancy of pile groups.

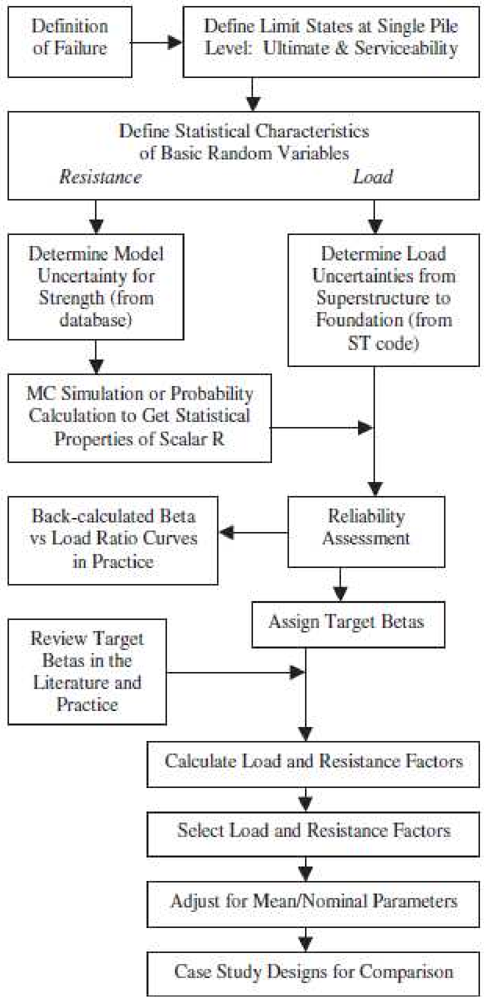

Figure 2: Resistance Factor Calibration Process (Paikowsky et al. (2004))

Evaluation of Recommended Resistance Factors

Recommended LRFD resistance factors as well as the Allowable Stress Design Factors of Safety (ASD FS) for calculating allowable uplift loads were considered in this report. The traditional ASD FS of 3.5 was applied as well as the resistance factors as specified in the Section 10 of AASHTO (2010) and in Table 26 of the NCHRP Report 507.

The current recommended resistance factors found in Section 10 of AASHTO (2010) Specifications for uplift resistance of a single pile using CAPWAP, Nordlund Method, and Beta Method are 0.50, 0.35, and 0.20 (φ) respectively. These resistance factors are based on the recommendations by Allen et al. (Feb 2005) in the FHWA Report No. FHWA-NHI-05-052. No real reliability-based calibrations were used to calculate this recommended resistance factor and it was based simply on reducing the resistance factors for compression loading by a value of 0.10. Most of the resistance factors for uplift resistance recommended by Allen et al. (Feb 2005), which are found in the current AASHTO (2010) Specifications, are deduced by the same mathematical reduction and are not based on reliability-based calibration of actual field data.

The recommended resistance factors for uplift found in Table 26 of the NCHRP Report 507, however, were recommended using reliability-based theory. The recommended resistance factors in Table 26 for non-redundant pile configuration are 0.15 and 0.20 (φ), for the Nordlund and Beta Methods, respectively. The factored shaft resistances using each method are shown in Table 5.

It should be noted that a calculated mean bias of one indicates that the predicted shaft resistance is equal to the actual measured shaft resistance. In addition, a mean bias less than one indicates that the predicted shaft resistance was over estimated and a mean bias greater than one indicates that the predicted shaft resistance was under estimated. The average factored tension loading test results will be considered in the next three sections.

CAPWAP

The mean bias pertaining to the average factored CAPWAP resistance and average factored tension loading test results was calculated to be 0.88. This indicates that the predicted factored tension load test results were calculated to be 0.88. This suggests that the predicted factored shaft resistance was slightly

Table 4: Unfactored Measured and Calculated Shaft Resistances							
Indicator Pile	Measured Capacity from Loading Test (kips)	CAPWAP Estimated Shaft Resistance (kips)	Estimated Shaft Resistance based on Nordlund Method (kips)	Estimated Shaft Resistance based on Beta Method (kips)	Bias= measured/calculated		
					CAPWAP	Nordlund Method	Beta Method
IP2A	184	300	1000.2	438.4	0.61	0.18	0.42
IP5	216	320	959.5	387.3	0.68	0.23	0.56
IP8	280	310	1315.4	493.2	0.90	0.21	0.57
Average	**226.7**	**310.0**	**1091.7**	**439.6**	**0.73**	**0.21**	**0.52**
STDEV	48.9	10.0	194.8	53.0	0.2	0.02	0.08
COV	0.2	0.03	0.18	0.12	0.21	0.10	0.16

Table 5 - Summary of Calculated Factored Pile Resistances							
Indicator Pile	Factored Resistances Based on ASD FS and LRFD Resistance Factors						
	Loading Test	CAPWAP	Nordlund Method			Beta Method	
	Based on AASHTO 2010 φ=0.60 (kips)	Based on AASHTO 2010 φ=0.50 (kips)	Based on ASD FS=3.5 (kips)	Based on AASHTO 2010 φ=0.35 (kips)	Based on NCHRP Report 507 φ=0.15 (kips)	Based on ASD FS=3.5 (kips)	Based on AASHTO 2010 and NCHRP Report 507 φ=0.20 (kips)
IP2-A	110	150	286	350	150	125	88
IP-5	130	160	274	336	144	111	77
IP-8	168	155	376	460	197	141	99
Average	**136**	**155**	**312**	**382**	**164**	**126**	**88**
Mean Bias to Factored Loading Test		**0.88**	**0.44**	**0.36**	**0.83**	**1.08**	**1.55**

** Table assumes a non-redundant pile configuration (4 or fewer per pile cap)*

over estimated as compared to the factored tension loading tests results. It should be noted, however, that when compared to the unfactored loading test results, the mean bias can be calculated as 1.46 which can be considered an accurate estimation.

Nordlund Method

The calculated shaft resistance using the Nordlund Method was factored using the ASD FS and LRFD resistance factors. In each case, the calculated shaft resistance was over estimated. The calculated factored shaft resistances when using both the recommended ASD FS and AASHTO (2010) was highly over estimated with a mean bias of 0.44 and 0.36 respectively. As mentioned previously, the recommended resistance factor of 0.35 from AASHTO (2010) was deduced by a mathematical reduction and was not based on reliability-based calibration of actual field data.

The calculated factored shaft resistance when using the recommended resistance factor found in the NCHRP Report 507, which was developed using reliability-based calibration, provides a mean bias of 0.83, which falls closely to that which was calculated by CAPWAP.

Beta Method

Similar to the Nordlund Method, the Beta Method was used to calculate the factored shaft resistance with a corresponding ASD FS of 3.5 and a LRFD resistance factor of 0.20. In using both the ASD FS and LRFD resistance factor, the factored resistance was slightly under estimated however remains a fairly accurate estimation when compared to the tension loading test results.

Conclusion

The recent development of geotechnical load and resistance factors for deep foundations design, through reliability-based calibration of large pile load test databases, has advanced the state of practice of geotechnical foundation design. The change in geotechnical engineering from ASD to LRFD based on statistical data is a major advancement that is slowly being adopted by the practicing engineer. It should be noted, however, that not all the recommend resistance factors found in the AASHTO (2010) Specifications were developed by reliability-based calibration and should be used with caution similar to when using the Factors of Safety from ASD.

It should be noted that the typical ASD FS listed in Table 3 lumps all uncertainty into one FS and does not provide a direct evaluation of the methods that are being used. This can be seen when comparing the results from the Nordlund and Beta Methods when applying the same FS of 3.5.

The AASHTO (2010) Specifications attempted to simplify the recommended resistance factors by neglecting the pile type and the lack of reliability-based calibration to develop the resistance factors for uplift. This simplification should be recognized by the practicing geotechnical engineer. The comparisons of the calculated factored capacities found in this report illustrate the importance of using reliability-based calibration to determine the resistance factors rather than using a simplified mathematical relation to reduce the resistance factor for compression loading by 0.10.

The practicing engineer should refer to the NCHRP Report 507 (Paikowsky et al 2004) for a more in-depth, case-specific, and reliability-based calibration of resistance factors. The authors strongly recommend using reliability-based resistance factors and anticipate that the measured and calculated data found in this case study and in future studies may pertain useful for the future calibration and modification of LRFD resistance factors.

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