Reliability-Based Geotechnical Design Code Development

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ABSTRACT

The evolution of geotechnical design codes, from traditional working stress design (factor of safety) to reliability-based design approaches, has been lagging well behind structural design codes. There is no question that this lag is due to the much larger uncertainty about the ground than exists with most other engineering materials. Nevertheless, significant efforts have been made in recent decades to provide insight into the probabilistic behaviour of the ground and allow for advances to be made in the development of reliability-based geotechnical design codes. This paper compares target reliability levels in existing codes from around the world and then presents the latest advances in consequence and resistance factors required for the design of both shallow and deep foundations which are scheduled to soon appear in Canadian geotechnical design codes.

INTRODUCTION

Geotechnical design codes have been migrating towards reliability-based design concepts for several decades now. This generally involves breaking up the traditional factor of safety into separate 'partial' factors applied to the various components in the design equations. In most modern code implementations, the resulting set of partial factors have been separated into two distinct groups. These are the load and resistance factors, which lead to a design methodology referred to as Load and Resistance Factor Design (LRFD). The partial factors are individually related to the variability of the quantity that they are factoring and are used to scale the characteristic design values to more conservative values such that the overall probability of design failure is acceptably small. Designs must satisfy an equation of the following generalized form,

$$\varphi_g \hat{R} \geq \sum_i I_i \eta_i \alpha_i \hat{F}_i \tag{1}$$

where φ_g is a geotechnical resistance factor, $\hat{R}$ is the characteristic geotechnical resistance (based on characteristic ground parameters), and, for the i^{th} load, I_i is a structure importance factor (reflecting failure consequence), η_i is a load combination factor (reflecting the likelihood of certain combinations occurring simultaneously), α_i is the load factor, and $\hat{F}_i$ is the characteristic load effect. In Europe, the resistance factors are divisive, rather than multiplicative, so that the resistance factors presented in this paper need to be inverted for Eurocode 7.

COMPARISON OF CODE SAFETY LEVELS

To compare design codes from around the world and their target safety levels, a simplified version of Eq. (1) is considered, involving just dead and live loads (which is Design Approach 2 in Eurocode 7);

$$\varphi_g \hat{R} \geq \alpha_L \hat{F}_L + \alpha_D \hat{F}_D \tag{2}$$

where the subscripts D and L denote dead and live loads. In this paper attention will be restricted to ultimate limit states, which will be denoted by a subscript u, where appropriate.

A comparison of the safety levels over a variety of design codes involves a careful consideration of how all of the parameters entering the design process are defined and factored, particularly with respect to characteristic values. Some codes specify that the characteristic load is equal to the mean, others suggest using a 'cautious estimate of the mean', while others specify the use of an upper (or lower) quantile. Similarly, the characteristic resistance may be computed using mean strength parameters, or using quantiles of the strength parameters or resistance. In general, the difference between the characteristic design value and its mean is usually captured by a bias factor defined as the ratio of the mean to characteristic value, i.e.,

$$k_R = \frac{\mu_R}{\hat{R}_u}, \quad k_L = \frac{\mu_L}{\hat{F}_L}, \quad k_D = \frac{\mu_D}{\hat{F}_D} \tag{3}$$

where k is the bias factor and μ is the mean of the subscripted variable. Introducing the dead to live load ratio, $R_{D/L} = \mu_D/\mu_L = 3$ in this paper , allows Eq. (2) to be re-expressed as

$$\mu_R \geq F_s \left(\mu_L + \mu_D\right) \tag{4}$$

where F_s is a global factor of safety, defined as

$$F_s = \left(\frac{k_R}{\varphi_{gu}}\right)\left(\frac{\alpha_L}{k_L} + \frac{\alpha_D R_{D/L}}{k_D}\right)\left(\frac{1}{1 + R_{D/L}}\right) \tag{5}$$

Note that F_s in Eq. (4) is equivalent to the traditional factor of safety used in working stress design approaches. If the coefficients of variation of loads and resistances are approximately the same worldwide, then the global factor of safety provides a simple measure of the relative safety of a code design which then allows the safety level of various codes to be compared. Ellingwood (1999) notes that probability models for loads collected in research programs in North America and Europe agree reasonably well, and so the assumption that coefficients of variation are similar, at least between North America and Europe, is deemed to be reasonable. In this paper the global factor of safety provided by the following design codes are compared for shallow foundations at the bearing capacity ultimate limit state: 1) The National Building Code of Canada (NBCC) (National Research Council, 2010); 2) The Canadian Highway Bridge Design Code (CHBDC) (Canadian Standards Association, 2006); 3) The Eurocodes, in particular BS EN 1990: Basis of structural design (British Standards Institution, BSI, 2002a), BS EN 1991-1-1: Eurocode 1 Part 1-1 General Actions (BSI, 2002b) and BS EN 1997-1: Eurocode 7 Geotechnical Design – Part 1: General Rules (BSI, 2004); and 5) AS 5100: Bridge Design (Standards Australia, 2004a).

Characteristic Loads and Bias Factors

BS EN 1990 (BSI, 2002a) states that the variability of permanent actions (dead loads) may be neglected if they do not vary significantly over the design working life. I.e., if the coefficient of variation of dead loads, v_D, is less than about 10%, then the dead loads can be considered to be non-random and $\hat{F}_D = \mu_D$ so that $k_D = 1.0$. Other codes are less specific about the definition of characteristic dead loads, but generally indicate that $\hat{F}_D$ is to be estimated using mean structural component weights. Bartlett et al. (2003) suggest that some dead load components are often forgotten or missed in the estimation process, so that in practice the characteristic (design) dead load is generally somewhat less than the true mean dead load and the dead load bias factor is more like 1.05 (see also Ellingwood et al., 1980). Since this error is probably common to all localities, it will be assumed here that $k_D = 1.05$ for all codes considered.

North American codes define the characteristic live load as the mean maximum live load exerted on the structure over its design lifetime. However, North American codes specify acceptable characteristic live load values which are typically somewhat higher than the actual mean maximum live load. For example, both the Canadian and US codes specify a uniform live load for office space of 2.4 kPa. Bartlett et al. (2003) suggest that, after reductions for tributary area, the code specified characteristic live load is typically about 10% higher than the actual mean value, so that $k_L = 0.9$ was adopted by Bartlett et al. in their calibration efforts for the 2005 edition of the NBCC. As also reported by Bartlett et al., this bias value is in reasonable agreement with ASCE-7 (American Society of Civil Engineers, 2010).

BS EN 1990 (BSI, 2002a) also states in Clause 4.1.2(7) that, for variable actions (live loads), the characteristic value shall correspond to one of; an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period; or a nominal value, which may be specified in cases where a statistical distribution is not known. This is a fairly vague definition, but Clause 4.1.2(4) suggests that an "upper value" (which would be of interest for loads) corresponds to a 5% probability of being exceeded (95% fractile). Clause 4.1.2(4) states that the action may be assumed to be Gaussian so that the 95% fractile is given by

$$\hat{F}_L = \mu_L \left(1 + 1.645 v_L\right) \quad \rightarrow \quad k_L = 1/\left(1 + 1.645 v_L\right) \tag{6}$$

where v_L is the coefficient of variation of the maximum lifetime live load. Both Allen (1975) and Bartlett et al. (2003) use $v_L = 0.27$. The authors are not sure what value of v_L was assumed in the Eurocodes, but Ellingwood (1999) suggests that Europe uses a similar value to that used in North America. If this is the case, then the Eurocodes are using $k_L = 0.69$, which is very close to Allen's (1975) suggested bias of 0.7.

Another approach to estimating the European live load bias factor is to consider the characteristic office occupancy uniform live load specified in the European and North American codes, which are 3.0 and 2.4 kPa, respectively. If the live load bias factor of $k_L = 0.9$, adopted by Bartlett et al. (2003), is assumed true for North America, then $\mu_L = 0.9(2.4) = 2.16$ kPa. If it is further assumed that this mean live load is at least approximately true in Europe, then the European live load bias factor is $k_L = 2.16/3.0 = 0.72$. On the basis of both of the above approximate calculations, it appears likely, then, that the Eurocode uses a live load bias factor of approximately $k_L = 0.70$.

The Australian Standard AS5100.1 (Standards Australia, 2004a) specifically defines load actions for ultimate limit state as "an action having a 5% probability of exceedance in the design life" in Clause 6.5. This is the same as used in the Eurocode (albeit more clearly specified). In addition, since the Australian- New Zealand "Structural Design Actions" Standard AS/NZS 1170 (Standards Australia, 2002a) specifies that the characteristic uniform live load for office buildings is 3.0 kPa, which is the same as the Eurocode, it appears that the live load bias factor for Australia is also $k_L = 0.70$.

Characteristic Resistance and Bias Factors

Eurocode 7-1, Clause 2.4.5.2 (British Standard, 2004) provides a number of requirements for the selection of characteristic properties, such as "The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state" and "If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%. NOTE: In this respect, a cautious estimate of the mean value is a selection of the mean value of the limited set of geotechnical parameter values, with a confidence level of 95%; where local failure is concerned, a cautious estimate of the low value is a 5% fractile." The Eurocode 0 (British Standard, 2002) states that "where a low value of material or product property is unfavourable, the characteristic value should be defined as the 5% fractile value." According to Schneider (2012), the characteristic ground parameters should be selected as a 5% fractile value of the sample mean, using the distribution of the sample mean, rather than that of the samples directly (the sample mean having standard deviation $s/\sqrt{n}$, where s is the sample standard deviation, and n is the number of samples used to estimate s). The author notes that a 5% fractile value based on the sample mean will generally be quite a bit less conservative than a 5% fractile based on the samples themselves.

Hicks (2012) interprets the intention of Clause 2.4.5.2 of Eurocode 7-1 to mean that the characteristic soil parameters are to be selected so as to ensure a 95% confidence in the geotechnical system being designed, and this appears to be the correct assumption. Unfortunately, this interpretation involves knowledge of the appropriate spatial averaging of geotechnical parameters over the actual failure surface (or failure domain). This knowledge will not be generally available and so the authors feel that it is probably easier to develop a design code using characteristic soil parameters based on fractiles of the soil parameter distribution at this point in time.

In any case, the above discussion about characteristic values used in the Eurocode refers to the selection of characteristic strength parameters (e.g. c_u or ϕ) rather than to the characteristic resistance appearing in Eq. (2). The characteristic geotechnical resistance, $\hat{R}_u$, would then be computed employing a (probably non-linear) model which uses these characteristic ground parameters. Thus, the final bias of the characteristic resistance depends not only on the distribution of the ground properties, but also on the model used to predict $\hat{R}_u$. It will be assumed here that the coefficient of variation, v_R, of $\hat{R}_u$ is approximately equal to the coefficient of variation of the ground parameters used in the model, which are typically in the range of 0.1 to 0.3 (e.g., Meyerhof, 1995). Note that geotechnical resistance often involves an average of ground properties, e.g. along a failure surface, which will have a smaller variability than the point variability suggested in the literature. Thus, a reasonable value for the resistance variability is deemed by the authors to be about $v_R = 0.15$, which will be assumed here. Similar to Eq. (6), the resistance bias factor assumed in the Eurocode can then be computed from

$$\hat{R}_u = \mu_R\left(1 - 1.645v_R\right) \quad \rightarrow \quad k_R = 1/\left(1 - 1.645v_R\right) \tag{7}$$

which for $v_R = 0.15$ gives $k_R = 1.33$.

The Australian Standard AS5100.3 (Standards Australia, 2004b) states that "the characteristic value of a geotechnical parameter should be a conservatively assessed value of the parameter." Although the authors were unable to find a more precise definition, the wording here suggests that the Australians are following the Eurocode approach. Thus, a bias factor of $k_R = 1.33$ will be assumed for Australia as well.

In North America, Commentary Clause C10.4.6.1 of AASHTO (2007) says that "For strength limit states, average measured values were used to calibrate the resistance factors", which suggests that $k_R = 1.0$. However, the commentary goes on to say that "it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the Engineer may have no choice but to use a more conservative selection of design properties" which suggests that in practice, $k_R > 1.0$.

Clause 8.5 of the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 2006) states that "Frequently, the mean value, or a value slightly less than the mean is selected by geotechnical engineers as the characteristic value." Commentary K of the NBCC User's Guide (National Research Council, 2011) says that "the [characteristic] resistance is the engineer's best estimate of the ultimate resistance." Becker (1996a) claims "The design values do not necessarily need to be taken as the mean values, although this is common geotechnical design practice." All of these statements suggest that $k_R = 1.0$, or perhaps slightly greater than 1.0. However, Becker (1996a) later argues that the characteristic resistance is typically selected to be somewhat below the mean, due to sampling uncertainties, and he subsequently uses $k_R = 1.1$ in his NBCC development paper (Becker, 1996b). Based on Becker's reasoning, the value of $k_R = 1.1$ will be assumed to apply to all of the North American design codes considered here.

Load Factors

Load factors are designed to reflect uncertainty in the lifetime loads experienced by a structure or foundation. The basic idea is to set the factored loads, $\alpha_L\hat{F}_L$ and $\alpha_D\hat{F}_D$, to values having sufficiently low probability of being exceeded by the true (random) lifetime loads. Considering, for example, live loads (with dead loads following the same reasoning), the factored live load which has a probability ϵ of being exceeded by the true live load over the design lifetime can be approximated as

$$\alpha_L\hat{F}_L = \mu_L\left(1 + z_\epsilon v_L\right) \tag{8}$$

in which z_ϵ is the standard normal point with exceedance probability ϵ, i.e. the point such that $\Phi(-z_\epsilon) = \epsilon$, where Φ is the standard normal cumulative distribution function. Note that Eq. (8) assumes that the live load is (at least approximately) normally distributed. Rearranging Eq. (8) leads to an expression for the load factor, which is

$$\alpha_L = \left(\frac{\mu_L}{\hat{F}_L}\right)(1 + z_\epsilon v_L) = k_L\,(1 + z_\epsilon v_L) \tag{9}$$

ASCE-7 (American Society of Civil Engineers, 2010) found that their load factors are well approximated by Eq. (9) when they set $z_\epsilon = \omega_L \beta$, where β is the target reliability index and $\omega_L = 0.8$ when L is a principal action or $\omega_L = 0.4$ when L is a companion action. Equation (9) can be used for other load types simply by changing the subscript. Note that Eq. (9) suggests that load factors are independent of the resistance distribution. It also states that the load factors are very dependent on how the characteristic load is defined, i.e., on the load bias factor, k. If designs have a common target reliability index, β, and $k_L = 0.9$ in North America and $k_L = 0.7$ in Europe and Australia, as suggested above, then one would expect the load factors in Europe and Australia to be lower than those used in North America if Eq. (9) is accurate. As will be seen, the European and Australian load factors are generally higher than those used in North America – the European and Australian codes compensate for their higher load factors through higher (albiet, inverse) resistance factors. In other words, Eq. (9) cannot be used as a general formula for load factors. The magnitude of the resistance factors (and bias factors) must still be considered.

Table 1 gives the load factors as specified by the various design codes considered here. The last column of the table gives the total load factor, α_T, for a given mean dead to live load ratio, $R_{D/L} = 3.0$, which scales the total mean load so that $\alpha_T(\mu_L + \mu_D)$, is equal to the sum of factored live and dead loads (see Eq. 2). The total load factor can be seen in Eq. (5) and is defined by

$$\alpha_T = \left(\frac{\alpha_L}{k_L} + \frac{\alpha_D R_{D/L}}{k_D}\right)\left(\frac{1}{1 + R_{D/L}}\right) \tag{10}$$

The dead load factor for the Eurocodes (1.35) is larger than the dead load factors used in North America (1.2 to 1.25) which, when combined with the smaller value of k_L, yields a final α_T value which is significantly larger than that appearing in the Canadian codes and in AASHTO. The Australian Standard AS5100 has an equally high α_T value because of their relatively high live load factor, α_L, and low live load bias factor, k_L.

Table 1 Load and bias factors for various design codes.

Source	k_L	k_D	α_L	α_D	α_T
NBCC 2012	0.9	1.05	1.50	1.25	1.31
CHBDC 2006	0.9	1.05	1.70	1.20	1.33
AASHTO 2007	0.9	1.05	1.75	1.25	1.38
Eurocode 7	0.7	1.05	1.50	1.35	1.50
AS5100.3	0.7	1.05	1.80	1.20	1.50

Table 2 shows the total effective load factor, the resistance bias, the resistance factor, and the global factor of safety for the five design codes considered with respect to shallow foundation bearing capacity (assuming Design Approach 2 for Eurocode 7).

Table 2 Global factor of safety for bearing capacity of a shallow foundation for various design codes.

Source	α_T	k_R	φ_{gu}	F_s
NBCC 2012[1]	1.31	1.1	0.50	2.88
CHBDC 2006	1.33	1.1	0.50	2.93
AASHTO 2007	1.38	1.1	$0.45 - 0.5$	$3.04 - 3.37$
Eurocode 7[2]	1.50	1.33	0.71	2.81
AS5100.3	1.50	1.33	$0.35 - 0.65$	$3.07 - 5.70$

[1] the NBCC itself does not specify resistance factors.The resistance factors shown above appear in Appendix K of the NBCC User's Guide (National Research Council, 2011).

[2] based on the Eurocode 7 Design Approach 2.

Perhaps unsurprisingly, and despite the considerable variation in implementation details, the five codes considered here all arrive at quite similar global factors of safety, F_s, as seen in the last column of Table 2. Many assumptions were made in arriving at Table 2 about how characteristic values are actually defined in the various codes, and so there may actually be more discrepancy between the codes for this particular limit state. However, it appears likely that codes are calibrated for much the same target failure probability regardless of the implementation details. The authors note that, if this is the case, there seems to be little justification in codes being different – we might as well all adopt the same model and work in common towards a safer and more economical design code. The model adopted worldwide should be the simplest and easiest to define.

GEOTECHNICAL DESIGN CODE DEVELOPMENTS IN CANADA

Because the ground is so highly uncertain, similarly to earthquake, snow, and wind loads, it makes sense to apply a partial safety factor to the ground that depends on both the resistance uncertainty and consequence of failure. This would be analogous to how wind load, for example, in North American codes has both a load factor associated with wind speed uncertainty as well as an importance factor associated with failure consequences. Increased site investigation and/or modeling effort should lead to lower uncertainty and thus a higher resistance factor and a more economical design. Similarly, for geotechnical systems with high failure consequences, e.g. failure of the foundation of a major multi-lane highway bridge in a capital city, the total resistance factor should be decreased to ensure a decreased maximum acceptable failure probability.

Rather than introducing myriad resistance factor tables for all possible combinations of site understanding and failure consequence, the multiplicative approach taken in structural engineering, where the load is multiplied by both a load factor and an importance factor, has been adopted in Canada for geotechnical resistance. In other words, the overall factor applied to geotechnical resistance is broken into two parts;

1. a resistance factor, φ_{gu} or φ_{gs}, which accounts for resistance uncertainty. This factor basically aims to achieve a target maximum acceptable failure probability equal to that used currently for geotechnical designs for typical failure consequences (e.g. lifetime failure probability of 1/5000 or less). The subscript g refers to 'geotechnical' (or 'ground'), while the subscripts u and s refer to ultimate and serviceability limit states, respectively.
2. a consequence factor, Ψ, which accounts for failure consequences. Essentially, $\Psi > 1$ if failure consequences are low and $\Psi < 1$ if failure consequence exceed those of typical geotechnical systems. For typical systems, or where system importance is already accounted for adequately by load importance factors, $\Psi = 1$. The basic idea of the consequence factor is to adjust the maximum acceptable failure probability of the design down (e.g. 1/10000) for high failure consequences, or up (e.g. 1/1000) for low failure consequences.

The geotechnical design would then proceed by ensuring that an equation of the form

$$\Psi\varphi_{gu}\hat{R} \geq \sum_i I_i\eta_i\alpha_{ui}\hat{F}_{ui} \tag{11}$$

is satisfied, where the overall resistance factor is now expressed as the product of the consequence factor, Ψ, and the ultimate geotechnical resistance factor, φ_{gu}, and the loads and load factors appearing on the right-hand-side are also those specific for the ultimate limit state under consideration (and, hence, the subscript u).

The geotechnical resistance factor, φ_{gu} or φ_{gs}, depends on the degree of site and prediction model understanding. Three levels will be considered in the future editions of the building and highway codes in Canada;

- *High understanding:* Extensive project-specific investigation procedures and/or knowledge are combined with prediction models of demonstrated (or proven) quality to achieve a high level of confidence in performance predictions,
- *Typical understanding:* Usual project-specific investigation procedures and/or knowledge are combined with conventional prediction models to achieve a typical level of confidence in performance predictions,
- *Low understanding:* Understanding of the ground properties and behaviour are based on limited representative information (e.g. previous experience, extrapolation from nearby and/or similar sites, etc.) combined with conventional prediction models to achieve a lower level of confidence with the performance predictions.

The consequence factor, Ψ, adjusts the maximum acceptable failure probability of the geotechnical system being designed to a value which is appropriate for the consequences. Three failure consequence levels will be considered in future Canadian geotechnical design codes;

- *High consequence:* the geotechnical system is designed to be essential to post-disaster recovery (e.g. hospital or lifeline bridge), and/or has large societal and/or economic impacts.

- *Typical consequence:* the geotechnical system is designed for typical failure consequences, e.g. the usual office building, bridge, etc. This will be the default failure consequence level.
- *Low consequence:* failure of the geotechnical system poses little threat to human or environmental safety, e.g. storage facilities, temporary structures, very low traffic volume bridges, etc.

CONCLUSIONS

With the above thoughts in mind, the next editions of the Canadian Highway Bridge Design Code and the National Building Code of Canada will include several philosophical changes to their geotechnical design provisions. These include;

- the introduction of three levels of site and model understanding – high, typical, and low – through the ULS and SLS resistance factors. These factors are intended to account for site and modeling uncertainties and are aimed at producing a design with a target maximum acceptable failure probability for typical geotechnical systems (i.e. systems having typical failure consequence levels). For example, ULS and SLS maximum acceptable lifetime failure probabilities might be 1/5000 and 1/500, respectively, and so these resistance factors would be targeted at these values.
- the introduction of three levels of failure consequence – high, typical, and low – through a consequence factor which multiplies the factored resistance. The basic idea of the consequence factor is to allow the target maximum acceptable lifetime failure probability provided by the resistance factor to be adjusted up or down depending on whether the failure consequences are lower or higher than typical.

Research into the determination of the required resistance and consequence factors for the Canadian codes is ongoing. The consequence factor is a new idea and work is still needed to determine when it should and should not be applied. For example, whether both the consequence factor and importance factors should be applied simultaneously is unknown, but initially, they will not be.

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