RELIABILITY INDEX FOR SERVICEABILITY LIMIT STATE OF DRILLED SHAFTS UNDER UNDRAINED COMPRESSION

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ABSTRACT

In recent years, reliability-based design (RBD) has gradually gained popularity in geotechnical engineering. Several RBD codes have been developed and implemented around the world that calibrate ultimate limit state (ULS) designs for a target ULS reliability index (β_{uls}). However, the serviceability limit state (SLS) design still is considered using conventional deterministic approaches with an unknown SLS reliability index (β_{sls}). This paper makes use of a relationship between β_{sls} and β_{uls} to infer the β_{sls} of drilled shafts under undrained compression from the β_{uls} that is specified already in the design codes. The values of β_{sls} are estimated for drilled shafts designed in accordance with three different design methods (*i.e.*, semi-empirical analysis using in situ and laboratory test data, analysis using static loading test results, and analysis using dynamic monitoring results) of the National Building Code of Canada (NBCC). The results indicate that, for the undrained compression capacity of drilled shafts designed in accordance with the NBCC, the designs automatically fulfill the corresponding SLS design requirements.

Key words: Serviceability limit state, ultimate limit state, drilled shafts, reliability-based design, undrained compression.

1. INTRODUCTION

Over the last two decades, reliability-based design (RBD) methodologies gradually have gained popularity in geotechnical engineering. Several RBD codes have been developed and implemented around the world, such as the load and resistance factor design (LRFD) code adopted by the American Association of State Highway and Transportation Officials (AASHTO 1997), the Canadian National Building Code (Becker 1996), Eurocode 7 [European Committee for Standardization (CEN) 2001], and the Japanese Geo-Code 21 (Honjo and Kusakabe 2002). A review of these RBD design codes shows that, although reliability principles are applied for ultimate limit state (ULS) designs to achieve a target ULS reliability index (β_{uls}), the serviceability limit state (SLS) designs still are evaluated using conventional deterministic approaches with an unknown SLS reliability index (β_{sls}). One exception is the RBD study for transmission line (and similar) structure foundations that was sponsored by the Electric Power Research Institute (EPRI) in North America (Phoon *et al*. 1995; Phoon *et al*. 2003a, 2003b; Phoon and Kulhawy 2008). However, in this EPRI study of SLS design, the limiting tolerable foundation settlement (y_{lt}) still was considered to be deterministic. Recognizing that SLS design is an indispensable aspect in various design codes (*e.g.*, AASHTO 1997; CEN 2001) underscores the need of proper estimate of the β_{sls} values.

This paper makes use of a relationship between β_{sls} and β_{uls} to infer the β_{sls} of drilled shafts under undrained compression from the β_{uls} that is specified already in the design codes. The probabilistic distribution of y_{lt} and the uncertainties associated with the calculation models of drilled shafts are accounted for explicitly in this paper. First, the β_{sls} and β_{uls} relationship is briefly described. Then, key variables in this relationship are explored, including the ratio (R) of the SLS capacity (Q_{sls}) to the ULS capacity (Q_{uls}), coefficient of variation of the ULS capacity (COV_{Quls}), and coefficient of variation of the design load F (COV_F). Capacity herein refers to the maximum soil resistance mobilized when a foundation is loaded to reach either the ultimate limit state (Q_{uls}) or serviceability limit state (Q_{sls}). Then, the relationship is used to estimate β_{sls} for drilled shafts under undrained compression that are designed in accordance with the National Building Code of Canada (NBCC), as described by Becker (1996). Finally, the effects of y_{lt} on β_{sls} are discussed.

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2. RELATIONSHIP BETWEEN β_{uls} AND β_{sls}

In RBD, design quantities, such as the load (F) and the capacity (Q), are commonly modeled as lognormal random variables (Ang and Tang 1975). The basic reliability problem is to evaluate the probability of failure (p_f) or β from some pertinent probabilistic characterizations of F and Q, which frequently include the mean (m_F and m_Q), standard deviation (s_F and s_Q), coefficient of variation (COV_F and COV_Q), and even probability density functions.

For lognormally distributed F and ULS capacity Q_{uls}, the β_{uls} can be expressed as (*e.g.*, Barker *et al.* 1991; Becker 1996; Phoon *et al.* 2003a, 2003b):

$$\beta_{uls} = \Phi^{-1}(1 - p_{fuls}) = \frac{\ln\left[\dfrac{m_{Q_{uls}}}{m_F}\sqrt{\dfrac{1+\mathrm{COV}_F^2}{1+\mathrm{COV}_{Q_{uls}}^2}}\right]}{\sqrt{\ln\left[(1+\mathrm{COV}_{Q_{uls}}^2)(1+\mathrm{COV}_F^2)\right]}} \quad (1)$$

in which Φ^{-1} = inverse standard normal probability distribution function, p_{fuls} = the probability of failure at ultimate limit state, m_{Quls} = mean of ULS capacity Q_{uls}, and COV_{Quls} = coefficient of variation of ULS capacity Q_{uls}. If the ratio of SLS capacity (Q_{sls})

to Q_{uls} is defined as R, the β_{sls} can be directly related to β_{uls} as (Wang and Kulhawy 2008a, b) :

$$\beta_{sls} = C_0 + C_1 \beta_{uls} \tag{2}$$

in which C_0 and C_1 = intercept and slope of this linear function, respectively, given by:

$$C_0 = \frac{\ln\left[\frac{m_R}{\sqrt{1+COV_R^2}}\right]}{\sqrt{\ln\left[(1+COV_R^2)(1+COV_{Q_{uls}}^2)(1+COV_F^2)\right]}} \tag{3}$$

$$C_1 = \sqrt{\frac{\ln\left[(1+COV_{Q_{uls}}^2)(1+COV_F^2)\right]}{\ln\left[(1+COV_R^2)(1+COV_{Q_{uls}}^2)(1+COV_F^2)\right]}} \tag{4}$$

where m_R = mean of R and COV_R = coefficient of variation of R. Derivations of Eqs. (2) ~ (4) are referred to Wang and Kulhawy (2008b).

3. KEY VARIABLES IN THE β_{uls} AND β_{sls} RELATIONSHIP

The β_{sls} and β_{uls} relationship aforementioned shows that β_{sls} is a function of COV_F, COV_{Quls}, β_{uls}, m_R, and COV_R. Each of them is discussed separately in this section. Table 1 summarizes the typical ranges of COV_F of various load effects for foundations. The COV_F of dead loads, live loads, and environmental loads lies in the ranges of 0.05-0.15, 0.2-0.6, and 0.3-0.5, respectively (Meyerhof 1993, 1995).

The values of COV_{Quls} depend on the design method used to calculate the Q_{uls}. Table 2 shows the values of COV_{Quls} adopted in NBCC (Becker 1996). When the Q_{uls} is calculated by semi-empirical analysis using in situ and laboratory test data, the value of COV_{Quls} is 0.40. In contrast, when the Q_{uls} is calculated by analyses using static loading test results or dynamic monitoring results, the values of COV_{Quls} are 0.25 and 0.30, respectively.

The values of β_{uls} are specified already in RBD codes, and Eq. (1) has been used as the basis in the RBD codes to achieve the target β_{uls} (*e.g.*, Barker *et al.* 1991; Becker 1996; Phoon *et al.* 2003a; 2003b). Using a set of calibrated resistance factors (ψ) which are reduction factors applied to the calculated resistance to account for its uncertainty, the RBD codes ensure that all ULS designs have a nominally consistent p_{fuls} or β_{uls}. Consider, for example, the National Building Code of Canada (Becker 1996, see Table 2), in which the proposed resistance factors are 0.4, 0.6, and 0.5 for three different design methods to achieve their respective target β_{uls} values of 3.4, 3.2, and 3.5. For reference, Table 3 correlates reliability indices for representative geotechnical components and systems and their corresponding probabilities of failure and expected performance levels. The reliability indices range from 1 to 5, corresponding to probabilities of failure varying from about 0.16 to 3×10^{-7}. A β_{uls} value larger than 3 is commonly adopted for ultimate limit state in RBD, and it corresponds to an expected performance level better than "above average".

A key variable in the relationship between β_{uls} and β_{sls} is $R = Q_{sls}/Q_{uls}$ and its mean (m_R) and coefficient of variation (COV_R).

Table 1 Coefficient of variation of load effects for foundations (after Meyerhof 1993 and 1995)

Load Type	Coefficient of variation, COV_F
Dead loads	0.05 ~ 0.15
Live loads	0.2 ~ 0.6
Environmental loads	0.3 ~ 0.5

Table 2 Summary of resistance factors for pile foundation in National Building Code of Canada (NBCC, after Becker 1996)

Design method	Resistance factor, ψ	ULS reliability index, β_{uls}	Coefficient of variation of Q_{uls}, COV_{Quls}
Semi-empirical analysis using in situ and laboratory test data	0.4	3.4	0.40
Analysis using static loading test results	0.6	3.2	0.25
Analysis using dynamic monitoring results	0.5	3.5	0.30

Table 3 Relationship between reliability index (β) and probability of failure (p_f) (U.S. Army Corps of Engineers 1997, p. B-11)

Reliability index β	Probability of failure $p_f = \Phi(-\beta)$	Expected performance level
1.0	0.16	Hazardous
1.5	0.07	Unsatisfactory
2.0	0.023	Poor
2.5	0.006	Below average
3.0	0.001	Above average
4.0	0.00003	Good
5.0	0.0000003	High

Note: $\Phi(\ )$ = standard normal probability distribution function

Probabilistic characterization of R requires a load-displacement model that relates the foundation displacement to load capacity and probabilistic characterization of the limiting tolerable foundation settlement (y_{lt}). A probabilistic load-displacement model for drilled shafts under undrained compression is described in next section, followed by probabilistic characterization of y_{lt} and closed-form approximations of m_R and COV_R in two respective sections.

4. LOAD-DISPLACEMENT MODEL FOR DRILLED SHAFTS UNDER UNDRAINED COMPRESSION

Accurate prediction of foundation movements is a difficult task, and most analytical attempts have met with only limited success, primarily because they can not include all the important factors, such as the in-situ stress state, soil behavior, soil-foundation interface characteristics, and construction effects (Kulhawy 1994; Becker 1996, and Phoon *et al*. 2003b). Alternatively, an empirical approach has been employed that utilizes load test results and normalizes the test load-displacement curves to obtain a single representative design curve (Phoon *et al*. 1995; Phoon *et al*. 2003b). For drilled shafts under undrained compression, the load-displacement curves can be represented reasonably well by the following hyperbolic model (Phoon *et al*. 1995; Phoon *et al*. 2003b):

$$\frac{Q}{Q_{\text{uls}}} = \frac{y/B}{a + b\ (y/B)} \tag{5}$$

in which Q = compression load, Q_{uls} = ULS capacity, y = axial butt displacement, B = drilled shaft diameter, and a and b = hyperbolic model parameters.

Phoon *et al*. (1995) compiled a database that includes load tests of 27 drilled shafts with pile diameter B varying from 0.18 m to 1.3 m. The drilled shafts were installed in clay and tested under axial compression. Figure 1 shows a scatter plot of the hyperbolic model parameters a and b. It is found that a and b are virtually uncorrelated and have the following statistics: Mean (m_a = 0.0040 and m_b = 0.7798), standard deviation (s_a = 0.0024 and s_b = 0.1492), and coefficient of correlation ($\rho_{a,b}$ = –0.05). For simplification herein, the value of $\rho_{a,b}$ is taken as 0.

5. LIMITING TOLERABLE SETTLEMENTS FOR FOUNDATIONS

The limiting tolerable foundation settlement (y_{lt}) is the maximum settlement that a foundation can sustain before causing any serviceability failure, and it corresponds to the SLS capacity Q_{sls}, expressed as:

$$R = \frac{Q_{\text{sls}}}{Q_{\text{uls}}} = \frac{y_{\text{lt}}/B}{a + b\ (y_{\text{lt}}/B)} \tag{6}$$

The y_{lt} for foundations has been examined by many researchers (*e.g.*, Skempton and MacDonald 1956; Lumb 1964; Grant *et al*. 1974; Wahls 1981, 1994; Zhang and Ng 2005), and deterministic y_{lt} values have been proposed and adopted in design codes around the world. Zhang and Ng (2005) synthesized the y_{lt} values for pile foundations supporting buildings or bridges and used fragility curves to represent the cumulative probability distribution of y_{lt}. The y_{lt} statistics, including the mean ($m_{y\text{lt}}$), standard deviation ($s_{y\text{lt}}$) and coefficient of variation ($COV_{y\text{lt}}$), were then obtained from the cumulative probability distribution of y_{lt}. Detailed development of these statistics is referred to Zhang and Ng (2005).

Table 4 summarizes the y_{lt} statistics, including the mean ($m_{y\text{lt}}$), standard deviation ($s_{y\text{lt}}$) and coefficient of variation ($COV_{y\text{lt}}$). For pile foundations supporting buildings, $m_{y\text{lt}}$ = 96 mm, $s_{y\text{lt}}$ = 56 mm, and $COV_{y\text{lt}}$ = 0.583. In contrast, for pile foundations supporting bridges, $m_{y\text{lt}}$ = 135 mm, $s_{y\text{lt}}$ = 89 mm, and $COV_{y\text{lt}}$ = 0.659. Note that, as the structure types are different for buildings and bridges, their respective $m_{y\text{lt}}$ are also different considerably. Nonetheless, these y_{lt} statistics are significantly larger than the allowable settlement limit of 25 mm that is used frequently in deterministic SLS designs of most foundation types (*e.g.*, Peck *et al*. 1974; Wahls 1994). The y_{lt} statistics summarized in Table 4 are therefore used as a starting point in this work. The effect of y_{lt} on β_{sls} is discussed later.

Table 4 Statistics of limiting tolerable settlement (y_{lt}) for pile foundations (after Zhang and Ng 2005)

Statistics	Supporting Buildings	Supporting Bridges
Mean, $m_{y\text{lt}}$ (mm)	96	135
Standard Deviation, $s_{y\text{lt}}$ (mm)	56	89
Coefficient of Variation, $COV_{y\text{lt}}$	0.583	0.659

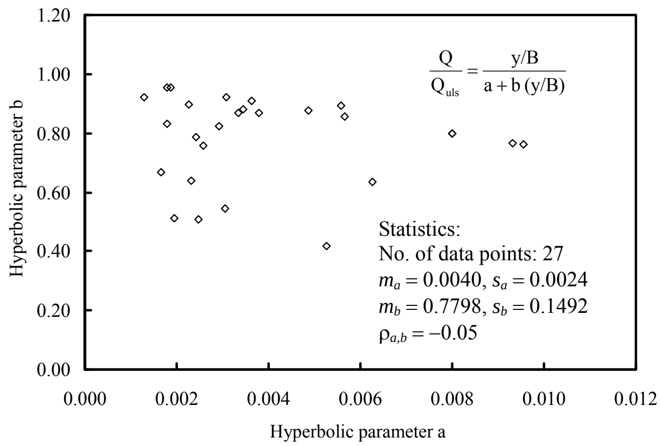


Fig. 1 Scatter plot of hyperbolic parameters *a* and *b*

6. CLOSED-FORM APPROXIMATIONS OF m_R AND COV_R

With the probabilistic load-displacement model for drilled shafts and mean and standard deviation of y_{lt}, the mean (m_R) and standard deviation (s_R) of R can be approximated by a Taylor series expansion as follows:

$$m_R \approx \frac{m_{y_{\text{lt}}}/B}{m_a + m_b\ (m_{y_{\text{lt}}}/B)} \tag{7}$$

$$s_R^2 \approx \left(\frac{\partial R}{\partial a}\right)^2 s_a^2 + \left(\frac{\partial R}{\partial b}\right)^2 s_b^2 + \left(\frac{\partial R}{\partial (y_{\text{lt}}/B)}\right)^2 s_{(y_{\text{lt}}/B)}^2$$

$$= \frac{m_{(y_{\text{lt}}/B)}^2\ s_a^2 + m_{(y_{\text{lt}}/B)}^4\ s_b^2 + m_a{}^2\ s_{(y_{\text{lt}}/B)}^2}{(m_a + m_b\ m_{(y_{\text{lt}}/B)})^4} \tag{8}$$

Table 5 summarizes statistics of R for drilled shafts supporting both buildings and bridges. The R statistics are calculated

Table 5 Statistics of $R = Q_{sls} / Q_{uls}$

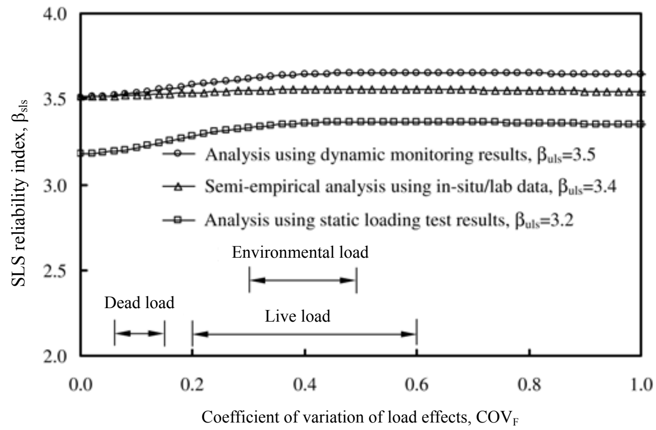
Type of Supporting structure		Mean, m_R	Standard deviation, s_R	Coefficient of variation, COV_R
Buildings	B = 0.18 m	1.27	0.24	0.19
	B = 1.30 m	1.20	0.22	0.19
	Average	**1.24**	**0.23**	**0.19**
Bridges	B = 0.18 m	1.27	0.24	0.19
	B = 1.30 m	1.22	0.23	0.19
	Average	**1.25**	**0.24**	**0.19**

for the values of shaft diameter B varying from 0.18 m to 1.30 m which are the respective minimum and maximum shaft diameters in the database compiled by Phoon *et al*. (1995) for the development of the hyperbolic model defined in the previous section. The R statistics vary slightly for different types of supporting structure and different B values. For drilled shafts supporting buildings, the value of m_R changes from 1.27 for B = 0.18 m to 1.20 for B = 1.30 m. The value of COV_R remains virtually constant at 0.19. For drilled shafts supporting bridges, the value of m_R changes from 1.27 to 1.22 with a COV_R value of 0.19, which equals to the COV_R for drilled shafts supporting buildings. The values of m_R and COV_R are insensitive to the variation of shaft diameters and types of supporting structures. Consequently, the average of m_R (*i.e.*, 1.24 and 1.25) and COV_R (*i.e.*, 0.19 and 0.19), as highlighted by the bold fonts in Table 5, are used in this work to estimate the values of β_{sls} for drilled shafts supporting buildings and bridges, respectively.

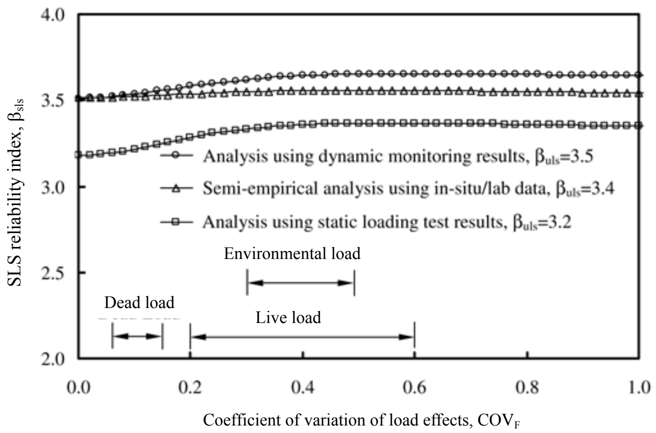
7. SLS RELIABILITY INDEX ESTIMATED FOR NBCC

Becker (1996) described the development of RBD methodologies for the NBCC and summarized the calibration process, corresponding ULS reliability index (β_{uls}), and proposed resistance factors (ψ), as shown in Table 2. The values of ψ are calibrated for three different design methods (*i.e.*, semi-empirical analysis using in situ and laboratory test data, analysis using static loading test results, and analysis using dynamic monitoring results) and their associated values of β_{uls} and COV_{Quls}. For example, the proposed resistance factor ψ = 0.6 for the axial compression capacity of a pile foundation, when interpreted from static loading tests. The corresponding β_{uls} = 3.2, and the coefficient of variation of Q_{uls} (COV_{Quls}) = 0.25.

Using Eqs. (2) ~ (4), the SLS reliability index (β_{sls}) can be estimated directly for NBCC. Figure 2 shows the estimated β_{sls} as a function of the coefficient of variation for load effects (COV_F) for drilled shafts under undrained compression and supporting either buildings (Fig. 2a) or bridges (Fig. 2a). As COV_F increases from 0 to 1.0, β_{sls} varies slightly and it is generally larger than 3,



(a) Supporting buildings



(b) Supporting bridges

Fig. 2 SLS reliability index β_{sls} estimated for NBCC

corresponding to an expected performance level of "above average" (see Table 3). As summarized in Table 1 and illustrated in Fig. 2, the COV_F of various load effects for foundations lies in the range of 0.05 to 0.6 (Meyerhof 1993, 1995).

For three different design methods (*i.e.*, semi-empirical analysis using in situ and laboratory test data, analysis using static loading test results, and analysis using dynamic monitoring results), the estimated β_{sls} is larger than their respective β_{uls} values of 3.4, 3.2, and 3.5 that are specified in the NBCC. This result indicates that, for the undrained compression capacity of drilled shafts designed in accordance with the NBCC, the designs automatically satisfy the SLS design requirements and have a β_{sls} larger than the β_{uls}. This result can be attributed to the probabilistic characterization of R, which shows that R is larger than 1 (see Table 5) and therefore the SLS capacity Q_{sls} is larger than Q_{uls}. Since Q_{sls} is larger than Q_{uls} and the probability of $Q_{uls} < F$ is $\Phi(-\beta_{uls})$, the $\Phi(-\beta_{sls})$, or the probability of Q_{sls} < F, is smaller than $\Phi(-\beta_{uls})$. Therefore, β_{sls} is larger than β_{uls}.

It should be pointed out that the values of β_{sls} reported in this paper are calculated using typical values of COV_{Quls}, COV_F, COV_R and m_R. They therefore are general estimates of the typical range of β_{sls}, which might not be necessarily valid for the β_{sls} value of a specific foundation design, particularly when the values of COV_{Quls}, COV_F, COV_R or m_R of the specific foundation design deviate significantly from their typical values used herein.

8. EFFECT OF LIMITING TOLERABLE SETTLEMENTS

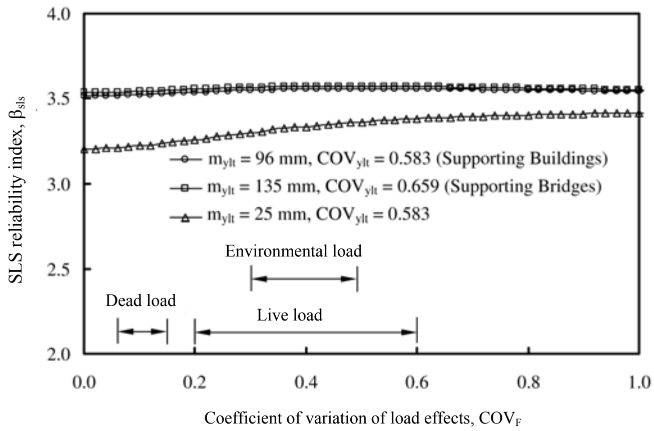
The y_{lt} statistics (*i.e.*, m_{ylt} = 96 mm and s_{ylt} = 56 mm for piles supporting buildings and m_{ylt} = 135 mm and s_{ylt} = 89 mm for piles supporting bridges) reported by Zhang and Ng (2005) and used herein are significantly larger than the displacement limit of 25 mm that is used frequently in deterministic SLS designs (*e.g.*, Peck *et al*. 1974; Wahls 1994). To explore the effect of y_{lt}, the values of β_{sls} are estimated using a m_{ylt} = 25 mm in combination with the COV_{ylt} = 0.583 reported by Zhang and Ng (2005) for piles supporting buildings. For m_{ylt} = 25 mm, $s_{ylt} = m_{ylt}$ (COV_{ylt}) = 25 (0.583) = 15 mm. Using Eqs. (7) and (8), the values of m_R and COV_R are taken as 1.124 and 0.210, respectively. Then, β_{sls} is estimated using these m_R and COV_R values.

Figure 3 shows the estimated β_{sls} as a function of the COV_F for m_{ylt} = 25 mm for three different design methods. As COV_F increases from 0 to 1.0, the values of β_{sls} increase gradually for three different design methods as shown in Figs. 3a, b, and c, respectively. For example, when the drilled shafts are designed by semi-empirical analysis using in-situ/lab data, the value of β_{sls} increases from 3.20 to 3.41 (see Fig. 3a). Although these values of β_{sls} are slightly smaller than β_{uls} = 3.4, they correspond to an expected performance level that is among "above average" (see Table 3). In addition, they are larger than the target β_{sls} = 2.6 that is given in the EPRI study (Phoon *et al*. 1995; Phoon *et al*. 2003a, 2003b). The values of β_{sls} similar to those for designs by semi-empirical analysis are shown in Figs. 3b and c for designs based on static or dynamic test results. Therefore, even with m_{ylt} = 25 mm and s_{ylt} = 15 mm, the NBCC designs of drilled shafts under undrained compression still are acceptable for the SLS design requirements.

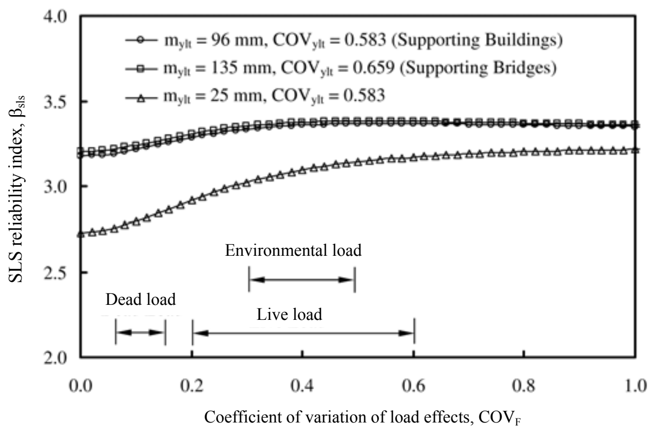
Figure 3 also includes the β_{sls} versus COV_F relationships for drilled shafts supporting buildings (m_{ylt} = 96 mm) and bridges (m_{ylt} = 135 mm) from Fig. 2. The β_{sls} versus COV_F relationships for m_{ylt} = 96 mm and m_{ylt} = 135 mm are virtually identical. The hyperbolic load-displacement model defined by Eq. (6) shows that, when y_{lt}/B is relatively large, the value of R approaches the asymptotic maximum of $1/b$ = 1.28. For m_{ylt} = 96 mm and m_{ylt} = 135 mm, the respective means of R value are estimated as 1.24 and 1.25, and they approach the maximum value of 1.28. Therefore, even the m_{ylt} = 135 mm for piles supporting bridges is significantly larger than the m_{ylt} = 96 mm for piles supporting buildings, only inconsiderable difference (*i.e.*, 1.25 − 1.24 = 0.01) is observed for the values of m_R. On the other hand, when m_{ylt} decreases from 96 mm to 25 mm, β_{sls} decreases significantly, particularly for relatively small COV_F. The value of β_{sls} may decrease to 2.7. This figure shows clearly that the limiting tolerable foundation settlement (y_{lt}) must be defined rationally and thoughtfully.

9. SUMMARY AND CONCLUSIONS

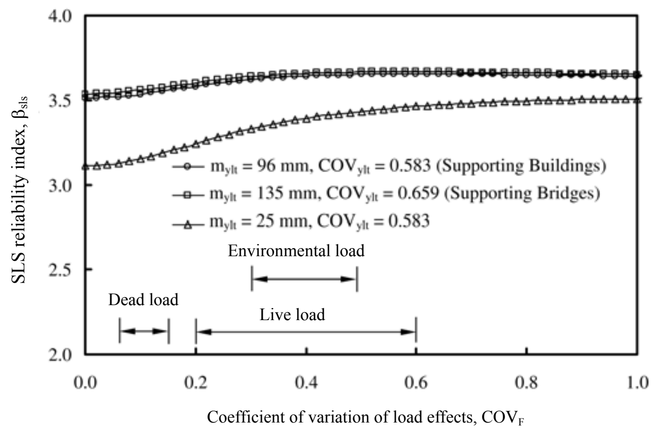
This paper dealt with the reliability index of serviceability limit state of drilled shafts under undrained compression. It made use of a relationship between β_{sls} and β_{uls} to infer the β_{sls} of drilled shafts under undrained compression from the β_{uls} that is specified already in the design codes. The probabilistic distribution of y_{lt} and the uncertainties associated with the hyperbolic load-displacement model of drilled shafts were accounted for explicitly in this paper. Key variables in this relationship were explored, including the ratio (R) of the SLS capacity (Q_{sls}) to the ULS capacity (Q_{uls}), coefficient of variation of the ULS capacity (COV_{Quls}), and coefficient of variation of the load effect F (COV_F).



(a) Semi-empirical analysis using in-situ/lab data (β_{uls} = 3.4)



(b) Analysis using static loading test results (β_{uls} = 3.2)



(c) Analysis using dynamic monitoring results (β_{uls} = 3.5)

Fig. 3 Estimated SLS Reliability Index β_{sls} for Varying m_{ylt} and COV_{ylt}

The values of β_{sls} were estimated for drilled shafts designed in accordance with the NBCC. For assumed input variables consistent with recent data summaries for piles supporting buildings

(m_{ylt} = 96 mm and COV_{ylt} = 0.583) or bridges (m_{ylt} = 135 mm and COV_{ylt} = 0.659), it was found that, for COV_F ranging from 0 to 1.0, β_{sls} varies slightly and it is generally larger than 3, corresponding to an expected performance level of "above average". For three different design methods (*i.e.*, semi-empirical analysis using in situ and laboratory test data, analysis using static loading test results, and analysis using dynamic monitoring results) in NBCC, the estimated β_{sls} is larger than their respective β_{uls} values of 3.4, 3.2, and 3.5 that are specified in the NBCC. This result indicates that, for the undrained compression capacity of drilled shafts designed in accordance with the NBCC, the designs automatically satisfy the SLS design requirements and have a β_{sls} larger than the β_{uls}. This result can be attributed to the input variables and the probabilistic characterization of R, which shows that R is larger than 1, and therefore Q_{sls} is larger than Q_{uls} and β_{sls} is larger than β_{uls}.

The effect of y_{lt} on the estimated β_{sls} also was examined. For m_{ylt} = 25 mm, β_{sls} increases gradually as COV_F increases. Although these values of β_{sls} are slightly smaller than their respective β_{uls} values, they are larger than the target β_{sls} = 2.6 that is given in the EPRI study. Therefore, even with m_{ylt} = 25 mm and COV_{ylt} = 0.583, the NBCC designs of drilled shafts under undrained compression still are acceptable for the SLS design requirements. It was also found that, the β_{sls} versus COV_F relationships for m_{ylt} = 96 mm and m_{ylt} = 135 mm are virtually identical because the values of R for these relatively large y_{lt}/B values already approach the asymptotic maximum of $1/b$ = 1.28 of the hyperbolic load-displacement model. On the other hand, when m_{ylt} decreases from 96 mm to 25 mm, β_{sls} decreases significantly, particularly for relatively small COV_F. The value of β_{sls} may decrease to 2.7. Clearly, the limiting tolerable foundation settlement must be defined rationally and thoughtfully.

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