Proc. of the 8th International Symposium on Geotechnical Safety and Risk (ISGSR)
Edited by Jinsong Huang, D.V. Griffiths, Shui-Hua Jiang, Anna Giacomini and Richard Kelly

doi: 10.3850/978-981-18-5182-7_18-008-cd

Risk Content in Some Existing Geo-Codes

Mabel Chedid[1] and Jean-Louis Briaud[2]

[1]WSP USA, New York, New York, 10119, USA.
E-mail: mabel.chedid@wsp.com
[2]Texas A&M University, College Station, Texas, 77843, USA. Word – word
E-mail: briaud@tamu.edu

Abstract: For many years, civil engineering design was based on the Allowable Stress Design (ASD) approach sometimes referred to as the Working Stress Design (WSD) approach. This traditional approach is deterministic and uses a single factor of safety defined as the ratio of the strength over the applied stress. In this way, ASD provides a certain level of safety by limiting the applied stress to a fraction of the maximum stress that the material can resist. In the 1980s, the emergence of structural reliability led to the development of Load and Resistance Factor Design (LRFD) as the basis of the new structural design codes. Consequently, the single factor of safety was replaced by a set of individual load and resistance factors to separately account for variability and uncertainty of the load and of the resistance. LRFD can be used to design structures with a desired reliability level or a target probability of failure. LRFD has also been used to develop geotechnical engineering codes, especially when it comes to foundations. Since the 1990s, the use of probabilistic methods in geotechnical engineering design has been increasing significantly. Today, the most commonly used tool for reliability-based design is LRFD, which has been adopted by many geo-structures design codes, including AASHTO LRFD Bridge Design Specifications, the Australian Standard for bridge design (AS 5100), the Canadian Highway Bridge Design Code (CHBDC), and Eurocode 7. Nevertheless, it has been recognized that the target reliability in LRFD should be adjusted to consider the consequences of a potential failure. These consequences can be described in terms of loss of life, environmental impact or economic loss or a combination of all three. A review of many existing geo-codes reveals a definite trend towards a risk-informed design where the risk includes not only the probability of failure but also the consequence of the failure. This paper defines risk in civil engineering, summarizes the extent to which the concept of risk is included in some existing civil engineering design codes, applies risk-based design concepts to a simple foundation design problem, and finally presents the authors' opinion on the development of risk-based designs as the next step beyond the reliability-based design era.

Keywords: Risk; reliability; factor of safety; design codes.

1 Risk Definition in Civil Engineering

The risk (R) associated with a civil engineering structure is defined here as the product of the probability of failure (P_f) of that structure times the value of the consequence (C) of that failure:

$$R= P_f \times C \quad (1)$$

The probability of failure (P_f) itself can be presented as the product of the probability of occurrence of an extreme event or a hazard, P(E), by the probability of failure if that event occurs, $P(F|E)$, also called vulnerability or fragility:

$$P_f=P(F)=P(E) \times P(F|E) \quad (2)$$

Eq. (1) above can be rewritten as:

$$Log\ (P_f)=-Log\ (C)+Log\ (R) \quad (3)$$

Thus, for a constant risk, the graph of the probability of failure versus the value of the consequence on log scales will be a straight line with a slope of -1 (Fig. 1). That line will depend on the chosen value of the annual risk R. Plots of log(P_f) versus log(C) are typically called f-N curves where N is the number of fatalities or the number of dollars lost and f is the annual probability that N fatalities or Ndollars are lost depending on how the consequence is quantified. In such a set of axes it becomes possible to locate zones or bubbles representing the performance of different civil engineering structures such as dams and offshore structures as well as general human activities such as car accidents and airplane crashes and even causes of human death due to illness such as cancer and heart attacks. Four lines are presented in Fig. 1, corresponding to risk levels of 1,000,000 \$/year, 100,000 \$/year, 10,000 \$/yr, and 1,000\$/year. The location of the bubbles in this chart helps defining the tolerable risk.

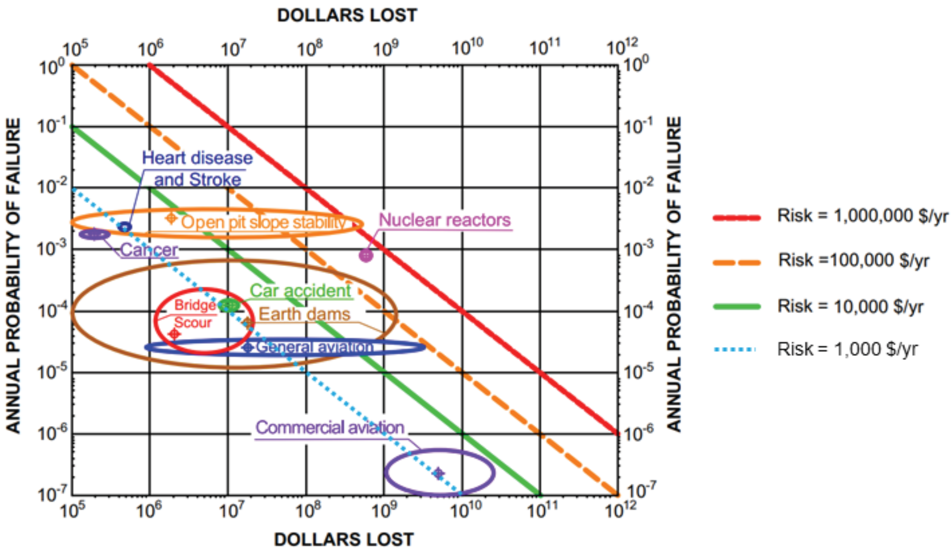

Figure1. Risk f-N chart for human activities (after the work of Timchenko et al. (2022))

2 Risk Content in Some Existing Geo-Engineering Design Codes

While many geotechnical engineering design codes have now adopted the LRFD approach, not all of these codes are truly based on the reliability-based design (RBD) approach. In reality, few countries, including Canada, Japan, the Netherlands and the USA have used reliability methods for calibrating the load and resistance factors to achieve a target reliability. Other countries have adopted factors that are calibrated empirically or based on traditional practice (ASD) and not by the means of a reliability analysis. Whether following a deterministic or a probabilistic approach, many codes have included some risk concepts and considerations in the design procedures.

2.1Slope stability

One major area of geotechnical engineering that has been remarkably slow in shifting from ASD to LRFD is slope stability analysis. Nonetheless, the ASD evaluation of slope stability can be made risk-informed by varying the recommended factor of safety as a function of the adverse consequences of the slope failure. For example, the Hong Kong geotechnical manual for slopes determines the design factor of safety on the basis of the severity of the associated loss of life and economic losses. The severity of the consequence is divided in 3 categories: negligible, low, and high (Geotechnical Engineering Office, 2011).

2.2 Foundation systems and soil retaining structures

On the other hand, LRFD has been better developed and possibly more accepted for the design of foundation systems and soil retaining structures. AASHTO LRFD bridge specifications and the Canadian Highway Bridge Design Code are two examples of codes containing a significant amount of risk considerations.
AAHSTO (2017) requires that all limit states satisfy the following basic design inequality:

$$\sum \eta_i \gamma_i Q_i \leq \phi\, R_n \tag{4}$$

Where Q_i is the effect of the ith load, R_n is the nominal resistance (often called characteristic resistances in geotechnical design codes), γ_i is the load factor of the ith load, ϕ is the resistance factor, and η_i is the load modifier of the ith load. The load modifier accounts for the bridge ductility, redundancy and failure consequences. It is calculated as the product of three factors: the ductility factor η_D, the redundancy factor η_R, and the importance factor η_I. η_I increases from 0.95 to 1.05 or greater as the operational classification of the bridge goes from relatively less important to critical or essential bridge. As expected, this classification is directly linked to the bridge role and consequences of failure.

The 2014 edition of the Canadian Highway Bridge Design Code (CHBDC) uses an advanced reliability-based model that considers spatial variability and failure consequence. As opposed to the AASHTO LRFD Bridge Design Specifications where the importance factor η_I is introduced on the load side, the CHBDC introduces a consequence factor on the resistance side to adjust the reliability level depending on the magnitude of the consequence. As a result, the overall resistance factor in the CHBDC accounts for both the resistance uncertainty and the failure consequences. The CHBDC adopts a three-tier classification scheme and provides three resistance factors for the following three different levels of failure consequences:

1. High consequence: associated with large safety and/or financial consequences. Examples include hospitals, schools, and lifeline highway bridges.

2. Typical consequence: characterizing most of the civil engineering designs.
3. Low consequence: associated with minor safety and/or financial consequences. Examples include low use structures.

The consequence factors are calibrated using the Random Finite Element Method (RFEM) for the bearing capacity design of shallow foundations and ultimate limit states (ULS) and serviceability limit states (SLS) designs of deep foundations to reach the target lifetime reliability in Table 1.The results of these RFEM studies indicate that the value of the consequence factor is independent of the limit state. The maximum failure probability p_m for high and low consequence levels for both limit states were calculated by scaling the typical value in the same way for both limit states;

$p_{m(\text{high consequence level})} = 0.5 \text{ x } p_{m(\text{typical})}$

$p_{m(\text{low consequence level})} = 5 \text{ x } p_{m(\text{typical})}$

Table 1 shows the consequence factor ψ specified in the 2014 CHBDC as 0.9 for high consequence designs and 1.15 for low consequence designs (Fenton et al., 2015).

Table1.2014 CHBDC consequence levels and associated target failure probability, reliability index and consequence factor

Consequence Level	ULS target lifetime failure probability	SLS target lifetime failure probability	ULS target lifetime reliability index	SLS target lifetime reliability index	Consequence Factor, **ψ**
High	1×10^{-4}	1×10^{-3}	3.7	3.1	0.9
Typical	2×10^{-4}	2×10^{-3}	3.5	2.9	1.0
Low	1×10^{-3}	1×10^{-2}	3.1	2.3	1.15

As a result, the ULS geotechnical design equation, within the LRFD framework, becomes:

$$\psi_u \varphi_{gu} \widehat{R}_u \geq \sum I_{iu} \alpha_{iu} \widehat{F}_{iu} \quad (5)$$

Where ψ_u is the consequence factor for the ultimate limit states, φ_{gu} is the geotechnical resistance factor, $\widehat{R}_u$is the characteristic ultimate geotechnical resistance,$I_{iu}$is the importance factor for the i^{th} load, and $\alpha_{iu}\widehat{F}_{iu}$ is the i^{th} factored load. For SLS, the subscript u is substituted by the subscript s.

The Eurocode follows the LRFD format and uses partial factors to transform the characteristic values of the basic variables (actions, ground parameters and resistances) into their corresponding design values. The Eurocode EN 1990:2002 (basis of structural design) introduces three reliability classes, going from RC1 with the lowest target reliability to RC3 with the highest target reliability (Table 2). These three reliability classes are associated with three consequence classes going from CC1 with low failure consequences (examples include agricultural buildings and greenhouses where people do not normally enter) to CC3 with high failure consequences (examples include highly populated buildings such as concert halls). It should be noted that the central values in the Eurocode are the 50 years values, whereas the annual year values are derived using the assumption of independence between years.

Table2. Reliability classes (Appendix B of EN 1990:2002)

Reliability Class	Minimum values for β	
	1 year reference period	50 years reference period
Reliability Class 3 (RC3)	5.2	4.3
Reliability Class 2 (RC2)	4.7	3.8
Reliability Class 1 (RC1)	4.2	3.3

The LRFD format is also adopted by the Australian Standard for bridge design AS 5100 where the geotechnical strength reduction ϕ_g for a certain geotechnical system and limit state is selected from a specified range to reflect the variations in soil conditions, the model used to evaluate geotechnical resistance, construction quality, importance of the structure and the consequence of its failure. For example, the geotechnical strength reduction factor used for the design of shallow foundations against ultimate limit states (overall stability, bearing capacity and sliding) are selected from the range of factors presented in Table 3 based on the guidelines in Table 4. The standard provides similar tables and guidelines for the selection of resistance factors associated with the ultimate limit states (strength and stability) of other geotechnical systems such as anchors, retaining walls and buried structures.

Table3. Ranges of geotechnical strength reduction factor (ϕ_g) for shallow footings (AS 5100.3)

Ultimate geotechnical strength assessment Method	Range of ϕ_g
Analysis using appropriate advanced in situ tests	0.50-0.65
Analysis using appropriate advanced lab tests	0.45-0.60
Analysis using Cone Penetration Test (CPT)	0.40-0.50
Analysis using Standard Penetration Test (SPT)	0.35-0.40

Table4. Guidelines for the selection of geotechnical strength reduction factor (ϕ_g) for shallow footings (AS 5100.3)

Lower end of ϕ_g range	Higher end of ϕ_g range
Limited site investigation	Comprehensive site investigation
Simple calculation methods	More sophisticated design methods
Limited construction control	Rigorous design control
Severe failure consequences	Less severe failure consequences
Significant cyclic loading	Mostly static loading
Permanent structure foundations	Temporary structure foundations
Design parameters using published correlations	Design parameters using site-specific correlations

2.3 Dam safety

A geotechnical area where a risk-informed framework is well advanced is dam safety. In addition to the traditional engineering analysis and judgment, dam safety programs incorporate risk analysis to economically reduce risk exposure levels below tolerable risk limits. The US Army Corps of Engineers (USACE) is currently working with the US Bureau of Reclamation (USBR) and the Federal Energy Regulatory Commission (FERC) to develop tolerable risk guidelines. Meanwhile, the USACE adopts a combination of the 2011 USBR guidelines, 2003 Australian National Committee on Large Dams (ANCOLD) guidelines and the 2006 New South Wales Governmental Dam Safety Committee guidelines (NSW-DSC). All of these tolerable risk guidelines share similar fundamental characteristics with some slight differences. For example, the USBR adopts two risk guidelines represented on a risk guideline chart (Fig. 2) known as the f-N chart. In this chart, N stands for the number of potential life losses and f stands for the annual probability that N lives are lost. The annualized failure probability guidelines are equivalent to the threshold level of individual risk adopted by the USACE and is presented by a horizontal line at 10^{-4} on the f-N chart in Fig. 2. The annualized life loss guideline is equivalent to the concept of societal risk by the USACE. This guideline sets a constant tolerable risk of 0.001 fatalities per year which can be translated into a straight line of slope -1 on the f-N chart. The USBR presents risk estimates as points on the f-N chart and compares them to these two risk guidelines. The goal is to ensure that the point(s) for the case at hand is below the criteria lines (USBR, 2011).

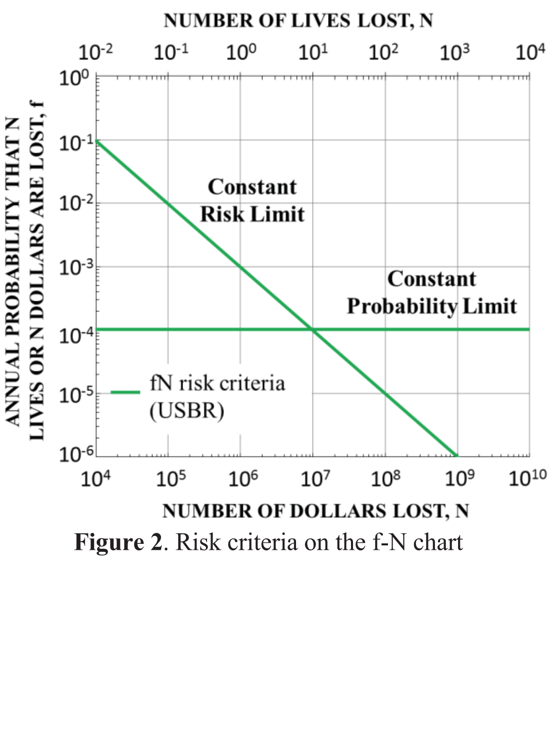

Figure 2. Risk criteria on the f-N chart

2.4Safety assessment of flood protection structures

The Netherlands'new legal safety standards for flood protection structures are based on the maximum allowable probabilities of flooding. These standards apply to segments ranging from about 5 to 40 km in length. Thesegment maximum allowable flooding probabilities are calculated based on flooding acceptable risk and estimated consequences and range from 1/100 per year to 1/100,000 per year. The "Legal Safety Assessment 2017", with the Dutch acronym WBI 2017, provides a set probabilistic and semi-probabilistic tools and procedures for assessing the compliance of flood defenses with these standards. To ensure consistency between the semi-probabilistic (or partial factor) approach and probabilistic approach, a standardized procedure for reliability-based calibration of the partial factors was developed for various failure mechanisms (Jongejan, 2017). A segment may include various sections, having different cross-sectional lengths. Thecross-sectional target reliabilities for the failure mechanism under consideration(β_T)is derived from the segment-level target reliability so that the combined probabilities of the various failure mechanisms for the segment do not exceed the flooding maximum allowable probability. Since it is not practical to vary all partial factors with varying values ofβ_T for different cross-sections and segments, WBI 2017 derives the values of all but one partial factor from a fixed reliability index (β_{basis}) and reconciliates the difference between β_{basis} and β_T with a β_T-dependent partialfactor ($\gamma_{\beta T}$). The factor $\gamma_{\beta T}$ can be either applied on the resistance side similarly to the consequence factor in the CHBDC, or on the load side similarly to the importance factor in theAASHTO LRFD Bridge Design Specifications.For failure mechanisms where a β_T-dependent partial factor complicates the semi-probabilistic assessment, such as the failure mechanisms of dune erosion and grass revetment failure, the design loads are defined as a function ofβ_T.

3 Example of Risk-Based Shallow Foundation Design

The concept of risk-based geotechnical engineering design is applied to a very simple example in which the width B of a spread footing, designed against bearing failure, is considered (Fig. 3). In this example, the spread footing is supporting a bridge pier generating a characteristic dead load L_D = 90000 kN and a characteristic live load L_L= 30000 kN. The length of this footing L is the width of the bridge or 30 m. The underlying soil has a characteristic pressuremeter limit pressure p_L = 1000 kPa.

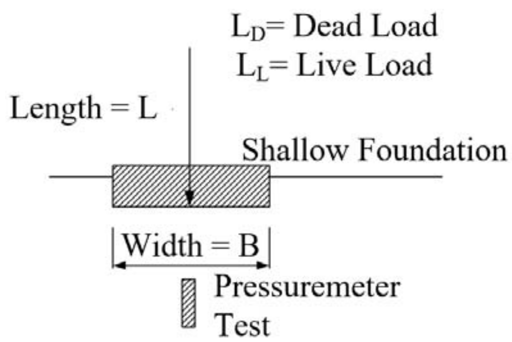

Figure 3.Spread footing design example

The design aims to satisfy the 2014 CHBDC equation:

$$\psi\varphi_u R_u \geq \alpha_D L_D + \alpha_L L_L \quad (6)$$

where ψ is the consequence factor, φ_u is the resistance factor for the ultimate limit state having a value of 0.5 for a typical site understanding, R_u is the characteristic ultimate geotechnical resistance, α_D is the dead load factor taken as 1.2, L_D is the characteristic dead load of 90000 kN, α_L is the live load factor taken as 1.7, and L_L is the characteristic live load of 30000 kN. The characteristic ultimate geotechnical resistance R_u is calculated by simply multiplying the characteristic pressuremeter limit pressure p_L by the footing area A=BL, i.e., $R_u = p_L BL$ The characteristic ultimate geotechnical resistance R_u can be expressed by: $R_u = 30000B$ where R_u is in kN and B is in m.

Replacing the known variables by their values in Eq. (6) results in:

$$B\ (m) \geq \frac{10.6}{\psi} \quad (7)$$

Eq. (7) shows how two bridges only differing by their failure consequences will have different consequence factors ψ and subsequently different design width B. To demonstrate this concept, two bridges are considered:

1. Bridge 1 with a failure consequence amounting to $ 75 M
2. Bridge 2 with a failure consequence amounting to $ 750 M

Referring to Fig. 1 and aiming to satisfy a tolerable risk level of 1000$/year (dottedblue line in Fig. 1), the annual probability of failures for Bridge 1 and Bridge 2 are P_{f1}= 1.33 x 10^{-5} and P_{f2}= 1.33 x 10^{-6}, respectively. The lifetime maximum acceptable failure probability, p_m, is calculated from the yearly maximum acceptable probability of failure, P_f, as follows:

$$p_m = 1 - (1 - P_f)^N \tag{8}$$

where N is the design life of the bridge in years, taken as 75 years. Eq. (8) assumes independence between years and results in the upper bound maximum acceptable lifetime failure probabilities of p_{m1}= 10^{-3} and p_{m2}= 10^{-4} for Bridges 1 and 2, respectively. These probabilities are used to determine the consequence factor (Fig. 4). Note that Fig. 4 is based on the results of random finite element method simulations (Fenton et al., 2015). For a typical site understanding and a coefficient of variation of the soil shear strength, CoV = 0.23, Fig. 4 gives $\psi_1 = 1.13$ and $\psi_2 = 0.93$ for Bridges 1 and 2, respectively. Applying Eq. 7 with $\psi_1 = 1.13$ and $\psi_2 = 0.93$, gives the minimum required footing width B for Bridges 1 and 2 as 9.4 m and 11.4 m, respectively, as shown in Table 5.

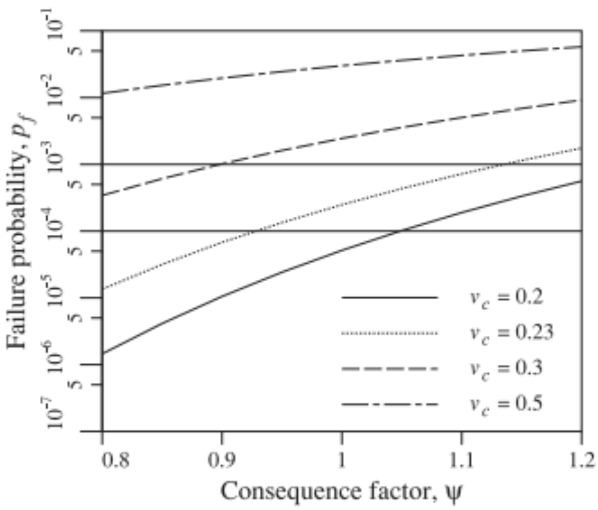

Figure 4.Failure probability versus consequence factor for a typical site understanding and φ_u=0.5 (Fenton et al., 2015)

Table5. Risk table and footing width for bridges 1 and 2.

Bridge	Failure Consequence, C ($)	Target Annual probability of Failure, P_f	Target Lifetime probability of Failure, p_m	Consequence Factor, ψ	Footing width, B (m)
1	75 M	1.33 x 10^{-5}	10^{-3}	1.13	9.4
2	750 M	1.33 x 10^{-6}	10^{-4}	0.93	11.4

This simple example illustrates how the geotechnical design should depend on the value of the consequence. This can be done by modifying the resistance factor based on the tolerable risk or by adding a consequence factor, as in the example above.

4 Our Opinion

The authors advocate for a rapid but thoughtful development of more risk content in civil engineering design codes. As a step towards more complex inclusion of risk in design approaches, the authors propose that all civil engineering structures be designed for the same risk. Furthermore, we propose that this risk should be $1,000/year.

Acknowledgments

The authors wish to thank the members of the Geo-Institute Risk Design Standard Task Force: Greg Baecher, Dennis Becker, Christi Bumpas, John Christian, Edward Clukey, Vic Donald, Robert Gilbert, Anand Govindasamy, Vaughan Griffiths, Jim

Harris, Karen Knight, Dimitris Konstantakos, Anna Lemnitzer, Mike Lewis, Erik Loehr, Allen Marr, Marty McCann, Zenon Medina-Cetina, Silas Nichols, Bob Patey, Rodolfo Sancio, Muhannad Suleiman, Bill Yu.

References

AASHTO. (2017). AASHTO LRFD Bridge Design Specifications.*American Association of State Highway and Transportation Officials, Washington, DC*.

Baecher, G. B., and Christian, J. T. (2003).Reliability and Statistics in Geotechnical Engineering,*Wiley, Chichester, U.K*.

Briaud J.-L., and Gardoni P., Yao C. (2013). Statistical, Risk, and Reliability Analyses of Bridge Scour.*Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, ISSN 1090-0241/04013011(10).

Chedid, M., and Briaud, J.-L. (2017). Risk Content in Existing Design Codes. *Report associated with the work of the Geo-Institute Risk Design Standard Task Force*, pp 74.

EN 1990:2002- Eurocode: Basis of structural design.*European Committee for Standardization (CEN), Brussels, Belgium*.

Fenton, G. A., Naghibi, F., Dundas, D., Bathurst, R. J., and Griffiths, D. V. (2015). Reliability-based Geotechnical Design in 2014 Canadian highway Bridge Design Code. *Canadian Geotechnical Journal*, 53(2), 236-251.

Geotechnical Engineering Office. (2011). Geotechnical Manual for Slopes.*The Government of the Hong Kong Special Administrative Region*.

Gilbert, R.B. (2017). Beyond Protecting the Public from Risk,*The Bridge, National Academy of Engineering, Washington, D.C.*, Volume 47, Number 1, 30-35.

Jongejan, R. (2017). Reliability-based code calibration and semi-probabilistic assessment rules for the WBI2017,*Rijkswaterstaat, Netherlands*.

Standards Australia. (2017). Bridge design. Part 3: Foundations and soil supporting structures.*Australian Standard AS 5100.3–2017 (second edition), Sydney, Australia*.

USBR. (2011). Interim Dam Safety Public Protection Guidelines, *U.S. Bureau of Reclamation, Denver, Colorado*, August 2011.

Timchenko, A., Shidlovskaya, A., and Briaud, J.-L. (2022). Tolerable Risk Chart. *Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering. Sydney, Australia*, May 2022.