Comparison of Geotechnical LRFD Implementations

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ABSTRACT
The goal of this paper is to compare the overall level of safety aimed at by geotechnical design codes from various locations around the world. In particular, the National Building Code of Canada (NBCC), the Canadian Highway Bridge Design Code (CHBDC), the American Association of State Highway and Transportation Officials (AASHTO), the Eurocode (EC), and the Australian Standards (AS) design codes are compared. The comparison is made by estimating a global factor of safety as a function of load and resistance factors, characteristic load and resistance biases, and dead to live load ratio, specifically for the shallow foundation bearing capacity ultimate limit state. The results of the paper can be used to aid in the calibration of the next generation of reliability-based geotechnical design codes of practice around the world.

RÉSUMÉ
Le but de cet article est de comparer le niveau global de sécurité visé par les codes de conception géotechniques provenant de différents endroits à travers le monde. En particulier, le National Building Code du Canada (NBCC), le Canadian Highway Bridge Design Code (CHBDC), l'American Association of State Highway et Transportation Officials (AASHTO), l'Eurocode (EC), et les Australian Standards (AS) de conception des codes sont comparées. La comparaison est faite par l'estimation d'un facteur global de la sécurité en fonction des facteurs de charge et de résistance, caractéristiques des préjugés de charge et de résistance, et morts pour vivre rapport de charge, en particulier pour le palier fondation d'utilité publique peu profonde limite la capacité ultime. Les résultats du document peut être utilisé pour aider à l'étalonnage de la prochaine génération de la fiabilité des codes axés sur la conception géotechniques de la pratique partout dans le monde.

1 INTRODUCTION

Geotechnical codes of practice around the world are migrating towards reliability-based design concepts, where the constructed geotechnical system is targeted to achieve a certain reliability level. This migration is partly in order to harmonize with structural design codes, which have been based on probabilistic methods for several decades now, and partly because the ground is one of the most highly variable, and thus uncertain, of engineering materials and so very much in need of probabilistic treatment. As with structural engineering, the common approach to achieving target reliabilities is to use Load and Resistance Factor Design (LRFD) concepts embedded in a Limit States Design (LSD) framework.

LRFD requires that the factored resistance be at least equal to the sum of the factored load effects. In this paper only the ultimate limit state (ULS) will be considered, in which case, the design must satisfy an equation of the form

$$\varphi_{gu}\hat{R}_u \geq \alpha_L \hat{F}_L + \alpha_D \hat{F}_D \qquad [1]$$

where φ_{gu} is the geotechnical resistance factor at ULS, $\hat{R}_u$ is the characteristic ultimate resistance, and the right-hand-side consists of a sum of factored load effects. For simplicity only the live and dead load combination is considered in this paper so that α_L and α_D are the live and dead load factors, respectively, and $\hat{F}_L$ and $\hat{F}_D$ are the characteristic live and dead loads, respectively. The characteristic dead load is a representation of the sum of weights of all permanently supported structural components and equipment, while the characteristic live load is a representation of the maximum non-permanent load that will be exerted over the lifetime of the system. The word 'representation' is used here because not all codes of practice use the same definition of characteristic values. For example, 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. In general, the difference between the characteristic design value and its mean is usually captured by a *bias* factor usually defined as the ratio of the mean to characteristic value, i.e.,

$$k_R = \frac{\mu_R}{\hat{R}_u}, \quad k_L = \frac{\mu_L}{\hat{F}_L}, \quad k_D = \frac{\mu_D}{\hat{F}_D} \qquad [2]$$

where k is the bias factor and μ is the mean of the subscripted variable. Introducing, the dead to live load ratio, $R_{D/L} = \mu_D / \mu_L$, allows eq. [1] to be re-expressed as

$$\left(\frac{\varphi_{gu}}{k_R}\right)\mu_R \geq \left(\frac{\alpha_L}{k_L}+\frac{\alpha_D R_{D/L}}{k_D}\right)\left(\frac{1}{1+R_{D/L}}\right)\left(\mu_L+\mu_D\right) \quad [3]$$

or

$$\mu_R \geq F_s\left(\mu_L+\mu_D\right) \quad [4]$$

where F_s is a global factor of safety, defined as

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

Note that in eq. [4] F_s is seen to take on a similar role (and definition as ratio of mean resistance to mean load) as does the traditional factor of safety used in working stress design approaches. If the coefficients of variation of the 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) published by the National Research Council of Canada (2010),
2. The Canadian Highway Bridge Design Code (CHBDC) published by the Canadian Standards Association (2006),
3. AASHTO LRFD Bridge Design Specifications (AASHTO), published by the American Association of State Highway and Transportation Officials (2007),
4. The Eurocode, in particular Eurocode 0 (Basis of Structural Design, British Standard, 2002a), Eurocode 1 (Actions on Structures – Part 1-1: General Actions, British Standard, 2002b) and Eurocode 7-1 (Geotechnical Design – Part 1: General Rules, British Standard, 2004),
5. Australian Standard AS5100 (Bridge Design, Standards Australia, 2004)

To compare the level of safety between each of these codes, a hypothetical geotechnical system will be considered which has dead to live load ratio $R_{D/L}=3.0$.

2 CHARACTERISTIC LOADS AND BIAS FACTORS

The Eurocode is reasonably specific as to how characteristic loads are defined. With respect to dead loads, the Eurocode 0 (British Standards, 2002a) states that the variability of permanent actions (i.e. dead loads) may be neglected if they do not vary significantly over the design working life. In other words, 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. The other codes considered 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 so 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.

With respect to live loads, the North American codes define the characteristic live load as the mean maximum live load exerted on the structure over its design lifetime – for example, Clause 4.3.1 of ASCE-7 (2010) states that uniformly distributed live loads are the mean of the maximum load over the design lifetime. Although the NBCC does not specifically define the characteristic live load, Bartlett et al. (2003) implies that it has the same definition as ASCE-7. Both 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 influence or 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. (2003) 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.

The Eurocode 0 (British Standard 2002a) states in Clause 4.1.2(7) that for variable actions, 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) further states that the action may be assumed to be Gaussian. If this is assumed, then the 95% fractile is given by

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

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 Eurocode, but Ellingwood (1999) suggests that Europe uses a similar value to that used in North America. If this is the case, then the Eurocode is using $k_L=0.69$, which is very close to Allen's (1975) suggested bias of 0.7.

Another approach to estimating the live load bias factor employed in Europe 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, 2002) specifies that the characteristic uniform live load for office buildings is 3.0, which is the same as the Eurocode, it appears that the live bias factor for Australia is also $k_L = 0.70$.

3 CHARACTERISTIC RESISTANCE AND BIAS FACTORS

The estimation of the resistance of the ground to imposed loads is generally a multi-step process: 1) take measurements of the ground properties, 2) correlate the measurements with characteristic engineering parameters (e.g. cohesion and friction angle), and 3) use the characteristic parameters in a prediction model. Each step introduces errors, and so the characteristic resistance and associated resistance factor (discussed later), along with the loads and load factors, must be found in such a way to ensure a safe design. 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 2002a) 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 Schnieder (2011), the characteristic ground parameters should be selected as a 5% fractile value of the sample mean (i.e. using the distribution of the sample mean, rather than of the samples).

Note that the above discussion about characteristic values used in the Eurocode refers to characteristic strength parameters (e.g. c_u or ϕ) rather than to the characteristic resistance appearing in Eq. [1], as far as the authors can determine. The characteristic geotechnical resistance, $\hat{R}_u$, is then computed employing a possibly 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 and Phoon and Kulhawy, 1999). 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 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 be computed from

$$\hat{R}_u = \mu_R\left(1-1.645v_R\right) \quad \Rightarrow \quad k_R = 1/\left(1-1.645v_R\right) \qquad [7]$$

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

The Australian Standard AS5100.3 (Standards Australia, 2004c) 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 they 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 of Canada, 2010) 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.

4 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 (and dead loads follow the same reasoning), the factored live load which has probability ε of being exceeded by the true live load over the design lifetime can be computed as

$$\alpha_L \hat{F}_L = \mu_L (1 + z_\varepsilon v_L) \quad [8]$$

In which z_ε is the standard normal point with exceedance probability ε, i.e. the point such that $\Phi(-z_\varepsilon) = \varepsilon$, 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

$$\alpha_L = \left(\frac{\mu_L}{\hat{F}_L}\right)(1 + z_\varepsilon v_L) = k_L (1 + z_\varepsilon v_L) \quad [9]$$

ASCE-7 (American Society of Civil Engineers, 2010) found that their load factors are well approximated by eq. [9] when they set $z_\varepsilon = \omega_L \beta$, where β is the target reliability index and $\omega_L = 0.8$ when L is a principle 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. However, the European and Australian codes compensate for their higher load factors through higher 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, which scales the total mean load, $\mu_L + \mu_D$, to be equal to the sum of factored live and dead loads.. The total load factor can be seen in eqs. [3] and [5] and is defined by

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

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

The dead load factor for the Eurocode (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 the second highest α_T because of their relatively high live load factor and low live load bias factor, k_L.

5 RESISTANCE AND GLOBAL SAFETY FACTORS

In order to compare Eurocode 7 to the other codes, attention must be restricted to a design approach which involves factoring the resistance, rather than factoring the ground strength parameters. The Eurocode 7 allows three design approaches;

1. Design Approach 1: partial factors are applied to actions and to ground strength parameters,
2. Design Approach 2: partial factors are applied to actions or to the effects of actions and to the ground resistance, or
3. Design Approach 3: partial factors are applied to actions or to the effects of actions from the structure and to ground strength parameters.

Design Approaches 1 and 3 involve factoring the ground strength parameters (i.e. factoring cohesion and friction angle directly), while Design Approach factors the final computed ground resistance. Since the latter is how the other codes proceed, only Design Approach 2 will be considered here. In addition, Eurocode 7 considers five limit states, of which the following two are of interest for the bearing capacity problem;

1. EQU – loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance, or
2. GEO – failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance

Evidently, in a bearing capacity problem, the strength of the ground is of significance, and so only the GEO limit state will be considered here.

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.

Table 2. Load and resistance factors 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[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 Eurocode 7 Design Approach 2 for the GEO limit state.

6 CONCLUSIONS

Perhaps unsurprisingly, and despite the considerable variation in details, the five codes considered here all arrive at quite similar global factors of safety, F_s, as seen in Table 2. The Australian Standard Bridge Design Code (AS5100) has the potential of being significantly the most conservative. However, this will only occur if the designer has a low understanding of the site and low confidence in the prediction model used, so that a low resistance factor (e.g. 0.35) is required. If the designer has a high degree of site understanding, a larger resistance factor can be used (e.g. 0.65). The advantage to providing a sliding scale for the resistance factor is that it allows the designer to show "proof" regarding the real benefit of improved site investigation and design modeling. At the high end of the resistance factor scale, the Australian code is only very slightly more conservative than the other codes considered – given all of the other approximations made in this study, the difference is considered to be negligible.

The Canadian codes (NBCC and CHBDC) seem to be between the Eurocode and AASHTO, with global factors of safety slightly below 3.0. However, the difference is minor. A change of about 5% in, say, the load factors, e.g. using $\alpha_D = 1.3$ instead of 1.25, would put the NBCC, and similarly the CHBDC, in or near the lower range of AASHTO. Likewise, a decrease of 5% in the dead load factors of the Canadian codes would .yield global factors of safety approximately the same as the Eurocode.

The similarity in overall safety levels suggests that

1. Codes are calibrated similarly, i.e., using similar target reliabilities, similar probability models, and similar databases.
2. Harmonization amongst codes is a possibility. Cross-border consulting is becoming increasingly common and there is a real desire amongst those engineers who practice in several countries to "unify" the design codes. If the end result is so similar in any case, there probably isn't a really compelling reason for the codes to be so different in their details.

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