Review of Reliability Levels Achieved by Geotechnical Design Codes

Gennaro Esposito
Shell, Engineering, 400 – 4th Avenue SW, Calgary, AB T2P 2H5, Canada
Gordon A. Fenton
Engineering Mathematics Department, Dalhousie University, Halifax, NS B3J 2X4, Canada;

ABSTRACT
This paper compares the overall true reliability levels achieved by a variety of design codes from around the world for the Ultimate Limit State of geotechnical systems. We compare the Canadian CHBDC code with the European EN 1997, the American AASHTO, and the Australian AS 4678 and AS 2159. All the codes in this study are reliability-based design codes and their use should produce designs characterized by very similar reliability levels. We use deep foundations and shallow foundations and consider the bias of the design methodology and the variability of true and predicted geotechnical resistance. In general, North-American codes target a lower reliability target than EN 1997 and Australian codes. The study revealed that, despite differences in format, choice of the characteristic values and material factors, all the codes achieve an acceptable overall reliability level for the geotechnical systems considered, with the exception of the EN 1997 for the design of deep foundations. We postulate that the observed consistency is due to implicit inclusion of many sources of uncertainty in the code calibration. Finally, the North-American codes are found to be the most robust due to the higher likelihood of achieving their target reliability level. The European and Australian codes are less robust, as they have a lower likelihood to meet a higher target reliability level.

1 INTRODUCTION

Traditionally, geotechnical engineers consider the effect of various sources of uncertainty by specifying the ratio of resistance to load - the factor of safety – to be larger than unity and its value is usually chosen by experience based on understanding of the potential risk and uncertainties about both load and resistance. The major drawback of this approach, called Working Stress Design (WSD), is that it does not properly account for the variability of the resistance and load, nor risk. Thus, the factor of safety cannot accurately reflect the probability of failure (unsatisfactory performance) of the geotechnical system (Fenton et al., 2015).

Modern design codes, such as the Eurocode (EN 1997 - CEN, 2004) in Europe, the AASHTO bridge design specifications (AASHTO, 2012) in the United States, the Australian codes AS 4678 (Standard Australia, 2002) and AS 2159 (Standard Australia, 2009), and the Canadian Highway Bridge Design Code (CHBDC – CSA, 2012), have adopted the reliability-based Limit State Design (LSD) approach where the variability of resistance and load is explicitly considered to achieve a prescribed target reliability level. It reasonable to expect that design of geotechnical systems executed with any of the considered design codes would achieve similar levels of reliability given that safety acceptance is based on factors widely shared by people in regions having similar standards, such as personal and community safety, potential losses, and amount of investments necessary to improve safety. The objective of this paper is to determine if the various codes actually achieve similar reliability targets by comparing a) the overall reliability levels targeted by the geotechnical codes and b) the overall reliability levels achieved by these codes. In this initial study, the comparison is restricted to the Ultimate Limit State (ULS) design of deep and shallow foundations.

To achieve this objective, we start with a short review of the different probabilistic formats adopted by the various geotechnical design codes. A review of the target reliability levels aimed at by each code is followed by a discussion of the difference between the true geotechnical resistance and the geotechnical resistance predicted through the design process. The true geotechnical resistance and the statistics of the load actions are then used to calculate the overall reliability level achieved by the various codes. Finally, the similarities and differences between the codes are discussed.

2 LIMIT STATE DESIGN FORMATS

In LSD, the limit state function, $Z(\boldsymbol{X})$ is a vector of basic variables $\boldsymbol{X} = X_1, X_2, \dots, X_n$, generally random, affecting the performance of a geotechnical system. Considering only two variables, resistance R and load S, the safe domain of the limit-state function $Z(\boldsymbol{X})$ can be written as:

$$Z(\boldsymbol{X}) = Z = R - S > 0 \qquad [1]$$

Given Z, the probability of failure of a geotechnical system is given by (Ang and Tang, 1984):

$$P_F = P[Z < 0] = P(R < S) = \int_0^{\infty} F_R(s) f_S(s) ds \qquad [2]$$

where S and R are independent random variables. At any given $S = s$, the area under R below s (Figure 1), where $r < s$, is $F_R(s)$ and indicates an unacceptable performance of the geotechnical system. This is the conditional failure probability given s.

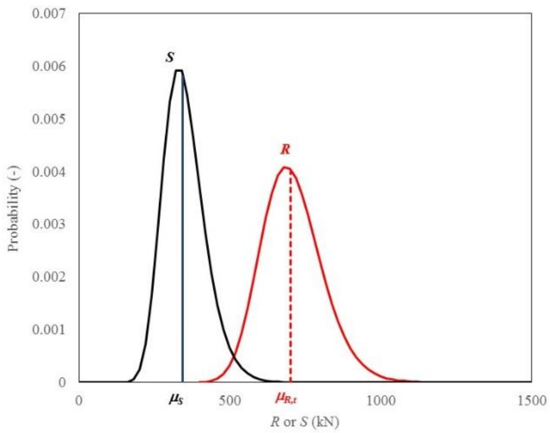

Figure 1 – Distribution of load S and resistance R

Instead of the probability of failure P_f, the reliability index is often used in codes as an equivalent quantity to P_f. The reliability index β is related to the probability of failure P_f as

$$P_f = \Phi(-\beta) \quad [3]$$

where Φ denotes the standardized normal cumulative distribution function. For the limit-state function $Z(\boldsymbol{X})$, the reliability index, β, is the distance, in units of standard deviation, from the origin of the space of the basic random variables to the failure surface at the most probable point on that surface (Figure 2). Note that when the vector of basic variables $\boldsymbol{X}$ is becomes independent from year to year, then the relationship between the lifetime reliability index β and the annual β_{ann}, can be written as (Fenton et al., 2015):

$$\beta = -\Phi^{-1}[1 - \Phi^T(\beta_{ann})] \quad [4]$$

where T is the design life of the geotechnical system in years. In this study, it is assumed that the basic variables in $\boldsymbol{X}$ are independent from year to year.

North American, Australian and European codes use different limit state design formats (Becker, 1996a). In North America, limit state design of geotechnical systems is based on the Load and Resistance Factor Design (LRFD) approach. In this approach, also called *factored resistance* approach (Becker, 1996a), an overall resistance factor is applied to the geotechnical resistance for each limit state. For instance, CHBDC (CSA, 2014) states that the characteristic value of the ultimate resistance (R_k) multiplied by a resistance factor (φ) and by an importance factor (Ψ), which thus becomes the factored resistance (R_d), must be greater than or equal to the summation of the characteristic values of load effects ($S_{k,i}$) multiplied by a structure importance factor (I_i), the corresponding load factors (α_i), and a modifier (η_i). For strength limit state corresponding to ULS, the following applies (Fenton et al., 2014):

$$R_d = \Psi\varphi R_k \geq \sum I_i\,\eta_i\,\alpha_i S_{k,i} \quad [5]$$

AASHTO (2012) and AS 2159 (Standard Australia, 2009) adopt a similar format. The European EN 1997 (CEN, 2004) allows both the factored resistance approach similar to Eq. 5 and a *factored strength* approach (Becker, 1996a), where partial material factors are applied either to the geotechnical strength parameters as:

$$R_d = \Psi R\{\varphi_X X_k\} \geq \sum I_i\,\eta_i\,\alpha_i S_{k,i} \quad [6]$$

or to the geotechnical strength and geotechnical resistance simultaneously as

$$R_d = \Psi\varphi R\{\varphi_X X_k\} \geq \sum I_i\,\eta_i\,\alpha_i S_{k,i} \quad [7]$$

where X_k is the geotechnical strength and φ_X is the partial material factor of the geotechnical strength. In EN 1997 (CEN, 2004), geotechnical design according to Eq. 6 is denoted as Design Approach 1 (DA1 - Bond et al., 2013). Design according to Eq. 5 (similar to the North American factored resistance) is denoted DA2. Finally, design according to Eq. 7 is denoted to DA3.

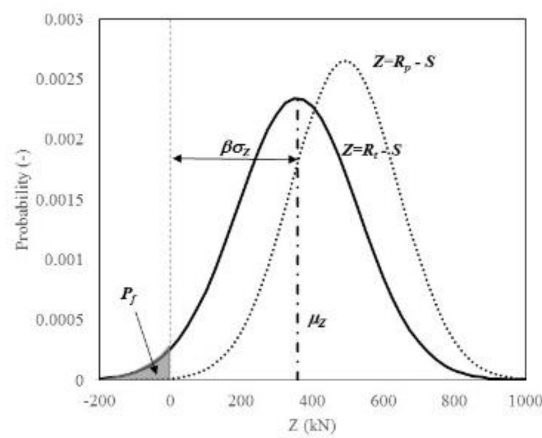

Figure 2 – Distribution of the limit-state function $Z(\boldsymbol{X})$

Note that in EN 1997 the resistance factors are applied inversely to the North American approach, i.e., by dividing rather than multiplying. In this study, we apply the European material factors in a multiplicative fashion. Also, EN 1997 (CEN, 2004) allows participating countries to adjust the material factors recommended in the code by publishing a National Annex (NA). In this study, we consider the British UK-NA which adopts DA1, and the Irish IR-NA which adopts DA 2. AS 4678 (Standard Australia, 2002) also adopts the factored strength approach.

3 TARGET RELIABILITY LEVEL IN DESIGN CODES

Target reliability indexes for geotechnical systems are generally selected to ensure consistency with the target reliability of the structure they are part of or support. This approach is not always simple due to the different level of uncertainty between structure and geotechnical system and the difficulty of controlling the construction quality of a geotechnical system.

In Canada, Fenton et al. (2015) considered the minimum annual reliability index of 3.75 prescribed for traffic load on bridges in CHBDC (CSA, 2015) as the starting point for the reliability target of geotechnical systems. Considering the possible decrease in reliability with time and the design life, Fenton et al. (2015) concluded that a lifetime reliability index between 3.0 and 3.5 for 75 years would conservatively correspond to an annual reliability index of 3.75.

The reliability level used to calibrate structural design in the AASHTO Bridge Design Specifications (AASHTO, 2012) was similar to the one used for CHBDC (CSA, 2015). For the Strength Limit State, equivalent to the ULS in CHBDC (CSA, 2015), AASHTO adopted a lifetime target reliability index of 3.5 during the 75-year design life of the bridge for calibration. It is interesting to note that, if each year is considered to be independent, with 75-year design life, this would correspond to an annual target reliability index of 4.6, well above the annual reliability index of 3.75 adopted in CHBD (CSA, 2014). The calibration of geotechnical systems was conducted by the Transportation Research Board (TRB) between 2002 and 2010 to update the LRFD version of the AASHTO Bridge Design Specifications (AASHTO, 2012). Two different calibration studies (TRB, 2004; TRB, 2010) introduced the concept of redundancy for deep foundations and adopted a lifetime target reliability between 2.0 and 2.5 for redundant deep foundations and between 3.0 and 3.5 for non-redundant foundation elements.

The basis of structural design Eurocode EN1990 (CEN, 2002) prescribes target reliability levels between 3.3 and 4.7 for a 50-year reference period, depending on the consequences of failure. For the class of structures in which failure would have considerable impact, the standard recommends a lifetime target reliability level of 3.8. This value does not necessarily represent the actual failure probability level, but gives a good comparison of reliability levels between structural and geotechnical systems and is therefore adopted as target for geotechnical systems (De Koker and Day, 2017).

In Australia, the target reliability level for structural design is given in the Building Code of Australia (BCA - ABCB, 2015), which prescribes an annual reliability level of 3.8 for buildings of any importance categories. For bridge design, the Australian Standard General Principles on Reliability for Structures AS 5104 (AS, 2017, adopted from ISO 2394-1998) recommends a lifetime target reliability level of 3.8 for ultimate strength limit states (ULS) design where consequences of failure are moderate and relative costs of safety measures are low. When considering the different design lives (50 years for BCA and 75 years for AS 5104), it appears that the annual reliability level of AS 5104 (2017) is higher than the one mandated by BCA (2015).

The target reliability levels from the codes considered in this paper are summarized in table 1. Comparing the columns of the annual reliability levels, the desire to match the target reliability levels of structures and geotechnical systems is evident, except for CHBDC (CSA, 2015) and BCA (2015), which prescribe target reliability levels for structural design lower that the other codes. However, the mismatch for these two codes is compensated when selecting a more conservative target reliability level for geotechnical systems (Fenton et al., 2015). It is interesting to observe that the annual target reliability level adopted in North-American practice is generally lower than the one adopted in European and Australian practice, apparently contradicting the expectation that the performance expectation concerning geotechnical systems should be the same.

Table 1 – Target reliability levels of the geotechnical codes considered in this study

		Structure		Geotechnical System	
Code	Design Life (yr)	Life Time β	Annual β	Life time β	Annual β
CHBDC	75	2.5*	3.75	3.0 - 3.5	4.1 - 4.5*
AASHTO	75	3.5	4.5*	3.0 - 3.5**	4.1 - 4.5*
EC7	50	3.8	4.7*	3.8	4.7*
BCA	50	2.7*	3.8	3.8	4.7*
AS 5104	75	3.8	4.8*	3.8	4.8*

*Derived using Eq. 4 considering that each year of the design life is independent

**Target reliability level for non-redundant foundations

4 METHODOLOGY TO ASSESS RELIABILITY LEVEL ACHIEVED BY GEOTECHNICAL DESIGN CODES

The geotechnical design process produces an estimate of the “true” resistance R_t of the geotechnical systems, which is the predicted resistance, R_p (Figure 3). True and predicted geotechnical resistances are related through the bias factor of the resistance λ_R as:

$$\lambda_R = R_t/R_p \quad [8]$$

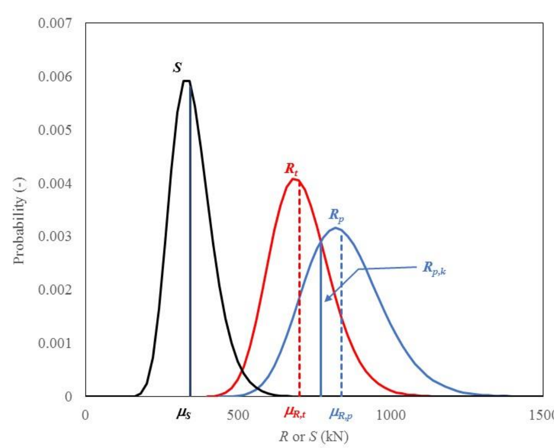

Figure 3 - Relationship between true geotechnical resistance and predicted geotechnical resistance

The variability of the true geotechnical resistance R_t corresponds to the inherent geological variability of the soil. The variability of the predicted resistance, R_p, includes the net effect of other sources of error such as measurement errors and model uncertainties in addition to the inherent geological variability. The bias factor of the resistance λ_R therefore represents the total error of the design process and is characterized by its mean, μ_{λ_R}, and its Coefficient of

Variation (COV), COV_{λ_R}. Assuming that the sources of uncertainty are statistically independent, a first order approximation (Ang and Tang, 1974) of the mean of the overall bias factor may be written as the product of the mean of individual bias factors:

$$\mu_{\lambda_R} = \mu_{\lambda_{R,inh}} \cdot \mu_{\lambda_{R,mea}} \cdot \mu_{\lambda_{R,mod}} \quad [9]$$

and the COV of the overall bias factor is the square root of the sum of the squares of the individual coefficients of variation:

$$COV_{\lambda_R} = \sqrt{COV^2_{\lambda_{R,inh}} + COV^2_{\lambda_{R,mea}} + COV^2_{\lambda_{R,mod}}} \quad [10]$$

where the subscripts inh, mea, and mod indicate inherent, measurement, and model uncertainty respectively. In the ideal case of a perfect design process, the only source of uncertainty is the inherent geological variability and $R_p = R_t$. The other extreme corresponds to the case where all the sources of uncertainty affect the estimate R_p. In this case, $\mu_{\lambda_R}$and COV_{λ_R} produce two distinct effects. The effect of μ_{λ_R} is to scale up or down the mean of the predicted geotechnical resistance. If μ_{λ_R} is less than unity, the design methodology produces an unconservative overestimate of R_t; conversely, if μ_{λ_R} is larger than unity, the design methodology conservatively underestimates R_t. The effect of COV_{λ_R} is to increase the variability of the predicted geotechnical resistance impacting the characteristic value of the geotechnical resistance. Geotechnical codes prescribe (or imply) a conservative characteristic value of the predicted geotechnical resistance $R_{p,k}$, usually taken as a lower (Figure 3) or higher (whichever yields the most conservative result) value than the mean of R_p, $\mu_{R,p}$, expressed as:

$$R_{p,k} = \mu_{R,p}/k_R \quad [11]$$

Where k_R is the bias factor of the characteristic value (Fenton et al., 2014) which scales up or down $\mu_{R,p}$ to obtain a certain fractile of R_p. The resulting fractile depends on the variability of R_p which, in turn, depends on COV_{λ_R}. Figure 3 shows an unconservative prediction of the geotechnical resistance (λ_R is less than unity) with COV_{λ_R} larger than $COV_{\lambda_{R,inh}}$. Note that in Figure 3 the predicted geotechnical resistance $R_{p,k}$ is larger than the mean of the true resistance $\mu_{R,t}$. The statistics of λ_R for many popular design methodologies are available in TBR (2004) for deep foundations and in TBR (2010) for shallow foundations, these values include all the sources of uncertainty present in the design process. The inherent geological uncertainty is described by Phoon et al. (1995) for many geotechnical parameters. The characteristic value of the predicted loads $S_{p,i,k}$ is derived in a similar manner from their predicted mean $\mu_{S,p,i}$, using the bias factor of the loads $k_{S,i}$. Fenton et al. (2015) summarized typical bias factors of the loads for many geotechnical codes.

To assess the reliability levels achieved by the codes, we assume that the predicted geotechnical resistance R_p is available for a certain site, through soil investigation and geotechnical analysis. We continue calculating the characteristic values of the predicted resistance $R_{p,k}$ using the code recommended (or implied) bias factor k_R. We also assume that the predicted loads $S_{p,i,k}$ are available and calculate the characteristic value of the loads using the bias factors $k_{S,i}$. We next apply code-prescribed material factors to conclude the design process and derive the typical dimension of the foundation under consideration. The calculated foundation dimensions are then used as deterministic variables to determine the reliability level of the design process. For this task, we derive R_t from R_p and $S_{t,i}$from $S_{p,i}$ using Equation (10), and solve the following limit state equation to find the reliability index β:

$$P_F = P\left[R_t - \textstyle\sum_i S_{t,i} > 0\right] \quad [12]$$

Where loads S_i and the true resistance R_t are expressed in terms of their statistical distribution and no material factor is applied. The reliability index β is calculated from P_F for different values of the geotechnical resistance variability, ranging from a minimum corresponding to the inherent geological variability, to a maximum where all the sources of variability impact the design process. Finally, we do not consider the effect of any bias affecting the loads and use the bias factors of the load characteristic values from Fenton et al. (2014) to derive the distributions F_i.

5 RELIABILITY OF DEEP FOUNDATIONS

Drilled shafts subjected to axial dead and variable loads and designed with static analysis are considered to determine the actual reliability level of deep foundations achieved by geotechnical codes. The case considered in this section is illustrated in Figure 4. It is assumed that the piles are not redundant, and that the geotechnical and structural importance factors Ψ and I_i are equal to one. It is also assumed that the shear stress along the shaft q_s and the bearing capacity of the pile toe q_b are estimated from the soil investigation, increase with the depth, and have mean values of 10 kPa/m and 150 kPa/m respectively. The mean predicted geotechnical resistance μ_{R_p} is obtained as (CFEM, 2006):

$$\mu_{R_p} = \textstyle\sum_z^L C\, \mu_{q_{s,p}} \Delta z + A_b \mu_{q_{b,p}} - W_P \quad [13]$$

where z is the depth, L is the pile length, C is the pile circumference, W_P is the pile weight, $\mu_{q_{s,p}}$is the mean predicted shaft friction, $\mu_{q_{b,p}}$ is the mean predicted base resistance, and A_b is the pile toe area. For each code, the characteristic value of the geotechnical resistance $R_{p,k}$ is obtained from Eq. 11 using the k_R factors in Fenton et al. (2015). Similarly, the characteristics values of the loads are obtained from the mean values using the k_D and k_L factors in Fenton et al. (2015). The last step to complete the code design process is to obtain the design values of loads and resistance using the material factors (α_i for loads and φ for the resistance, in Fenton et al., 2015) and verify the limit state, Eqs. 5 - 7, depending on the code. Note that for EN

1997 DA1-C2 the total material factor of the resistance is used. The design pile lengths are shown in Table 2 together with μ_{R_p}, the mean of the predicted loads $\mu_{S_{p,tot}}$, and the factored predicted load $S_{p,tot}$. EN 1997 DA1-C2 produces the shortest pile while AASHTO and AS 2159 produce the longest. This due to the combination of the small geotechnical resistance factors and large load factors.

Table 2 – Results for deep foundations

Code	L (m)	$\mu_{S_{p,tot}}$ (MN)	$S_{p,tot}$ (MN)	μ_{R_p} (MN)
CHBDC	12.8	0.8	1.06	2.95
AASHTO	13.7	0.8	1.08	3.05
EN-DA1	10.1	0.8	0.94	1.90
EN-DA1- UKNA	11.8	0.8	0.94	2.54
EN-DA2-IRNA	13.1	0.8	1.20	3.08
AS 2159	13.7	0.8	1.20	3.35

To determine the reliability of the design, we obtain the mean of the true geotechnical resistance $\mu_{R,t}$ using Eq. 13 and calculate the reliability index β for a range of COV_{R_t}.The mean and the COV of the bias factor λ_R of several static analysis methodologies are given in TBR (2004) for frictional and cohesive soils. Inspection of TBR (2004) reveals that most of the available methodologies have the mean of the bias factor μ_{λ_R} ranging between 0.8 and 1.7, and the coefficient of variation of the bias factor COV_{λ_R} ranging between 0.3 and 0.7. For this example, we use an extended range of COV_{R_t}, from 0.05, representing the case where only the inherent variability is affecting the geotechnical resistance, to 0.7, representing the case where other sources of uncertainty affects the geotechnical resistance. We also consider μ_{λ_R} ranging between 0.8 and 1.2. The variability of the load, COV_{S_D} and $COV_{S_L}$are taken as 0.1 and 0.27 respectively (Bartlet et al., 2004, Fenton et al., 2015). The First Order Reliability Method (FORM, Low and Phoon, 2002) is used to solve Eq. 12. The results are shown in Figure 8, where the range of reliability indexes β is plotted against COV_{R_t}.

We define the reliability of the code as the reliability index achieved for a certain value of the resistance variability COV_{R_t}, and the robustness of the code as the ability to meet the target reliability level at the largest resistance COV_{R_t} for the lowest bias factor of the resistance λ_R. Figure 8 shows that for the same COV_{R_t}, the reliability index β depends on μ_{λ_R}, the larger μ_{λ_R} (underprediction of the true resistance), the larger the reliability index β.

AASHTO (Figure 8b) and AS 2159 (Figure 8f) followed by CHBDC (Figure 8a) are the codes that produce the most robust design. When considering their respective reliability target, AASHTO and CHBDC are the geotechnical codes with the higher chances of meeting the target. With μ_{λ_R} as low as 0.8, AASHTO meets the reliability target until COV_{R_t} exceeds 0.25, whereas CHBDC meets the reliability target until COV_{R_t} exceeds 0.20. With a μ_{λ_R} of 0.9, CHBDC meets the reliability target until COV_{R_t} exceeds 0.25, confirming the robustness of the code that was calibrated using a resistance variability of 0.15. Figure 8c reveals that EN 1997 DA1 never achieves the target reliability level irrespective of the design approach μ_{λ_R}. When using EN 1997 DA1-UK-NA (Figure 8d), if COV_{R_t} exceeds 0.2, design cannot achieve the target reliability level, irrespective of μ_{λ_R}. The results of this study confirm that the more conservative material resistance factors introduced in several national annexes are indeed needed to raise the reliability of the geotechnical design executed with EN 1997. See, for instance, Figure 8e, the Irish National Annex, which adopts DA2.

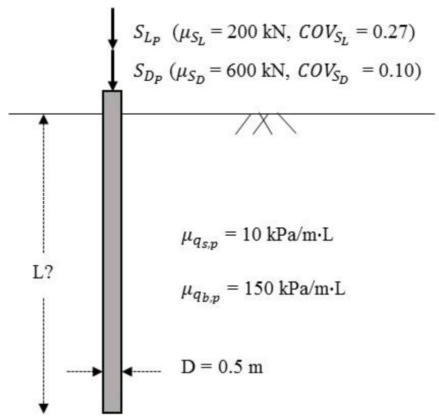

Figure 4 – Geometry of drilled shaft used to assess the reliability level of deep foundations

6 RELIABILITY OF SHALLOW FOUNDATIONS

To assess the actual level of reliability of shallow foundation, we use the same approach adopted for deep foundation. Also, we use a similar case as the one in Fenton et al. (2014), where a footing on weightless soil (with no embedment, nor surcharge) having mean predicted cohesion and friction angle of $\mu_{c,p}$ = 110 kPa and $\mu_{\phi,p}$ = 33° respectively has to be designed to resists mean predicted dead and live loads of $\mu_{S_{D,p}}$ = 4,500 kN and $\mu_{S_{L,p}}$ = 1,500 kN, respectively (Figure 5). The geotechnical and structural importance factors Ψ and I_i are set equal to one. The mean predicted geotechnical resistance is therefore (Fenton et al., 2014):

$$\mu_{R_p} = A\mu_{c,p}\mu_{N_{c,p}} \quad [14]$$

Where A is the area of the footing and $\mu_{N_{c,p}}$ is the mean of the predicted bearing capacity factor, which depends on the mean predicted friction angle $\mu_{\phi,p}$. Both the code design and the reliability analysis follow the same procedure used to assess the reliability of deep foundation. For the shallow foundation designed with EN 1997 (CEN, 2004), we consider the material factors given in Appendix D of the code for DA1 and DA2, as neither UK-NA nor IR-NA prescribe different material factors. Also, for the shallow foundation designed with EN 1997 DA1 (CEN, 2004) and with AS 4678(SA, 2004), the material factors are applied to the soil properties (factored strength approach), thus two factors are used, one for the cohesion and one for the friction angle. The calculated footing areas are in Table 3, which reveals that the smallest footings are designed using

CHBDC (CSA, 2014) and AASHTO Bridge Design Specifications (AASHTO, 2012).

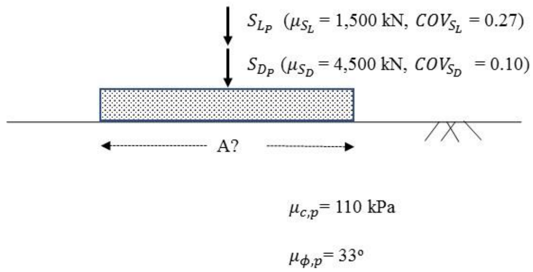

Figure 5 - Geometry of footing used to assess the reliability level of shallow foundations

The results of the reliability analysis are shown in Figure 9, where the reliability index β is plotted against COV_{R_t}, which increases from 0.05 to 0.7. For the same COV_{R_t}, AS 4678 (SA, 2004) produces the most reliable (largest β) and most robust (able to meet the target with the largest COV_{R_t}) design as the target reliability is met when λ_R = 0.8 and COV_{R_t} is 0.3. For the same COV_{R_t}, CHBDC (CSA, 2014) and AASHTO Bridge Design Specifications (AASHTO, 2012) produce a robust design when λ_R = 0.8 and COV_{R_t} is 0.25, however the reliability level of these two codes is the lowest. EN 1997 - DA1 (CEN, 2004) produces the least robust design, only meeting the target reliability level when COV_{R_t} is less than 0.2. As was found for the reliability of deep foundations, the actual reliability level achieved by each code depends on how the characteristic values of loads and resistance are defined and on the material factors mandated by the codes.

Table 3 – Material Factors, bias factors, and footings area

Code	A (m^2)	$\mu_{S_{p,tot}}$ (kN)	$S_{p,tot}$ (kN)	μ_{R_p} (kN)
CHBDC	5.3	6,000	7,976	22,526
AASHTO	5.4	6,000	8,120	22,951
EN 1997 - DA1	6.4	6,000	7,071	27,201
EN 1997 - DA2	7.5	6,000	9,000	31,877
AS 4678	8.0	6,000	8,571	34,214

7 DISCUSSION

Except for EN 1997 (CEN, 2004) when applied to deep foundations, all the codes considered in this study meet their reliability target when COV_{R_t} is reasonably small. The maximum value of the resistance variability to achieve the target reliability level depends on the code and on the geotechnical system. The robustness also changes between codes and geotechnical system considered. Note that these results are based on the assumption that load~~s~~ variability and bias are as prescribed (or implied) in the codes.

The North-American codes are robust for both deep and shallow foundations as they meet the target reliability level provided that the resistance variability COV_{R_t} is within the limits used for calibration. The Australian codes are robust for shallow foundations and less robust for deep foundations, requiring COV_{R_t} to be less than 0.15. EN 1997 (CEN, 2004) is not robust for deep foundation but is robust for shallow foundations. For deep foundations designed with EN 1997 (CEN, 2004), the findings of this study agree with those from other authors (Wang et al., 2011; De Koker and Day, 2017; Orr et al., 2005).

In this study we did not vary k_R and used the values mandated by (or implied from) the codes. Selection of a different characteristic value would produce different results as stressed by many authors (Forrest and Orr, 2010, and De Koker and Day, 2017). For instance, review of the reliability levels achieved by of EN 1997 (CEN, 2004) for shallow foundation (Hara et al., 2011, Forrest and Orr, 2010) and De Koker and Day, 2017) is summarized in Figure 6 showing the effect of the characteristic value choice on the achieved reliability index. Figure 6 shows that use of higher fractile for the characteristic value than that prescribed by EN 1997 (CEN, 2004) would decrease the likelihood of achieving the target reliability index.

Despite the exception of EN 1997 DA1 (CEN, 2004) for deep foundations, the reliability levels achieved by the codes are close to the range that many authors consider as the minimum to achieve a societal acceptable level of structural safety (Ellingwood et al., 1980; Madsen et al.,1986; Bartlett et al., 2004; Fenton et. al., 2014), which corresponds to an overall β ranging between 3 and 3.5. Figure 7 shows the true reliability levels, for λ_R = 1 and COV_{R_t} = 0.2, of the codes considered in this study. This seems to confirm that, irrespective of the code target, the overall reliability level β between 3 and 3.5 is an acceptable target for well-designed geotechnical system.

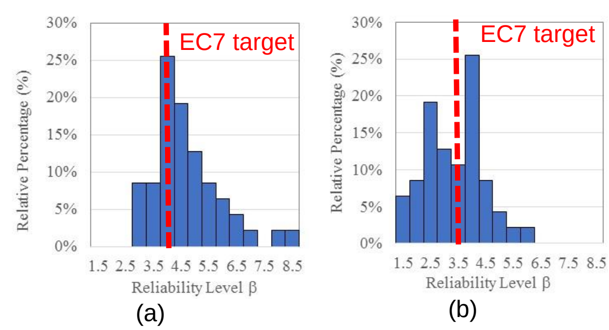

Figure 6 – Actual reliability levels achieved for shallow foundations using EN1997 (CEN, 2004); (a) characteristic value is 5% fractile, (b) characteristic value of the resistance is cautious estimate of the mean.

8 CONCLUSIONS

In this paper, we use a methodology to assess the true reliability levels achieved by international geotechnical codes based on the difference between true and predicted reliability level and compare the former with the codes target. The true reliability level depends on many factors. We considered the effect of the geotechnical resistance variability summarized by the coefficient of variation COV_{R_t}, and of the total bias of the design methodology expressed as the λ_R ratio. With exception of EN 1997 DA1, the geotechnical codes considered herein achieve reliability levels above 3 for maximum values of COV_{R_t} between 0.15 and 0.3. To use the terminology adopted in CHBDC (CSA, 2014), this COV_{R_t} value is consistent with an understanding of the geotechnical system ranging from typical to poor (Fenton et al., 2014), thus confirming the robustness of many codes. Reliability levels above 3 can be achieved using design methodologies having a total bias λ_R between 0.8 and 1.2, which is in the typical range of many of the most popular design methodologies. This study confirms that a minimum reliability level between 3 and 3.5 is what several authors consider an adequate level of safety for many geotechnical systems. Finally, this study stresses the need of having a profound understanding of the basis and the limitations of each geotechnical code to achieve the desired reliability level.

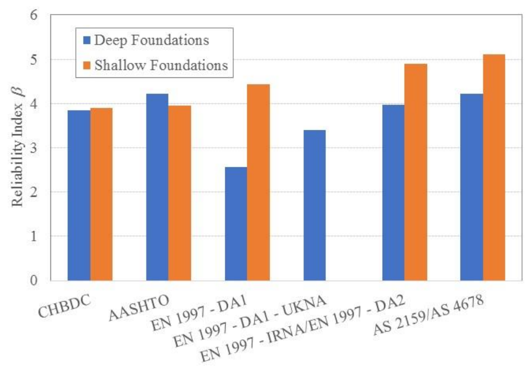

Figure 7 – comparison of the true reliability levels achieved by the geotechnical codes for λ_R = 1 and COV_{R_t} = 0.2

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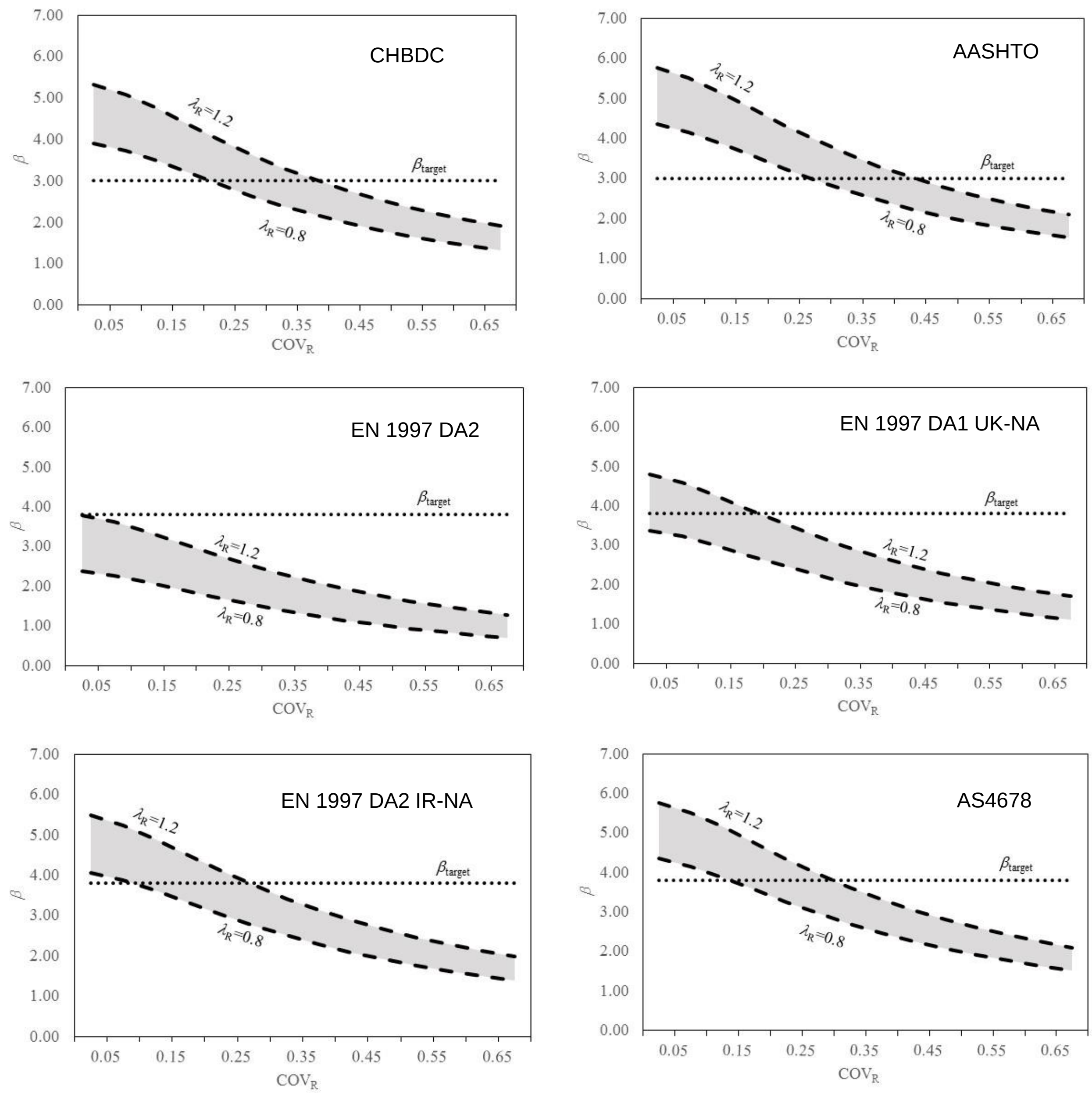

Figure 8 – Reliability level of deep foundations achieved by the codes versus the total variability of the resistance

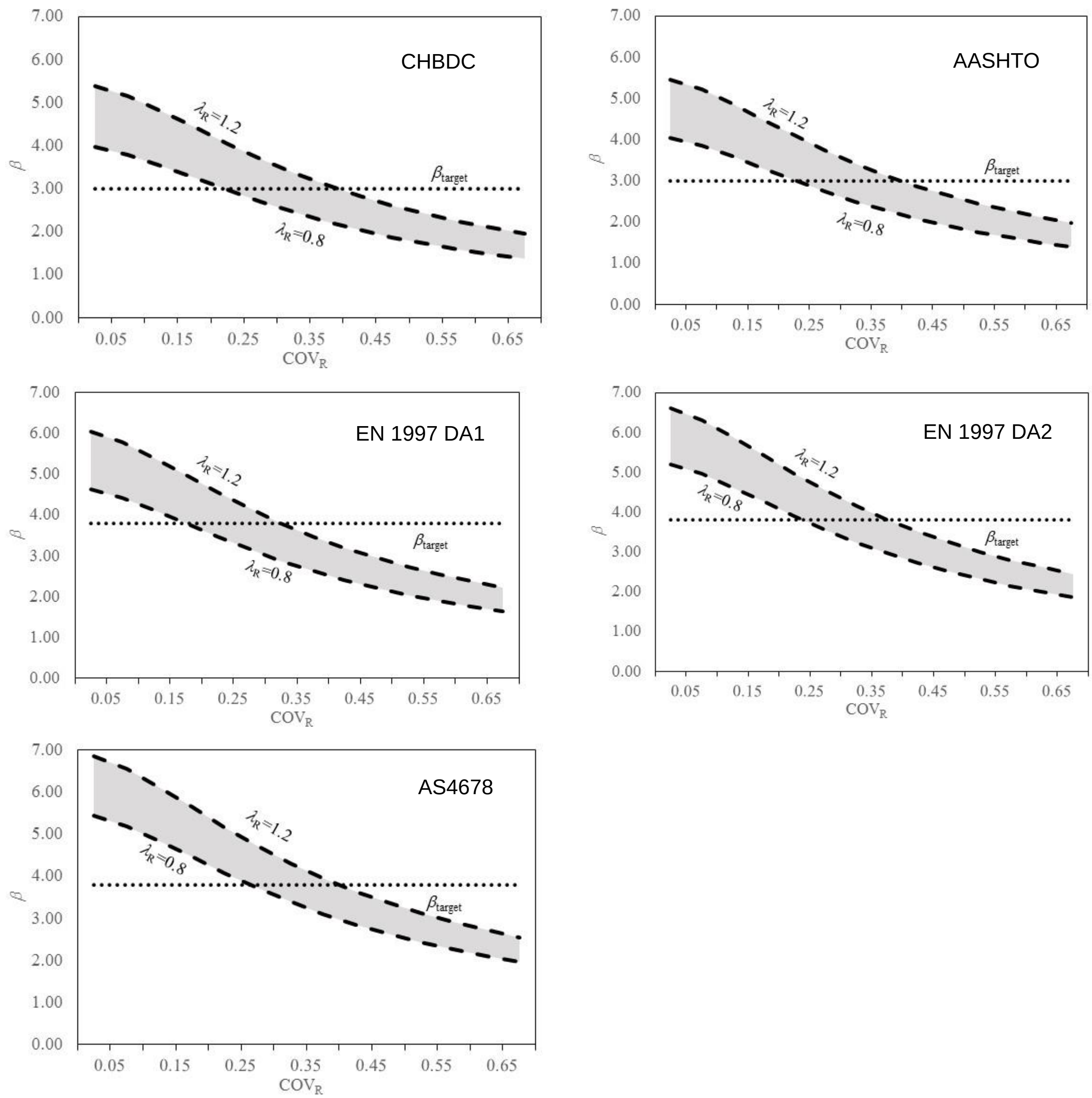

Figure 9 – Reliability level for shallow foundations achieved by the codes versus the total variability of the resistance