Reliability-Based Performance Indicators for Structural Members

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Abstract: The implementation of reliability methods for designing new structures and assessing the safety and evaluating the performance of existing structures and infrastructure systems has gained widespread acceptance. Consequently, reliability-based design specifications in the form of load and resistance factor design (LRFD) methods have dominated the development of current codes and standards. This paper reviews the reliability-based performance criteria used to calibrate design and evaluation codes and standards for assessing the strength, serviceability, and fatigue resistance of structural components. The review shows that large differences exist in the target reliability levels adopted for evaluating the strength of various types of structural members and materials. These differences result from many factors, including (1) intended structure design and service life; (2) expected member modes of failure (e.g., ductile or brittle); (3) importance of the individual member to overall system integrity (secondary member, column, or connection); (4) experiences with previous designs; (5) material and construction costs; (6) structure type and occupancy; and (7) risk tolerance of the engineering community and the public within a code's jurisdiction. For other than seismic hazards, current specifications remain primarily focused on the evaluation of individual structural members and components, although recently proposed performance-based design (PBD) procedures apply varying target member reliability levels that depend on structure categories, modes of failure, and required levels of structural performance. The implementation of reliability-based durability criteria in design standards is still a subject of research owing to difficulties encountered in modeling material degradation mechanisms and their interactions and in the collection and mapping of long-term site-specific data on degrading agents. Because of large epistemic uncertainties, the evaluation of the fatigue safety of structural components in engineering practice still relies on conservative basic models of damage accumulation using *S-N* curves or basic fracture mechanics crack growth models. Overall, reliability-calibrated structural standards are producing designs that offer a good balance between safety and cost. The future implementation of risk-based methods will further enhance the ability to meet structure-specific performance requirements set by owners and users. **DOI: 10.1061/(ASCE)ST.1943-541X.0001546.**

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Introduction

Over the last four decades, the implementation of reliability methods for assessing the safety and evaluating the performance of structural systems has gained widespread acceptance in the structural engineering community. In particular, reliability-based specifications have been adopted for the routine design of many structural systems, including buildings, offshore platforms, nuclear power plants, highway bridges, and wind turbines. Although there are many similarities between existing reliability-based specifications, the inherent performance indicators and criteria may differ. The objective of this paper and performance indicators for structural systems and infrastructure networks is to review the development and implementation of reliability-based performance criteria

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(RBPC) for civil structures and infrastructure systems. This paper reviews reliability-based performance criteria and indicators for individual members and discusses their merits and limitations. Member reliability is consistent with the design basis for most structures, with the exception of seismic design because recent seismic design standards are based on system performance rather than on member performance. Recently adopted and proposed system performance indicators for seismic and other hazards are addressed by Ghosn et al. (2016).

As the implementation of structural reliability methods expands to new applications, a review of the performance indicators and criteria in use, and those upon which existing specifications are based, is offered to help engineers assess the adaptability of current approaches to new situations. Approaches adopted in the United States and other countries to implement reliability-based performance criteria (RBPC) for various structural systems are compared. Specifically, indicators used to evaluate the performance of a structural member for strength, serviceability, durability, and the effect of cyclic fatigue are reviewed. Recently introduced modifications to member strength criteria based on concepts of risk are also discussed. Risk methods address the consequences of failures, which in this context, are related to component strength and member reliability.

Background

Structures are designed to perform in a satisfactory manner by meeting serviceability and functionality requirements over their intended service lives while ensuring the safety of the public at minimum construction, maintenance, and other costs. Thus, the design process aims to (1) ensure adequate performance under service load conditions; (2) reduce the probability of localized failures; (3) prevent structural collapse or irreversible serious damage under natural and human-made loads and hazards; (4) assure structural durability; and (5) minimize costs. Early design codes and guidelines, such as AISC (1989) and AASHTO (2002), aimed to achieve these goals by specifying prescriptive design requirements using safety factors selected based on experience with successful designs and engineering judgment. Although these codes and standards usually produced functional and safe structures, the degree of safety achieved was not consistent because these codes did not adequately account for the variability in uncertainties associated with the strength of structures, or the intensities and effects of gravity loads and hazard events.

Reliability-based approaches provide a framework for assessing and quantifying the safety of structures by explicitly accounting for aleatory and epistemic uncertainties. Aleatory uncertainty refers to the natural randomness or inherent variability associated with a parameter that cannot be reduced. Epistemic uncertainty reflects imperfect knowledge of any physical phenomenon or property/characteristic that affects structural performance; this type of uncertainty can be reduced with improved knowledge, advanced modeling, further experimentation, and additional data.

Reliability methods have been used to calibrate load and resistance factor design (LRFD) codes and standards that provide adequate safety margins and reduce the probability of failure initiation resulting from inadequate member capacities or excesses in loads or demands when designing new structures or assessing the safety of existing structures. Target reliability levels for members are set based on experience with the performance of existing structures, the consequences of member failures, and the cost of construction. In addition to the possibility of increases and changes in loads and hazards over time, structural members with adequate performance at the start of service or at particular points in time may, eventually, become subject to reduced safety levels because of degradation mechanisms that affect their structural components. These mechanisms may be related to environmental phenomena that reduce structural strength or repetitive loads that lead to fatigue and fracture. Reliability-based codes and standards can also consider the effect of these changes on structural safety.

Although reliability-based design and performance evaluation methods are sufficiently general to be applicable for all types of structures and infrastructure systems, actual implementation in engineering practice and codified design has advanced at different rates depending on the availability of data, the type of application, and the engineering community and industry. The following sections present a review and critical appraisal of member-oriented reliability-based performance assessment methods, their implementation in different codes, standards, and specifications, and their application in engineering practice. The paper addresses reliability-based performance criteria in current practice for evaluating the strength of individual members or components and the consideration of a component's failure. Approaches for including serviceability, durability, and cyclic fatigue in current structural design specifications are also discussed.

Reliability-Based Design Methods for Structural Members

Reliability of structural members is expressed in terms of a margin of safety, or limit state function, Z

$$Z = R - Q \tag{1}$$

where R = resistance or load-carrying capacity; and Q = maximum load effect that the member may be exposed to within its expected design and service life. The probability of member failure, P_f, or limit state exceedance when $Z \leq 0$, may be determined through a convolution integral

$$P_f = \Pr[Z \leq 0] = \int_0^\infty F_R(q) f_Q(q) dq = 1 - \Re = \Phi(-\beta) \tag{2}$$

where $\mathcal{R}$ = reliability, or the complement of the probability of limit state exceedance, P_f; $\Phi(.)$ = standard normal cumulative distribution function; β = reliability index; $F_R(.)$ = cumulative distribution function of the resistance R; and $f_Q(.)$ = probability density function of the load effect, Q. The last equality, which is exact for normal distributions, is often used to obtain an approximate value of the reliability index for all distributions.

Various statistical analysis techniques are available to model the probability distributions of the random variables in the safety margin, Z, based on measured experimental and field data (Ang and Tang 2006). Generally speaking, there are limited numbers of laboratory model or full-scale tests for the resistance, R, of structural members and components because of costs and equipment requirements. Also, scaled model tests may not represent the actual behavior of a member within a structure because of the effect of scaling, boundary conditions, detailing, and attachments. Combined with the scarcity of data on the occurrence of extreme hazards and the difficulty of modeling their load effects, the issue with representing the behavior of structural members leads to problems when seeking to model the tail ends of resistance and load probability distributions. For these reasons, Thoft-Christensen and Baker (1982) recommend using physical reasoning about the nature of each particular random variable to guide the choice of a probability distribution. For example, a lognormal distribution may be adequate when R can be modeled as the product of a number of underlying independent random variables. A normal distribution is used when R is obtained from the sum of independent random variables. When the resistance is governed by the size of the largest defect, a Weibull distribution may be a good choice (Thoft-Christensen and Baker 1982). Using similar reasoning, the effects of permanent (dead) loads are usually modeled by normal distributions. Live loads and transient load effects are commonly modeled by normal, lognormal, or extreme value distributions when the structure is expected to safely support the largest extreme load from successive reference periods. For example, the extreme value Type I distribution, also known as the Gumbel distribution, is used when modeling the largest value of underlying independent identically distributed load effects having an exponentially decaying tail (Ang and Tang 2006).

If R and Q are functions of several basic independent or correlated random variables, it can be very demanding to evaluate P_f exactly by analytical methods. Instead, various numerical algorithms and simulation techniques are often used. Structural reliability textbooks, such as those by Ang and Tang (2006), Melchers (1999b), Thoft-Christensen and Baker (1982), and Nowak and Collins (2012) describe the most classic reliability analysis methods, and researchers are continuously introducing improvements to their efficiencies and accuracies.

Among the various available techniques, first-order reliability methods (FORM) have been regularly applied, although more advanced techniques including second-order methods (SORM) are also available. Because of advances in computer speeds and efficiencies, variations on the Monte Carlo–simulation (MCS) method are now commonly used by researchers and practitioners. Specifically, FORM and MCS algorithms were used by many code writing agencies to develop load and resistance factor design (LRFD) methods.

LRFD standards apply separate factors on the resistance and load effects of design equations to reflect the uncertainties in each parameter while providing a proper balance between the reliability and cost of a structural design. The LRFD safety-check equation, which must be satisfied for all specified load effects and their combinations, takes the following form:

$$\phi R_n \geq \sum \gamma_i Q_{ni} \tag{3}$$

where R_n = nominal strength; ϕ = resistance factor; Q_{ni} = nominal load effect for load i; and γ_i = associated load factor. Load and resistance factors (ϕ and γ_i) are calibrated so that structural components and their connections have adequate strength to resist the applicable load combinations and provide consistent levels of reliability for each limit state (e.g., flexure and buckling) by meeting a target reliability index, β_T. In most cases, appropriate target reliability index values have been based on existing practice. That is, if the safety performance of a representative set of existing structures has been found satisfactory, then the average reliability index obtained from this set is used as the target that an LRFD code or standard should satisfy.

Transient loads in Eq. (3) are usually set in terms of a return period, T_r (in years), which for a Poisson process is the inverse of the mean occurrence rate, ν, of the loading event. For small values, ν is approximately equal to the annual probability of load level exceedance, P_a such that $P_a \approx \nu = 1/T_r$. A hazard follows a Poisson process if the occurrences of loading events are statistically independent and the probability of simultaneous occurrences is negligibly small. Although the occurrence of certain hazards such as seismic events may not be strictly speaking independent, the Poisson model is assumed when no specific data is available for simplicity.

LRFD standards are calibrated for structure service lives that typically vary between 20 and 100 years even if, in some instances, the reliability index, β, or the probability of limit state exceedance, P_f, are expressed in terms of annual values. Yet, return periods for nominal design loads are usually much higher than service lives to provide an appropriate margin of safety. For example, the ASCE 7-10 (ASCE/SEI 2010) wind load provisions for a 50-year design life are based on 700-year wind speed maps associated with a load factor $\gamma_W = 1.0$. This combination of return period and load factor leads to the same designs and reliabilities that were obtained when previous editions used 50-year wind speed maps associated with a load factor $\gamma_W = 1.6$. The primary reason for the change is to provide consistent probability of occurrence of loads in hurricane prone regions and in nonhurricane regions. If statistical independence applies, the probability, P_N, that the nominal load will be exceeded within the design or service life (say, N years) can be approximated by

$$P_N = 1 - (1 - P_a)^N \tag{4}$$

To account for the possibility of simultaneous occurrences of different hazards, many codes and standards, such as ASCE/SEI 7-10 (ASCE/SEI 2010) designate load factors for load combinations using the principle of "principal action–companion action" (Galambos et al. 1982; Ellingwood et al. 1980, 1982). The approximate but simple to apply principle is based on the notion that the maximum combined load (or load effect) during a service period occurs when one time-varying load attains its maximum value while the remaining time-varying loads are at their frequent (or arbitrary-point-in-time) values (Turkstra and Madsen 1980).

In addition to sets of load and resistance factors that cover pertinent members and hazards, a few design manuals, such as ASCE 7-10 (ASCE/SEI 2010) and CAN/CSA-S6-06 (CAN/CSA 2012), provide equations for calculating load and resistance factors for components and loads not specified in the standards. The equations are also applied when more stringent target reliabilities than those in the standards are stipulated by structure owners for special projects. For example, Eq. (5) is provided in ASCE/SEI 7-10 (ASCE/SEI 2010) to determine the load factor γ_i for a given target reliability, β_T

$$\gamma_i = \frac{\mu_{Q_i}}{Q_{ni}}(1 + \alpha_{Qn}\beta_T V_{Qi}) \tag{5}$$

where μ_{Qi} = mean of load i; Q_{ni} = nominal value of load i as specified in the standard; V_{Qi} = coefficient of variation in the load; and α_{Qi} = sensitivity coefficient that is approximately equal to 0.8 when Q_i = principal action and 0.4 when Q_i = companion action. Eq. (5) was derived for cases in which the load effect Q_i is normally distributed (Melchers 1999b) but is still used otherwise. Eq. (6) is proposed by ASCE/SEI 7-10 (ASCE/SEI 2010) to determine the resistance factor ϕ

$$\phi = \frac{\mu_R}{R_n}\exp(-\alpha_R \beta_T V_R) \tag{6}$$

where μ_R = mean strength; R_n = code-specified nominal strength; V_R = coefficient of variation in strength; and α_R = sensitivity coefficient approximately equal to 0.7. Eq. (6) is valid when the distribution is lognormal.

Code Calibration Process

Despite the simplifying assumptions and data limitations, LRFD standards calibrated to achieve target reliabilities based on past performance were found to be robust as they minimize the effects of inadequacies in the statistical databases and probability models (Ghosn and Moses 1986). For these reasons, target reliability levels have been based on successful previous designs rather than on actuarial evaluations of the probability of limit state exceedance, P_f. Also, because of reserve strength and multiple load paths, exceeding the limit state function for components does not necessarily imply that the structure will actually collapse. Therefore, many codes and standards establish performance criteria in terms of the reliability index, β, rather than P_f because of the different connotations that the term, failure, associated with P_f may imply.

The process of calibrating an LRFD procedure involves the following steps: (1) identify a representative set of structural designs that have demonstrated adequate performance by providing good safety at reasonable cost; (2) assemble statistical data for all the variables in Eq. (1); (3) analyze the reliability of the members of the representative set; and (4) apply the target reliability extracted from the population to find the resistance ϕ and load factors γ_i for use in Eq. (3).

The calibration process is illustrated using the following simplified numerical example:

1. Assume that three existing structures having components with nominal resistance to load ratios $R_n/Q_n = 1.57$, 1.74, and 1.94 have shown satisfactory performance.
2. A statistical analysis of the resistances and loads shows that the resistance follows a lognormal distribution with a bias between the mean and the nominal value $b_R = \mu_R/R_n$ equal to 1.15 and a coefficient of variation (COV) $V_R = 10\%$ while the load follows a normal distribution with a bias $b_Q = \mu_Q/Q_n = 1.0$ and a COV, $V_Q = 20\%$.
3. The reliability analysis of the three structural members shows that the reliability indexes are $\beta = 2.98$, 3.53, and 4.04 for an average value $\beta = 3.52$.
4. Taking the average value from the three structures as the target reliability $\beta_T = 3.52$ that a new design code aims to match, Eqs. (5) and (6) lead to a resistance factor $\phi = 0.90$ and a load factor $\gamma = 1.56$.

To demonstrate the robustness of the calibration process, it is assumed that further investigation of additional data showed that the resistance bias and COV are actually $b_R = 1.0$ and $V_R = 14\%$. Using the new data, the reliability indexes of the original three structures' components would change to $\beta = 1.92$, 2.35, and 2.72 for an average value $\beta = 2.33$. These reliability indexes are significantly lower than those obtained with the original data set. Yet, if the three structures have performed satisfactorily and the new average is set as the target index $\beta_T = 2.33$, Eqs. (5) and (6) would lead to $\phi = 0.80$ and $\gamma = 1.37$. Adjusting these factors by scaling the right and left sides of Eq. (3) by the same value would lead to a resistance factor $\phi = 0.90$ and a load factor $\gamma = 1.55$, which are essentially the same as the factors obtained earlier with the different statistical database.

This simple exercise demonstrates the importance of using past experience with successful designs as part of the calibration of LRFD equations, especially in an environment in which the statistical databases have known limitations.

Implementation in Design Standards

Table 1 summarizes some of the reliability targets and intended design or service lives for component strength limit states for a sample of LRFD codes and standards calibrated using processes similar to that described in the simplified illustrative example. Building standards are typically based on a 50-year design or service life, standards for offshore and wind turbines are based on a 20-year service life, the AASHTO LRFD bridge specifications (AASHTO 2012) and the Canadian CSA-S6 code (CAN/CSA 2012) are based on a 75-year design life and the Load and Resistance Factor Rating (LRFR) method for evaluating existing bridges in the AASHTO MBE (AASHTO 2011) is based on a 5-year rating period. The service life only pertains to the probabilistic determination of the intensity of the maximum transient load that the structural member is expected to withstand with cumulative distributions approximated per Eq. (4).

One of the earliest comprehensive structural standards based on LRFD methods was developed by AISC's load and resistance factor design method (AISC 2011) that was calibrated to meet a reliability index target $\beta_T = 3.0$ for structural members (Ravindra and Galambos 1978) and a target reliability index of $\beta_T = 4.5$ for connections (Fisher et al. 1978). American Concrete Institute ACI-318 (ACI 2011) standards are based on a target reliability index $\beta_T = 3.5$ for reinforced concrete beams in bending and shear, whereas reinforced concrete slabs are based on a target reliability index $\beta_T = 2.5$. For tied concrete columns, the reliability index is $\beta_T = 4.0$ (Szerszen and Nowak 2003). A target reliability index β_T on the order of 2.3 to 2.5 was selected for wood flexural members subject to dead plus live loads or dead plus snow loads, with duration of load (DOL) effects included for the bending moment limit state (American Wood Council 2012; Ellingwood and Rosowsky 1991). Aluminum standards were calibrated with a target reliability index $\beta_T = 2.5$ for yielding and buckling and $\beta_T = 3.0$ for fracture limit states. Values between $\beta_T = 2.0$ and 2.5 were chosen for secondary members subjected to wind or seismic loads, e.g., Aluminum Association's ADM-105 (AA 2010; Kissell and Ferry 2002). All U.S. standards are closely coordinating their activities with those undertaken under the auspcies of ASCE 7-10 (ASCE/SEI 2010). The trend in these U.S. standards is that the target reliability levels for concrete and steel structures that are primary materials in large buildings are higher than those of wood and aluminum, which are usually used in smaller structures whose failure may be less dramatic and affect fewer potential occupants. Also, it is clear that connections, fracture, and buckling limit states have higher target reliabilities because their failures are brittle leading to severe consequences. Increasing the reliability levels of connections will entail minimal additional cost and lead to measurable improvements in structural safety. Secondary members by their very nature are less important to the integrity of the structure and can be designed with lower reliability targets.

As presented by Arangio (2012), a main target annual reliability index $\beta_T = 4.7$ corresponding to $\beta_T = 3.8$ in 50 years is used in Eurocode 1 (CEN 1993) for the design of typical structures. The target is allowed to vary between $\beta_T = 4.3$ and 3.3 in 50 years, depending on the structure's importance. An annual reliability index $\beta_T = 4.2$ is specified in the document issued by the Joint Committee on Structural Safety (JCSS) for the assessment of existing structures, (JCSS 2001). ISO 13822 (ISO 2010) specifies a reliability index that varies between $\beta_T = 2.3$ and 4.3 in 50 years,

Table 1. Sample of Target Member Reliability Indexes for Strength Limit States in Various Codes and Standards

Code	Target reliability index	Design or service life (years)
AISC (2011) (U.S.)	3.0–4.5	50
ACI (2011) (U.S.)	2.5–4.0	50
AASHTO LRFD (2012) (U.S.)	3.5	75
AASHTO MBE (2011) (U.S.)	2.5	5
CAN/CSA-S6 (CAN/CSA 2012) (Canada)	3.75	75
CEN (2002) Eurocode (Europe)	3.3, 3.8 or 4.3	50
ISO 13822 (ISO 2010)	2.3–4.3	50
AIJ (2002) (Japan)	1.5–2.5	50
Chinese Standards (2011) (China)	2.7–4.2	50

depending on the consequences of failure. As reported by Takada and Wang (2006), the Architectural Institute of Japan (AIJ) standards for the limit state design of steel frames (AIJ 2002) uses a reliability index between $\beta_T = 1.5$ and 2.5 for a 50-year design life, and the Australian Standard AS-5104 (AS 2005) proposes a 50-year target reliability index β_T between 3.1 and 4.3. The Chinese Standards GB 50068-2001 (Chinese Standards 2011) propose reliability index values between 2.7 and 4.2 for a 50-year design life as reported by Takada and Wang (2006).

The aforementioned international standards show very wide variations. On the one hand, the European standards (Eurocode and JCSS) have slightly higher reliability indexes than those in U.S. standards. This is primarily because of the different approach taken in which the codes apply what is known as partial factors to each parameter in the design equations rather than load and resistance factors. For example, although the U.S. standards calculate the bending moment capacity of a reinforced concrete beam and apply the resistance factor on the final calculated nominal capacity, the Eurocode would apply a partial factor on the concrete strength and another factor on the steel yielding stress and calculates the moment capacity from these factored variables. The European approach misses the epistemic uncertainties associated with the equations and models used to find the nominal capacities. Thus, the estimate of the reliability level is higher than what would have been obtained using the U.S. calibration approach. The Australian code development history is closely associated with the North American process, which led to similar reliability levels. Although the calibration process adopted in the Japanese standards is similar to that practiced in the U.S., AIJ (2002) has specified significantly lower target reliabilities than any of the other standards. A review of Takada and Wang (2006) shows that the COVs used for the loads are significantly higher than those used during ASCE 7-10 (ASCE/SEI 2010) calibrations. The exact reasons for these higher COVs is not clear, but it is common for code writers to use conservative estimates when data in the tail ends of the probability distributions are scarce.

The AASHTO (2012) bridge design specifications were calibrated for $\beta_T = 3.5$ for an anticipated 75-year design life, whereas the load and resistance factor rating (LRFR) method of the AASHTO (2011) manual for bridge evaluation (MBE) used for existing bridges was calibrated for a reliability index $\beta_T = 2.5$ for a 5-year rating period (Nowak 1999; Moses 2001; Sivakumar and Ghosn 2011). A target reliability index is specified as $\beta_T = 3.75$ for new designs and 3.25 for the assessment of existing bridges in the Canadian bridge design code [CAN/CSA-S6-06 (CAN/CSA 2012)]. The development of LRFD bridge codes in Canada preceded that in the United States, and there has been much coordination between the two countries during the original development of the AASHTO LRFD specifications (AASHTO 2012). Distinct tracks however have been taken with respect to the evaluation of existing bridges. The AASHTO MBE (2011) LRFR's use of a target reliability index $\beta_T = 2.5$ for a 5-year rating period rather than the $\beta_T = 3.5$ or 3.75 over a 75-year for new designs is justified based on a qualitative cost-benefit analysis. In fact, the costs of constructing a new bridge to higher reliability criteria are only a fraction of the costs needed to replace an existing bridge to meet the higher requirements. Because U.S. bridges are regularly inspected, the risk of failures between inspection cycles is considered acceptable given the costs that would have been required to maintain the higher target reliabilities (Moses 2001).

The American Petroleum Institute (API) Recommended Practice (RP) (API 2003) for offshore structures uses a target annual probability of failure $P_f = 10^{-3}$, which for a 20-year service life, approximately gives $\beta_T = 2.0$, for manned installations and a target annual probability of failure $P_f = 5 \times 10^{-2}$ for unmanned structures (Det Norske Veritas 1995, 2011). For the design of wind turbines, it is common to assign β_T values based on failure type (ductile with reserve capacity; ductile with no reserve capacity; brittle). For a 20-year service life, annual reliability index, β_T, values can then range between 3.09 and 5.20 (Det Norske Veritas 2002; Petrini et al. 2010). Offshore platforms and wind turbines are unique and very costly structures with low exposure to the general public. Yet, their target reliability levels are significantly different. Although the consequences of their failure may be significant in terms of the cost of the structure, economic losses, and potential damage to the environment, using lower reliability levels for offshore platforms has the same justification provided previously for the evaluation of existing bridges—the cost of their construction at higher reliability levels will be exorbitant, they are regularly inspected, and can be quickly shut down and evacuated upon the detection of major hazards. On the other hand, there is little long-term experience with the performance of wind turbines, and it is natural to use conservative target reliability levels when designing these new types of structures.

Although calibration efforts have traditionally relied on historic data on hazard occurrences and intensities, it is understood that climate change would impact structural safety owing to changes in wind, temperature, rain, snow, wave heights, and flood intensities (Steenbergen et al. 2009; Meyer et al. 2014). Also, increased populations and economic activities will lead to increased load demand on infrastructure systems such as bridges (FHWA 2007). Accounting for such changes in design standards is still difficult at this time because of limitations in available data.

Commentary

The principles of reliability-based design are well established and have been used to calibrate numerous LRFD structural codes, standards, and specifications for component strength limit states. The review of a sample of current reliability-based structural design and evaluation methods leads to the following observations:

1. Member reliability: most LRFD codes and standards are based on a margin of safety formulation for the different expected modes of member failure, in which member capacity must be equal to or greater than the demand imposed by gravity loads and hazards. Methods for computing reliabilities are well established and range from analytical approaches to simulation techniques that can efficiently analyze limit state expressions. The scarcity of data on the occurrence of extreme hazards and the complexity of modeling their load effects combined with uncertainties in isolating the behavior of structural members from the behavior of the entire system have necessitated the use of simplifying assumptions to facilitate the calibration process. Unless these issues are specifically addressed, member reliabilities should not be interpreted in an actuarial sense but simply used as means for comparing alternate designs.
2. Target reliability: because of the implication on public safety and construction costs, engineers are reluctant to implement drastically different safety levels than those that have historically been found to meet the public's expectation. Therefore, the goal of standards has been to ensure that the safety level is consistent over its range of applicability. The target reliability index values used in different design codes and standards for similar types of limit states and materials seem to differ. The differences are the result of factors that affect the reliability calculations, such as intended service life, modes of failure (e.g., ductile or brittle), importance of member to system integrity (secondary member, column, or connection). The practice

of estimating the target reliability index from the successful performance of previous designs also influences the selected values because these experiences may differ between jurisdictions and industries.

Target reliability indexes also reflect a balance between safety and material and construction costs and consider the consequences of a member's failure, the type and occupancy of the structures addressed by the specific code, and the risk tolerance of the engineering community and the public within a code's jurisdiction. For example, it is reasonable to have a code that addresses the design of small-scale structures with low occupancy, constructed with high-quality controls in an environment which is nonaggressive to the construction materials, use a lower target reliability index than a code that deals with large-scale structures built for harsh environments. Similarly, it is natural to apply higher target reliabilities for connections because increasing their safety levels will not require a major increase in the structure cost but will clearly reduce the chances of a major collapse.

3. Direct reliability analysis: because LRFD factors are calibrated to meet their target reliability indexes for a range of typical structural configurations, in some instances, engineers have resorted to performing a direct reliability analysis to evaluate the safety of a particular structure. This is usually done when engineers believe that the evaluation performed using the LRFD procedures does not reflect the actual safety of a specific structure because of different load intensities, the particular condition of its materials, or its distinct geometry. Although the tools, procedures, and simulation algorithms needed for performing direct reliability analyses are widely available, the results of such analyses greatly depend on properly identifying and including the important random variables and the supporting statistical data. An important issue is the interpretation of the results. Although it is logical to compare the reliability index obtained from such direct analyses to the target reliabilities for the LRFD codes applicable to the jurisdiction where the structure is located, this comparison can only be made after careful evaluation of the probabilistic and random variable models used during the direct analysis with those used during the code calibration process in order to have a common benchmark as illustrated in the simplified illustrative example presented in this section.

Consideration of Risk in Member Design

Traditional structural design processes are based on checking the safety of each individual member for a single limit state or one failure mode at a time. However, structures are composed of many members, each of which may fail in a different mode, and the reliability of the structural system is a function of the reliability of all its components. The interdependency between the reliability of the system and its members results from (1) common actions that produce load effects and stresses in different elements; (2) effect of member properties such as stiffness, strength, and ductility on the initial distribution and subsequent redistribution of loads throughout the structure as the load intensity increases and the members undergo nonlinear deformations; (3) correlation between member properties; and (4) the geometric configuration of the structural system and the presence of multiple load paths. Risk analysis methods provide a means to consider the performance of the structural system including the probability of structural failure, P_f, and the resulting consequences (Ang and Tang 1975). Risk may be represented as

$$\text{Risk} = P_f \times \text{consequence of failure} \tag{7}$$

where the consequence of failure can be expressed in terms of costs that may include direct costs, such as the cost of replacing or repairing the structure and loss of life, and indirect and user costs including downtime, economic losses, environmental, societal, and political costs.

The expression for risk, or the probability of loss owing to damage caused by a single or multiple hazards, can be expanded to Ellingwood (2001)

$$\text{Risk} = P(\text{Loss}) = \sum_H \sum_{LS} \sum_D P(\text{Loss}|D) P(D|LS) P(LS|H) P(H) \tag{8}$$

where Loss is any appropriate loss metric; $P(H)$ = probability of occurrence of an input intensity level associated with hazard (H); $P(LS|H)$ = conditional probability of exceeding a structural limit state (LS) given the hazard (H); $P(D|LS)$ = conditional probability of a damage state (D) given the exceedance of the structural limit state (LS); and $P(\text{Loss}|D)$ = conditional probability of a loss (L) given the damage state (D).

By considering the effects of multiple hazards, Eq. (8) forms the basis for an integrated multihazard design approach that identifies and eliminates potentially conflicting effects of certain design features or hazard mitigation measures that could improve the performance under one hazard but aggravate the vulnerability to other hazards (FEMA 2003).

In a first step toward the implementation of comprehensive multihazard design methods, risk-informed performance-based designs (PBD) address structural performance for each hazard intensity level separately. PBD provides a more transparent design process than traditional prescriptive component-based methods. It requires explicitly stated performance objectives for each hazard level, transforming these objectives to structural response requirements, and assessing whether the structure meets the stated objectives. PBD guidelines have been proposed for seismic hazards and are under consideration for other hazards (PEER 2010). The formal implementation of PBD methods requires advanced nonlinear structural system analysis tools, detailed damage estimation models, and approaches for loss quantification. Until such tools are made available for all pertinent hazards, interim PBD procedures have been proposed as an extension to member-based design methods by making a distinction in the reliability requirements of components of different categories of structures, depending on the consequences of member failure (Ellingwood 2001).

ASCE/SEI Standard 7-10 (ASCE/SEI 2010) PBD option classifies buildings into four risk categories:

I. Buildings and other structures that represent low risk to human life in the event of failure.
II. All other buildings and other structures not in Risk Categories I, III, and IV.
III. Buildings and other structures that could pose a substantial risk to human life or cause a substantial economic impact or disruption of everyday life in the event of failure.
IV. Buildings and other structures designated as essential facilities or the failure of which could pose a substantial hazard to the community.

For the loads addressed by ASCE/SEI 7-10, other than earthquake, including dead, live, wind (not tornado), rain, flood (not tsunami), snow, and atmospheric ice loads, the load factors and combinations have been calibrated for structural members for the target annual probabilities of failure shown in Table 2.

Table 2. ASCE/SEI-7 Target Member Reliability for Loads Other Than Earthquakes [Data from ASCE/SEI 7-10 (ASCE/SEI 2010)]

Basis	Occupancy category			
	I	II	III	IV
Failure that is not sudden and does not lead to widespread progression of damage	$P_f = 1.25 \times 10^{-4}$ $\beta = 2.5$	$P_f = 3.0 \times 10^{-5}$ $\beta = 3.0$	$P_f = 1.25 \times 10^{-5}$ $\beta = 3.25$	$P_f = 5.0 \times 10^{-6}$ $\beta = 3.5$
Failure that is either sudden or leads to widespread progression of damage	$P_f = 3.0 \times 10^{-5}$ $\beta = 3.0$	$P_f = 5.0 \times 10^{-6}$ $\beta = 3.5$	$P_f = 2.0 \times 10^{-6}$ $\beta = 3.75$	$P_f = 7.0 \times 10^{-7}$ $\beta = 4.0$
Failure that is sudden and results in widespread progression of damage	$P_f = 5.0 \times 10^{-6}$ $\beta = 3.5$	$P_f = 7.0 \times 10^{-7}$ $\beta = 4.0$	$P_f = 2.5 \times 10^{-7}$ $\beta = 4.25$	$P_f = 1.0 \times 10^{-7}$ $\beta = 4.5$

Note: P_f = annualized probability of failure; β = reliability index for a 50 year service period.

Table 3. Structure, System, or Component (SSC) Performance Goals for Nuclear Power Plants [Data from DOE-STD-1020 (DOE-STD 2002)]

Performance category	Performance goal description	NPH performance goal annual probability of exceeding acceptable behavior limits (P_f)
0	No safety, mission, or cost consideration	No requirements
1	Maintain occupant safety	10^{-3} of the onset of SSC damage to the extent that occupants are endangered
2	Occupant safety, continued operation with minimum interruption	5×10^{-4} of the onset of SSC damage to the extent that the component cannot perform its function
3	Occupant safety, continued operation, hazard confinement	10^{-4} of the onset of SSC damage to the extent that the component cannot perform its function
4	Occupant safety, continued operation, confidence of hazard confinement	10^{-5} of the onset of SSC damage to the extent that the component cannot perform its function

Table 4. Values of Acceptable Annual Failure Probability and Target Member Reliability Index for Offshore Jacket Platforms (Data from Det Norske Veritas 1995)

Class of failure	Less serious consequence	Serious consequence
Redundant structure	$P_f = 10^{-3}$ $\beta = 3.09$	$P_f = 10^{-4}$ $\beta = 3.71$
Significant warning prior to occurrence of failure in a non-redundant structure	$P_f = 10^{-4}$ $\beta = 3.71$	$P_f = 10^{-5}$ $\beta = 4.26$
No warning before the occurrence of failure in a non-redundant structure	$P_f = 10^{-5}$ $\beta = 4.26$	$P_f = 10^{-6}$ $\beta = 4.75$

Table 5. ISO 13822 (ISO 2010) Values of Acceptable Member Target Reliability Index

Ultimate limit states	β	Reference period
Very low consequences of failure	2.3	Design working life (e.g., 50 years)
Low consequences of failure	3.1	Design working life (e.g., 50 years)
Medium consequences of failure	3.8	Design working life (e.g., 50 years)
High consequences of failure	4.3	Design working life (e.g., 50 years)

The U.S. DOE established the natural phenomena hazard (NPH) performance categories and associated target probabilistic performance goals for DOE facilities as shown in Table 3 (U.S. DOE 2002). These goals are expressed as mean annual probabilities of onset of significant damage (but not necessarily collapse) to structures, systems, and components (SSC) subjected to natural hazards such as wind, earthquake, and floods.

Table 6. Target Reliability Index in Australian Standards [Data from AS-5104 (AS 2005)]

Relative costs of safety measures	Consequences of failure			
	Small	Some	Moderate	Great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

The offshore industry developed a set of target annual failure probabilities and reliability indexes for structural members as shown in Table 4 (Det Norske Veritas 1995). ISO 13822 (ISO 2010) recommends the reliability index targets given in Table 5 for the assessment of members of existing structures, including buildings, bridges, and industrial structures. The Australian standards (AS 2005) also use a set of reliability indexes varying between $\beta_T = 0$ to 4.3 for a 50-year service period depending on the consequences of failure and the costs of failure prevention measures as shown in Table 6.

The Canadian Highway Bridge Design Code (CAN/CSA 2012) recommends adjusting the target reliability index β_T values of bridge members depending on the failure mode, member behavior, system behavior, and member inspectability. The recommended member target reliabilities vary between $\beta_T = 2.50$ and 4.0 as shown in Table 7 for a 75-year service life and the member resistance factor is changed based on the target reliability level.

The approach adopted by the AASHTO LRFD (AASHTO 2012) bridge design specifications is less explicit. Although the original load and resistance factors were calibrated to provide a member reliability index, $\beta_T = 3.5$ for a 75-year design life assuming that the system provides sufficient levels of redundancy, members of nonredundant systems are penalized by requiring that they be more conservatively designed so that their member reliability indexes are higher than $\beta_T = 3.5$. This is achieved by applying a

Table 7. Target Member Reliability Index for 75-year Design Period for Normal Traffic in Bridge Canadian Code [Data from CAN/CSA-S6-06 (CAN/CSA 2012)]

Class of failure	Element behavior	Inspection level		
		INSP1	INSP2	INSP3
S1	E1	4.00	3.75	3.75
	E2	3.75	3.50	3.25
	E3	3.50	3.25	3.00
S2	E1	3.75	3.50	3.50
	E2	3.50	3.25	3.00
	E3	3.25	3.00	2.75
S3	E1	3.50	3.25	3.25
	E2	3.25	3.00	2.75
	E3	3.00	2.75	2.50

Note: E1 = sudden loss of capacity with no warning; E2 = sudden failure with no warning but with some postfailure capacity; E3 = gradual failure; INSP1 = component not inspectable; INSP2 = inspection records available to the evaluator; INSP3 = inspections of the critical and substandard members directed by the evaluator; S1 = element failure leads to total collapse; S2 = element failure does not cause total collapse; S3 = local failure only.

load modifier, η, specified in the LRFD code based on member ductility, system redundancy, and bridge importance. Accordingly, the design/check equation is presented as

$$\phi R_n \geq \eta \sum \gamma_i Q_{ni} \tag{9}$$

For example, by applying a load modifier $\eta = 1.10$, the code effectively raises the target member reliability index to $\beta_T = 4.0$. The member reliability index is reduced to $\beta_T = 3.0$ for ductile members of redundant bridges by assigning the load modifier, $\eta = 0.95$.

AASHTO (2011) assigns a system factor (inverse of load modifier) applied on the member resistance, based on specific bridge geometries and on structural characteristics and member types.

In summary, this section demonstrates that, at this stage of development, all design manuals are converging on using member-oriented performance-based design methods. Each matrix in Tables 2–7 is centered on a main target reliability index that corresponds to the target index in the earlier LRFD version of the standard (e.g., $\beta_T = 3.0$ for Category II structures when member failure is not sudden and does not lead to widespread damage). Although as explained earlier, the main target in each case has been calibrated to match successful existing designs, the other targets are usually based on the code writer's experience rather than a formal calibration process.

Also, the factors adopted to define performance levels differ between standards. ASCE 7-10 (ASCE/SEI 2010) addresses different types of structures. Therefore, it categorizes structures based on risk to daily human life and the community. For example, the failure of a warehouse would have low risk to human lives, whereas the failure of a hospital would affect an entire community. When the structure category is well defined, such as bridges, nuclear power plants, or offshore structures, performance levels are set in relationship to the type of member failure and its consequence to the entire system. AASHTO (2012) adds an importance factor that relates to the bridge size and its criticality to a community's economic activity and security concerns. Important bridges are thus required to have higher safety levels. The Canadian bridge code (CAN/CSA 2012) adds inspectability as a factor for determining appropriate reliability targets. This is because if a member is inspectable, engineers will be able to alleviate any potential failures if the member is observed to be undergoing deterioration. Interestingly, the Australian matrix in AS (2005) explicitly includes cost as one of the parameters that determines the target reliability.

Commentary

There has long been a consensus that structural designs should aim to minimize risk by considering the probability of failures and their consequences. Yet, the routine design of structural systems based on formal risk assessment methods for all pertinent hazards remains a challenge because of (1) the difficulty of applying probabilistic analyses techniques when evaluating the performance of complex structural systems; (2) limited statistical data to model the intensities of extreme hazards and their effects on structural systems; (3) the lack of calibrated criteria that relate analysis results to physical structural damage; and (4) the difficulty of enumerating the consequences of failures and the allocation of quantifiable measures for these consequences. Ongoing research is making progress in resolving these challenges and is developing the necessary tools that will eventually facilitate the use of formal risk assessment methods on a regular basis. In the meantime, recent structural design codes and standards have introduced risk-informed performance-based design methods to support the application of varying target member reliability levels that depend on structure categories, modes of failure, and required levels of structural performance. This approach provides a transition between the traditional LRFD approach and a full-fledged risk analysis of the structural system. Accordingly, design standards and codes are still calibrated on member reliability criteria but the target is adjusted to reflect a subjective or an objective evaluation of the consequences of member failure on the overall system's integrity. The paper by Ghosn et al. (2016) describes advanced methods that directly address system performance.

Serviceability and Durability

Although the primary objective of structural design is to provide structural safety, another important objective is to ensure structural serviceability. Serviceability is normally considered by controlling deformations caused by applied loads, effect of temperature changes, moisture, shrinkage or creep, as well as cracking and stresses in concrete members and other critical components. Although reliability-based concepts and methods for evaluating structural safety can also be used for serviceability, most current serviceability criteria including those in LRFD codes and standards are based on past practice rather than reliability assessments. In many cases, the origins of the existing criteria are not known and the relationships between them and the structure's reliability, service life, functionality, or user comfort are not well established.

ASCE 7-10 (ASCE/SEI 2010) does not impose mandatory deflection limits but only provides guidance for design for serviceability to maintain the function of a building and the comfort of its occupants during normal usage. However, in an early attempt at establishing reliability-based deflections criteria for buildings, it was suggested that visible deflections and architectural damage can be avoided if vertical deflections in floors or roofs remain within $1/300$ of the span (or $1/150$ of the span for cantilevers) under loads that have 5% annual probability of being exceeded. A limit of $1/200$ times the span would maintain the functionality of moveable components such as doors and windows. Lateral deflection or drift limits at $1/400$ of the building or story height would minimize damage to cladding (Galambos and Ellingwood 1986).

Similarly, existing AASHTO (2012) specifications require that bridges be designed to avoid undesirable deflection-induced structural or psychological effects and to control bridge vibration and reduce the discomfort of pedestrians and occupants of stopped vehicles without giving specific requirements. Although previous AASHTO (2002) standard specifications did impose deflection limits on the order of $1/800$, several studies have demonstrated that vertical deflection limits will not necessarily lead to improvements in vibration control because much of the dynamic amplification of live load effects results from deck surface roughness and discontinuities at expansion joints rather than to bridge flexibility (Darjani et al. 2010).

On the other hand, design standards do include serviceability criteria that control crack width and concrete stresses and cover to ensure the durability of reinforced and prestressed concrete members. However, no specific information is available on the relationship between the specified crack widths or stress limits and concrete member durability or on how often these criteria can be exceeded within the service life of the structure.

A reliability-based serviceability limit versus demand format can be formulated using Eq. (1) with Q representing the serviceability demand that may be set as the computed deflection, crack size, or stress under service load, and R representing the serviceability limit that may be the allowed deflection, crack size, or stress limit. However, despite the simplicity of the reliability formulation, only a few existing codes and standards have established their serviceability criteria based on reliability analyses. Among these, ISO 13822 (ISO 2010) proposes a target reliability index $\beta_T = 0.0$ for the remaining service life of up to 50 years, if the serviceability parameter being checked is reversible. For irreversible parameters, a target $\beta_T = 1.5$ is proposed. According to the British Standard BS EN 1990 (BS 2002), which follows the Eurocode, the target reliability index for serviceability limit states should be set at $\beta_T = 1.5$ for a 50-year reference period and $\beta_T = 2.9$ for a 1-year reference. To control corrosion of reinforcing steel bars, the reliability index proposed for cracks in concrete cover is set at values of 0.5, 1.5, and 2.0, depending on whether the structure is constructed in a dry environment, moderate environment with cyclic wetting and drying, or a wet environment as explained in BS EN 1990 (BS 2002). With similar reasoning, higher reliability indexes have been applied in chloride environments owing to the risk of higher corrosion rates compared with carbonation-induced corrosion. Table 8 gives a summary of some current reliability index targets set by a few codes and standards that are known to have established reliability-based serviceability criteria using information reported by Takada and Wang (2006). Research is ongoing in the United States to establish reliability-based serviceability criteria. For example, proposals have been made to control concrete tension stresses by achieving a target reliability index $\beta_T = 1.0$ for a 1-year service period (Nowak et al. 2008).

Material durability is another issue that is often discussed in codes and standards but in general terms, without giving specific instructions on how it can be checked. The lack of specificity in current standards notwithstanding, research on developing models to account for material deterioration for structural elements over time has been underway for a number of years (Kayser and Nowak 1989; Mori and Ellingwood 1993; Melchers 1999b; Estes and Frangopol 2001; Biondini and Frangopol 2008). The problem can be formulated as a time-dependent reliability analysis by adjusting Eq. (2) to include time, t, such that (Melchers 1999b)

$$P_f(t) = \Pr[Z(t) \le 0] = \int_0^\infty F_R(t,q) f_Q(q) dq = 1 - \mathcal{R}(t) = \Phi[-\beta(t)] \qquad (10)$$

The goal of a time-dependent reliability analysis is to verify that structural members meet the target reliability index β_T for all time periods $t \le T_L$, where T_L is the specified design life of the structure (Corotis et al. 2005; Frangopol 2011).

Although the conceptual formulation of the problem has been well established and demonstrated for the evaluation of existing deteriorating structures, the implementation of the concepts to explicitly account for material degradation within structural design codes and standards is still a work in progress. There are a number of difficulties still encountered in collecting data and modeling the various random parameters that influence degradation processes, their interactions, and their effect on structural safety especially under the effect of climate changes (Li et al. 2015; Stewart et al. 2011). In the interim, methods have been developed and proposals have been made to standardize the use of field measurements when assessing the reliability of structures exposed to degradation mechanisms and changing load demands (Ghosn et al. 1986; Frangopol et al. 2008; Catbas et al. 2008; Chung et al. 2006; Arangio and Bontempi 2010; Wisniewski et al. 2012).

Table 8. Comparison of Target Reliability Indexes for Serviceability for Cracks in Concrete Cover

Code	Target reliability index	Design or service life
BS EN 1990 and Eurocode (BS 2002)	2.9	1 year
ISO 13822 (ISO 2010)	0.0–1.50	Remaining life
AIJ (2002) (Japan)	1.1–2.0	1 year
Chinese Standards (2011) (China)	0.0–1.50	1 year

Commentary

Although conceptual models have long been available, the implementation of reliability-based performance indicators for serviceability limit states in current codes, standards, and engineering practice has lagged for a number of reasons. A primary reason is that engineers are reluctant to codify serviceability issues because of a concern that the imposition of serviceability limits may restrict design options, stifle creativity, and remove engineering judgment from design issues not related to safety (Griffis 1993). Consequently, many design codes and standards do not have mandatory serviceability criteria.

Also, the large differences in the topologies and behavior of structures produce hard to quantify uncertainties that affect predicting deflections and deformations for buildings. These are attributable to (1) irregular floor plans and placement of columns; (2) effect of cracking and unaccounted-for restraints; (3) long-term deformations in concrete, masonry, and wood structures resulting from creep, shrinkage, or temperature effects that occur after the initial application of the dead and live loads depending on humidity, temperature, age, moisture, and the level to which the member is stressed; and (4) different levels of conservatism in existing deflection models (Ad Hoc Committee on Serviceability Research 1986; Stewart 1996).

Serviceability specifications for deflection limits in design codes and standards have remained unchanged and are based on past experience. However, this experience has not been documented in a systematic manner, so it is unclear how these specifications compare to actual field data of damaging deflections (Hossain and Stewart 2001; Ad Hoc Committee on Serviceability Research 1986; Azevedo and Diniz 2008).

Although controlling crack width is intended to improve the durability of concrete structures, studies on long-term corrosion processes of steel bars in reinforced concrete beams show that existing bending cracks and their widths do not significantly influence the service life of concrete structures. Also, reinforcement cover depths required by standards are found to be a necessary but not a sufficient parameter to define reinforced concrete service life (Vidal et al. 2007).

The causes and catalysts that control structural degradation mechanisms are numerous. They range from chemical reactions, thermal and environmental factors, consumption by living organisms, erosion or mechanical wear as well as load actions that affect materials in different ways. Accelerated testing in laboratory settings may identify the most important factors that affect the long-term behavior of specific materials. However, modeling the in situ real-time conditions and the interaction of these factors remain an important challenge.

Reliability analyses for serviceability and durability criteria need to address time-dependent processes, which are more difficult than reliability analyses for point-in-time ultimate strength limit states (Melchers 1999a). For example, time-varying capacities and stochastic load processes are needed to characterize the effects of vibrations from wind gusts on structures, combinations of vehicular traffic on bridges, humans on footbridges and floors, and material degradation processes.

Fatigue Safety

Structural fatigue, which is a primary cause of failure of bridges, aircraft, offshore platforms, and wind turbines, is the accumulation of damage from the application of repetitive cycles of loads that may lead to brittle fracture of structural components with little prior warning or signs of distress. Reliability-based methods are necessary to account for the randomness of applied loads, the analysis of the stress response, and uncertainties in modeling fatigue accumulation processes and fatigue strength. Using the most basic approach, the fatigue properties of structural materials are represented by empirical *S-N* multilinear log-log relationships in which the maximum number of cycles to failure N_f for a given stress range S is obtained by equations of the form

$$N_f(S) = KS^{-m} \tag{11}$$

where m = slope of the linear plot of $\log(S)$ versus $\log(N_f)$; and K is related to the intercept.

Reliability methods for assessing the fatigue performance of members are often expressed in terms of the service life in which the safety margin, Z, takes the form

$$Z = Y_S - Y_F \tag{12}$$

where Y_F = accumulated fatigue life of the detail; and Y_S = required service life. For variable-amplitude stress cycles, fatigue life is obtained using the Palmgren-Miner rule, which assumes that every stress cycle having a range, S_i, causes a damage, D_i, equal to the reciprocal of the number of cycles to failure for that stress range. Cycle counting for variable-amplitude stresses can be based on either time-domain or frequency-domain representations of the stochastic loading process (Lutes and Sarkani 2003). The total accumulated damage, D, can then be obtained from the sum of all D_i

$$D = \sum D_i = \sum \frac{1}{N_f(S_i)} \tag{13}$$

The accumulated fatigue life is obtained as

$$Y_F = \frac{1}{D} \tag{14}$$

Table 9. Comparison of Target Member Reliability Indexes for Fatigue

Code	Target reliability index	Design or service life
AASHTO LRFD (2012) (U.S.)	2.0–3.0	75 years
CAN/CSA-S6 (CAN/CSA 2012) (Canada)	3.5	75 years
CEN (2002) Eurocode (Europe)	1.5–3.8	50 years
ISO 13822 (ISO 2010)	2.3–3.1	50 years
API (2003) (U.S.)	2.0	20 years

A sample of standards that provide reliability-based fatigue design criteria is provided in Table 9. Because of its simplicity, the *S-N* approach is the most widely used method for evaluating the fatigue performance of structural members. For example, the AASHTO LRFD (AASHTO 2012) uses a reliability-based *S-N* analysis to develop fatigue design criteria for steel bridge components. The reliability-based calibration of the criteria considered the variability in the truck weight spectra and their load effects, including dynamic amplification and epistemic uncertainties, in the structural analysis process as well as variability in the *S-N* curves and accuracy of the Palmgren-Miner rule. The recommended criteria for checking the fatigue performance of details for a 75-year service life are based on a target reliability index $\beta_T = 2.0$ for redundant bridges and $\beta_T = 3.0$ for nonredundant bridges (Moses et al. 1987). CAN/CSA (2012) uses a target $\beta_T = 3.5$ for a 75-year service period. CEN (2002) specifies a target reliability index for fatigue that ranges from $\beta_T = 1.5$ to 3.8 for a 50-year service life. ISO (2010) recommends that fatigue-prone details be designed to meet a target reliability index, $\beta_T = 2.3$, for members that can be inspected and $\beta_T = 3.1$ for members that cannot be inspected for the remaining service life of the structure for up to 50 years. A similar approach and criteria are used in the offshore industry (Det Norske Veritas 1995), in which an annual probability of failure on the order of 10^{-3} is specified which, for a 20-year service life, leads to an approximate reliability index $\beta_T = 2.0$. Similar criteria have been proposed for analyzing the reliability of wind turbine blades (Lange 2007).

The more fundamental fracture mechanics approach for fatigue life evaluation starts by recognizing that flaws or cracks are inherent in any component owing to material defects including voids, brittle inclusions, second phase particles, or corrosion pits as well as manufacturing and installation processes, including welding and machining. The fatigue crack driving force at the crack tip region can be expressed in terms of the stress intensity factor ΔK for relatively brittle materials with small scale yielding, the energy release rate factor (related to the J integral) ΔJ for ductile materials. The crack intensity factor ΔK and the energy release rate factor ΔJ are defined as

$$\Delta K = y(a)S\sqrt{\pi a} \tag{15a}$$

$$\Delta J = y(a, c, t, W, n)S\sqrt{\pi a} \tag{15b}$$

where S = nominal applied stress range; a = crack size; and $y(a, c, t, W, n)$ = geometry function related to crack length c, crack depth a, specimen thickness t, specimen width W, and strain hardening exponent, n. For components at high temperature, where creep is dominant, ΔJ is replaced by a creep driving force, C^*, with a geometry function that includes additional terms related to time-dependent and heat-dependent material properties.

The driving forces are related to the rate of crack growth per load cycle as expressed by Paris-Ergodan's law, which for small scale yielding governed by ΔK can be written as

$$\frac{da}{dN} = C_0(\Delta K)^{m_0} \quad (16)$$

where C_0 and m_0 = material constants that depend on loading frequency, stress ratio, temperature and environment. Crack growth is obtained by integrating Eq. (16) or similar relationships obtained from the results of experimental measurements starting from the initial flaw size. Failure occurs when the crack length reaches a critical size.

In the reliability analysis, the applied stress range is represented by a stochastic process, which may be characterized by its power spectral density and all the terms in Eqs. (15) and (16) as well as the initial flaw size and critical crack length are treated as random variables.

Although researchers have developed advanced reliability models for fracture mechanics fatigue crack growth analysis, routine implementation in engineering practice has remained relatively limited outside the offshore, aeronautic, and aerospace industries. A linear elastic model based on Eq. (15*a*) is recommended as an option for the analysis of offshore jacket platform components under sea wave loadings (Det Norske Veritas 1995). In the aircraft industry, nonlinear and creep based models are used for engines under high temperature as a function of start and flight times. To estimate the total fatigue life, the initial crack length is assumed based on measurements or the probability of detection for a flaw size. For critical parts, 90% probability of crack detection with 95% confidence is becoming the norm in the aircraft industry to establish initial crack size (FAA 2011).

Commentary

Models for the reliability-based evaluation of the fatigue performance of structural components have been developed using a limit state approach analogous to the one used for strength limit states. In civil infrastructure systems such as bridges, current standards use the *S-N* curve approach because of its simplicity despite its known limitations. The offshore industry has implemented both the *S-N* approach and the linear elastic version of the fracture mechanics approach. The aircraft industry on the other hand is heavily involved in developing and implementing advanced probabilistic fatigue analysis models based on theoretical and experimental findings.

Both the *S-N* curve approach and fracture mechanics models involve large levels of aleatory and epistemic uncertainties. These result from the difficulty of estimating changes of the applied loads during the long service lives of structures as well as the difficulty of modeling the fatigue phenomenon itself. Issues with the validity of the Miner-Palmgren rule or the criteria used to distinguish between the crack initiation and propagation phases of the fracture mechanics model and the size of the initial flaws are widely discussed in the research literature. The collection of fatigue life data is especially hampered by the required long service lives under varying environmental conditions.

From Tables 1 and 9, it is observed that the target reliabilities adopted for fatigue of offshore and bridge systems are lower than those for strength limit states in spite of the brittle nature of fatigue failures. These lower target reliabilities result from efforts to match the reliability of successful previous designs and should not be interpreted as an intentional tolerance for lower fatigue safety levels. In fact, fatigue evaluation models used in practical applications have been shown to be conservative. For example, comparisons between results of analytical fatigue assessments of several steel bridge details using established *S-N* curves and field inspections showed that many of these details are free of cracks, although the remaining life calculations predicted that they would have cracks (Kwon et al. 2012).

Current research focuses on reducing epistemic uncertainties by improving existing fatigue models and developing advanced probabilistic analysis tools to help obtain better correlation between estimated fatigue lives and observed lives, including (1) improved *S-N* curves and damage accumulation models; (2) empirical, analytical, and numerical micromechanical models which describe the initiation and propagation of fatigue cracks; and (3) methods to update prior estimates of the uncertainties, as more information becomes available from experimental observations (Rebba et al. 2006).

Conclusions

An overview of current reliability-based methods to evaluate the performance of structures revealed the following points:

1. Reliability-based design and safety assessment LRFD methods have dominated the development of current structural codes and standards. These codes are calibrated to uniformly match the reliability levels of various types of structural members and components that have historically demonstrated adequate performance in terms of providing good safety levels at minimum cost.
2. Large differences are observed in the target reliability levels adopted by various countries and types of structures and materials. These differences are attributed to many factors, including (1) intended structure design and service life; (2) expected member modes of failure (e.g., ductile or brittle); (3) importance of member to system integrity (e.g., secondary member, column, or connection); (4) experiences with previous designs; (5) material and construction costs; (6) structure type and occupancy; and (g) risk tolerance of the engineering community and the public within a code's jurisdiction.
3. For other than seismic hazards, current specifications remain primarily focused on evaluating the strength of individual structural members and components, although recently proposed PBD procedures apply varying target member reliability levels that depend on structure categories, modes of failure, and required levels of structural performance. This approach provides a transition between the traditional LRFD approach and a full-fledged risk analysis of the structural system. Accordingly, PBD standards are still calibrated to meet member reliability criteria, but the target is adjusted to reflect a subjective or an objective evaluation of the consequences of member failure on the overall system's performance.
4. The implementation of reliability-based durability criteria in design standards is still a subject of research owing to difficulties encountered in (1) modeling material degradation mechanisms and their interactions; (2) relating degradation processes to member detailing and in situ conditions; and (3) collection and mapping of long-term site-specific data on degrading agents.
5. Because of large epistemic uncertainties, the evaluation of the fatigue safety of structural components in engineering practice still relies on conservative basic models of damage accumulation using S-N curves or basic fracture mechanics crack growth models.
6. Ideally, future generations of structural design guidelines should seek to implement formal risk analysis methods that account for member and system strength and take into consideration serviceability and durability of structures subjected to repetitive service loads, extreme hazards, and degradation mechanisms. To reach that goal, research is needed on (1) development of

efficient and accurate probabilistic analyses techniques for evaluating the performance of complex structural systems; (2) collection of long-term statistical data to model the intensities of extreme hazards and degradation mechanisms and their effects on structural systems; (3) development of models to relate structural analysis results to physical damage of various structure types; (4) investigating approaches for enumerating the consequences of local damage, partial failures, and structural collapse; and (5) development of measures for quantifying losses and consequences of failures.

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