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ABSTRACT

To achieve engineered designs with consistent levels of reliability, the Federal Highway Administration (FHWA) mandated that all new bridges initiated after October 1, 2007, including those founded upon drilled shafts, be designed according to the Load and Resistance Factor Design (LRFD) approach. As the first step in developing efficient regional LRFD procedures for drilled shafts, the Drilled SHAft Foundation Testing (DSHAFT) database was formulated. DSHAFT was aimed at assimilating high quality, historical drilled shaft test data from Iowa and the surrounding states, and it presently contains data from 41 drilled shaft load tests, 38 of which are O-cell load tests, along with subsurface information and structural details. Following an introduction to DSHAFT, several challenges associated with subsurface investigations, measurement of geomaterial properties, and test methods employed in current practice for drilled shaft capacity estimations are discussed. An improved procedure is then proposed featuring three different cases for establishing the equivalent top load-displacement response of drilled shafts. Using the proposed procedure and LRFD framework, it is shown that robust, more efficient regional LRFD resistance factors can be established for drilled shafts with a target displacement limit.

INTRODUCTION

Drilled shafts have been used as a cost-effective deep foundation alternative for bridges over many decades. Drilled shafts are relatively easy to construct in firm cohesive soils, provide a deep foundation alternative in areas requiring a minimal foundation footprint or locations with low overhead clearance, and may not require design and construction of pile cap or pile-to-cap connections. Drilled shafts were traditionally designed using the Allowable Stress Design (ASD) philosophy, in which the uncertainties associated with loads and resistances are considered by a single factor of safety that is insensitive to bias and level of reliability. To achieve engineered designs with consistent levels of reliability, the Federal Highway Administration (FHWA) mandated that all new bridges initiated after October 1, 2007, including those founded upon drilled shafts, be designed according to the Load and Resistance Factor Design (LRFD) philosophy. This mandate initiated an effort to assimilate, evaluate and analyze historical drilled shaft test data necessary for the development of regional LRFD procedures that reflect local conditions and practices.

With support from the Iowa Department of Transportation (Iowa DOT), an electronic database for Drilled SHAft Foundation Testing (DSHAFT) was developed. DSHAFT is currently comprised of 41 drilled shaft load tests conducted in eleven states (Colorado, Iowa, Illinois, Kansas, Kentucky, Minnesota, Missouri, Nebraska, Nevada, South Dakota, and Tennessee), and is available in electronic form at the project website (http://srg.cce.iastate.edu/dshaft/). DSHAFT embodies a model for efficient LRFD analysis on the amassed dataset, laying the groundwork for improving the LRFD procedure for drilled shafts. Utilizing this data, several challenges associated with subsurface investigations, measurement of geomaterial properties and load test methods employed in the current design practice for drilled shafts are then discussed in this paper. These challenges affect the quantification of both estimated and measured resistances and restrict the calibration of LRFD resistance factors using a probability-based reliability theory. Assumptions made to alleviate the challenges of quantifying the relevant geomaterial properties and estimating drilled shaft resistances are presented. One of the most challenging tasks of using the historical test data is the generation of equivalent top load-displacement curves from the Osterberg load cell (O-cell) test results. Most of the top load-displacement curves generated using the current approach suggested by Loadtest, Inc. [2006], as shown in Fig.

1, often fail to provide sufficient information on potential displacement limits, which may be necessary for defining the measured resistance in the LRFD calibration. To overcome this limitation, a procedure is suggested which improves upon the existing methodology. Using the proposed procedure and considering the assumptions made in the resistance estimation, LRFD resistance factors are calculated and compared with those recommended in the reports by Barker et al. [1991], Paikowsky et al. [2004] and Allen [2005], as well as the recommendations in the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications [2010]. Since the development of LRFD procedures for drilled shafts is an ongoing effort, the paper also highlights the importance of determining and collecting more high-quality data for future calibration of LRFD resistance factors.

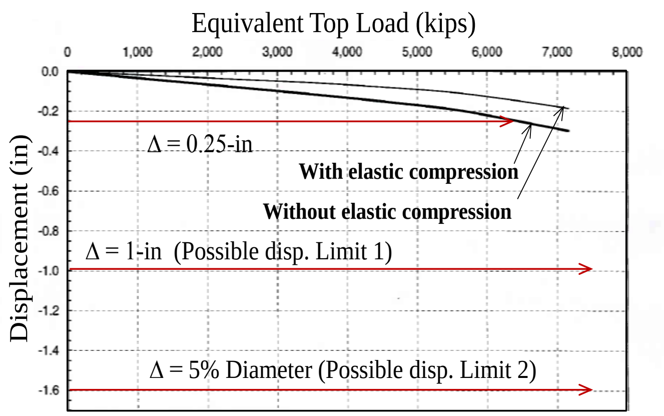


Fig. 1. Typical equivalent top load-displacement curve from O-cell test

DSHAFT DATABASE

A quality assured, electronic database for drilled shafts was developed by Garder et al. [2012] using Microsoft Office Access™. Available site investigation and static load test results were collected, reviewed to ensure, and integrated into DSHAFT, which has an efficient, easy-to-use filtering capability and provides easy access to original field records in electronic format. DSHAFT currently contains 41 drilled shaft tests performed in 11 states as illustrated in Fig. 2(a). Each data set is associated with an identification number (ID) starting from 1 to 41. Out of the 41 tests, 28 are usable tests and their distribution by states are shown in Fig. 2(b). Each usable test includes the structural, subsurface, testing and construction details necessary for the establishment of the LRFD resistance factors. The drilled shaft data are also distributed according to 1) three construction methods (i.e., dry, casing and wet) as shown in Fig. 3; 2) four geomaterials at the shaft base as shown in Fig. 4; and 3) thirteen combinations of geomaterials along the shaft as summarized in Fig. 5. Among the 41 test shafts, 38 were tested using O-cells and the remaining three (IDs 9, 10 and 11) were tested using the Statnamic method. Table 1 shows the summary of the drilled shaft data.

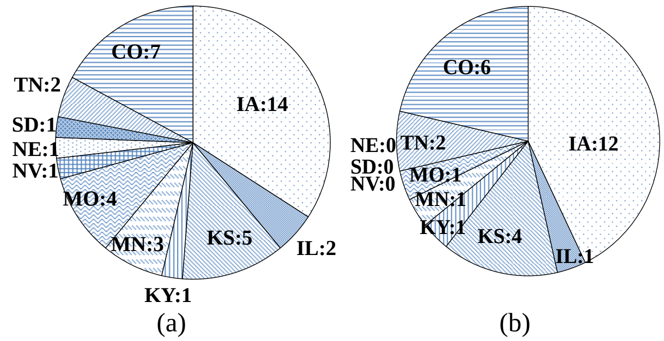


Fig. 2. Distribution of drilled shafts in DSHAFT by states (a) available data, (b) usable data

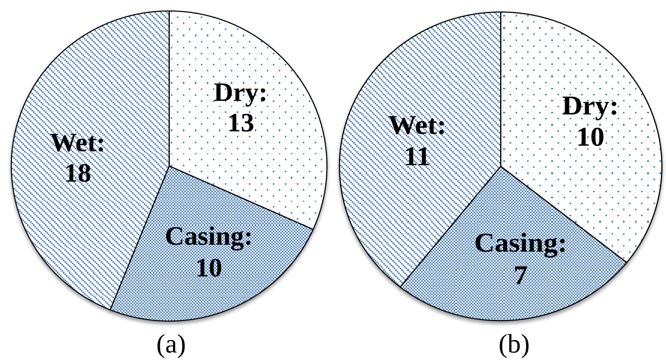


Fig. 3. Distribution of drilled shafts in DSHAFT by construction methods (a) available data, (b) usable data

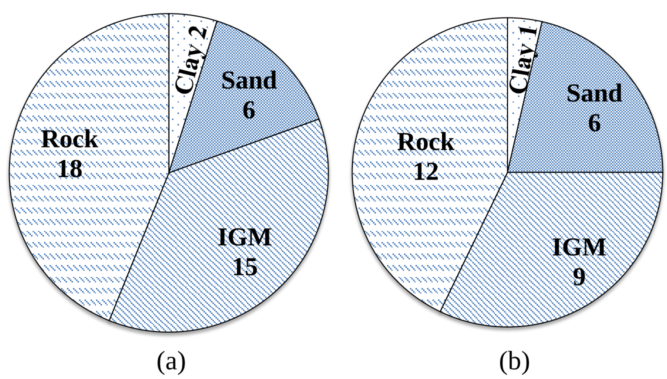


Fig. 4. Distribution of drilled shafts in DSHAFT by geomaterials at base (a) available data, (b) usable data

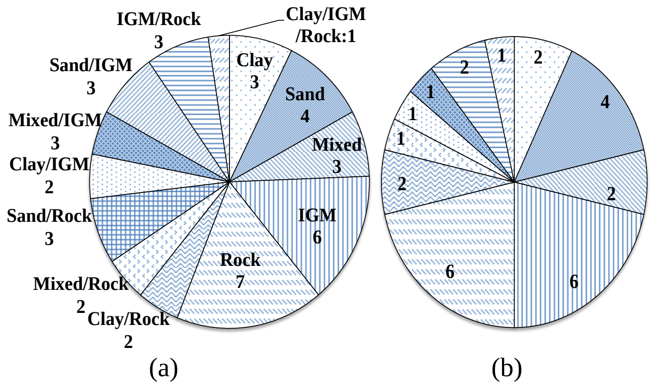


Fig. 5. Distribution of drilled shafts in DSHAFT by geomaterials along shafts (a) available data, (b) usable data

Table 1. Summary of drilled shaft data

ID	State	Dia. (ft)	L (ft)	Geomaterial		Con. Met.	Usable (Y/N)
				Shaft	Base		
1	IA	4	67.9	C	I	W	N
2	IA	3	12.7	R	R	W	Y
3	IA	4	65.8	C+R	R	W	Y
4	IA	3.5	72.7	M+I	I	CA	Y
5	IA	4	79.3	C+I+R	R	W	Y
6	IA	2.5	64.0	C	C	CA	Y
7	IA	3	34.0	C+R	R	W	Y
8	IA	5.5	105.2	M+R	R	CA	Y
9	IA	5	66.3	S	S	W	Y
10	IA	5	55.4	M	S	W	Y
11	IA	5	54.8	M	S	W	Y
12	MN	6.5	93.9	S+R	R	W	N
13	KS	6	49.0	I	I	D	Y
14	MO	6	40.6	I+R	I	D	Y
15	KS	3.5	19.0	I	I	W	Y
16	KS	6	34.0	I	I	D	Y
17	KY	8	105.2	I+R	R	W	Y
18	MO	6.5	69.5	S+I	I	W	N
19	KS	6	26.2	I	I	D	Y
20	MN	6	55.3	S	S	CA	Y
21	KS	5	94.0	S+I	I	D	N
22	MO	3.83	32.0	M+R	R	W	N
23	MN	4	28.0	S+R	R	CA	N
24	IL	5.17	75.1	I+R	R	D	N
25	IL	3.5	37.5	C+I	R	D	Y
26	IA	5	75.2	S	S	W	Y
27	IA	5	75.0	S	S	W	Y
28	TN	4	16.0	R	R	D	Y
29	TN	4	23.0	R	R	D	Y
30	NV	4	103.0	M	C	W	N
31	NE	4	69.1	M+I	I	W	N
32	SD	8	107.3	S+I	I	W	N
33	CO	3.5	22.6	I	I	D	Y
34	CO	3.5	16.0	C	I	D	Y
35	CO	4	25.3	I	I	CA	Y
36	CO	3.5	40.6	R	R	CA	Y
37	CO	4.5	39.7	S+R	R	D	N
38	CO	3	11.3	R	R	D	Y
39	CO	4	20.0	R	R	CA	Y
40	IA	4	59.5	C+I	I	CA	N
41	MO	4.5	28.4	R	R	CA	N

Dia. – shaft diameter; L – embedded length; Con. Met. – construction method; Usable (Y/N) – usable data (Yes/No?); C – clay; S – sand; M –Mixed soil; I – Intermediate Geo Material (IGM); R – rock; W – wet; CA – casing; D – dry

Geotechnical Information and Challenges

The Standard Penetration Test (SPT) N-value of soil, unconfined compressive strength (q_u) and Rock Quality Designation (RQD) of rock and IGM are the three most common geotechnical parameters provided in test reports and are significant to calibration. In the estimation of unit side resistance (q_s) and unit end bearing (q_p) in cohesive soil using the α-method proposed by Tomlinson [1971], undrained shear strength (S_u), used as the main soil parameter, was usually not available and was estimated using the correlation established by Bjerrum [1972] as

$$S_u = \frac{f_1 N_{60} P_a}{100} \qquad (1)$$

where, f_1 is an empirical factor (4.5 for PI = 50 and 5.5 for PI = 15), PI is the plasticity index, N_{60} is the SPT blow count corrected to 60% hammer efficiency, and P_a is the atmospheric pressure. In most tests, PI values were not reported and the reported SPT N-values were not corrected for 60% hammer efficiency, which led to assumed f_1 values based on the reported cohesive soil description. Furthermore, the uncorrected SPT N-values were the primary soil parameters used in the estimation of q_s and q_p in cohesionless soil using the β-method by O′Neill et al. [1999]. The unit weight (γ) required in the estimation of q_s in cohesionless soil and q_p in cohesionless IGM is yet another soil parameter that was not typically available and was estimated based on the correlation suggested by Bowles [1996]. Estimation of q_p in rock and cohesive IGM requires knowledge of mass conditions, which is typically determined by performing extensive site investigations involving multiple boreholes or even geophysical investigations. Unfortunately, these mass conditions were rarely obtained in the field, and the estimation of q_s and q_p in rock and cohesive IGM must therefore rely on the given q_u and RQD values. In this analysis, closed joints were assumed and the pile-cohesive IGM interface friction angle (ϕ_{rc}) was assumed to be 30 degrees. The aforementioned correlations and assumptions will produce additional uncertainties associated with the estimated geotechnical parameters, which can be minimized by directly determining them from in-situ tests, site investigations, and laboratory tests. The probable reason for the lack of adequate geotechnical information is that current drilled shaft design practice and verification relies on the performance of static load tests on test or production shafts. However, detailed geotechnical investigations are essential for the development of the LRFD procedure.

Estimation of Resistances

Using the shaft and subsurface information provided in DSHAFT as briefly summarized in Table 1, unit side resistance (q_s) and unit end bearing (q_p) in each geomaterial layer were estimated using the static methods summarized in Tables 2 and 3, respectively. Although various analytical methods for estimating q_p in cohesive IGM and rock were applied, a combination of methods by Rowe et al. [1987] for intact mass and Carter et al. [1988] for fractured mass is presented due to limited space. The results of the analysis are reported by Ng et al. [2012a].

Table 2. Summary of static methods for estimating q_s

Geomaterial	Method for q_s
Clay	$q_s = \alpha S_u$ (Tomlinson1971)
Sand	$q_s = \beta \sigma'_v$ (O'Neill et al. 1999)
Cohesive IGM	$q_s = \alpha \phi q_u$ (Hassan et al. 1997)
Cohesionless IGM	$q_s = K \tan(\phi') \sigma'_v$ (O'Neill et al. 1999)
Rock	$q_s = 0.65 \alpha_E P_a \left(\frac{q_u}{P_a}\right)^{0.5}$ (Horvath et al. 1979)

α – adhesion factor; β – load transfer coefficient; σ'_v – vertical effective stress; ϕ – correction factor; q_u – uniaxial compressive strength; K – coefficient of horizontal stress; ϕ' - effective friction angle; and α_E – reduction factor

Table 3. Summary of static methods for estimating q_p

Geomaterial	Method for q_p
Clay	$q_p = N_c S_u$ (O'Neill et al. 1999)
Sand	$q_p = 1.2 N_{60}$ (Reese et al. 1989)
Cohesionless IGM	$q_p = 0.59\left[N_{60}\left(\frac{P_a}{\sigma'_v}\right)\right]^{0.8} \sigma'_v$
Cohesive IGM	A combination of $q_p = 2.5 q_u$ (Rowe et al. 1987) and
Rock	$q_p = \left[\sqrt{s} + \sqrt{\left(m\sqrt{s} + s\right)}\right] q_u$ (Carter et al. 1988)

N_c – bearing capacity factor; and s, m – fractured rock mass parameters

O-cell Load Test Results

Since only a single O-cell was routinely used, drilled shaft tests either produced the ultimate side resistance or the ultimate end bearing, but not both. In some cases, the maximum O-cell capacity occurred before either the ultimate side resistance or the end bearing was fully mobilized, as shown in Fig. 6. When these test results are used to generate the equivalent top load-displacement curve as is routinely done [Loadtest 2006], it was shown in Fig. 1 that it does not yield the resistance at 1 in. of top displacement–a criterion adopted by the Iowa DOT, nor at a displacement equal to 5% of the shaft diameter–the displacement criterion recommended by AASHTO [2010]. The O-cell load test results as obtained with one cell thus present new challenges for estimating the ultimate measured resistance using displacement-based design criteria, such as the top displacement limit of 1 in or 5% diameter (i.e., 0.05D displacement limit).

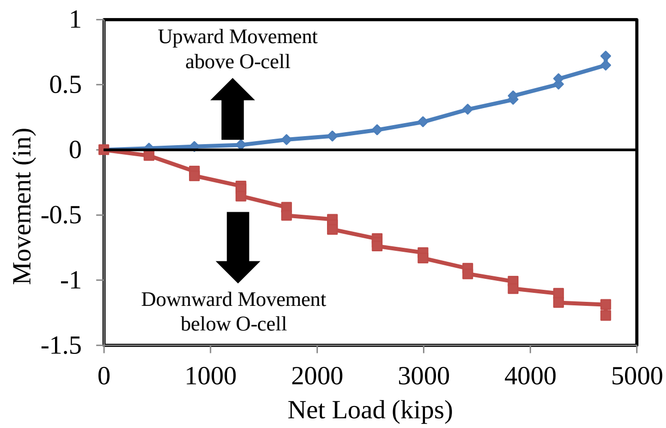


Fig. 6. Neither end bearing nor side resistance reached ultimate for test ID 39

PROPOSED PROCEDURE

Overview

To overcome the limitation of the existing methodology in generating equivalent top load-displacement curves, an improved procedure is proposed with three different shaft response scenarios typically observed in O-cell tests, and they are categorized as Cases A, B and C. Case A corresponds to O-cell tests, in which side resistance (R_s) reaches its ultimate value with an excessive upward displacement before ultimate end bearing (R_p) occurs, as illustrated in Fig. 7 for test ID 2. Case B is the opposite of Case A, in which the end bearing and/or the lower side resistance below the O-cell reaches the ultimate value with an excessive downward displacement occurring before side resistance above the load cell is fully mobilized. When neither the measured side resistance nor the end bearing reaches its respective ultimate value, the shaft response is categorized as Case C as illustrated in Fig. 6 for test ID 39. In each case, the improved procedure can be described in a separate flowchart. For Case A, as shown in Fig. 9, the flowchart at the top describes the current approach suggested by Loadtest, Inc., the flowchart on the left describes the approach to determine the ultimate side shear and extend the measured upward load-displacement curve, and the flow chart on the right describes the approach to determine the ultimate end bearing and extend the measured downward load-displacement curve. After extending the upward and downward curves and identifying the ultimate side shear and end bearing, equivalent dependable top load-displacement curve is reconstructed and adjusted to account for shaft elastic compression. To demonstrate the application of the improved procedure, an example of Case A is presented below while demonstrations of Cases B and C may be found in Ng et al. [2012a].

Example of Case A

Test ID 2 is a 3-ft diameter drilled shaft socketed 12.7 ft in rock with an RQD of 93%. The O-cell test response shown in Fig. 7 was categorized as Case A. A maximum O-cell load (Q_m) of 4,845 kips mobilized an excessive upward movement of 2.63 in. and a minimal downward movement of 0.19 in. Following the proposed method (i.e., left flowchart in Fig. 9), the ultimate side resistance was limited to the maximum upward applied O-cell load of 4,845 kips. Since the ultimate side resistance was smaller than the structural side resistance of 39,488 kips calculated using Eq. (2) for concrete compressive strength (f'_c) of 5.86 ksi and a circumferential area (A_c) of 119.7 ft^2, the original upward load-displacement curve was used in reconstructing the top load-displacement curve.

$$R_s \text{ (structural limit)} = q_s A_c = 7.8P_a\left(\frac{f'_c}{P_a}\right)^{0.5} A_c \qquad (2)$$

In contrast, the end bearing indicated by the downward load-displacement curve did not reach its ultimate resistance; only a very small downward movement was mobilized and the load-displacement curve remained almost elastic. Hence, the ultimate end bearing was determined by following the orange flowchart given in Fig. 9. Having a rock socketed shaft and the RQD smaller than 100%, the ultimate end bearing was limited to either an end bearing of 8,250 kips estimated using the proposed analytical method given in Table 3 or the maximum downward applied O-cell load of 4,845, whichever is larger. In this comparison, the ultimate end bearing of 8,250 kips was preliminarily chosen and compared with the structural capacity of 5,996 kips calculated using Eq. (3) for end bearing and Eq. (2) for 1.7 ft of side friction below the O-cell.

$$R_p \text{ (structural limit)} = 0.85f'_c\,(A_g - A_s) + A_s f_y \qquad (3)$$

where, A_g is the gross cross-sectional area of shaft, A_s is the area of steel reinforcement, and f_y is the yield strength of steel.

Since the preliminary value of 8,250 kips was larger than the structural capacity, the downward load-displacement curve was extended following the best-fit dashed line and the end bearing was limited to the structural capacity shown in Fig. 8. Using the modified downward curve and measured upward curve, the equivalent top load-displacement curve was reconstructed as shown by the dashed line in Fig. 10. The shaft elastic compression was then accounted for, giving the solid line of Fig. 10. Comparing the improved curve given in Fig. 10 with that shown in Fig. 1, the improved curve enables the determination of total measured resistance of 9,698 kips corresponding to the 1-in displacement limit and 10,285 kips corresponding to the 0.05D displacement limit. These total measured resistances were compared with the estimated resistance of 8,741 kips using methods given in Tables 2 and 3, which was later used in the statistical analysis performed to determine the LRFD resistance factors.

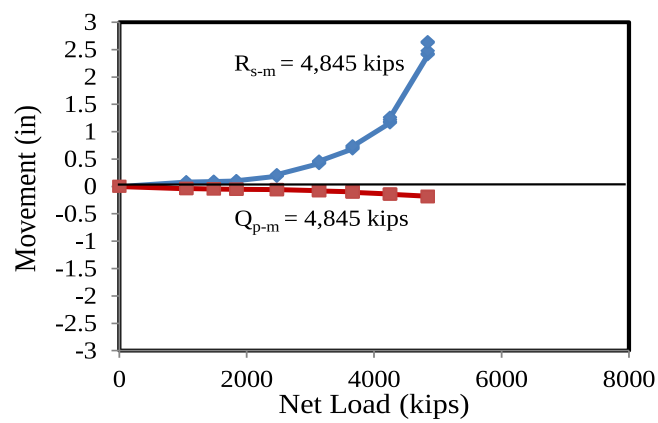


Fig. 7. Original O-cell measurement curves for test ID 2

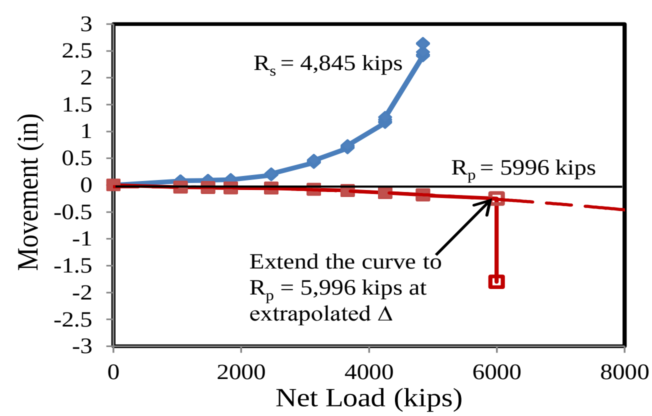
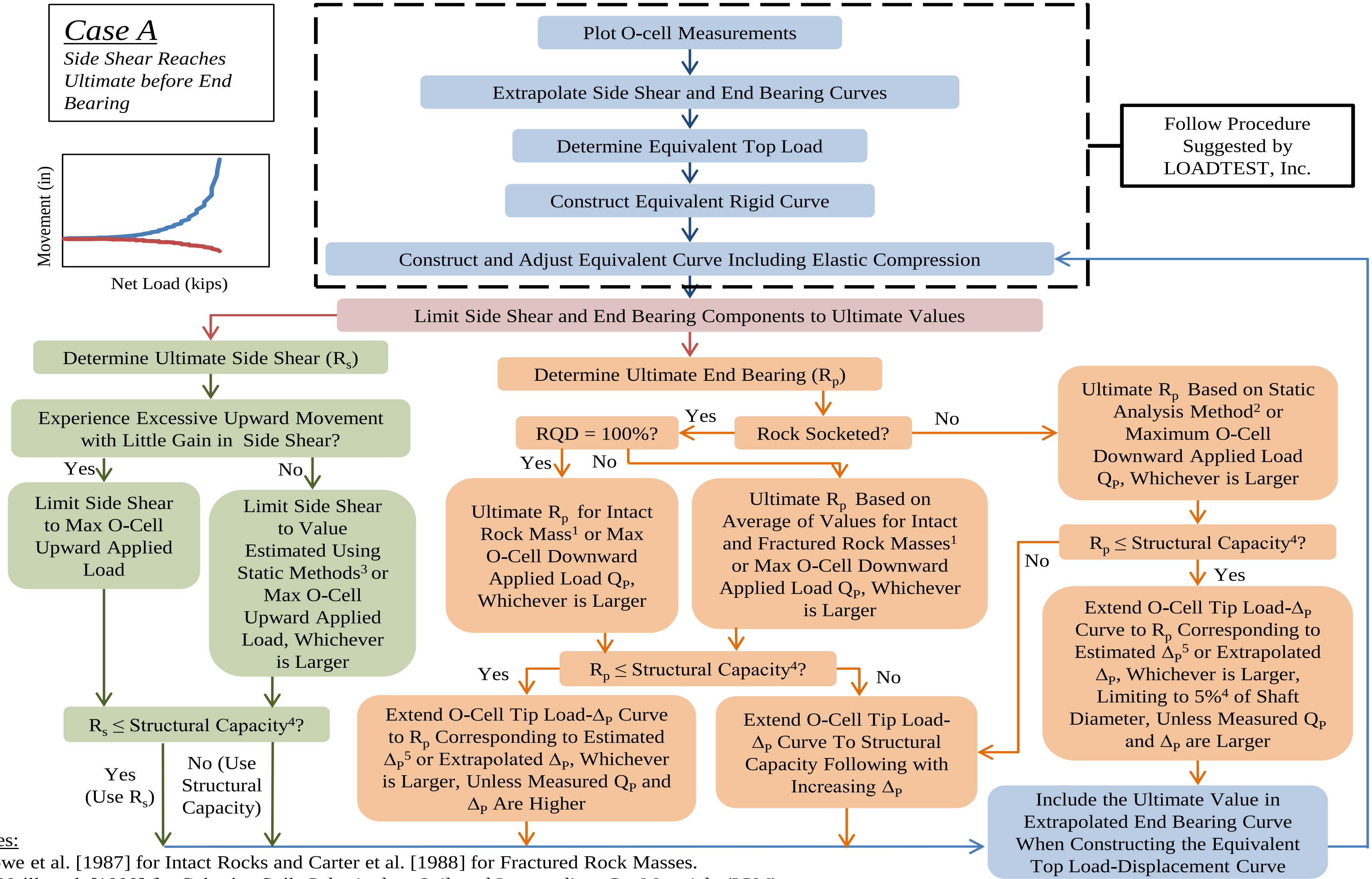


Fig. 8. Modified O-cell measurement curves for test ID 2

Measured Resistances

Adopting the aforementioned proposed procedure, measured resistances were determined for the DSHAFT corresponding



Notes:
[1] Rowe et al. [1987] for Intact Rocks and Carter et al. [1988] for Fractured Rock Masses.
[2] O'Neill et al. [1999] for Cohesive Soil, Cohesionless Soil, and Intermediate-GeoMaterials (IGM).
[3] Alpha-method for Cohesive Soil, Beta-Method for Cohesionless Soil, O'Neill et al. [1996] for IGM, Horvath et al. [1979] for Rocks, or a Combination of All.
[4] AASHTO LRFD Bridge Design Specifications [2010].
[5] Estimated Tip Displacement: Vesic [1977] for Soils, O'Neill et al. [1996] for Cohesive IGM, Mayne et al. [1993] for Granular IGM, and Kulhawy et al. [1992] for Rock

Fig. 9. Proposed procedure to generate an equivalent top load-displacement curve for Case A

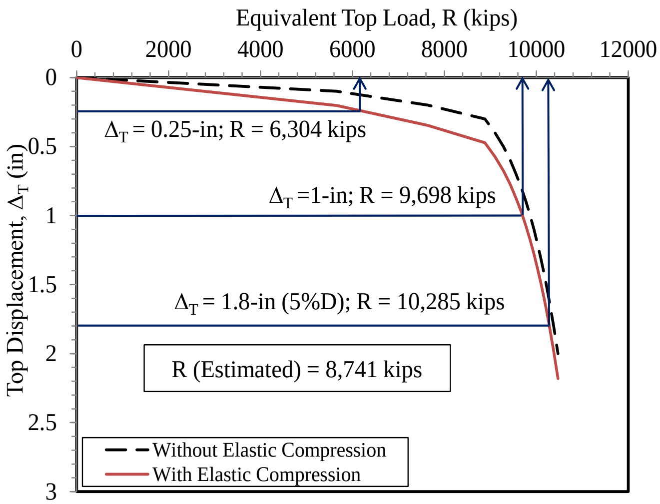


Fig. 10. Equivalent top load-displacement curves generated for a Case A (test ID 2) using the proposed procedure

Table 4. Summary of measured shaft resistances corresponding to 5% of diameter as the displacement limit

ID	Case	Measured Resistance (kips)		
		Total	Side	End Bearing
2	A	10,285	4,289	5,996
3	A	4,422	2,495	1,927
4	A	8,142	4,059	4,083
5	C	5,160	2,322	2,839
6	B	751	411	340
8	C	27,102	8,629	13,350
9	Statnamic	2,530	1,799	731
10	Statnamic	2,285	1,560	725
11	Statnamic	1,950	1,576	374
13	A	2,327	1,297	1,030
14	A	7,594	7,594	Neglected
15	B	4,602	1,412	3,189
16	C	7,594	7,594	0
17	A	2,820	1,499	1,321
19	A	17,363	8,024	9,339
20	A	3,811	2,258	1,553
25	C	3,160	1,580	1,580
26	B	14,238	7,857	6,381
27	B	3,160	1,580	1,580
28	A	13,034	4,323	8,711
29	C	14,836	5,486	9,350
33	N/A	1,067	N/A	N/A
34	B	1,220	660	560
35	N/A	6,504	N/A	N/A
36	N/A	14,218	N/A	N/A
38	A	9,283	3,108	6,175
39	C	10,769	4,708	5,805

N/A – not available

to the displacement-based design criteria. Table 4 summarizes the measured total resistance, side resistance and end bearing corresponding using the 5% shaft diameter as the displacement limit.

To assist with the calibration of resistance factors for various geomaterials along a drilled shaft, the measured side resistances summarized in Table 4 for the 0.05D displacement limit were proportioned according to the fraction of side resistance measured from the O-cell load test in each geomaterial layer.

DEVELOPMENT OF REGIONAL LRFD RESISTANCE FACTORS

The regional LRFD resistance factors (φ) are calibrated following the reliability-based framework adopted by AASHTO. Among the various methods that fit into this framework, the modified FOSM method proposed by Bloomquist [2007], which is given in Eq. (4), was selected to determine the resistance factors for total resistance (R), side resistance (R_s) and end bearing (R_p) with respect to four different geomaterials (i.e., clay, sand, IGM and rock)

$$\varphi = \frac{\lambda_R\left(\frac{\gamma_D Q_D}{Q_L}+\gamma_L\right)\sqrt{\frac{\left(1+\frac{\frac{Q_D^2}{Q_L^2}\lambda_D^2 COV_D^2+\lambda_L^2 COV_L^2}{\frac{Q_D^2}{Q_L^2}\lambda_D^2+2\frac{Q_D}{Q_L}\lambda_D\lambda_L+\lambda_D^2}\right)}{(1+COV_R^2)}}}{\left(\frac{\lambda_D Q_D}{Q_L}+\lambda_L\right)\exp\left\{\beta_T\sqrt{\ln\left[(1+COV_R^2)\left(1+\frac{\frac{Q_D^2}{Q_L^2}\lambda_D^2 COV_D^2+\lambda_L^2 COV_L^2}{\frac{Q_D^2}{Q_L^2}\lambda_D^2+2\frac{Q_D}{Q_L}\lambda_D\lambda_L+\lambda_D^2}\right)\right]}\right\}} \tag{4}$$

where, COV is the coefficient of variation, β_T is the target reliability index, λ is the resistance bias factor, γ is the load factor, Q is the applied load, R is the resistance in consideration, D is the dead load, and L is the live load. This method was chosen, due to its simplicity and because it provides comparable results to other more rigorous reliability methods [Bloomquist 2007; Ng et al. 2012b]. Using the estimated resistances and the measured resistances corresponding to the failure criterion, statistical measures (i.e., mean and standard deviation) of resistance ratio, which is the ratio of measured to estimated resistance, were determined for each resistance component and geomaterial. To verify whether the drilled shaft resistances follow a lognormal distribution, a hypothesis test based on the Anderson-Darling (AD) [1952] normality method was used to assess the goodness-of-fit of the assumed distribution. The Anderson-Darling method was preferred because of it appropriateness for normality tests when the same size is relatively small sample size [Romeu, 2010].

With the focus on the axial resistance of a drilled shaft, the AASHTO [2010] Strength I load combination was selected in the calibration process, in which only dead load (Q_D) and live load (Q_L) were considered in the limit state equation ($\sum\gamma Q \leq$

φR). The probabilistic characteristics of dead and live loads were adopted after AASHTO [2010] in the calibration framework. Since the dead to live load ratio (Q_D/Q_L) has no significant influence on the resistance factor [Paiskowsky et al. 2004], a Q_D/Q_L ratio of 2.0, the same ratio used in the calibration of resistance factors for driven piles in Iowa [AbdelSalam et al. 2012], was selected to remain consistency between foundation types. Additionally, the calibration of resistance factors requires a proper selection of a set of target reliability levels represented by a series of target reliability indices (β_T) that correspond to a range of probability of failure (p_f). Resistance factors recommended in AASHTO [2010] for drilled shafts were determined based on a β_T of 3.0, because a bridge foundation normally has four or fewer drilled shafts per cap. However, for a redundant foundation with five or more drilled shafts per cap, a lower β_T of 2.33 may be used.

To cover a wide range of design possibilities, reliability indices of 2.00, 2.33, 2.50, 3.00 and 3.50 were selected as shown in Fig. 11 through Fig. 14. Due to space limitations, only results corresponding to the 0.05D displacement limit are presented. To evaluate the efficiency of the analytical methods for different geomaterials, efficiency factors (φ/λ), the ratio of resistance factor to resistance bias, were calculated over the range of reliability indices as shown in Fig. 12 for side resistance and Fig. 14 for end bearing. These figures show that the resistance and efficiency factors decrease with increasing reliability indices. Fig. 12 shows that the analytical methods for side resistances in sand and rock given in Table 2 have the highest efficiency while the α-method for clay has the lowest efficiency. Since only one drilled shaft was found to have bearing in clay (i.e., ID 6 given in Table 1), a statistical analysis on end bearing in clay couldn't be performed and no results are reported for clay in Figs. 13 and 14. Fig. 14 shows that the analytical method for rock given in Table 3 has the highest efficiency factors despite having the lowest resistance factors in Fig. 13 while the method for sand has the lowest efficiency factors. The analysis confirms that the efficiency of an analytical method is not judged by its resistance factor but by its efficiency factor.

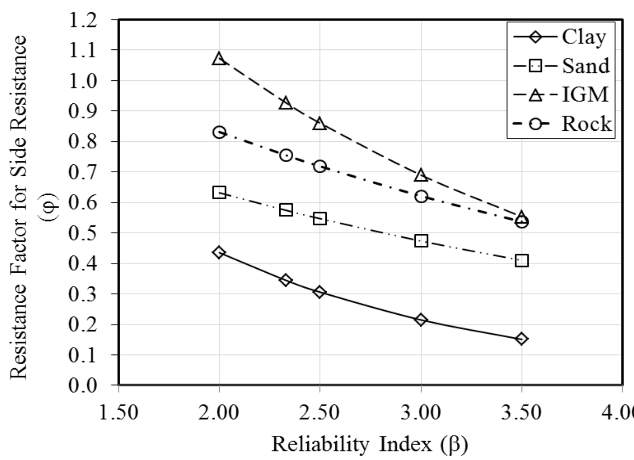


Fig. 11. Resistance factors for side resistance

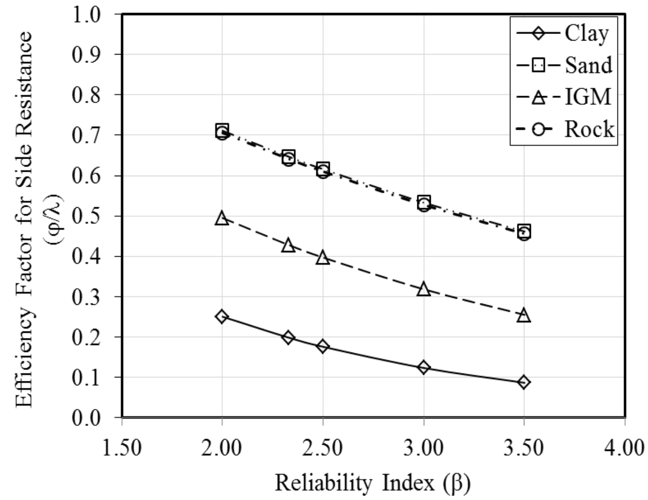


Fig. 12. Efficiency factors for side resistance

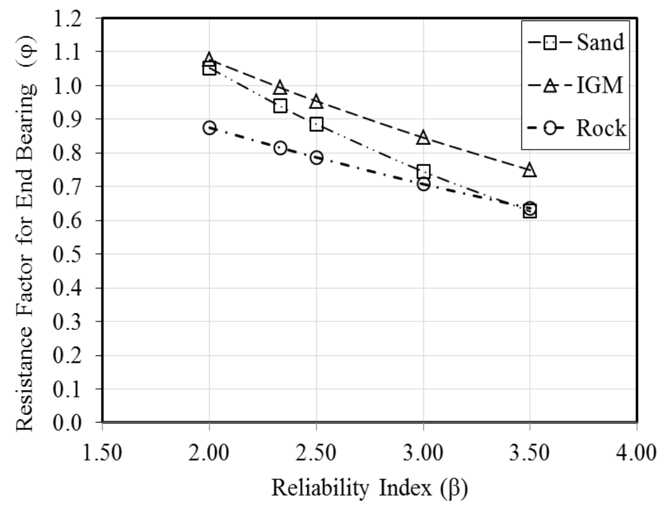


Fig. 13. Resistance factors for end bearing

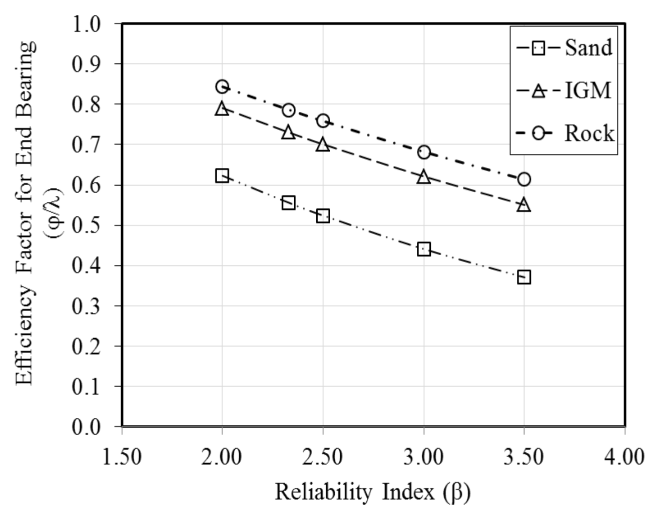


Fig. 14. Efficiency factors for end bearing

The calculated resistance factors for four geomaterials based on a β_T of 3.00 are compared with those recommended in the National Cooperative Highway Research Program (NCHRP) Report 343 by Barker et al. [1991], NCHRP Report 507 by Paikowsky et al. [2004], National Highway Institute (NHI) Report No. 05-052 by Allen [2005] and AASHTO [2010] as shown in Table 5 for side resistance and Table 6 for end bearing. The efficiency factors (φ/λ) from NCHRP Report 507 are also included for comparison. The analytical methods and the calibration procedures used in the literature are described in superscripted notes below Tables 5 and 6. Table 5 shows that resistance and efficiency factors of the side resistance in clay obtained from DSHAFT are lower than those recommended in the literature. This is possibly attributed to the high uncertainty associated with the estimation of undrained shear strength of clay from SPT N-values using Eq. (1). Table 5 indicates that the regionally calibrated resistance factors for side resistance in sand, IGM and rock are either comparable to or higher than those recommended in the literature. Similarly, Table 6 indicates that regionally calibrated resistance and efficiency factors for end bearing are higher than those recommended in the literature. The regional LRFD calibration reflects local soil conditions and provides a cost-effective design for drilled shafts.

Although the 1-in. displacement limit and the 0.05D displacement limit were adopted in this paper, the proposed procedure to generate equivalent top load-displacement curves enables the development of LRFD resistance factors in accordance with any displacement-based design criterion, in which drilled shaft designs not only satisfy the strength limit state but also will ensure a desirable and tolerable serviceability displacement.

Table 5. Comparison of resistance factors of side resistance corresponding to 0.05D displacement limit

Geo	Resistance Factor (φ) for β_T = 3.00				
	NC-343	NC-507	NHI	A	DSHAFT
Clay	0.65	0.36 (φ/λ=0.41)	0.60(c)	0.45(d)	0.22 (φ/λ=0.12)
Sand	N/A	0.31 (φ/λ=0.28)	0.55(b)	0.55(b)	0.47 (φ/λ=0.53)
IGM	N/A	0.51 (φ/λ=0.41)	0.55(b)	0.60(d)	0.69 (φ/λ=0.32)
Rock	0.65	0.38(a) (φ/λ=0.32)	0.55(c)	0.55(d)	0.62 (φ/λ=0.53)

Geo – geomaterials; NC-343 – NCHRP Report 343; NC-507 – NCHRP Report 507; NHI – NHI Report 05-052; A – AASHTO [2010]; (a) – based on analytical method by Carter et al. [1988]; (b) – based on calibration by fitting to ASD; (c) – calibration performed using Monte Carlo Method; (d) – based on combined recommendations from NCHRP 343, NCHRP 507 and NHI 05-052

Table 6. Comparison of resistance factors of end bearing corresponding to 0.05D displacement limit

Geo	Resistance Factor (φ) for β_T = 3.00				
	NC-343	NC-507	NHI	A	DSHAFT
Clay	0.55	0.24–0.28 (φ/λ = 0.29–0.31)	0.60(e)	0.40(b)	N/A
Sand	N/A	0.25–0.73 (φ/λ = 0.15–0.32)	0.55(b)	0.50(b)	0.70(f) (φ/λ=0.44)
IGM	N/A	0.57–0.65(a) (φ/λ = 0.44–0.48)	0.55(a)	0.55(a)	0.70(f) (φ/λ=0.62)
Rock	N/A	0.45–0.49(c) (φ/λ = 0.37–0.38)	0.55(d)	0.50(d)	0.70(f) (φ/λ=0.68)

Geo – geomaterials; NC-343 – NCHRP Report 343; NC-507 – NCHRP Report 507; NHI – NHI Report 05-052; A – AASHTO [2010]; (a) – based on analytical method by O′Neill et al. [1999]; (b) – based on calibration by fitting to ASD; (c) – based on analytical method by Carter et al. [1988] (d) – based on analytical method by the Canadian Geotechnical Society [1985]; (e) – calibration performed using Monte Carlo Method; (f) – resistance factors were reduced to maximum AASHTO value of 0.70 from 0.75 for sand, 0.85 for IGM and 0.71 for rock.

CONCLUSIONS

In response to the FHWA’s mandate, the DSHAFT database has been developed to establish groundwork for improving current LRFD procedures for drilled shafts. DSHAFT is aimed at assimilating high quality, historical drilled shaft test data from Iowa and the surrounding states. DSHAFT presently comprises historical data from 41 drilled shaft load tests, along with subsurface information and structural details. The application of the historical data was restricted by several challenges associated with subsurface investigations, determination of geomaterial properties and test methods employed in the current practice for drilled shaft capacity estimations. Due to limited geotechnical information, correlations and assumptions were made to quantify required geomaterial parameters, which created unnecessary uncertainties in the statistical analysis. These parameters can be easily and economically determined from in-situ investigations and laboratory tests, which should be performed whenever possible. Hence, it is important to highlight that detailed and appropriate site investigations are essential for further advancements of LRFD resistance factors.

Another challenge identified in this study is attributed to typical O-cell test results from which the equivalent top load-displacement curves are generated. Many of these curves do not pass the desired displacement-based design criteria that define the ultimate measured resistance. To overcome this challenge, a procedure to generate equivalent top load-displacement curves from O-cell tests was introduced to facilitate the development of regionally calibrated LRFD resistance factors in accordance with any displacement-based design criterion. This procedure is demonstrated to three different load test cases, referred as A, B and C. Following the flowcharts described in the procedure, ultimate side resistance corresponding to the modified upward O-cell test curve and ultimate end bearing corresponding to the modified downward O-cell test curve were determined and used in the generation of a top load-displacement curve. Using the proposed procedure, measured resistances corresponding to a desired displacement-based design criterion were determined and used in LRFD resistance factor calibration. With the exception of side resistance in clay, the results demonstrated that the regional LRFD resistance factors and their efficiencies were higher than those recommended in the literature. In summary, the proposed procedure improved the development of the LRFD approach and facilitated more efficient and more dependable regional LRFD calibration for drilled shafts that accounted for the local soil conditions.

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