Determining the Capacity of Reused Bridge Foundations from Limited Information

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Abstract: Reuse of bridge foundations often requires determining the capacity of existing foundations, including driven piles. These piles have a proven history of load-carrying capacity but may lack test data from which the capacity can be obtained. In general, pile capacity can be estimated from the pile geometry and soil properties using empirical calculations. However, these produce highly uncertain results due to the variable nature of soil and pile-driving methods. As a result, LRFD practice uses low resistance factors for pile design based on these calculations, which can produce inefficient designs, sometimes with lower capacity than the loading observed during the initial service life. This research proposes a novel reliability-based methodology to update the capacity of individual piles based on previously observed loading to the pile group. A likelihood function is developed that accounts for the probability that a single pile has low capacity given that the total group capacity was greater than the observed loading during its initial service life. The results are tabulated to show the increases to the LRFD resistance factors that are possible while maintaining the reliability index (and corresponding probability of failure) for standard LRFD designs.

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Introduction

When an existing in-service bridge is being rehabilitated, widened, or replaced, significant economic advantages can be realized through alternatives that reuse substructure and foundation components (Boeckmann and Loehr 2017). Foundation reuse can be particularly beneficial for accelerated and low-traffic-impact construction methods by maintaining the existing alignment and reducing construction activities. Many foundations that may be candidates for reuse are supported by driven piles, potentially with limited or nonexistent test data, installation records, and quality control. These foundations may have questions surrounding their durability, capacity, or integrity. This research proposes a novel approach to determining the geotechnical capacity, which can only be used when durability and integrity questions have been answered.

Modern construction of new foundations supported by driven piles will typically employ static or dynamic testing to determine the capacity of piles. Techniques for pile capacity estimation, such as static analysis or dynamic analysis (based on driving criteria), typically provide highly variable estimations of pile capacity, as shown by Paikowsky (2004). These equations are frequently used for sizing piles and determining preliminary capacity but are assigned low resistance factors for design by the LRFD Bridge Specifications (AASHTO 2014), which can lead to highly inefficient designs. For reused foundations, the factored capacity determined using design equations may be less than the load previously applied to the bridge. A common formalized procedure for accounting for the previous loading history when estimating pile capacity does not currently exist.

This article proposes a reliability-based methodology that updates resistance factors prescribed by the LRFD Specifications (AASHTO 2014) using the previous loading history. The proposed methodology considers the previous loading applied to the pile cap to determine the likelihood of one of the piles having a low pile capacity. A distribution of ultimate pile capacity is then updated considering this knowledge of the previous loading. The updated distribution is used in a reliability analysis to produce designs with consistent probabilities of failure. A second methodology is explored that does not consider a method for pile capacity prediction but assigns a capacity based solely on the previously applied loading. The former methodology is appropriate in situations where the capacity can be estimated using design equations, whereas the latter method does not require prior estimation of the pile capacity.

Background

The LRFD approach seeks to maintain a constant probability of failure for components by prescribing load and resistance factors based on the expected variation of load and resistances (Barker et al. 1991; Nowak 1999). The LRFD Bridge Specifications (AASHTO 2014) provide resistance factors for the geotechnical capacity of driven piles that vary based on the calculation method used to determine that pile’s capacity. These factors account for two main sources of pile capacity variability: the variation of pile capacities on a single site (onsite variation) and the inherent uncertainty associated with the prediction method. When pile capacities are based on the results of static testing to failure, a resistance factor of 0.75–0.80 is recommended to account for onsite pile capacity variation. A slightly lower factor ranging from 0.65 to 0.75 is recommended for
capacities derived from dynamic test data obtained during pile driving. For new foundations, one of these forms of testing is frequently employed. Static analysis methods, commonly used for preliminary pile sizing, predict capacity from empirical equations based on soil properties and produce highly variable results. When not augmented with test data, capacities derived from empirical equations require low resistance factors ranging from 0.25 to 0.50. In addition, pile-driving formulas that estimate capacity on driving performance allow for resistance factors ranging from 0.10 to 0.50. The guidance given by AASHTO (2014) largely follows the recommendations of Allen (2005), who summarized prior work done by Paikowsky (2004) and Barker et al. (1991) to determine resistance factors that provide a consistent probability of failure.

Ellingwood et al. (1980) conducted early research into calibrating load and resistance factors for structural components using reliability analysis. Six load cases were considered that included combinations of live, dead, wind, and earthquake loading. Ellingwood and Galambos (1982) detail this methodology and provide summaries of target reliabilities for various structural components. Moses and Verma (1987) applied reliability-based design principles to evaluate in-service bridges, including a methodology that load-rated components by their reliability indexes.

Barker et al. (1991) applied reliability analysis to driven pile capacity using mean value first-order second-moment (MVFOSM) reliability analysis. This research calculated the required resistance factors for driven piles using then-current load-factor design (LFD), which used load factors of 1.3 and 2.17 for dead and live loads, respectively. It was observed that previous working stress design (WSD) codes provided reliability indexes between 1.6 and 3.1, with long-span bridges having higher reliabilities due to a higher ratio of dead to live load. Target reliabilities between 2.5 and 3.0 were suggested for axially loaded driven piles, although it was suggested that 2.0 to 2.5 might be appropriate due to the ductile nature of pile failure and the effects of group redundancy.

Nowak (1999) studied the reliability of bridge structural components designed using LFD and allowable stress design (ASD) for spans of various lengths. Like Barker et al. (1991), it was shown that long-span bridges were typically built to higher reliabilities than short spans, due to the higher ratio of dead to live load. A parametric study of the variability of dead and live loads on bridges was performed by Nowak (1999), who found that these loads followed lognormal distributions with the biases and coefficient of variations (COVs) as listed in Table 1.

Ayoub and Assakkaf (1999) have presented a method that uses the first-order reliability method (FORM), pioneered by Hasofer and Lind (1974), to determine partial safety factors (PSFs) for the loading and resistance of bridge components. This methodology allows for all required safety factors to be computed at once, or for the resistance factor to be computed, given predetermined load factors. Paikowsky (2004) followed this methodology to produce resistance factors for use in LRFD along with the dead- and live-load factors of 1.25 and 1.75, respectively. Paikowsky (2004) followed the lognormal distributions presented by Nowak (1999) for dead and live loading on the bridge foundation.

To develop pile capacity distributions, Paikowsky (2004) compiled test data on the performance of static analysis and dynamic capacity estimation methods. Static methods do not typically account for installation, whereas dynamic capacity estimation methods are primarily based on the details associated with driving, such as hammer energy, pile penetration, and driving criteria. The compiled test data consisted of hundreds of piles where the nominal capacity was calculated using static and dynamic capacity estimation methods and the ultimate capacity was found through static testing to failure. From the results, a lognormal distribution of the ratio of ultimate capacity to nominal capacity was calculated for each prediction method. Target reliabilities of 2.33 for redundant piles (pile caps with four or more piles) and 3.0 for three or fewer piles were suggested.

Allen (2005) compared the resistance factors found by calibrating existing ASD practice (following Barker et al. 1991) with those found through reliability analysis (Paikowsky 2004). From these results, recommendations were made for resistance factors to be used in LRFD analysis alongside dead- and live-load factors of 1.25 and 1.75, respectively. The recommended resistance factors were not generally the explicit result of a reliability analysis but were in line with the calibrated factors from Paikowsky (2004). Hence, the proposed methodology will update the resistance factors provided by Allen (2005) and AASHTO (2014) using an updated reliability analysis that follows from the data compiled by Paikowsky (2004).

Zhang and Tang (2002) have proposed a method for updating the reliability analysis proposed by Barker et al. (1991). This methodology uses Bayesian sampling to update the distribution of the mean pile capacity using load test results from individual piles. The original mean capacity distribution is derived from a prior distribution of pile capacities governed by the capacity estimation technique and an assumed onsite variance. A new resistance factor is found using the MVFOSM approach. Park et al. (2015) followed this approach to update resistance factors on a site with 10 individual pile tests.

Abdallah et al. (2015) also proposed a new updating strategy using individual pile tests. This strategy defines a lower bound of pile capacity that is utilized in a reliability analysis to update the required resistance factor. Huang et al. (2016) developed an approach to updating individual pile capacity without considering the distribution of the mean pile capacity. This methodology still relies on load tests of individual piles. Zhang (2017) discusses the implications of Bayesian updating techniques for geotechnical engineering practice. Several barriers to widespread usage of Bayesian methods were identified, including a lack of available computer codes and unfamiliarity with Bayesian methods.

### Proposed Methodology

To conform with the current code approach, the proposed methodology provides a method for updating the distribution of individual pile capacity on a site. This new distribution will be used in a reliability analysis to determine new resistance factors for driven piles considering the past loading on a bridge foundation. A major advantage of this approach is that the results can be employed by practicing engineers by utilizing the prescribed resistance factors provided later in this article.

Because the loading history considered in this research consisted of loading on a set of piles simultaneously, it is useful to consider that set to be a population of piles in which each individual pile is simply a sample. Individually, these piles will all have the same probability density function (PDF) that links their ultimate capacity to predicted (nominal) capacity. However, it can be further assumed that the capacities of the piles will be related to each other. In the previous research from Paikowsky (2004) and Zhang and Tang

<table>
<thead>
<tr>
<th>Load type</th>
<th>Bias of prediction (λ)</th>
<th>COV of distribution (σ/μ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live loads</td>
<td>1.15</td>
<td>0.2</td>
</tr>
<tr>
<td>Dead loads</td>
<td>1.05</td>
<td>0.1</td>
</tr>
</tbody>
</table>
(2002), this relation has been defined as a COV between individual piles on the site.

Previously observed loading on bridge foundations consists of a mixture of dead loads and live loads. During reuse, a portion of the existing dead load corresponding to the reused components will remain on the foundation. Another portion will be removed and replaced, and this replacement load may be lower or higher than the original dead load. Live loading can consist of operational trucks, past permit loading, or test trucks. Loading from past permitting or test trucks can be used to update pile capacity because it can be observed that such loading did not cause significant distress (e.g., as excessive settlement), and the magnitude of the load is reported in permit documents. A computer model may be needed to determine how much of the load reaches each pile group. It cannot be known from this loading how much load was transferred into each individual pile because piles may have varying stiffness, and the ductile nature of pile failure will cause a pile near failure to increasingly lose stiffness and transfer loads to the adjacent piles. It cannot necessarily be determined through observation that any individual pile failed or did not take up adequate load unless structural distress has occurred.

Instead, it can be determined from the previous loading that the average capacity of the piles being loaded is at least as great as the average load that was applied to that population. A population of piles refers to a group of piles that were similarly driven and similarly loaded during testing, some of which may be battered. Battered piles subjected to purely vertical load will experience greater axial loading than vertical piles because the axial force in battered piles will contain a lateral component as well. However, this effect will be minor in comparison to the loading history, especially for low batter angles. The proposed approach of updating pile capacity by considering the average vertical load will yield a slightly conservative result because it does not include any axial loading caused by lateral loads. The new resulting capacity will be the axial capacity of all piles (including battered).

It would be impossible to measure the actual onsite variation between pile capacities without obtaining test data of individual pile capacities. However, statistical data have been compiled in the literature that attempt to quantify the onsite variance of pile resistance. Based on research by Kay (1976), Evangelista et al. (1977) and Zhang and Tang (2002) have adopted a blanket site variance of 0.20. Paikowsky (2004) classified sites into three categories of variability: low, medium, and high. Based on the research by Phoon and Kulhawy (1996), these three site categories were taken to have onsite pile capacity COVs of 0.15, 0.25, and 0.35, respectively. Selection of an appropriate variability for a site always requires engineering judgment because the actual variability of the site will depend on the soil variability, the variability of installation techniques, and various other factors that are difficult to quantify. This method is applicable to sites with any selected variability, although only sites with low, medium, and high (COVs of 0.15, 0.25, and 0.35) variabilities are presented in this article.

**Distribution of Individual Pile Capacity**

Paikowsky (2004) considered the ratio of ultimate pile capacity to the nominal capacity of an installed pile to be a random variable following a lognormal distribution. This distribution is defined by its bias (ratio of mean ultimate capacity to nominal capacity) and COV. Eq. (1) shows a lognormal distribution typically used to model individual pile capacity (Paikowsky 2004; Nowak 1999).

\[
f_X = LN\left(\mu_x, \sigma_x\right)
\]

Fig. 1 shows an example distribution of pile ultimate capacity normalized by the capacity predicted by the \( \beta \)-method for an H-pile in clay where \( \mu_x = 0.61 \) and \( \sigma_x = 0.37 \). Due to the variability of this prediction method, a resistance factor of 0.25 is specified by AASHTO (2014) when pile capacity is determined only from its results.

**Population Capacity Distribution**

For updating purposes, it is convenient to transform the parameters in Eq. (1) using the logarithmic transformations (Ang and Tang 1975) shown in Eqs. (2) and (3).

\[
\sigma_{x,\ln} = \sqrt{\ln\left(\sigma_x^2 + 1\right)}
\]

\[
\mu_{x,\ln} = \ln(\mu_x) - \frac{\sigma_{x,\ln}^2}{2}
\]

Zhang and Tang (2002) have proposed a methodology to calculate the distribution of the population mean using the distribution of individual pile capacity and the expected onsite variability. The distribution of the mean is given by Eq. (4) for normally distributed pile capacity and site variability. The parameters governing the PDF of the population mean are given by Eqs. (5) and (6).

\[
f'(\mu) = N_{\mu'}(\mu', \sigma')
\]

\[
\mu' = \mu_{x,\ln}
\]

\[
\sigma' = \sqrt{\sigma_{x,\ln}^2 - \sigma^2}
\]
with respect to both the nominal pile capacity and the number of piles so that the distribution represents the ratio of the population mean capacity to the predicted capacity. Fig. 2 shows three possible distributions of population mean capacity corresponding to low-, medium-, and high-variability sites for a site where the individual pile capacities follow the distribution in Fig. 1 (which represents an H-pile in clay with capacity determined using the \( \beta \)-method).

### Updating Population Mean Capacity Distribution

Zhang and Tang (2002) have employed Bayesian sampling theory and have provided a new distribution of the population mean, given the results of a limited number of pile tests to failure. The new population is normally distributed (or lognormally distributed after conversion) and is given by

\[
f^x(\mu) = N_{\mu_x}(\mu^x, \sigma^x) \tag{7}
\]

where \( \mu^x \) and \( \sigma^x \) are given by Eqs. (8) and (9).

\[
\mu^x = \frac{x(\sigma^2_x + \mu^2_x)}{(\sigma^2_x + \mu^2_x)} \tag{8}
\]

\[
\sigma^x = \sqrt{\frac{(\sigma^2_x + \mu^2_x)}{(\sigma^2_x + \mu^2_x)}} \tag{9}
\]

In Eq. (8), \( x \) is the average test result (pile capacity) from an \( n \)-sized sample of piles. The novel aspect of this research is that no individual testing is required to be performed on piles. Instead, the distribution described by Eq. (7) are utilized to determine the likelihood of the population mean capacity being at least as large as observed, considering all possible pile capacities for a single pile (\( n = 1 \)). Hence, the distribution of the population mean can become a function of the individual capacity of an arbitrary pile, \( x \); the mean and variance of individual pile capacity (\( \mu_x \) and \( \sigma_x \)); and the onsite variance, \( \sigma \), following Eqs. (1)–(6). Figs. 3(a and b) show the PDF of the population mean (for the example case of H-piles in clay, \( \beta \)-method) for a site variance (\( \sigma \)) of 0.15, given two possible values of individual pile ultimate capacity, \( x \) (shown as a percentage of the nominal capacity).

### Bayesian Updating of the Pile Capacity Distribution

Following Bayes’ theorem (Ang and Tang 1975), an updated probability distribution can be obtained using

\[
f_{updated}(x) = \int_{-\infty}^{\infty} f(x|x)(x) \, dx \tag{10}
\]

where \( f_{updated}(x) \) is the new PDF that describes the probability of a pile having a specific capacity, given the observations on past loading history. Here, \( P(x|x) \), commonly referred to as a likelihood function, represents the probability of observation \( x \) being made, given \( x \); and \( f(x) \) is the prior distribution of pile capacity, as shown in Fig. 1. In this methodology, this specifically refers to the likelihood that the population mean was at least as large as the observed loading for a given \( x \), as shown in Eq. (11).

\[
P(x|x) = L(x) = P(f^x(\mu|x) \geq \text{past loading}) \tag{11}
\]

where \( f^x(\mu) \) is a function of \( x \), as shown by Eqs. (7)–(9).

Figs. 4(a and b) show the population mean distributions [from Figs. 3(a and b)] along with the maximum past loading, shown by a vertical dashed line. In these figures, the solid line represents the \( f_{updated}(x) \) obtained in Eq. (10), and the shaded region represents the range of possible population means that are equal to or greater than the previously observed loading. The likelihood that this observed population mean could occur along with an arbitrary possible single pile capacity, \( x \), is denoted by the shaded area and is noted in the legend of these figures as Likelihood. The area of the shaded region is always between 0 and 1.0 and forms the likelihood function \( L(x) \).

![Fig. 2. Three possible distributions of population mean.](image-url)

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Figs. 4(c and d) show these same distributions and likelihoods if a higher maximum past loading of 30% of the nominal capacity has been previously observed.

As can be seen in Fig. 4, the likelihood function generally decreases as \( x \) decreases, although the magnitude depends on the magnitude of the past loading. The likelihood function in Eq. (11) was numerically integrated using the MATLAB \textit{integral} function to determine the constant that forms the denominator of Eq. (10). The equation for the posterior pile capacity PDF given in Eq. (10) was solved again using numerical integration. In practice, the primary difference between the prior and posterior distributions is the reduction in probability of low values of \( x \). This can be observed by comparing the PDF of the prior distribution from Fig. 1 with updated distributions considering a past loading of 20% and 30% of the nominal pile capacity, as shown in Fig. 5.

The solid line in Fig. 5 shows the PDF of the ultimate capacity normalized with respect to the nominal capacity for a pile with its capacity predicted by the \( \beta \)-method. The dashed line in Fig. 5 accounts for a past loading of 20% of the nominal pile capacity being applied to the pile population, on average. The dotted line accounts for a previous past loading of 30% of the nominal pile capacity. The updating process accounted for the unlikelihood of a single pile having an ultimate resistance far below that of the population mean and provided a new distribution that can be used to

![Fig. 3. Distribution of population mean: (a) pile with 15% of the nominal capacity; and (b) pile with 30% of the nominal capacity.](image1)

![Fig. 4. Distributions shown in Figs. 3(a and b): (a and b) with a past loading of 20% of the nominal capacity; and (c and d) with a past loading of 30% of the nominal capacity.](image2)
estimate the reliability of the pile with respect to future uncertain loading. The new distribution is numerical, rather than a defined distribution (i.e., lognormal).

### Determining a New Resistance Factor

A new resistance factor is calculated using the updated pile capacity distribution described previously. A partial safety factor (PSF) formulation, as described by Ayyub et al. (2002) and followed by Paikowsky (2004) and Nowak (1999), was used to find the resistance factor required to reach a target reliability index of 2.33. The PSF formulation can be used with an arbitrary limit state, and the limit-state equation solved to derive this factor is given by

\[ g = R_{nom} \cdot x_1 - LL_{nom} \cdot x_2 - DL_{nom} \cdot x_3 \]  \tag{12}

where \( R_{nom}, LL_{nom}, \) and \( DL_{nom} \) = nominal resistance, live load, and dead load effects, respectively. Variables \( x_1, x_2, \) and \( x_3 \) in Eq. (12) represent the distributions that describe the ultimate pile capacity (updated), live loads, and dead loads, respectively, normalized with respect to their respective nominal values. The parameters governing the distributions of \( x_2 \) and \( x_3 \) are given in Table 1. To solve the general limit-state equation in Eq. (12), \( LL_{nom} \) is set to be 1, and \( DL_{nom} \) is equal to the ratio of dead to live load \( (Q_d/Q_L) \). This ratio is taken to be 2.5 for this analysis, following Paikowsky (2004), although the methodology allows the selection of any ratio. A reliability analysis following Hasofer and Lind (1974) is then performed with the value of \( R_{nom} \) iterated until the target reliability index is achieved. Because the load factors to be used on the live and dead loads are set by AASHTO (2014) to be 1.75 and 1.25, respectively, the formula for the appropriate resistance factor is given by

\[ \phi = \frac{1.75(1) + 1.25 \left( \frac{Q_d}{Q_L} \right)}{R_{nom}} \]  \tag{13}

where \( Q_d/Q_L \) = ratio of dead load to live load; and \( R_{nom} \) = nominal resistance value converged during the analysis. Because the updated distribution has a lower likelihood of the ultimate capacity being far lower than the nominal capacity, a lower \( R_{nom} \) is required to achieve the same reliability index, leading to higher \( \phi \) values. Fig. 6 shows the calculated required resistance factor for various levels of past loading observed. This figure is only for a single distribution, denoted as Original \( \phi \) value, given by Paikowsky (2004), and does not correspond to a specific resistance factor given in the LRFD Bridge Specification (AASHTO 2014).

Fig. 7 shows the resistance factor as a function of the largest previously applied loading for three cases of site variability. Original \( \phi \) value refers to a constant value of \( \phi \) as per Paikowsky (2004). The increase in resistance factor \( \phi \) with an increase in the largest previously applied loading becomes lesser as the site-variability COV increases from 0.15 to 0.35. Nevertheless, the benefits of a higher value of resistance factor \( \phi \) due to consideration of the largest previously applied load for all three site variabilities are obvious from Fig. 7.

### Impact of Uncertainty in Previously Applied Loading

A sensitivity analysis has been performed to understand the impact of uncertainty in the previously applied loading on the proposed
methodology. An updated resistance factor, $f$, was found considering two scenarios: (1) the actual previous loading was deterministic but was higher or lower than estimated, and (2) the previously loading was treated as a single probabilistic value with a variety of possible variances. The updated resistance factor ($f$) for both analyses determines the resistance factor ($f$) that is required to meet the target reliability (2.33), considering the actual (not estimated) loading. This procedure is useful for determining how sensitive the results are to inaccurate estimations of past loading. Fig. 8(a) shows the impact of the previously applied loading being higher or lower than that expected for the $\beta$-method example on a site with an in-site COV of 0.15 (the solid top line in Fig. 7). Fig. 8(b) shows the impact of a probabilistic loading being used to update the required resistance factor ($f$) to maintain a reliability of 2.33 for the $\beta$-method example. This analysis finds the likelihood function shown graphically in Fig. 4 using a second reliability analysis that determines the probability of the group capacity being greater than the applied load. This greatly increases the computational time because the methodology provided by Hasofer and Lind (1974) needs to be computed for many possible values of individual pile capacity, $x$, to numerically determine the integral in the denominator of Eq. (10).

From Fig. 8, a biased estimate of the previous loading applied impacts the updated curve even at very low loads (relative to nominal pile capacity). As expected, if the actual applied load is lower than estimated, a lower resistance factor ($f$) is required to maintain a reliability index of 2.33, and a higher resistance factor ($f$) maintains this index if the load was underestimated. Thus, an underestimate of the previous loading is a conservative choice.
previous loading is treated as probabilistic, there is little impact at low previous loads relative to nominal pile capacity). If the COV of the previous load is 0.2 (the value typically used for live loads), then the reduction to the required resistance factor ($\phi$) roughly matches that of a 20% reduction in deterministic load. However, permit loads are expected to have a much lower uncertainty than typical live loads due to reporting requirements and verification of truck weights. It is therefore recommended that the previous loading be treated as a deterministic load with an appropriately conservative estimate of load used as the updating load. The level of conservatism should account for Fig. 8. This allows the use of Table 2 in the next section for updating purposes, rather than implementation of the entire methodology.

Application of Findings to LRFD Code

The statistical data on pile capacity distributions presented by Paikowsky (2004) are divided into 41 static capacity prediction methods and 14 dynamic prediction methods. Each separate distribution will produce a new resistance factor, which may vary from the code-prescribed resistance factor. The LRFD Bridge Specifications (AASHTO 2014) lump these into seven resistance factors for static capacity calculation methods, three for dynamic calculation methods and four for dynamic and static testing. The resistance factor obtained for each pile distribution does not necessarily match the corresponding LRFD resistance. Therefore, it is necessary to normalize Fig. 6 to the prior code factor obtained for the various distributions in Paikowsky (2004) that fit into that category.

For example, the $\alpha$-method has a code-prescribed resistance factor of 0.35 in clay and mixed soils. Paikowsky (2004) lists 11 different distributions for driven piles in clay, each with its own calibrated $\phi$ factor (ranging from 0.24 to 0.54, with an average of 0.38). To apply the updating procedure, the curve shown in Fig. 6 is obtained for each of these distributions and normalized by dividing by the nonupdated (prior) $\phi$ factor for each distribution (the dashed line in Fig. 6). The new curve represents the multiple of $\phi$ (as a function of permit load magnitude) that can be used while maintaining a minimum reliability of 2.33. To combine the various distributions provided in Paikowsky (2004) into the LRFD (AASHTO 2014) categories, the Paikowsky (2004) data are aggregated into the respective design categories. A weighted average (to number of piles used in

### Table 2. Updated pile resistance factors for LRFD (low variability)

<table>
<thead>
<tr>
<th>Method type</th>
<th>Condition/resistance determination method</th>
<th>Resistance factor (LRFD)</th>
<th>Updated resistance factor Maximum past load (% of nominal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic</td>
<td>WEAP</td>
<td>0.50</td>
<td>0.50 0.53 0.64 0.78 0.92 1.07</td>
</tr>
<tr>
<td></td>
<td>FHWA-modified gates</td>
<td>0.40</td>
<td>0.40 0.41 0.46 0.55 0.64 0.75</td>
</tr>
<tr>
<td></td>
<td>ENR formula</td>
<td>0.10</td>
<td>0.11 0.14 0.19 0.24 0.30 0.35</td>
</tr>
<tr>
<td>Static</td>
<td>$\alpha$-method</td>
<td>0.35</td>
<td>0.35 0.37 0.43 0.52 0.61 0.71</td>
</tr>
<tr>
<td></td>
<td>$\beta$-method</td>
<td>0.25</td>
<td>0.25 0.28 0.34 0.41 0.49 0.58</td>
</tr>
<tr>
<td></td>
<td>$\lambda$-method</td>
<td>0.40</td>
<td>0.40 0.43 0.50 0.59 0.70 0.81</td>
</tr>
<tr>
<td></td>
<td>Nordlund/Thurman method</td>
<td>0.45</td>
<td>0.45 0.47 0.54 0.62 0.70 0.81</td>
</tr>
<tr>
<td></td>
<td>SPT method</td>
<td>0.30</td>
<td>0.30 0.32 0.34 0.40 0.46 0.53</td>
</tr>
<tr>
<td></td>
<td>CPT method</td>
<td>0.50</td>
<td>0.50 0.50 0.50 0.52 0.57 0.63</td>
</tr>
<tr>
<td></td>
<td>End bearing in rock</td>
<td>0.45</td>
<td>—————— —————— —————— —————— —————— ——————</td>
</tr>
</tbody>
</table>

Note: WEAP = Wave Equation of Pile Driving; FHWA = Federal Highway Administration; ENR = Engineering News Record; SPT = Standard Penetration Test; and CPT = Cone Penetration Test.
the statistical data) is taken of this collection of curves to find the percentage of increase in $\phi$ allowable that maintains the target reliability. The updating curve for the $\alpha$-method is given in Fig. 9.

This updating procedure is performed for all prediction methods where a prior distribution is obtainable. Table 2 provides the LRFD resistance factor alongside updated resistance factors using the previously described methodology and a site variance of 0.15 (the same tables for site variance of 0.25 and 0.35 are available in the Supplemental Data). The maximum past load should be a conservative estimate of the average past load applied to each pile from all sources (accounting for the effects of load variance in Fig. 8). The updated resistance factor for each method maintains a reliability index of 2.33 for each individual pile. From Table 2, it is evident that the updating methodology does not have a consistent effect on piles with capacities predicted by various methods. For piles with capacities predicted using wave-equation analysis, it is possible to get a resistance factor above 1, higher than either the predicted capacity or previously applied load. This is due to the Wave Equation Analysis of Pile Driving (WEAP) prediction method having a high bias of 1.656 and a high COV of 0.724. The updated PDF has an especially high likelihood of values greater than the predicted value, which is reflected in the updated reliability analysis. It may be desirable in practice to limit the capacity to the previous average loading applied. Conversely, the Cone Penetration Test (CPT) method is least suited to this updating method, although an increase of 20% in the capacity is possible when the pile group withstood the loading of 60% of the nominal capacity. The static capacity calculation methods can all have their resistance factors raised substantially, especially in low-COV populations. Piles that are end bearing in rock cannot be updated using this methodology because little statistical data on these piles is available.

### Determining Capacity without Prior Distribution

In some cases, obtaining a prior distribution of pile capacity may not be possible due to limited or unreliable subsurface information. For driven piles end bearing on rock, statistical data on capacity estimation are not readily available. For these scenarios, the first methodology cannot be employed, and a second methodology is proposed to verify the capacity of existing piles, given only the previous loading history or the magnitude of test loads applied to the group of piles being investigated. The test load or previous loading occurs in conjunction with dead loads, earth-pressure loads, and other permanent loads acting on the pile group being considered. The magnitude of the test load or previous load is defined as the excess population mean (EPM), or the average excess pile capacity beyond those permanent loads. This load is considered to be the average excess capacity of the pile group, although individual pile capacities will vary from each other and may be greater or lower than average.

A PDF of the excess capacity of an individual pile is obtained by assuming the piles of a single population are lognormally distributed with parameters $\mu$ and $\sigma$, obtained from Eqs. (14) and (15). For this methodology, the previously applied load or a test load is treated as deterministic, but all other loads acting on the pile (during current service life or reuse) can be assumed to be probabilistic. The total distribution of excess pile capacity is then given by Eq. (16).

$$\sigma^2 = (\text{site COV}^2 + 1)$$

$$\mu = -\frac{1}{2} \sigma^2$$

$$\text{excess pile capacity} = \text{EPM} \times x_{epc}$$

where EPM has been defined as the magnitude of the test load or previous load. In Eq. (16), $x_{epc}$ follows a lognormal distribution with parameters $\mu$ and $\sigma$. A limit-state equation is assembled in Eq. (17) for just the live loading to determine the nominal live loading that achieves the target reliability index.

$$G = \text{EPM} \times x_{epc} - L_{nom} \times x_{LL}$$

where $L_{nom}$ is nominal live load; and $x_{LL}$ is lognormal distribution of live loads described in Table 1. Following Ayyub and Assakkaf (1999), a reliability analysis can be performed using Eq. (17), with

![Fig. 9. Updating curve for piles with capacities calculated using alpha method (site COV = 0.15).](image-url)
capacity is also considered to be applicable to all limit states. Capacities calibrated to this limit state for all limit states, this limit state, as shown in Eq. (19). Because AASHTO (2014) uses DLnom = nominal dead load acting on the pile; DLrem = dead load that will be removed; and DLnew = dead load of the bridge because of bridge repurposing, this approach can also be used to design the maximum increase in the dead load that will allow the reuse of the bridge foundation. Then the design of superstructure can be optimized through analysis, modeling, and innovative materials to limit increase in the dead load to this maximum allowable increase for reuse. Fig. 12 shows plots for a pile foundation where 20% of the existing dead load was removed and replaced. In Fig. 12, five different options are plotted: two with decreased new load, two with increased new dead load, and one that maintains the nominal dead load (although with uncertainties introduced by the swapping of 20% of the dead load). The y-axis in Fig. 12 shows the live-load capacity (on top of new dead load) as a percentage of the test load applied to the original bridge. Note that the black line in Fig. 12 is identical to the solid line (20% replaced) in Fig. 11.

Examples

Route 2A Bridge: Haynesville, Maine

The Route 2A Bridge in Haynesville, Maine, is a three-span steel girder bridge supported by mass concrete stub abutments and solid wall piers (Krusinski 2015). Both piers and abutments are founded on driven timber piles installed through a loose silty sand layer that ranges in thickness from 4.6 to 10.4 m (15 to 34 ft). Both piers and one of the abutments were driven into a dense glacial till layer as indicated by borings and historical records of pile cutoff elevations. The second abutment was believed to be terminated in the silty sand layer. The nominal pile capacity was calculated to be 329 kN (74 kips) per pile, including only 31 kN (7 kips) of end-bearing resistance. The nominal capacity is based on the Nordlund method for side resistance and the Thurman method for end-bearing capacity. The factored capacity was found to be 147 kN (33 kips), nearly the same as the allowable strength design nonfactored load capacity of 142 kN (32 kips) per pile found in historical records. Using the updated resistance factors in Table 2 and assuming low variability between the piles (COV = 15%), the resistance factor can be updated to include the maximum past loading, as shown in Fig. 13. From this figure, the new LRFD capacity for each individual pile in the pile group

$$L_{nom}$$, iterated until the target reliability is reached. The allowable live-load capacity ($$C_{LL}$$) (LRFD capacity beyond what is used for dead loads) is then found using Eq. (18). The total geotechnical capacity of the piles can be calculated by considering the Strength I limit state, as shown in Eq. (19). Because AASHTO (2014) uses

$$C_{LL} = 1.75 \times (L_{nom})$$

(18)

$$C_{test} = 1.75(L_{nom}) + 1.25(DL_{nom}) + \sum \gamma_p(P_{nom})$$

(19)

where $$DL_{nom}$$ = nominal dead load acting on the pile; $$P_{nom}$$ = other nominal permanent loads acting on the pile; and $$\gamma_p$$ = load factors for those loads. Fig. 10 shows the ratio of $$C_{LL}$$ to the test-load magnitude or largest permit load for a target reliability of 2.33.

To account for modified loading after reuse, a new limit-state function is proposed to estimate the reliability of the piles after removal and replacement of the dead load, as shown in Eq. (20).

$$G = (EPM)x_{epc} + DL_{rem}x_{DL1} + DL_{new}x_{DL2} - L_{nom}x_{LL}$$

(20)

where $$DL_{rem}$$ = dead load that will be removed; and $$DL_{new}$$ = dead load that will be added during reuse. These two numbers can sum to zero if the total dead load acting on the piles does not change during reuse. The change in dead load, however, will cause a reduction in the available live-load capacity because both the removed and new dead loads are variable. Fig. 11 presents the factored LL capacity as a function of the test load or previously applied load, normalized with respect to the magnitude of the existing dead load, for a site COV of 0.15.

It should be noted that this approach is able to explicitly include changes in the dead load during reuse in Eq. (18). It reduces to the live-load testing shown in Fig. 10 (for the 0.15-COV point) when there is no change in dead load. In the case of an increase in the dead load of the bridge because of bridge repurposing, this approach can also be used to design the maximum increase in the dead load.

![Fig. 10. LL capacity of existing piles for various site COVs.](image-url)

**Fig. 10.** LL capacity of existing piles for various site COVs.
(with onsite COV of 15%) is given as a function of the total loading, including dead loads and previous permit loading.

Because the factored geotechnical capacity of 147 kN (33 kips) was not sufficient for design, the pile cap was excavated, and a single pile was load tested with a hydraulic jack. The pile failed at 534 kN (120 kips) of load, allowing for a factored capacity of 374 kN (84 kips). To achieve this factored capacity using this method, a loading of approximately 347 kN (78 kips) per pile would need to be applied to the abutments, for a total of 8,327 kN (1,872 kips) on the entire 24-pile abutment. This is approximately 2.4 times the ASD design load of 3,416 kN (768 kips) on this abutment.

**Jackson Road over Route 2: Lancaster, Massachusetts**

The Jackson Road Bridge in Lancaster, Massachusetts (GTR 2014), was considered for replacement due to the age of the superstructure and accelerated deterioration. The two abutments and center pier of the original bridge were founded on timber piles. No load test data were available for the piles, although extensive driving logs,
including details of the pile hammer, blow counts, and end-of-drive penetration, were found in the documentation for the bridge. A range of possible capacities for each pile was determined using wave analysis, which were then compared to the expected future loading on each individual pile. It was determined that the total geotechnical capacity of the foundation was sufficient but with several piles being overloaded when compared to their factored geotechnical capacity. The bridge was reconstructed with geofoam being used to replace the abutment soil. The replacement of this soil with geofoam lowered the loading on the overloaded pile enough to reuse the pile without modification.

A resistance factor of 0.5 is prescribed by the LRFD Bridge Design Specifications (AASHTO 2014) for use on piles with their capacity determined using wave analysis. Had the previous loading been considered, a higher resistance factor and LRFD capacity could be specified while maintaining a reliability index of 2.33 for the piles. A plot of the resistance factor that achieves a target reliability of 2.33 versus the maximum past loading is shown in Fig. 14. From this figure, the required resistance factor to maintain the target reliability is plotted against possible values of the maximum previously applied loading, in kips.

**Route 1A Viaduct, Bath, Maine**

The Route 1A viaduct in Bath, Maine (Haley and Aldrich 2013), consisted of 19 piers and abutments, supported by a mixture of driven steel H-piles and spread footings bearing on rock. The H-piles were visually examined to be in good condition. The parallel seismic testing ensured that the piles were driven to the top of bedrock. The piers were reused with the original foundations, with the pile capacity considered to be governed by the structural capacity of the piles. Without making any assumptions about the soil or rock conditions, each group could be analyzed for capacity based solely on the maximum previous loading. Assuming a 30% replacement of dead load on the bridge, a graph of total pile capacity versus total previous load can be made, as shown in Fig. 15. This figure provides the total LRFD capacity of the individual piles in the foundation as a function of the total previously applied loading.

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![Fig. 13. New factored capacity as a function of maximum past loading.](image1)

![Fig. 14. Updated resistance factors based on past loading.](image2)

**Conclusions**

The proposed methodology allows engineers to consider the previous loading on pile groups when determining the capacity of a single pile. By considering the maximum previous loading applied to an entire pile group, a higher resistance factor for individual pile geotechnical capacity can be used by the design engineer while maintaining a constant reliability index. The use of a higher resistance factor and capacity may allow the design of a reused foundation to forego expensive strengthening, load-reduction techniques, or individual pile testing to ensure piles have adequate capacity. A second methodology is provided that allows engineers to determine the capacity of piles bearing on rock with no available static test data. This method effectively applies a resistance factor that ranges from 0.5 to 0.9 on the live-load capacity, as determined through the placement of a known weight. The effective code factor on dead loads is...
determined considering the amount of load removed and replaced and the total change in dead load. The capacity obtained using this method can be applied to other load cases, although the capacity determined does not consider past earthquake or wind loading.

Future Work

Further work is needed to determine appropriate probabilistic distributions for lateral earth-pressure loading and other loading. The inclusion of past earthquakes, downdrag, and other loading can be considered as other sources of previous loading and as potential sources of future loading. Hazards like downdrag and scour that reduce the pile length available for resistance can also be considered. This method can be adapted to be used with sensors, such as strain gauges, to measure the forces going into individual piles. Further statistical data can be gathered on the resistance of piles end bearing in rock. This methodology can be further explored to include measurements of bridge movement during loading or unloading. Research is needed on determining the impacts of group effects from closely spaced piles. The methodology can be adapted to include data from embedded sensors at the base of the piles to determine the amount of strain being transferred to the pile tip.

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Supplemental Data

Tables S1–S3 are available online in the ASCE Library (www.ascelibrary.org).

References


