Load and resistance factor design for axially loaded drilled shafts

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ABSTRACT
Resistance factors are developed for drilled shafts (bored piles) in sand and clay for design methods based on soil variables. A rigorous analysis was performed in which the uncertainties of the design variables and equations were systematically quantified. Monte-Carlo simulations were then performed to obtain the distributions of the shaft and base capacities and applied dead and live loads. The limit state and nominal resistances and loads were identified, and the optimal dead and live load factors calculated. The optimal resistance factors were then adjusted for use with load factors recommended by FHWA. Use of these factors in the design of drilled shafts using LRFD is then discussed.

RÉSUMÉ
Les facteurs de résistance sont développés pour des puits forés dans le sable et d'argile pour les méthodes de conception basées sur les variables du sol. Une analyse rigoureuse a été effectuée dans laquelle les incertitudes des variables de conception et les équations ont été systématiquement quantifiés. Simulations Monte-Carlo ont été effectuées pour obtenir les distributions de l'arbre et des capacités de base et les charges appliquées morts et vivants. Les résistances état limite et nominales et les charges ont été identifiés, et les facteurs de charge optimal mortes et vivantes ont été obtenus d'eux. Les facteurs de résistance optimale ont ensuite été ajustés pour une utilisation avec des facteurs de charge recommandées par la FHWA. Conception des piles en utilisant LRFD est discutée.

1 INTRODUCTION

Design of pile foundation solutions can best be done by clearly defining limit states and then configuring the piles in such a way as to prevent the attainment of these limit states. There are three approaches to do this; in order of complexity, they are: working stress design (WSD), load and resistance factor design (LRFD) and reliability-based design (RBD). All three account in some way for the fact that foundation engineering problems are not deterministic, and most if not all variables in the problem are random or have a random component. In this study, we develop LRFD methodology for ultimate limit states related to axial loading of single drilled shafts (bored piles); sands and clays are the two soil types we have considered.

2 LRFD FRAMEWORK

In the LRFD framework, the capacity (total resistance) and demand (applied loads) are related by:

$$(\mathrm{RF})R^{(\mathrm{n})} \geq \sum (\mathrm{LF})_i L_i^{(\mathrm{n})} \qquad [1]$$

where (RF) = resistance factor, $R^{(\mathrm{n})}$ is the nominal resistance, $(\mathrm{LF})_\mathrm{i}$ = load factor corresponding to the i^{th} nominal (or characteristic) load $L_\mathrm{i}^{(\mathrm{n})}$, and the superscript (n) represents nominal loads and resistances. The deterministic loads and resistances estimated by design engineers based on procedures prescribed by codes, manuals and books or on experience are referred to as nominal loads and resistances, respectively.

For pile problems, we have two sources of resistance: base resistance and shaft resistance. The base and shaft resistances of piles are calculated separately, and the mechanisms by which these resistances develop are quite different. The loading that develops along the pile shaft closely approximates simple shear loading and, at an ultimate limit state, corresponds to critical state values of shear stress. The loading around the base is much more complex, with mean stress increasing at almost every point around the base and shear stresses developing to different degrees and at different rates depending on the point considered. Consequently, the two capacity equations are subjected to different sets of uncertainties.

We define an ultimate limit state of a pile as any combination of loads and resistances such that the sum of the loads (i.e., live, dead, etc.) equals the ultimate capacity Q_{ult} of the pile, which is itself the sum of the ultimate base resistance $Q_{b,\mathrm{ult}}$ and limit shaft resistance Q_{sL} (Salgado 2008). A "safe" or acceptable design state, therefore, satisfies inequality. In this paper, inequality [1] is used as the LRFD design relationship, which, for applied dead and live loads, can be rewritten as:

$$(\mathrm{RF})_b Q_{b,\mathrm{ult}}^{(\mathrm{n})} + (\mathrm{RF})_s Q_{sL}^{(\mathrm{n})} \geq (\mathrm{LF})_{\mathrm{DL}} (\mathrm{DL})^{(\mathrm{n})} + (\mathrm{LF})_{\mathrm{LL}} (\mathrm{LL})^{(\mathrm{n})} \qquad [2]$$

where $Q^{(\mathrm{n})}_{b,\mathrm{ult}}$ and $Q^{(\mathrm{n})}_{sL}$ are the nominal values of $Q_{b,\mathrm{ult}}$ and Q_{sL}, $(\mathrm{DL})^{(\mathrm{n})}$ = nominal dead load, $(\mathrm{LL})^{(\mathrm{n})}$ = nominal live load, $(RF)_b$ = base resistance factor, $(RF)_s$ = shaft resistance factor, $(LF)_{DL}$ = dead load factor and $(LF)_{LL}$ = live load factor.

3 UNIT BASE AND SHAFT RESISTANCE

3.1 Drilled Shaft (Bored Pile) in Sand

The unit shaft resistance q_{sL} in sand is often determined using the β method, according to which:

$$q_{sL} = (K \tan \delta)\sigma_v' = \beta \sigma_v' \quad [3]$$

where σ_v' is the *in situ* vertical effective stress, K is a coefficient and δ is the friction angle mobilized along the pile-soil interface. Recently, Loukidis and Salgado (2008) performed finite element analysis coupled with an advanced constitutive model to investigate the mechanics of load transfer at the interface of non-displacement piles in sands. Based on their analysis, Loukidis and Salgado (2008) proposed the following equation for K:

$$K = \frac{K_0}{e^{0.2\sqrt{K_0 - 0.4}}} C_1 e^{\frac{D_R}{100}\left[1.3 - 0.2\ln\left(\frac{\sigma_v'}{p_A}\right)\right]} \quad [4]$$

where D_R is the relative density of sand and p_A is a reference stress (100 kPa). Loukidis and Salgado (2008) suggested a value of C_1 = 0.7 for clean sands in general. Loukidis and Salgado (2008) also found that the angle δ is approximately equal to the triaxial-compression critical-state friction angle ϕ_c; thus, $\delta = \phi_c$ can be assumed in calculations without any significant error.

The ultimate unit base resistance $q_{b,ult}$ is related to the limit bearing capacity. Lee and Salgado (1999) performed nonlinear finite element analysis and used plate load tests in calibration chambers to find that $q_{b,ult}/q_{bL}$ depends primarily on D_R. Based on this analysis, Salgado (2008) proposed an analytical expression for $q_{b,ult}$ corresponding to 10% relative settlement:

$$q_{b,ult} = q_{b,10\%} = 0.23 e^{-0.0066 D_R} q_{bL} \quad [5]$$

where the limit unit base capacity q_{bL} can be expressed, based on rigorous cavity expansion analysis (Salgado and Randolph 2001, Salgado and Prezzi 2007) as:

$$\frac{q_{bL}}{p_A} = 1.64 e^{0.1041\phi_c + (0.0264 - 0.0002\phi_c)D_R} \left(\frac{\sigma_h'}{p_A}\right)^{0.841 - 0.0047 D_R} \quad [6]$$

where $\sigma_h' = K_0 \sigma_v'$ is the *in situ* effective horizontal stress and K_0 is the coefficient of earth pressure at rest.

3.2 Drilled Shaft in Clay

To calculate the unit shaft resistance q_{sL} in clay, we used the α method, according to which:

$$q_{sL} = \alpha s_u \quad [7]$$

Based on finite element analyses coupled with an advanced constitutive model, Chakraborty et al. (2011) proposed the following equation for α:

$$\alpha = \left(\frac{s_u}{\sigma_v'}\right)^{-0.05} \left[A_1 + (1 - A_1)\exp\left\{-\left(\frac{\sigma_v'}{p_A}\right)\left(\phi_c - \phi_{r,\min}\right)^{A_2}\right\}\right] \quad [8]$$

where σ_v' is the *in situ* vertical effective stress; ϕ_c is the critical-state friction angle; $\phi_{r,\min}$ is the minimum residual state friction angle; p_A is a reference stress (100*k*Pa); A_1 is a coefficient equal to 0.4 for $\phi_c - \phi_{r,\min} \geq 12°$, 0.75 for $\phi_c - \phi_{r,\min} \leq 5°$; and A_2 is a coefficient determined by $A_2 = 0.4 + 0.3 \ln(s_u / \sigma_v')$.

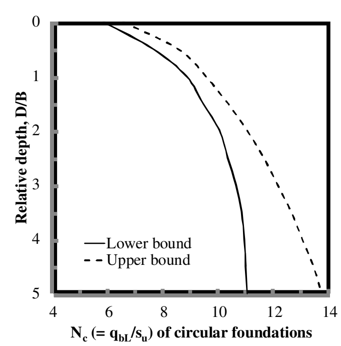

Figure 1 Limit unit base resistance of circular foundation versus depth (Salgado et al. 2004)

Salgado et al. (2004) used finite element limit analysis to investigate the upper and lower bound of the ratio of net bearing capacity q_{bL}^{net}, defined as

$$q_{bL}^{net} = q_{bL} - q_0 \quad [9]$$

where q_0 is the surcharge at the pile base level. Based on their analysis, Salgado et al. (2004) found that the lower and upper bound values of N_c, (= q_{bL}^{net} / s_u) increases with increasing relative depth D/B (the ratio of the length of the pile to the width of the pile), as Figure 1 shows. By substituting N_c into equation [9], the equation of the limit unit base resistance is:

$$q_{bL} = N_c s_u + q_0 \qquad [10]$$

4 UNCERTAINTIES OF VARIABLES AND MODEL

4.1 Uncertainties in Soil Variables and Models

4.1.1 Drilled shafts in sand

The soil variables required for pile capacity calculations in this analysis are ϕ_c, D_R, K_0 and soil unit weight γ. K_0 is difficult to estimate in the field, and not much information is available regarding its variability. Accordingly, we assumed K_0 deterministic (but performed calculations for values in the 0.4-0.5 range).

Kim (2008) estimated the COV of ϕ_c to be in the 0.0081-0.0172 range based on the experimental studies by Verdugo and Ishihara (1996) and Negussey et al. (1987). In this study, we conservatively assume the COV of ϕ_c to be equal to 0.02. We also assume that ϕ_c follows a normal distribution.

Baecher and Christian (2003) reported, based on studies by Lee et al. (1983), Lacasse and Nadim (1996) and Lumb (1974), that the COV of unit weight does not exceed 0.1. In this research, we assumed that γ follows a normal distribution with a COV = 0.1.

A reliable and practical method of calculating D_R is from CPT results, as proposed by Salgado (2008), which is a result of rigorous cavity expansion analysis (Salgado and Prezzi 2007):

$$D_R(\%) = f_{DR}(q_c,\ \phi_c,\ \sigma') = \frac{\ln\left(\dfrac{q_c}{p_A}\right) - 0.4947 - 0.1041\phi_c - 0.841\ln\left(\dfrac{\sigma'_h}{p_A}\right)}{0.0264 - 0.0002\phi_c - 0.0047\ln\left(\dfrac{\sigma'_h}{p_A}\right)} \qquad [11]$$

In this research, we assume cone resistance q_c for a soil profile as the starting point instead of D_R and calculate the PDF of D_R from the PDF of q_c equation [11]. In this research, we assumed that q_c follows a normal distribution with COV = 0.08 (following Foye 2005).

The model uncertainty in the $q_c \rightarrow D_R$ relationship was investigated by Foye (2005) using results of twenty-five well-controlled calibration chamber tests (Salgado 1993). Based on the study by Foye (2005), we found that D_R is over-predicted by 3% using equation [11]. Foye (2005) calculated the standard deviation of D_R to be equal to 10%, with q_c as a deterministic variable. Foye (2005) also observed that the normalized error of D_R follows a normal distribution. The incorporation of the $q_c \rightarrow D_R$ model error in the M-C simulations was done by introducing a bias factor $M_{DR}^{\text{bias}} = 0.97$ and a new random variable M_{DR} that follows a normal distribution with standard deviation S_{MDR} = 0.1/0.97$E(f_{DR})$ and a mean $E(M_{DR})$ = 1.0.

$$D_R(\%) = M_{DR}^{\text{bias}} M_{DR} f_{DR}(q_c,\ \phi_c,\ \sigma') = 0.97 M_{DR} f_{DR}(q_c,\ \phi_c,\ \sigma') \qquad [12]$$

The model uncertainty associated with q_{sL} arises in the estimation of $\beta = K\tan\delta$ in equation [3]. To estimate the uncertainty in equation [4], results of eight centrifuge tests by Fioravante (2002) and Colombi (2005) were used. The standard deviation of the normalized error between data from centrifuge test and the values from equation [4] was about 0.2 with no bias. Therefore, in order to incorporate the q_{sL}-model uncertainty in our analysis, we introduce a new random variable M_β that follows a normal distribution with expectation $E(M_\beta)$ = 1.0 and standard deviation $S_{M\beta}$ = 0.2. The equation used to calculate q_{sL} in the Monte-Carlo simulations is:

$$q_{sL} = M_\beta \frac{0.7K_0}{e^{0.2\sqrt{K_0 - 0.4}}} e^{\frac{D_R}{100}\left[1.3 - 0.2\ln\left(\frac{\sigma'_v}{p_A}\right)\right]} \sigma'_v \tan\phi_c \qquad [13]$$

The model uncertainty associated with unit base resistance (equations [5] and [6]) was estimated from twenty-one well-controlled "deep" plate load tests performed within a calibration chamber (Lee and Salgado 1999). The plate load tests in the calibration chamber were done for two relative densities: D_R = 50% and D_R = 90%. We found that the model over-predicts $q_{b,10\%}$ by 3% for D_R = 50% and under predicts $q_{b,10\%}$ by 16% for D_R = 90%. In order to incorporate the model uncertainty of $q_{b,10\%}$ in calculations, we introduced a bias factor which equals to 0.97 for $D_R \leq 50\%$, equals to 1.16 for $D_R \geq 90\%$ and equals to a linearly interpolated value between 0.97 and 1.16 for 50% < D_R < 90%. We also introduced a new random variable M_{qb}, following a normal distribution with mean $E(M_{qb})$ = 1 and standard deviation S_{Mqb} equal to 0.1.

$$q_{b,10\%} = M_{qb}^{\text{bias}} M_{qb} 0.38 p_A e^{-0.0066 D_R} e^{0.1041\phi_c + (0.0264 - 0.0002\phi_c) D_R} \left(\frac{\sigma'_h}{p_A}\right)^{0.841 - 0.0047 D_R} \qquad [14]$$

4.1.2 Drilled shafts in clay

For drilled shafts in clay, the soil variables required ϕ_c, $\phi_{r,\min}$, s_u and soil unit weight γ. We assumed that soil unit weight γ follows a normal distribution with a COV = 0.1 for clay as well as for sand.

Based on experimental observations of Bolton (1986), the expectation of the maximum error in the estimation of ϕ_c at a particular site is ±1°. Assuming that ϕ_c follows a normal distribution, the spread of 2° results in a standard deviation of 0.33° (=Range/6.0=2.0/6.0) for ϕ_c based on the 6σ method (Foye 2005). Because ϕ_c of different clays typically lie within the 15°-30° range (Salgado 2008), the maximum and minimum values of COV of ϕ_c are 0.33°/15° = 0.022 and 0.33°/30° = 0.011 at a particular site. In this study, we conservatively assume the COV of ϕ_c to be equal to 0.03.

We also assume that the maximum error in the estimation of $\phi_{r,min}$ is ±1°. For a clay such as London Clay, however, $\phi_{r,min}$ is within the 7.5°-9.4° range (Bishop et al. 1971).So, we assume that the minimum value of $\phi_{r,min}$ is 7.5°. Assuming that $\phi_{r,min}$ follows a normal distribution with a standard deviation of 0.33°, the maximum COV of $\phi_{r,min}$ is 0.33°/7.5° = 0.044. We conservatively assume the COV of $\phi_{r,min}$ to be equal to 0.05.

Equations [7], [8] and [10] show that the limit unit base and shaft resistances are functions of undrained shear strength s_u. The equation for s_u in terms of cone resistance is:

$$s_u = \frac{q_c - \sigma_v}{N_k} \quad [15]$$

where q_c is cone resistance, σ_v is vertical stress and N_k is the cone factor. In this research, we assume cone resistance q_c for a soil profile as the starting point in design. According to Foye (2006), the COV of q_c in clay is 0.06 and q_c follows a normal distribution. In this study, we worked with COV = 0.06.

Equation [15] already implies the level of uncertainty in the $q_c \rightarrow s_u$ relationship since the cone factor N_k is expressed as a range. The upper and lower bound values of N_k are 13.7 and 11.0, respectively, with a range for N_k of 2.7 (Salgado et al. 2004). We can estimate the standard deviation for a variable that has a range and follows a normal distribution using the 6σ method. (Withiam et al. 1997, Foye et al. 2006). In this study, we assumed that the mean value of N_k is 12.3 and assumed that N_k follows a normal distribution with standard deviation of 0.45, which is equal to the range divided by 6.0.

The relationship $s_u \rightarrow q_{bL}$ for a pile is similar physically to the $s_u \rightarrow q_c$ relationship. If the dimensions of the pile are given, we can obtain the lower and upper bound values of N_c from Figure 1. Typically, the relative depth of a pile is much greater than 5.0, so the maximum and minimum values of N_c are taken as 13.7 and 11.0. From this range, it is reasonable to set the mean and the standard deviation of N_c as 12.3 and 0.45, respectively.

The model uncertainty associated with q_{sL} arises in the estimation of α using equation [8]. To estimate this, we used the elemental simulations of undrained triaxial compression tests and direct simple shear tests using the two-surface plasticity model for clay proposed by Basu et al. (2009). From K_0-consolidated undrained triaxial compression test (CK0UTXC) simulations with random values of ϕ_c and $\phi_{r,min}$, we obtained the distribution of undrained shear strength of the model, $s_{u,model}$. Since the limit shaft resistance of drilled shafts is arrived at through a simple shear loading path (Basu et. al 2009), the limit shaft resistance q_{sL} would be equal to the simple shear strength τ_{ss} of the soil. Thus, we estimated the distribution of q_{sL} using the residual shear strength at very large strains resulting from CK0UDSS with random values of ϕ_c and $\phi_{r,min}$. From the distributions of $s_{u,model}$ and q_{sL}, we estimated the distribution of α_{model} (= q_{sL} / $s_{u,model}$).

The upper bound of absolute relative error (1- $|\alpha_{model}$ / $\alpha_{equation}|$) is 0.2. Consequently, we can assume that the range of the ratio $\alpha_{equation}/\alpha_{model}$ is approximately 0.4. The 6σ method leads to an estimate of the standard deviation of $\alpha_{equation}/\alpha_{model}$ as 0.067 (=0.4 / 6.0). Therefore, to incorporate the model uncertainty of shaft resistance in our analysis, we introduced a new variable M_α that follows a normal distribution with expectation $E(M_\alpha)$ = 1.0 and standard deviation $S_{M\alpha}$ = 0.1, which is conservative.

$$q_{sL} = M_\alpha \left(\frac{s_u}{\sigma_v'}\right)^{-0.05} \left[A_1 + (1-A_1)\exp\left\{-\left(\frac{\sigma_v'}{p_A}\right)\left(\phi_c - \phi_{r,\min}\right)^{A_2}\right\}\right] s_u \quad [16]$$

4.2 Uncertainties in applied loads

According to Ellingwood and Tekie (1999), the dead load distribution is approximately normal with a bias factor of 1.05 and COV equal to 0.1, and that is the assumption we made. Live load is generally described using a lognormal distribution (Foye et al. 2006). According to FHWA (2001), live load has a bias factor of 1.1-1.2 and a COV of 0.18. In this study, we used the lognormal distribution to describe live loads. We conservatively chose a COV = 0.25 and the corresponding bias factor = 1.0, as recommended by Ellingwood and Tekie (1999).

4.3 Uncertainties in pile dimensions

Drilled shafts are constructed by removing soil from the ground by drilling and filling the resulting cylindrical void with concrete and reinforcement. The construction process is controlled. Based on typical construction tolerances, we assume that the drilled shaft diameter B_p follows a normal distribution with a COV = 0.02. Additionally, we assume that the pile length L_p is deterministic.

5 MONTE-CARLO SIMULATION

Monte-Carlo (M-C) simulations were performed to obtain the probability distributions of the resistance ($Q_{b,ult}$ + Q_{sL}), the load (DL + LL) and their difference. We start with a soil profile with an assumed mean trend of CPT profile $q_c(z)$, where z is the depth and with assumed values of pertinent soil variables (ϕ_c, K_0 and γ for sand and ϕ_c ,$\phi_{r,min}$ and γ for clay). We also assume a mean value of applied dead load (DL)$^{(mean)}$ and a (LL)/(DL) ratio. Then we consider a drilled shaft with an assumed length and an assumed mean diameter. We start by taking a random value of B_p and proceeding with shaft capacity calculations from depth z = 0 to z = L. First, a random value of q_c is generated for a particular depth along with random values of the aforementioned soil variables. Then we calculate the soil variables that follow from q_c (D_R for sand, s_u for clay). Subsequently, we determine the uncertainties in the shaft resistance model and calculate a random value of unit shaft resistance for that depth. The calculated shaft capacities at the different depths are added over the entire pile length to obtain the random value of the total shaft resistance Q_{sL}. Then, as we reach the pile base, a random value of the base resistance $Q_{b,\mathrm{ult}}$ is calculated using the random values of soil variables and B_p and the variables representing model uncertainties for base resistance.

After calculating the random $Q_{b,\mathrm{ult}}$ and Q_{sL}, random values of DL and LL are generated and the difference ($Q_{b,\mathrm{ult}}$ + Q_{sL}) − (DL + LL) between the random values of resistance R = $Q_{b,\mathrm{ult}}$ + Q_{sL} and load Q = (DL + LL) is calculated. The above set of calculations completes one run of the M-C simulations. It is repeated n_{total} times (the value of n_{total} depends on the target probability of failure). The number of runs n_f for which ($Q_{b,\mathrm{ult}}$ + Q_{sL}) − (DL + LL) is less than zero is recorded. The ratio n_f / n_{total} approximates the probability of failure p_f. If the calculated p_f does not fall in the range $p_{f,\mathrm{target}}$ ±10%, DL and LL are adjusted until it does. After that, we locate the ultimate limit state values of the base and shaft capacities and dead and live loads as those corresponding to the Monte-Carlo run for which |C − D| is the minimum. The nominal values of resistances and loads are calculated separately by using the nominal values of all variables. Optimum factors of base and shaft resistances and dead and live loads are calculated by dividing the ultimate limit state values by the corresponding nominal values.

Because of the non-uniqueness of the ultimate limit state, the calculations of optimum factors are repeated 200 times and their average values are proposed as the final values of the optimal resistance and load factors. This has been shown to provide an excellent estimate of the most probable ultimate limit state.

6 ANALYSIS RESULTS

6.1 Drilled shafts in sand

The soil profiles considered in this paper are those of Basu and Salgado (2011): (1) a homogeneous, completely dry deposit of sandy soil with a mean relative density $D_{R,\mathrm{mean}}$ = 70%; (2) the same homogeneous sand deposit described in (1) with a water table located at the ground surface; (3) a completely dry sand deposit with a loose layer ($D_{R,\mathrm{mean}}$ = 50%) overlying a strong bearing layer ($D_{R,\mathrm{mean}}$ = 80%) that extends to great depth; (4) a two-layer system, as in (3), with a water table located at a depth of 2 m below the ground surface; (5) a two-layer system with the top layer consisting of extremely loose sand having $D_{R,\mathrm{mean}}$ = 20% and the bearing layer consisting of dense sand having $D_{R,\mathrm{mean}}$ = 80% and with a water table located at the ground surface; and (6) a four-layer, completely dry deposit consisting of a loose top layer with $D_{R,\mathrm{mean}}$ = 30% spanning 0−5 m down from the ground surface, a second layer with $D_{R,\mathrm{mean}}$ = 45% spanning 5−10 m below the ground surface, a third layer with $D_{R,\mathrm{mean}}$ = 60% spanning 10−15 m below the ground surface and a bearing layer with $D_{R,\mathrm{mean}}$ = 75% that lies below the third layer and extends down to great depth. The thicknesses of the soil layers in profiles (3)−(6) were assumed to be deterministic variables. In deciding the thicknesses of the top layers of the two-layer profiles (3), (4) and (5), we assumed that the depth of pile embedment H_{bearing} in the bearing layer is two times the mean pile diameter $B_{p,\mathrm{mean}}$

For these deposits, we considered three different sand types with mean critical-state friction angle $\phi_{c,\mathrm{mean}}$ = 30°, 33° and 36° and, for each type, we assume three different values of K_0: 0.4, 0.45 and 0.5. The mean values of sand unit weight γ_{mean} was calculated using the mean relative density $D_{R,\mathrm{mean}}$, $e_{\max}$ (=0.9) and $e_{\min}$ (=0.45). The $q_{c,\mathrm{mean}}(z)$ curves were initially back-calculated from pre-assumed values of $D_{R,\mathrm{mean}}$, γ_{mean}, $\phi_{c,\mathrm{mean}}$ and K_0 using the inverse of equation [12], and then given as input to the Monte-Carlo analysis code.

We studied the responses of four drilled shafts with (A) mean diameter $B_{p,\mathrm{mean}}$ = 0.3 m and length L_p = 10 m, (B) $B_{p,\mathrm{mean}}$ = 1.5 m and L_p = 10 m, (C) $B_{p,\mathrm{mean}}$ = 0.3 m and L_p = 30 m and (D) $B_{p,\mathrm{mean}}$ = 1.5 m and L_p = 30 m for the profiles (1)−(5). For profile (6), we considered a fifth drilled shaft (E) with $B_{p,\mathrm{mean}}$ = 1.0 m and L_p = 20 m.

Titi et al. (2004) tabulated the (LL)/(DL) ratios recommended by AASHTO and FHWA for design of bridge structures; the recommended values vary over a wide range of 0.28−1.92. Accordingly, we considered (LL)/(DL) = 0.25, 1.0 and 2.0 in our analysis.

We found that the optimal load and resistance factors are independent of soil variables for all practical purposes. This is evident from a comparison of the results in Figure 2, in which the optimal resistance and load factors of drilled shaft (B) are plotted for soil profiles (1) through (5) and for (LL)/(DL) = 1.0 and target p_f = 10^{-3}. This invariance of the factors with soil profiles was also observed for the other drilled shafts and for other values of (LL)/(DL) and p_f. The invariance exists not only for soil profiles but also for pile dimensions as is evident from Figure 3, which shows the optimal resistance and load factors for drilled shafts (A)−(D) installed in soil profile (3) and for (LL)/(DL) = 1.0 and p_f = 10^{-4}.

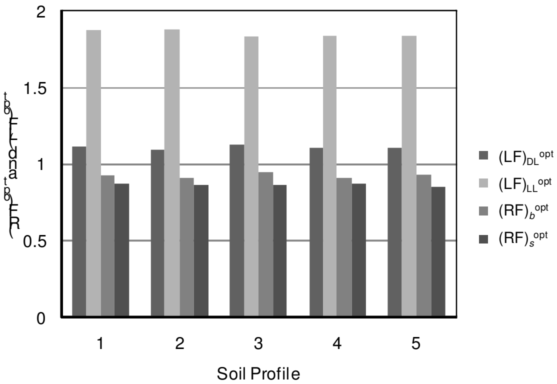

Figure 2 Optimal resistance and load factors for different soil profiles

The (LL)/(DL) ratio has a non-negligible effect on the live load factor. As Figure 4 shows, the increase in the optimal live load factor with increase in (LL)/(DL) ratio is significant. Figure 4 was plotted for drilled shaft (D) installed in soil profile (1) for different values of (LL)/(DL) and p_f = 10^{-4}; the trend was consistent for all the other cases.

Since live load has the most effect on the results of our reliability study, we investigated how much the resistance factors would change if, instead of our choice of live load COV = 0.25 and bias factor = 1.0 (Ellingwood and Tekie 1999), we used the live load COV = 0.18 and bias factor = 1.15 recommended by FHWA (2001).

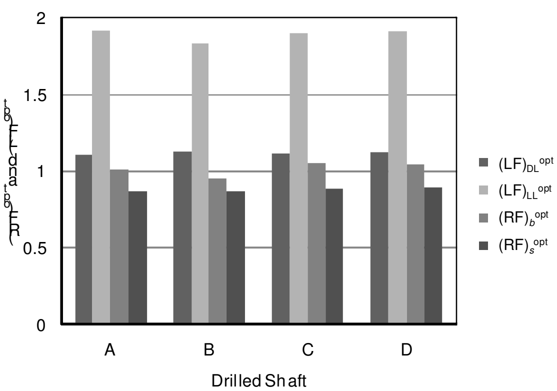

Figure 3 Optimal resistance and load factors for different drilled shaft dimensions

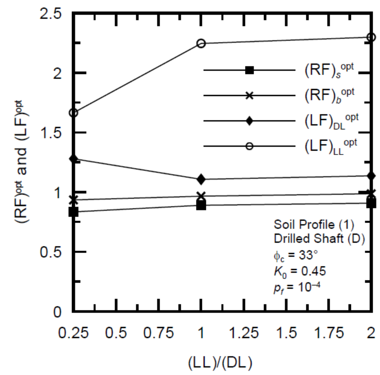

Figure 4 Variation of optimal resistance and load factors with live load to dead load ratio

Figure 5 shows the optimal resistance factors and also the resistance factors adjusted to the dead load and live load factors of 1.25 and 1.75 recommended by AASHTO (2007) for drilled shaft (D) in soil profile (1) and for p_f = 10^{-4}. The figure shows that the resistance factors obtained by using both sets of live load uncertainty parameters are nearly identical. Figure 5 also shows that the adjusted resistance factors do not vary much with (LL)/(DL) ratio. Since the code-adjusted resistance factors do not vary significantly for the different drilled shafts, it is relatively simple to propose values that apply reasonably well to any of the soil profiles/situations studied. Based on our study, we recommend the following values for use in design: $(RF)_b^{code}$=0.85 and $(RF)_s^{code}$=0.75 for p_f = 10^{-3} and $(RF)_b^{code}$=0.70 and $(RF)_s^{code}$=0.65 for p_f = 10^{-4}.

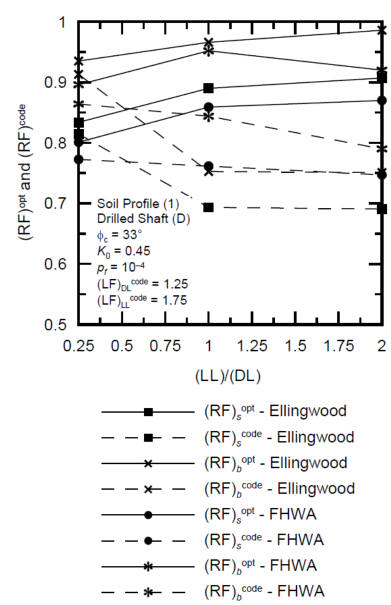

Figure 5 Variation of optimal and code-adjusted resistance factors with live load to dead load ratio

6.2 Drilled shafts in clay

The soil profile is assumed as a homogeneous, completely saturated deposit of normally consolidated clay with $s_{u,\text{mean}}$ / $\sigma'_{v0,\text{mean}}$ = 0.17 and with a water table located at the ground surface. For this deposit, we considered three different clay types. All clay deposits have identical mean critical-state friction angle $\phi_{c,\text{mean}}$ = 21°. However, we considered three different values of mean minimum residual-state friction angle $\phi_{r,\text{min,mean}}$: 21°, 16°, and 9°. We estimated the mean unit weight, the mean void ratio and the mean stress at depths along the pile using the relation between unit weight and void ratio in clay and the relation between mean effective stress and void ratio. Note that, in this research, the mean cone resistance profiles $q_{c,\text{mean}}(z)$ are given as inputs that produce the mean undrained shear strength s_u using equation [15]. The $q_{c,\text{mean}}(z)$ curves were initially back-calculated from pre-assumed ratio of undrained shear strength to effective vertical stress (s_u / σ'_{v0}), void ratio and stress.

The relative depth or slenderness ratio (D/B = L_p/B_p) and diameter and length of drilled shafts that we considered in this research ranged from values that would be considered very low to values considered high for real field conditions. We studied the responses of three drilled shafts with (a) mean diameter $B_{p,\text{mean}}$ = 0.3 m and length L_p = 10 m, (b) mean diameter $B_{p,\text{mean}}$ = 0.9 m and length L_p = 10 m and (c) mean diameter $B_{p,\text{mean}}$ = 1.5 m and length L_p = 10 m. Additionally, we considered (LL)/(DL) = 0.25, 1.0 and 2.0 for clay as well as for sand.

Figure 6 shows the plots of the load and resistance factors with respect to ($\phi_{c,\text{mean}}$ - $\phi_{r,\text{min,mean}}$) for drilled shafts (a) with (LL)/(DL) = 0.25 with target failure probability p_f = 10^{-3}. In Figure 6, it is evident that the optimal load and resistance factors are independent on ($\phi_{c,\text{mean}}$ - $\phi_{r,\text{min,mean}}$). The minor variations in the resistance and load factors observed in Figure 6 are typical of all other drilled shafts.

Figure 7 shows the plots of the load and resistance factors with respect to relative width (B_p/L_p) for all drilled shafts installed in the assumed soil profile with $\phi_{c,\text{mean}}$ - $\phi_{r,\text{min,mean}}$ = 12°, with (LL)/(DL) = 2.0 and with target failure probability p_f = 10^{-3}. Figure 7 shows that the load factors are nearly independent of the relative width (inverse of slenderness ratio) B_p/L_p; the minor variations in the load factors observed in Figure 7 are typical of all the other drilled shafts and are random in all cases.

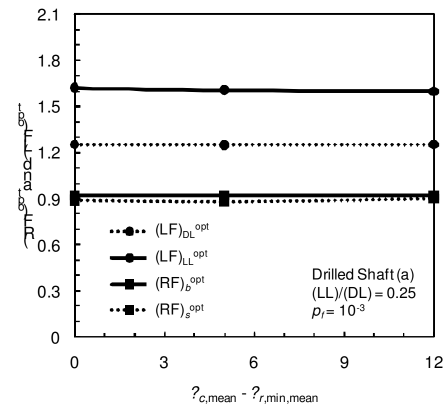

Figure 6 Variation of optimal load and resistance factors to ($\phi_{c,\text{mean}}$ - $\phi_{r,\text{min,mean}}$)

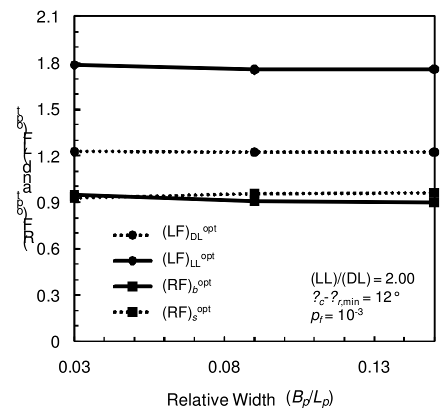

Figure 7 Variation of optimal load and resistance factors to relative width

The soil properties have practically no effect on the resistance and load factors, and the pile dimensions affect only slightly the resistance and load factors. The (LL)/(DL) ratio, however, has a non-negligible effect on the live load factor. Figure 8 show the plots of the load and resistance factors with respect to the (LL)/(DL) ratio for drilled shaft (c) (mean diameter $B_{p,\text{mean}}$ = 1.5 m and length L_p = 10 m) when $\phi_{c,\text{mean}}$ - $\phi_{r,\text{min,mean}}$ = 12° and target failure probability p_f = 10^{-3}. In Figure 8, the optimal resistance factors increase slightly and the optimal dead load factor decreases slightly with increasing (LL)/(DL) ratio; however, the optimal live load factor increases significantly with increasing (LL)/(DL) ratio.

Figure 9 shows the optimal and adjusted resistance factors (for the dead load and live load factors of 1.25 and 1.75 recommended by AASHTO (2007)) for drilled shaft (a) (with mean diameter $B_{p,\text{mean}}$ = 0.3 m and length L_p = 10 m) when p_f = 10^{-3}. As can be seen, the code-adjusted resistance factors do not vary as much as the optimal resistance factors with (LL)/(DL) ratio.

Since the code-adjusted resistance factors do not vary significantly for the different drilled shafts, we can use the results of our calculations to propose resistance factors that will apply reasonably well within wide ranges of values for all the variables, at least for soil profiles that resemble those assumed here. We calculated the mean, standard deviation (SD), maximum and minimum of the resistance factors obtained for the different drilled shafts installed in a given soil profile. Based on these statistics, we calculated the code-adjusted resistance factors with 99% confidence to obtain reasonable and conservative resistance factors: $(RF)_b^{code}$=0.70 and $(RF)_s^{code}$=0.73 for p_f = 10^{-3} and $(RF)_b^{code}$=0.66 and $(RF)_s^{code}$=0.69 for p_f = 10^{-4}. These could be rounded to 0.70 and 0.75 and 0.65 and 0.70.

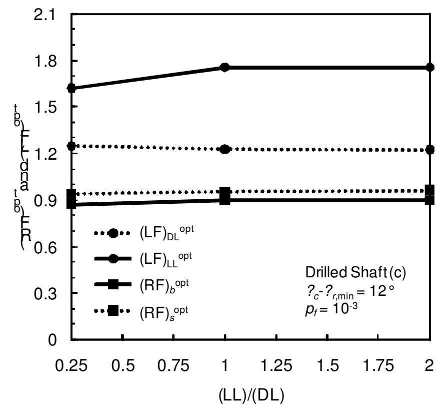

Figure 8 Variation of optimal load and resistance factors with live load to dead load ratio

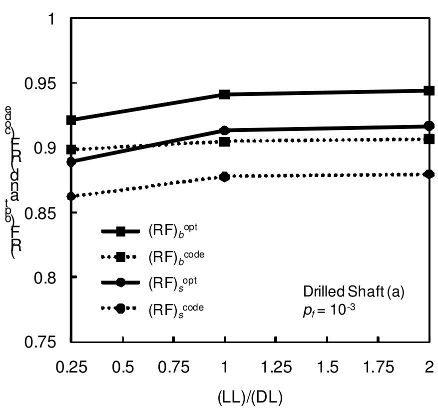

Figure 9 Variation of optimal and code resistance factors with live load to dead load ratio

7 CONCLUSIONS

We performed a systematic probabilistic analysis to develop the resistance factors for drilled shafts in normally consolidated sand and clay for a soil variable-based design method. The analysis involved identification of robust design methods, quantification of the uncertainties associated with the design variables and the design equations and subsequent performance of Monte-Carlo simulations to generate the probability distributions of the pile capacities and applied loads. The limit state loads and shaft and base capacities can be identified from these distributions based on a target probability of failure. From the calculated limit state and nominal values of shaft and base capacities and dead and live loads, the optimal resistance and load factors are obtained. The optimal resistance factors are then adjusted to make them compatible with the dead and live load factors recommended by AASHTO (2007).

In the course of the study, we found that the resistance and load factors did not vary to any significant extent between the different soil profiles and drilled shafts considered. The ratio of live to dead load was identified as the only variable that affected the results; however, it affected mostly the live load factor, with minimal effect on the resistance and dead load factors.

Based on the study, we recommended base and shaft resistance factors that can be used in design with the AASHTO (2007) recommended dead load and live load factors of 1.25 and 1.75, respectively. For drilled shafts in sand the recommended base and shaft resistance values are 0.85 and 0.75 for a probability of failure of 10^{-3} and 0.70 and 0.65 for a probability of failure of 10^{-4}. For drilled shafts in clay, the recommended base and shaft resistance values are 0.70 and 0.75 for a probability of failure of 10^{-3} and 0.65 and 0.70 for a probability of failure of 10^{-4}.

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REFERENCES

AASHTO 2007. *AASHTO LRFD Bridge Design Specifications,* AASHTO.

Baecher, G. B. and Christian, J. T. 2003. *Reliability and Statistics in Geotechnical Engineering.*, John Wiley & Sons Ltd.

Basu, D. and Salgado, R. 2011. Load and Resistance Factor Design of Drilled Shaft in Sand, *Working Paper.*

Basu, P., Salgado, R., Prezzi, M. And Chakraborty, T. 2009. A method for accounting for pile set up and relaxation in pile design and quality assurance, Joint Transportation Research Program, INDOT, SPR-2930.

Bishop, A.W. 1971. Shear Strength Parameters for Undisturbed and Remoulded Soil Specimens. *Stress-Strain Behavior of Soils, Proceedings of the Roscoe Memorial Symposium*, Cambridge University, G.T. Foulis & Co. Ltd.

Bolton, M.D. 1986. The strength and dilatancy of sands. *Geotechnique*, 36(1): 65 - 78.

Chakraborty, T., Salgado, R., Basu, P., and Prezzi, M. 2011. The shaft resistance of drilled shafts in clay. *Working paper.*

Colombi, A. 2005. Physical Modelling of an Isolated Pile in Coarse Grained Soils. *Ph.D. Thesis*, Univ. of Ferrara.

Ellingwood, B. R. and Tekie, P. B. 1999. Wind load statistics for probability-based structural design. *Journal of Structural. Engineering*, 125(4), 453-463.

Fioravante, V. 2002. On the shaft friction modelling of non-displacement piles in sand. *Soils and Foundations*, 42(2), 23-33.

FHWA 2001. *Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures*, FHWA HI-98-032, FHWA.

Foye, K. C. 2005. A rational probabilistic method for the development of geotechnical load and resistance factor design. *Ph. D. Thesis*, Purdue University.

Foye, K. C., Salgado, R. and Scott, B. 2006. Assessment of variable uncertainties for reliability-based design of foundations. *Journal of Geotechnical and Geoenvironmental Engineering*, 135(1), 1-13.

Kim, D. W. 2008. Load and resistance factor design of slopes and MSE walls. *Ph.D. Thesis*, Purdue University.

Lacasse, S. and Nadim, F. 1996. Uncertainties in characterising soil properties. *Uncertainty in the Geologic Environment*, Proc. Uncertainty 96, 49-73.

Lee, I. K., White, W. and Ingles, O. G. 1983. *Geotechnical Engineering*, Pitman.

Lee, J. H. and Salgado, R. 1999. Determination of pile base resistance in sands. *Journal of Geotechnical and Geoenvironmental Engineering*, 125(8), 673-683.

Loukidis, D. and Salgado, R. 2008. Analysis of shaft resistance of non-displacement piles in sand. *Géotechnique,* 58(4), 283-296.

Lumb, P. 1974. Application of statistics in soil mechanics. *Soil Mechanics – New Horizons*, Ed. Lee, I. K., American Elsevier Publishing Company.

Negussey, D., Wijewickreme, W. K. D. and Vaid, Y. P. 1987. Constant-volume friction angle of granular materials. *Canadian Geotechnical Journal*, 25, pp50-55.

Salgado, R. 1993. Analysis of Penetration Resistance in Sands. *Ph.D. Thesis*. Univ. of California, Berkeley.

Salgado, R. and Randolph, M. F. 2001. Analysis of cavity expansion in sand. *International Journal of Geomechanics*, 1(2), 175-192.

Salgado, R., Lyamin, A. V., Sloan, S. W., and Yu, H. S. 2004. Two- and three-dimensional bearing capacity of foundations in clay. *Géotechnique*, 54(5), 297-306.

Salgado, R. and Prezzi, M. 2007. Computation of cavity expansion pressure and penetration resistance in sands. *International Journal of Geomechanics*. 7(4), 251-265.

Salgado, R. 2008. *The Engineering of Foundations*, McGraw Hill.

Titi, H. H., Mahamid, M., Abu-Farsakh, M. Y. and Elias, M. 2004. Evaluation of CPT methods for load and resistance factor design of driven piles. *GeoTrans 2004*, ASCE.

Verdugo, R. and Ishihara, K. 1996. The steady state of sandy soils. *Soils and Foundations*, 36(2), 81-91.