# Considerations for Resistance Factor Calibration in the National Building Code of Canada

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ABSTRACT
The Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-14, adopted a resistance and consequence factor design approach in their 2014 edition. The resistance and consequence factors were calibrated for the CHBDC using probabilistic models capturing the Load and Resistance Factor Design (LRFD) framework. The National Building Code of Canada (NBCC) is beginning to develop its own geotechnical LRFD framework. However, the resistance and consequence factors have yet to be calibrated specifically for the NBCC. The NBCC differs from the CHBDC in at least three areas which are important to this calibration process: 1) The load factors and load combinations used in the NBCC are different than those used in the CHBDC, 2) The design life in the NBCC is 50 years and 75 years in the CHBDC, 3) The target reliability index in the NBCC is 3.0 over a 50-year design life, while the CHBDC specifies an annual reliability index of 3.75. The changes in these factors will affect the resistance and consequence factors required to achieve the specified reliability. This paper presents the results of a preliminary calibration of resistance factors for the NBCC.

RÉSUMÉ
Le Code de conception des ponts routiers canadiens (CHBDC), CAN/CSA-S6-14, a adopté une approche de conception des facteurs de résistance et de conséquence dans leur édition de 2014. Les facteurs de résistance et de conséquence ont été calibrés pour le CHBDC à l'aide de modèles probabilistes capturant le cadre de conception de facteurs de charge et de résistance (LRFD). Le Code national du bâtiment du Canada (CNBC) commence à mettre au point son propre cadre géotechnique LRFD. Cependant, les facteurs de résistance et de conséquence n'ont pas encore été calibrés spécifiquement pour le NBCC. Le NBCC diffère du CHBDC dans au moins trois domaines importants pour ce processus d'étalonnage: 1) Les facteurs de charge et les combinaisons de charge utilisés dans le NBCC sont différents de ceux utilisés dans le CHBDC, 2) La durée de vie nominale du NBCC est de 50 ans et 75 ans dans le CHBDC, 3) L'indice de fiabilité cible du NBCC est de 3,0 sur une durée de vie nominale de 50 ans, tandis que le CHBDC spécifie un indice de fiabilité annuel de 3,75. Les modifications de ces facteurs affecteront les facteurs de résistance nécessaires pour atteindre la fiabilité spécifiée. Cet article présente les résultats d'un étalonnage préliminaire des facteurs de résistance et de conséquence pour le NBCC.

## 1 INTRODUCTION

The NBCC (NRC, 2015) differs from the CHBDC (CSA, 2014) in at least three areas: 1) the load factors and load combinations, 2) the design lifetime, and 3) the target reliability index (Fenton et al., 2015). In this paper, the CHBDC geotechnical resistance factor for various limit state designs of Shallow Foundations, Deep Foundations, Ground Anchors, Internal MSE reinforcement, Retaining Systems, and Embankments are used to calculate the equivalent geotechnical resistance factors for the NBCC, considering the design parameters summarized in the following table:

Table 1. Design parameters

| Parameter | CHBDC | NBCC |
|---|---|---|
| ULS[1] live load factor, $\alpha_L$ | 1.70 | 1.50 |
| ULS dead load factor, $\alpha_D$ | 1.20 | 1.25 |
| ULS earth pressure load factor, $\alpha_E$ | 1.25 | 1.5 |
| ULS wind load factor, $\alpha_W$ | 1.40 | 1.40 |
| Live load bias factor, $k_L$ | 1.05 | 1.05 |
| Dead load bias factor, $k_D$ | 0.9 | 0.9 |
| Earth pressure bias factor, $k_E$ | 1.0 | 1.0 |
| Wind load bias factor, $k_W$ | 1.0 | 1.0 |
| Live load CoV[2], $v_L$ | 0.27 | 0.27 |
| Dead load CoV, $v_D$ | 0.10 | 0.10 |
| Wind load CoV, $v_W$ | 0.135 | 0.135 |
| Earth pressure CoV, $v_E$ | 0.14 | 0.14 |
| Design lifetime, $n$ | 75 years | 50 years |
| Lifetime Reliability index, $\beta_n$ | 2.5 | 3.0 |
| Annual Reliability index, $\beta_1$ | 3.75 | 4.03 |

[1] Ultimate Limit State
[2] Coefficient of Variation

## 2 THEORY

In this section, the theory used to calibrate the geotechnical resistance factors for the NBCC is summarized. Three examples for shallow foundations—bearing capacity, sliding, and settlement—will be used to illustrate the theory behind the prediction of resistance factors for a given limit state. The resulting theory will then be used to calculate the resistance factors for the NBCC which lead to the same reliability as those used in the CHBDC for all of the limit states listed in Table 6.2 of the CHBDC. The goal is to see how the NBCC resistance factors must be changed in order to achieve similar reliability levels as targeted by the CHBDC.

## 2.1 Bearing Capacity

This limit state is governed by vertical loads and, for shallow foundations, the geotechnical resistance factor is calculated by Fenton et al. (2008), assuming only live and dead loads, to be

$$\phi_{gu} = \frac{\hat{F}}{\exp\{\mu_{\ln Y} + \beta\sigma_{\ln Y}\}} \qquad [1]$$

where

$$\hat{F} = \alpha_L \hat{F}_L + \alpha_D \hat{F}_D \qquad [2]$$

is the factored design load, and

$$Y = F\frac{\hat{c}\hat{N}_c}{\overline{c}\overline{N}_c} \qquad [3]$$

has mean and variance

$$\begin{aligned} \mu_{\ln Y} &= \mu_{\ln F} \\ \sigma^2_{\ln Y} &= \ln\left(1+v_Y^2\right) \end{aligned} \qquad [4]$$

In Eq.'s [1]-[3], $\alpha_L$ and $\alpha_D$ are the live and dead load factors, $\hat{F}_L$ and $\hat{F}_D$ are the characteristic live and dead loads, $\beta = -\Phi^{-1}\left(p_{\max}\right)$ is the reliability index corresponding to maximum acceptable failure probability $p_{\max}$, $F = F_L + F_D$ is the true (random) load defined as the sum of the maximum lifetime live and dead loads, $F_L$ and $F_D$, $\hat{c}$ is the characteristic cohesion, $\hat{N}_c$ is the characteristic bearing capacity factor, $\overline{c}$ is the geometric average of the cohesion field over domain D underneath the footing, and $\overline{N}_c$ is the actual bearing capacity factor (Fenton et al., 2008).

Assuming that

$$\begin{aligned} \hat{F}_L &= \mu_L / k_L \\ \hat{F}_D &= \mu_D / k_D \\ R_{D/L} &= \mu_D / \mu_L \\ \mu_F &= \mu_L + \mu_D = \mu_L\left(1+R_{D/L}\right) \end{aligned} \qquad [5]$$

where $\mu_L$ and $\mu_D$ are the mean live and dead loads, $k_L$ and $k_D$ are the live and dead bias factors, and $R_{D/L}$ is the dead to live load ratio, then Eq. [1] reduces to

$$\begin{aligned} \phi_{gu} &= \frac{\alpha_L\mu_L / k_L + \alpha_D\mu_D / k_D}{\exp\{\mu_{\ln F} + \beta\sigma_{\ln Y}\}} \\ &= \frac{\alpha_L\mu_L / k_L + \alpha_D R_{D/L}\mu_L / k_D}{\left[\mu_F / \sqrt{\ln\left(1+v_F^2\right)}\right]\exp\{\beta\sigma_{\ln Y}\}} \\ &= \frac{\left[\alpha_L / k_L + \alpha_D R_{D/L} / k_D\right]\sqrt{\ln\left(1+v_F^2\right)}}{\left(1+R_{D/L}\right)\exp\{\beta\sigma_{\ln Y}\}} \end{aligned} \qquad [6]$$

where

$$v_F = \frac{\sqrt{\sigma_L^2 + \sigma_D^2}}{\mu_L + \mu_D} \qquad [7]$$

is the coefficient of variation of the total load, $F$, and $\sigma_L$ and $\sigma_D$ are standard deviations of the live and dead loads. It is assumed in Eq. [7] that live and dead loads are independent.

It is apparent from Eq. [6] that the geotechnical resistance factor depends on the dead and live load factors, bias factors, dead to live load ratio, the reliability index, and the load variability. The resistance factor also depends on $\sigma_{\ln Y}$, which includes both resistance and load variability, and will be discussed later in Section 3.

## 2.2 Sliding

It is assumed that this limit state is governed by lateral load due to wind with load factor $\alpha_W = 1.40$ used in both the CHBDC and the NBCC.

For cohesive soils, the probability of sliding failure is

$$p_f = \mathrm{P}\left[W > A\overline{c}\right] = 1 - \Phi\left(\beta\right) \qquad [8]$$

where $W$ is the horizontal wind load, $A$ is the footing area, and $\overline{c}$ is the average cohesion between the footing and the soil. Following a procedure similar to that developed by Fenton et al. (2008), the required geotechnical resistance factor for cohesive soil can be found as follows. If sliding governs the design, the area $A$ is determined to be $A = \alpha_w\hat{W} / \phi_{gu}\hat{c}$ and Eq. [8] leads to

$$\phi_{gu} = \frac{\alpha_W\sqrt{\ln\left(1+v_W^2\right)}}{k_W\exp\{\beta\sigma_{\ln Y}\}} \qquad [9]$$

where $Y = W\hat{c}/\overline{c}$ and $k_W = \mu_w / \hat{W}$ is the bias factor for wind, assumed to be $k_W = 1.0$ (see Bartlett et al, 2003). For simplicity, it has been assumed that the maximum wind load over the design lifetime of the geotechnical system is lognormally distributed with mean $\mu_W$. In the NBCC, this mean is assumed to be the wind speed having annual probability of exceedance equal to 1/50. Bartlett et al (2003) suggest that the error in this assumption is less than 5% when the annual maximum windspeed coefficient of variation is set to the average value of 0.135 that they obtained. It is assumed in this paper that the lifetime maximum windspeed coefficient of variation is also $v_W = 0.135$.

An entirely similar procedure and resistance factor equation can be found for sliding on frictional soils except that $Y = W \tan\hat{\delta} / \tan\bar{\delta}$ where $\delta$ is the friction angle between the footing and the soil.

For sliding of retaining walls, the only difference in Eq. [9] is that the horizontal driving force is now earth pressure with load factor $\alpha_E$ and coefficient of variation $v_E \approx 0.14$ (estimated by the authors for friction angle of $\phi = 25°$ and coefficient of variation of $v_\phi = 0.2$). Also, $k_E = \mu_E / \hat{E}$ is the bias factor for earth pressure, assumed to be $k_E = 1.0$.

Similarly, for global stability of embankments, Eq. [9] applies except that the driving force is vertical dead load with load factor $\alpha_D$, bias factor $k_D$, and coefficient of variation $v_D$ (see Table 1).

## 2.3 Settlement and Lateral Movement

The settlement serviceability limit state for shallow foundations is governed by vertical live and dead loads with load factors $\alpha_L = \alpha_D = 1.0$. The geotechnical resistance factor is calculated by Fenton et al. (2005a) to be

$$\phi_{gs} = \frac{\hat{F}}{\exp\{\mu_{\ln Y} + \beta\sigma_{\ln Y}\}} = \frac{[\alpha_L / k_L + \alpha_D R_{D/L} / k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}} \qquad [10]$$

where

$$Y = F\frac{\hat{E}}{\bar{E}} \qquad [11]$$

with mean and variance

$$\mu_{\ln Y} = \mu_{\ln F}, \qquad \sigma^2_{\ln Y} = \ln(1+v_Y^2) \qquad [12]$$

In Eq.'s [10] and [11], $\hat{F}$ is the design load, $F$ is the true (random) load, $\hat{E}$ is the soil's characteristic elastic modulus derived from soil samples, and $\bar{E}$ is the effective elastic modulus that, if underlying the footing, would yield the same settlement as actually observed. Notice that Eq. [10] is identical to Eq. [6], the only difference being the definition of the random variable $Y$.

For the lateral movement serviceability limit state of shallow foundations, the movement is assumed to be due to lateral wind load with serviceability load factor $\alpha_W = 1.0$ (CHBDC). The geotechnical resistance factor is given by Eq. [9] with $Y = W\hat{E}/\bar{E}$ where $E$ is now the elastic modulus resisting lateral movement.

## 2.4 Other Geotechnical Systems

All other limit states considered in the CHBDC follow similar equations for the resistance factor as presented in the three previous examples and are shown in Table 2. Please note that the random variable $Y$ is not actually needed as explained in Section 3. See Fenton and Naghibi, 2011, Naghibi and Fenton, 2011, and Naghibi et al., 2014, for reliability-based deep foundation design.

Table 2. Resistance factor equations

| Application | Limit state | Equation |
|---|---|---|
| Shallow Foundation | Bearing | $\phi_{gu} = \frac{[\alpha_L / k_L + \alpha_D R_{D/L} / k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Sliding and Passive Resistance | $\phi_{gu} = \frac{\alpha_W\sqrt{\ln(1+v_W^2)}}{k_W\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Settlement | $\phi_{gs} = \frac{[1/k_L + R_{D/L}/k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Lateral movement | $\phi_{gs} = \frac{\alpha_W\sqrt{\ln(1+v_W^2)}}{k_W\exp\{\beta\sigma_{\ln Y}\}}$ |
| Deep foundations | Compression | $\phi_{gu} = \frac{[\alpha_L / k_L + \alpha_D R_{D/L} / k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Tension/Lateral | $\phi_{gu} = \frac{\alpha_W\sqrt{\ln(1+v_W^2)}}{k_W\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Settlement | $\phi_{gs} = \frac{[1/k_L + R_{D/L}/k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Lateral deflection | $\phi_{gs} = \frac{\alpha_W\sqrt{\ln(1+v_W^2)}}{k_W\exp\{\beta\sigma_{\ln Y}\}}$ |
| Ground Anchor | Pull-out | $\phi_{gu} = \frac{\alpha_E\sqrt{\ln(1+v_E^2)}}{k_E\exp\{\beta\sigma_{\ln Y}\}}$ |
| MSE Reinforcement | Rupture and Pull-out | $\phi_{gu} = \frac{\alpha_E\sqrt{\ln(1+v_E^2)}}{k_E\exp\{\beta\sigma_{\ln Y}\}}$ |
| Retaining systems | Bearing | $\phi_{gu} = \frac{[\alpha_L / k_L + \alpha_D R_{D/L} / k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Overturning, Sliding, and Connections | $\phi_{gu} = \frac{\alpha_E\sqrt{\ln(1+v_E^2)}}{k_E\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Settlement | $\phi_{gs} = \frac{[1/k_L + R_{D/L}/k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Deflection | $\phi_{gs} = \frac{\alpha_E\sqrt{\ln(1+v_E^2)}}{k_E\exp\{\beta\sigma_{\ln Y}\}}$ |
| Embankments | Bearing | $\phi_{gu} = \frac{[\alpha_L / k_L + \alpha_D R_{D/L} / k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Sliding | $\phi_{gu} = \frac{\alpha_E\sqrt{\ln(1+v_E^2)}}{k_E\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Global Stability | $\phi_{gu} = \frac{\alpha_D\sqrt{\ln(1+v_D^2)}}{k_D\exp\{\beta\sigma_{\ln Y}\}}$ |
| | Settlement | $\phi_{gs} = \frac{[1/k_L + R_{D/L}/k_D]\sqrt{\ln(1+v_F^2)}}{(1+R_{D/L})\exp\{\beta\sigma_{\ln Y}\}}$ |

## 3 NBCC RESISTANCE FACTORS

It will be assumed that the resistance variability is the same for both bridges and buildings so that the term $\sigma_{\ln Y}$, appearing in the resistance factor equations, which includes the resistance variability, can be determined from the CHBDC calibration and then used to estimate the required NBCC resistance factors. Since the variability of $Y$ also includes the load variability, the load variability assumed by CHBDC will also creep into the NBCC resistance factor computation. However, the explicit use of load coefficient of variation (e.g., $v_F$ in Eq. [6]) is assumed to dominate the results, so that the approximation of the variability in $\ln Y$ using the CHBDC factors is assumed to be reasonable.

The resistance factors for the NBCC which correspond to the safety levels targeted in the CHBDC are thus obtained as follows;

1) For a given limit state, use the CHBDC resistance factor value along with the CHBDC design parameters (load and bias factors as well as coefficients of variation shown in Table 1) to solve for the unknown $\sigma_{\ln Y}$ value appearing in the resistance factor equations in Table 2.
2) Assuming that $\sigma_{\ln Y}$ is the same for both the CHBDC and the NBCC, as discussed above, compute the NBCC resistance factors using equations in Table 2, the NBCC design parameters (Table 1), and the common value of $\sigma_{\ln Y}$.

The resulting resistance factors are listed in Table 3. The lifetime target reliability, $\beta_{50}$, for the NBCC was held constant at a value of 3.0 (Becker, 1996). The CHBDC specifies an annual reliability index of 3.75 with the corresponding lifetime reliability index of 2.5 calculated assuming years are independent and using lifetime $n = 75$ years according to

$$\Phi(\beta_n) = \Phi^n(\beta_1) \rightarrow \beta_n = \Phi^{-1}\left(\Phi^n(\beta_1)\right) \qquad [13]$$

where $n$ is the design lifetime in years. However, it is unlikely that the resistances from year to year are actually independent. Therefore, the effective value of $n$ may be less than the actual design lifetime of the structure. The lifetime reliability index for the CHBDC could thus be anywhere between 2.5 and 3.75. Two lifetime target reliability indices for the CHBDC are considered in Table 3, the lowest level, $\beta_n = 2.5$, and a level assuming reasonably strong correlation between years of $\beta_n = 3.0$ to agree with the NBCC lifetime target. For $\beta_n = 3.0$ the effective number of years is $n = 15$.

## 4 DISCUSSION

When a target lifetime reliability of $\beta_n = 2.5$ for the CHBDC is achieved using the resistance factors shown in columns 4-6 of Table 3, then an increased lifetime target reliability in the NBCC of $\beta_{50} = 3.0$ necessarily results in smaller resistance factors than used in the CHBDC. This can be seen in columns 10-12 in Table 3. However, if it is assumed that the CHBDC target lifetime reliability is actually the same as the NBCC ($\beta_n = 3.0$), then the resistance factors are similar, if not identical. This can be seen in columns 7-9 of Table 3. When the limit state involves both dead and live loads, the effect of the load factors between the two codes cancel and the resistance factors become the same (i.e. $\alpha_L$ decreases while $\alpha_D$ increases in the NBCC). When the limit state involves loads other just dead plus live, the resistance factors differ due to differing load factors.

For example, when the limit state involves only earth pressure, such as overturning or sliding of a retaining system, the differing load factors between the two codes (see Table 1) lead to significantly different resistance factors regardless of the target reliability considered. Because the NBCC has a higher load factor for earth pressure ($\alpha_E = 1.5$ vs. the CHBDC load factor of 1.25), the NBCC resistance factors are higher by a factor of 1.5/1.25 = 1.2 for the same target lifetime reliabilities.

For limit states that involve wind loads alone, the resistance factors for $\beta_n = 3.0$ are the same since the wind load factors are identical (see Table 1).

## 5 CONCLUSIONS

The NBCC design parameters differ from the CHBDC in load factors, load combinations, design lifetime, as well as the target reliability. Changes in these factors will affect the resistance factors required to achieve a specified reliability.

In this paper, a Load and Resistance Factor Design (LRFD) framework is employed to calculate the NBCC resistance factors that lead to a target lifetime reliability of $\beta_{50} = 3.0$, assuming that the CHBDC resistance factors achieve one of two lifetime reliability targets. The results of this calibration are listed in Table 3. One critical issue is the choice of the lifetime reliability target for the CHBDC. The CHBDC itself specifies only an annual reliability target, while the literature relating to the calibration of the NBCC suggest a lifetime reliability target. To make these measures consistent, assumptions need to be made about how the statistics of annual load and resistance design factors relate to statistics of lifetime design factors. Since it is unlikely that loads and resistances are independent from year to year, it is unlikely that the lifetime target reliability index for the CHBDC is actually as low as $\beta_n = 2.5$. It is more likely to be at least equivalent to the reliability index assumed for the NBCC. As a result, the resistance factors specified in columns 7-9 in Table 3 are probably the most reasonable for the

NBCC, pending rigorous individual calibration exercises involving detailed probabilistic models of the geotechnical systems and limit states in question.

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Table 3. Table 6.2 of the CHBDC along with calibrated resistance factors for the NBCC (superscripts indicate column numbers)

| | | | CHBDC | | | NBCC ( $\beta_{50} = 3.0$ ) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Degree of Understanding | | | Degree of Understanding $\beta_n = 3.0$ for CHBDC | | | Degree of Understanding $\beta_n = 2.5$ for CHBDC | | |
| **Application**[1] | **Limit state**[2] | **Test Method/Model**[3] | **Low**[4] | **Typical**[5] | **High**[6] | **Low**[7] | **Typical**[8] | **High**[9] | **Low**[10] | **Typical**[11] | **High**[12] |
| Shallow foundations | Bearing, $\phi_{gu}$ | Analysis | 0.45 | 0.50 | 0.60 | 0.45 | 0.50 | 0.60 | 0.36 | 0.40 | 0.50 |
| | | Scale model test | 0.50 | 0.55 | 0.65 | 0.50 | 0.55 | 0.65 | 0.40 | 0.45 | 0.55 |
| | Sliding, $\phi_{gu}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 | 0.61 | 0.71 | 0.82 |
| | Frictional | Scale model test | 0.75 | 0.85 | 0.95 | 0.75 | 0.85 | 0.95 | 0.66 | 0.77 | 0.88 |
| | Sliding, $\phi_{gu}$ | Analysis | 0.55 | 0.60 | 0.65 | 0.55 | 0.60 | 0.65 | 0.46 | 0.51 | 0.56 |
| | Cohesive | Scale model test | 0.60 | 0.65 | 0.70 | 0.60 | 0.65 | 0.70 | 0.51 | 0.56 | 0.61 |
| | Passive resistance, $\phi_{gu}$ | Analysis | 0.40 | 0.50 | 0.55 | 0.40 | 0.50 | 0.55 | 0.31 | 0.41 | 0.46 |
| | Settlement, $\phi_{gs}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 | 0.64 | 0.75 | 0.87 |
| | | Scale model test | 0.80 | 0.90 | 1.00 | 0.80 | 0.90 | 1.00 | 0.75 | 0.87 | 0.99 |
| | Lateral movement, $\phi_{gs}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 | 0.61 | 0.71 | 0.82 |
| | | Scale model test | 0.80 | 0.90 | 1.00 | 0.80 | 0.90 | 1.00 | 0.71 | 0.82 | 0.93 |
| Deep foundations | Compression, $\phi_{gu}$ | Static analysis | 0.35 | 0.40 | 0.45 | 0.35 | 0.40 | 0.45 | 0.26 | 0.31 | 0.36 |
| | | Static test | 0.50 | 0.60 | 0.70 | 0.50 | 0.60 | 0.70 | 0.40 | 0.50 | 0.61 |
| | | Dynamic analysis | 0.35 | 0.40 | 0.45 | 0.35 | 0.40 | 0.45 | 0.26 | 0.31 | 0.36 |
| | | Dynamic test | 0.45 | 0.50 | 0.55 | 0.45 | 0.50 | 0.55 | 0.36 | 0.40 | 0.45 |
| | Tension, $\phi_{gu}$ | Static analysis | 0.20 | 0.30 | 0.40 | 0.20 | 0.30 | 0.40 | 0.14 | 0.22 | 0.31 |
| | | Static test | 0.40 | 0.50 | 0.60 | 0.40 | 0.50 | 0.60 | 0.31 | 0.41 | 0.51 |
| | Lateral, $\phi_{gu}$ | Static analysis | 0.45 | 0.50 | 0.55 | 0.45 | 0.50 | 0.55 | 0.36 | 0.41 | 0.46 |
| | | Static test | 0.45 | 0.50 | 0.55 | 0.45 | 0.50 | 0.55 | 0.36 | 0.41 | 0.46 |
| | Settlement, $\phi_{gs}$ | Static analysis | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 | 0.64 | 0.75 | 0.87 |
| | | Static test | 0.80 | 0.90 | 1.00 | 0.80 | 0.90 | 1.00 | 0.75 | 0.87 | 0.99 |
| | Lateral deflection, $\phi_{gs}$ | Static analysis | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 | 0.61 | 0.71 | 0.82 |
| | | Static test | 0.80 | 0.90 | 1.00 | 0.80 | 0.90 | 1.00 | 0.71 | 0.82 | 0.93 |
| Ground Anchor | Pull-out, $\phi_{gu}$ | Analysis | 0.35 | 0.40 | 0.50 | 0.42 | 0.48 | 0.60 | 0.32 | 0.38 | 0.50 |
| | | Test | 0.55 | 0.60 | 0.65 | 0.66 | 0.72 | 0.87 | 0.56 | 0.62 | 0.68 |
| Internal MSE reinforcement | Rupture, $\phi_{gu}$ | Analysis | 0.75 | 0.80 | 0.85 | 0.90 | 0.96 | 1.02 | 0.81 | 0.88 | 0.94 |
| | | Test | 0.85 | 0.90 | 0.95 | 1.02 | 1.08 | 1.14 | 0.94 | 1.01 | 1.08 |
| | Pull-out, $\phi_{gu}$ | Analysis | 0.35 | 0.40 | 0.50 | 0.42 | 0.48 | 0.60 | 0.33 | 0.38 | 0.50 |
| | | Test | 0.55 | 0.60 | 0.65 | 0.66 | 0.72 | 0.78 | 0.56 | 0.62 | 0.68 |
| Retaining systems | Bearing, $\phi_{gu}$ | Analysis | 0.45 | 0.50 | 0.60 | 0.45 | 0.50 | 0.60 | 0.36 | 0.40 | 0.50 |
| | Overturning, $\phi_{gu}$ | Analysis | 0.45 | 0.50 | 0.55 | 0.54 | 0.60 | 0.66 | 0.45 | 0.50 | 0.56 |
| | Base sliding, $\phi_{gu}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.84 | 0.96 | 1.08 | 0.75 | 0.88 | 1.01 |
| | Facing interface sliding, $\phi_{gu}$ | Test | 0.75 | 0.85 | 0.95 | 0.90 | 1.02 | 1.14 | 0.81 | 0.94 | 1.08 |
| | Connections, $\phi_{gu}$ | Test | 0.65 | 0.70 | 0.75 | 0.78 | 0.84 | 0.90 | 0.68 | 0.75 | 0.81 |
| | Settlement, $\phi_{gs}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 |
| | Deflection, $\phi_{gs}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.84 | 0.96 | 1.08 | 0.75 | 0.88 | 1.01 |
| Embankments (fill) | Bearing, $\phi_{gu}$ | Analysis | 0.45 | 0.50 | 0.60 | 0.45 | 0.50 | 0.60 | 0.36 | 0.40 | 0.50 |
| | Sliding, $\phi_{gu}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.84 | 0.96 | 1.08 | 0.75 | 0.88 | 1.01 |
| | Global stability – temporary condition, $\phi_{gu}$ | Analysis | 0.70 | 0.75 | 0.80 | 0.73 | 0.78 | 0.83 | 0.64 | 0.70 | 0.75 |
| | Global stability – permanent condition, $\phi_{gu}$ | Analysis | 0.60 | 0.65 | 0.70 | 0.62 | 0.68 | 0.73 | 0.53 | 0.59 | 0.64 |
| | Settlement, $\phi_{gs}$ | Analysis | 0.70 | 0.80 | 0.90 | 0.70 | 0.80 | 0.90 | 0.64 | 0.75 | 0.87 |
| | | Test | 0.80 | 0.90 | 1.00 | 0.80 | 0.90 | 1.00 | 0.75 | 0.87 | 0.99 |