Uncertainty in Structural Engineering

William M. Bulleit, M.ASCE[1]

Abstract: Structural engineering design is replete with uncertainties, some of which are obvious and some of which many engineers may never have considered. This paper is an examination of the uncertainties facing structural engineers and the ways that engineers have developed to handle those uncertainties. Uncertainty can be separated into two categories: *Aleatory*, related to luck or chance, and *epistemic*, related to knowledge. This breakdown has an impact on how we handle the various types of uncertainty. Uncertainty has a range of sources; we consider five broad sources: time, statistical limits, model limits, randomness, and human error. These five sources will be examined using examples from structural engineering, particularly with respect to the allowable stress design and LRFD code formats. Some uncertainties are explicitly dealt with in design codes, some are dealt with through quality control measures, and some are dealt with in implicit ways that we often do not think much about, e.g., heuristics. Design codes can deal with uncertainties caused by randomness, statistical limits, some aspects of time, and modeling. Other uncertainties such as human error must be dealt with using quality control methods, such as peer reviews and construction inspection.

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Introduction

Structural engineers make many decisions during the design and, often, the construction of a structural system. Most of these decisions are performed under uncertainty, although we often do not think about that uncertainty because we have techniques for dealing with them. We use design codes that include techniques for handling the uncertainty that arises from variable material properties, e.g., nominal capacities and resistance factors, and from uncertainty that arises from variable loads, e.g., design loads and load factors. In the allowable stress design format we deal with both load and capacity variability using a single variable, a factor of safety; in the load and resistance factor design format, we use multiple factors. But some of the uncertainties cannot be dealt with using design code criteria. The possibility of design or construction errors cannot be incorporated into a design code, but the ever present possibility of such errors induce uncertainty into the design of a structural system. These uncertainties are controlled using quality assurance techniques. When you choose a model for the analysis of a structural system, there is uncertainty about whether the model you choose is adequate to predict the behavior of the not-yet-built structure under not-yet-to-have-occurred load events. Some aspects of the model are based on code criteria, but others are based on the structural engineer's professional judgment about what aspects of the structure need to be included in the model and how rigorously the behavior of the structural elements must be modeled.

The objective of this paper is to examine the above topics, and more, in a way that will allow a practicing structural engineer to think about uncertainty and its implications in a fairly deep manner. One aspect of structural design that I hope the reader will consider is the potential benefits, from the standpoint of a reduction of implicit uncertainty, of the load and resistance factor design (LRFD) format versus the allowable stress (strength) design (ASD) format.

Uncertainty

Pliny the Elder, 1st Century military officer, and author, is reputed to have said, "The only thing that is certain is that nothing is certain." This statement is of course true at the most fundamental level, although most of us would probably argue that each of us will die and the sun will rise tomorrow. Those near-certain events notwithstanding, certainty, for all practical purposes, is impossible to achieve. But, we as structural engineers must still make design decisions. We do this by reducing the uncertainty as far as we can. Of course, it never goes to zero, but we are used to making decisions under some level of uncertainty.

Uncertainty can be separated into two broad categories (Ayyub and McCuen 2003): *Aleatory* and *epistemic*. Aleatory means dependent on luck or chance. So, aleatory uncertainty is that uncertainty that arises from randomness inherent in nature. Epistemic means dependent on human knowledge. Thus, epistemic uncertainty is uncertainty that could, in theory, be reduced by increasing the profession's knowledge about the area of interest. Flipping a fair coin is generally thought of as aleatory uncertainty, but it might be possible to model the coin flipping process so well, including coin unbalance, air resistance, etc., that the results of the flip would be nearly predictable. If that prediction were possible, then much of the uncertainty in flipping the coin would not be aleatory, but would be epistemic. Clearly, in practice there are

[1]Professor, Dept. of Civil and Environmental Engineering, Michigan Tech, 1400 Townsend Dr., Houghton, MI 49931-1295. E-mail: wmbullei@mtu.edu

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uncertainties where it is not obvious how to categorize them. The distinction between the two categories will be discussed throughout this paper and often arises in the context of what particular uncertainty can be accounted for in a design code and which one cannot.

Probability

E. W. Howe, the American editor, novelist, and essayist, wrote that, "A reasonable probability is the only certainty." This statement assumes that the reader has a good idea what *probability* is. Even if that assumption is true, allow me to take some time to examine certain aspects of probability a bit closer. Probability has been broken down into a number of types (Ayyub and McCuen 2003). For our purposes, we'll consider only two of them: *Frequentist* and *subjective*. Frequentist probability is the one you think of the most when you think about probability. Given that there are a total number of events that can occur and some subset of that number are the ones you are interested in, then the probability of the events that you are interested in is the number of those divided by the total number. If I flip a coin 10,000 times and get H number of heads, then the probability of a head is $H/10{,}000$. But, in reality it is only an estimate of the probability of a head, and it is *only* for that coin. The coin you flipped 10,000 times may have had some nicks in it and may have had some small fabrication errors that make it behave in its own unique way. But as there are two sides to a coin and it *appears* that each one should be equally likely, we *say* that the probability of a head is $\frac{1}{2}$. This is subjective probability. It is related to your belief that a head should come up $\frac{1}{2}$ the time. It is probably not far from the truth, but we really do not know how far since we have not flipped enough different coins enough times to be certain. Whichever of these two types of probability you believe is true, for engineering problems probability theory and statistics still work.

So, let us consider one aspect of probability theory that we will need later. That aspect is the distribution of a random variable. Consider a design variable, say the ultimate stress of a concrete cylinder, f_c'. For any given concrete mix, this value can have a range of possibilities, thus f_c' is a *random variable*. As f_c' can, theoretically, take on any value in its range of possibilities, it is a *continuous* random variable. Random variables are defined by probability distributions, e.g., normal, lognormal, and Weibull. For a continuous random variable, the cumulative distribution function, $F(x)$, and the probability density function, $f(x)$, are defined as

$$F(x) = P[X \leq x]$$

$$f(x) = \frac{dF(x)}{dx} \qquad (1)$$

where X=random variable; x=specific value of the random variable; and $P[\cdot]$ means probability of the statement inside the square brackets. A histogram is a discrete version of the probability density function. These concepts are shown in Fig. 1. The reader should consult a probability book for more details, e.g., Ayyub and McCuen (2003).

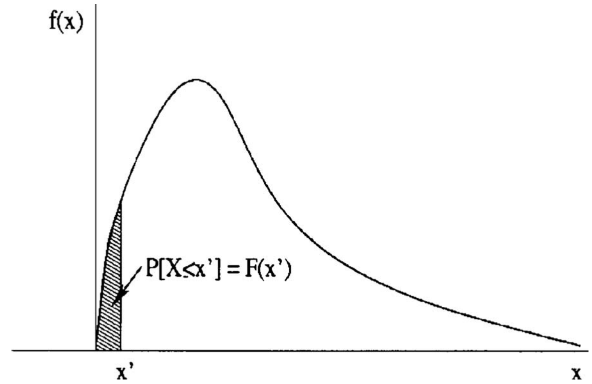

Fig. 1. Probability density function

Causes of Uncertainty

The uncertainties that a structural engineer encounters during a design come from a range of sources. The following five sources of uncertainty cover the vast majority of examples that we will examine in this paper.

- **Time:** There is uncertainty in predicting the future (e.g., how much snow load will our structure experience?) and uncertainty in knowing the past (e.g., what was the concrete strength in the old building we need to renovate?);
- **Statistical limits:** We never can get enough data. (I took some cores from the old building and tested the concrete. Do these test values truly represent the concrete strength?);
- **Model limits:** The structural model used in the analysis and design leaves out or simplifies many aspects of the structure, and it is possible that the model is not conceptually correct;
- **Randomness:** The structural properties (e.g., modulus of elasticity, concrete strength) are not a single number but vary over some range. The properties are random variables;
- **Human error:** It is possible that an error was made during the design or the construction.

None of these five causes of uncertainty separate uncertainties cleanly into aleatory or epistemic. Generally there are aspects of both in each of the five causes. But, the five causes allow us to categorize the uncertainties that we will encounter subsequently, and whether an uncertainty is aleatory or epistemic will allow us to focus on which uncertainties can be reduced through probabilistic techniques and which must be dealt with in other ways.

Design

In the late 1970s and early 1980s, members of the structural design community proposed a change from ASD to LRFD (Ellingwood et al. 1980). Many of the arguments for this change were directly related to how well each code format was able to handle uncertainty, with LRFD doing a better job than ASD. For the purposes of this paper, we will examine how uncertainty is included in the ASD code format and then discuss the ability of the LRFD format to better include certain sources of uncertainty.

Allowable Stress Design

Allowable stress design code formats (also called allowable strength design in some recent incarnations) have been used for many years. ASD is often perceived as being easier to use than

LRFD because ASD uses fewer factors, but it pays for that advantage with some simplifications in how uncertainty is included. The general form for ASD is

$$\frac{R_n}{FS} \geq Q_d + \gamma(Q_{t1} + Q_{t2}) \tag{2}$$

where R_n=nominal resistance; Q_d=nominal dead load effect; Q_{t1} and Q_{t2}=nominal transient load effects; γ=load combination factor; and FS=factor of safety. I have only shown two transient load effects, primary and secondary, for discussion purposes, but there can be only one or more than two.

Consider first the Q's, the load effects. They are the effects that the loads cause, e.g., moment or shear. Typically the load effects are linear functions of the load, e.g., $wL^2/8$ or $wL/2$, where w=uniformly distributed load and L is the beam length. Based on linearity and the small variability in member lengths, we assume that the load effect variability is the same as the load variability. Thus, we have introduced a small uncertainty into the design process, a very small uncertainty, for all practical purposes negligible, but an uncertainty nonetheless.

Dead loads are usually the least uncertain of the loads. The causes of uncertainty in the dead loads will be examined in the manner described previously. An occupancy change of the building could change the dead load, a time uncertainty. We only have so much data on the distribution of concrete unit weight, a statistical limit, and the concrete weight in the building is an unknown value from that distribution, randomness. We might for a wood joist floor say that the total dead load is 479 Pa (10 psf) including the joists even though the actual dead load is distributed differently than that, a model uncertainty. Last, the designer may believe that the flooring will be vinyl tile when the owner actually will use ceramic tile, a human error.

Transient loads and their combinations, e.g., snow load and wind load, are a much larger contributor to the uncertainty. In the vast majority of cases, transient loads are the primary contributor to the uncertainties that can be incorporated directly into the design code. First consider the causes of the uncertainties in the transient loads and their associated load effects. For environmental loads, e.g., snow loads, we base future loads on data from past loads. That practice causes a time-based uncertainty. We have a statistical limit uncertainty because we have only 100 or so years of data with which to work. We induce model uncertainties when we make occupancy live loads uniformly distributed over the floor area. Clearly live loads are not uniformly distributed. We model wind as a static, stepped uniformly distributed load, when in reality the wind is random both in space and time. And, of course the last uncertainty arises from human error. An extreme example is the error made in the design of the 59-story Citicorp Building in New York City (Morgenstern 1995). The building was supported on 9-story tall columns that were placed on the sides of the building rather than at the corners, Fig. 2. During the wind load analysis, the wind was applied to the wall faces as is usually done. Shortly after the structure was completed the structural engineer realized that quartering winds (Fig. 2) would control the wind load effects in the building connections. A heroic effort was required to upgrade the building before the high-wind season descended on New York (Morgenstern 1995).

The nominal loads, dead and transient, are prescribed by a design code, either in the code or by referring to a document like ASCE 7 (ASCE 2005). In the case of dead load, the uncertainty that can be accounted for in the code, statistical limits and randomness, is fairly small so the mean dead load is used as the nominal. But for transient loads, the variability is great enough that the nominal value should be on the high end of the distribution. For our discussion, consider snow load. The nominal snow load is based on a return period R=50 years. The return period means that on average the nominal value will be exceeded once in R years, or, in other words, the probability of exceedance of the nominal load in any given year is $1/R$. The use of a return period is an effort to account for time as a cause of uncertainty. From studies of past snow load data, the distribution of maximum annual ground snow load for various parts of the United States has been determined (Ellingwood and Redfield 1983). Maximum annual snow load in the contiguous states is assumed to be lognormally distributed with parameters dependent on location. Examination of Ellingwood and Redfield (1983) will show that the Type I extreme value distribution (Gumbel distribution) fit some data better than did lognormal, but lognormal was chosen because it worked best for the majority of locations and the difference between the two for this data was small. Uncertainty in this case has been slightly increased due to the selection of distribution type. The nominal snow load for a return period of R is S_R and can be found by solving

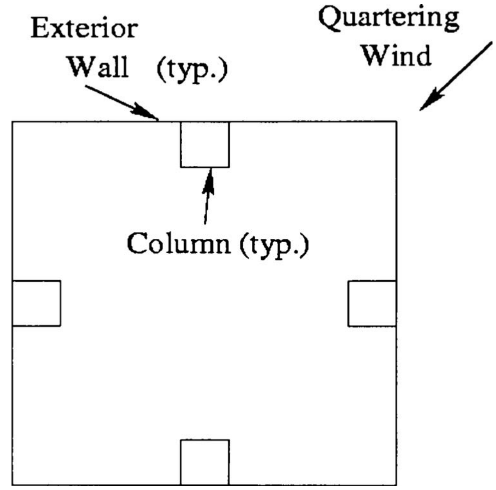

Fig. 2. Plan view of the Citicorp Center

$$1 - \frac{1}{R} = \Phi\left(\frac{\ln S_R - a}{b}\right) \tag{3}$$

where R=50 in the United States; a and b=lognormal parameters found from fitting the distribution to ground snow load data; and $\Phi(\cdot)$=standard normal cumulative distribution function. As stated earlier, the probability that the nominal ground snow load is exceeded in any given year is $1/R$; what is the probability that the design load is exceeded over the design life? Let the design life be N years (typically N=50 years in the United States). The nominal load could be exceeded in the first year, or the second year, or the third year, etc. If we assume that each yearly event is independent, i.e., the maximum snow load in year i is not affected by the maximum snow load in years i-1 or i-2 or i-3 etc., then the probability that S_R is exceeded over N years is (Ayyub and McCuen 2003)

$$\mathrm{P}(S_m > S_R) = 1 - \left(1 - \frac{1}{R}\right)^N \tag{4}$$

where S_m=maximum annual ground snow load. If R=50 and N=50, then this probability is 0.64. There is better than a 60%

chance that the design snow load will be exceeded over the design life.

Another source of uncertainty is the combination of load effects, e.g., the primary load effect is due to occupancy live load and the secondary load effect is due to snow load. We certainly do not want to consider the worst occupancy live load in 50 years occurring at the same time as the worst snow load in 50 years; that is far too unlikely to be useful or economical. We need to examine the two load processes to determine what the worst *combined* load effect is. ASD codes simply sum the load effects from the two nominal transient loads [Eq. (2)] and multiply by γ, which has a value of 0.75 or 1.0 depending on the loads being combined. The potential impact of this practice on design will be examined when we discuss LRFD codes.

The resistance can have a fairly large uncertainty depending on the material. All of the causes of uncertainty influence the magnitude of the resistance uncertainty. A steel member may rust or a wood member may decay; these are time effects. An example of statistical limits is the inability to test all wood joists from all stands and mills. We just get a sample. When we predict the capacity of a structural member, we use a model that is based on assumptions that produce model error. For instance, the base capacity of a wood beam is $F_b S$, where F_b=extreme fiber bending stress capacity and S=section modulus. This equation assumes that the stress distribution through the beam is linear, but that is not necessarily the case for a wood member. All material properties used in calculating a resistance have some inherent randomness, even if we could remove all the statistical limits. We partially account for the statistical limits and randomness by using a nominal strength value that is on the low end of the distribution. One approach is to use a 5% exclusion value; this is a value that has 5% of the population of strength values less than it. In Fig. 1, if $P[X \leq x'] = 0.05$, then x' would be the 5% exclusion value. The 5% exclusion value would be the nominal value. A human error problem that has arisen in various parts of the United States is inadequate structural bracing for metal-plate connected wood trusses used in a piggy-back configuration, in which a triangular shaped truss is placed on top of a truss with parallel bottom and top chords. The failure occurs when the top chord (compression chord) of the lower truss is not adequately braced allowing it to buckle under loads significantly lower than the designer anticipated.

The last item to discuss is the factor of safety, FS in Eq. (2). The factor of safety must account for all variability in the resistance and the loads that are not accounted for in the selection of the nominal values. For example, we saw that the design snow load has a 64% chance of being exceeded during the life of the structure, so the design capacity must be larger than that. Also, if we use 5% exclusion resistance with no other reduction, then we would expect 5% of our members to be below that value. We do not want 5% of our beams or columns to fail. And speaking of columns, a column failure is more catastrophic than a typical beam failure, so a column should have a higher level of safety than a beam. This type of information should also be embedded in the FS. Any model uncertainty from the resistance calculation and from load effect calculations must also be accounted for by the FS. So, the variability of each load, the uncertainty produced by load combinations, the variability in the resistance, some of the model uncertainty, and the relative safety levels for different failure modes must all be accounted for by one number, FS.

Load and Resistance Factor Design

A form of LRFD, called ultimate strength design, was introduced into reinforced concrete design as an appendix in 1956 (ACI 1956), shared equal footing in the body of the code to ASD in the 1963 code (ACI 1963) and became the preferred method in the 1971 version (ACI 1971). The load factors and resistance factors (called strength reduction factors) used in ACI-318 changed somewhat between 1963 and 1971 (ACI 1963, 1971) and then remained essentially unchanged until the factors suggested by Ellingwood et al. (1980) were introduced in an appendix in 1999 (ACI 1999). They became the preferred factors in 2002 (ACI 2002). The American Institute of Steel Construction (AISC) introduced the LRFD format to steel design, incorporating the factors suggested by Ellingwood et al. (1980), in 1986 (AISC 1986) and wood design followed suit in 1995 (ASCE 1995). The present design codes for steel and wood have combined ASD/LRFD (AISC 2005; AF & PA 2005) with no preference between the two formats implied in the code for steel design and an implied preference for ASD in wood design, where LRFD is in an appendix.

Let us examine the basic ideas that went into the 1980 LRFD effort (Ellingwood et al. 1980) and how that work affected the way uncertainty is accounted for in the LRFD format. The general design equation for LRFD is

$$\phi R_n = \gamma_d Q_d + \gamma_{t1} Q_{t1} + \gamma_{t2} Q_{t2} \tag{5}$$

where ϕ=resistance factor and γ_i=load factor associated with the ith load effect. The other terms are defined under Eq. (2). The description in the ASD section about the nominal loads and resistance applies to the nominal values in the LRFD format. The primary difference between the two formats is that the FS in ASD has been replaced with load and resistance factors. So, where ASD had one variable, FS, to account for a range of uncertainties and one variable, γ, to account for the effects of load combinations LRFD has a few variables to account for those uncertainties. First consider the load factors, which was the primary concern of the 1980 effort by Ellingwood and co-workers.

The load factors account for the uncertainty in the load effects that the nominal load (e.g., return period value) cannot. The magnitude of the load factor is a function of the nominal value choice and the variability. For instance, the dead load has a small variability so a mean value is used for the nominal, and the load factor, when dead load is combined with one or more transient loads, has a magnitude of 1.2. Contrast that with snow load where a 50-year return period value is used and the load factor, when snow load is the primary transient load, is 1.6. So, load variability is accounted for using both the selection of the nominal value and the magnitude of the associated load factor. I have been talking about primary and secondary loads and mentioned load combinations. But in ASD there was no real distinction between primary and secondary transient loads, just a factor that accounted for the load combination. LRFD approaches the problem by considering a primary transient load, i.e., the load that is at its maximum lifetime (design life) value plus a secondary transient load that is at its maximum arbitrary-point-in-time value, typically its maximum annual value. If there are more than two transient loads, then all but the primary load are at their maximum arbitrary-point-in-time values. This approach is based on load combination work performed by Turkstra and Madsen (1980). As the nominal loads do not change, the primary load factor is greater than or equal to 1.0 and the secondary (and others if present) are less than or equal to 1.0. An example of this concept is the load combinations that include occupancy live load and snow load (ASCE 2005)

$$\phi R_n \geqslant 1.2D_n + 1.6S_n + L_n$$

$$\phi R_n \geqslant 1.2D_n + 1.6L_n + 0.5S_n \tag{6}$$

So the load factors account for both load variability and the impact of load combinations on uncertainty. The resistance factor, ϕ, accounts for uncertainty in the resistance, including randomness, statistical limits, and model error. It implicitly accounts for structural element importance and whether a failure mode is ductile or brittle. Consider next how these factors were developed.

The load factors for LRFD came from the work of Ellingwood et al. (1980). The factors were developed using reliability analyses incorporating probability information and model error for resistances of reinforced concrete, prestressed concrete, masonry, steel (hot rolled and cold formed), aluminum, and wood structural elements. Probability information for maximum lifetime and maximum arbitrary-point-in-time loads was used in conjunction with the Turkstra and Madsen (1980) approach to load combinations. One key question was: What probability of failure should be used for each of the member types and load combinations? The approach to dealing with this question was to *calibrate* the new LRFD code to the ASD code, in a reliability sense. So, *why* calibrate a new code to an old code? There a number of reasons why code calibration makes sense. First, the ASD code had evolved for a significant amount of time so there was a large amount of implicit design knowledge built into the code. Second, calibration implicitly accounts for the amount of risk that society is willing to accept with respect to structural failures since that information would have evolved into the ASD code. And third, structural engineers are more comfortable with small changes (and rightfully so). Thus, calibration reduced some of the uncertainty in the code transition that otherwise might have increased the risk to structures designed to the new code. Let us consider some details of *how* the calibration was performed. For more in depth information, the interested reader should consult Ellingwood et al. (1980).

The calibration consisted of three steps: (1) determine the reliability levels inherent in the existing ASD codes; (2) select a target reliability for each material type and mode of failure based on the information obtained in Step 1; and (3) select load and resistance factors that minimize the difference between the reliability of designs made to the LRFD code and the target reliabilities. Even though Ellingwood et al. (1980) found some resistance factors, they only recommended load factors, leaving resistance factors up to the various material code bodies. Consider some of the effects of that work. The early calibrations showed that the dead load factor should be 1.1, but this was believed to be unacceptable to the design community, so they chose a dead load factor of 1.2 and performed the calibration with that value fixed. At the time of that work, the value of 1.2 was even somewhat controversial because ACI was using 1.4. ACI would not adopt this new set of load factors until 2002. Following the publication of these load factors, the efforts to develop corresponding resistance factors began. These efforts showed some problems with the existing ASD codes that hadn't been apparent in the past. For instance, wood beam design exhibited fairly large discrepancies between the reliability levels for different species designed for the same mode, e.g., bending (Bulleit 1985). In order to go to LRFD or to become more confident in the ASD code, better lumber data was necessary. The efforts to convert to LRFD also caused some code bodies to reduce epistemic uncertainty by developing more accurate resistance models. This occurrence is evident in, for example, the yield model connection design equations in wood design (NFPA 1991), and a number of equations in the steel design code, e.g., the equations for column capacity (AISC 1986). The efforts to develop a LRFD code helped reduce epistemic uncertainty in the ASD codes.

Model Uncertainty

For this paper, we will consider two types of model uncertainty: (1) the uncertainty related to how well a prediction equation models test data; and (2) the uncertainty about how a structure model, e.g., a finite element model, predicts how the structure behaves. These two types are similar, but one distinction is that the first one can be incorporated into the development of a resistance factor. When performing a reliability analysis, the capacity calculated using the predictive equation is multiplied by a model error factor, which is simply $R_{\text{test}}/R_{\text{pred}}$, where R_{test}=capacity (resistance) from a test of the structural element and R_{pred}=capacity determined using the predictive equation. The model error factor is a random variable so if enough test data is available for the type of structural element whose capacity prediction is to be performed, then the distribution of model error can be incorporated into the reliability analysis. Model error was incorporated into the effort to develop probability based load factors (Ellingwood et al. 1980). The second model uncertainty is a function of how well the structural engineer models the structure. Issues that affect this uncertainty include how well the engineer understands the modeling technique being used, how much effort the engineer can afford in order to model the structure accurately, and how important it is to the design to use a refined model. Efforts to reduce this type of model uncertainty are more apparent on large, complex, expensive structures, such as tall buildings or offshore oil rigs, since a reduction in model uncertainty can maintain safety while reducing costs. To quote George E. P. Box, former Vilas Research Professor of Statistics (now Professor Emeritus) at the University of Wisconsin at Madison, "All models are wrong, some are useful." A corollary to this statement is that all models are wrong, some are just more subtly wrong.

Heuristics

Koen (2003) has defined a heuristic as "anything that provides a plausible aid or direction in the solution of a problem but is in the final analysis unjustified, incapable of justification, and potentially fallible." Heuristics are techniques we as structural engineers use to help us solve problems and perform designs that would otherwise be intractable or too expensive. According to Koen, all parts of the design process are heuristics. At the limit, his thesis is likely to be true, but let's consider some heuristics that are a bit more obvious. As will become evident, most tools and ideas you use to design structures are heuristics.

Consider a few common heuristics. (1) The yield stress for high-strength steel is the 0.2% offset stress. This is a heuristic; it helps us solve problems using high strength steel. (2) The dynamics of wind loads can be ignored in the design of most buildings. If you are designing a low-rise building, you use equivalent static wind loads. You do not include the dynamic effects of the wind directly. (3) Occupancy live load can be modeled as a uniformly distributed static load. Look around you, the live load is not uniformly distributed and is not usually static. (4) The design spectral acceleration is 2/3 of the maximum considered spectral acceleration. This equation is based on a *judgment* that the lower bound for the overload capacity of a well designed structure is 1.5 times the design capacity (BSSC 2003). (5) As a final example, consider the determination of the effective flange width of reinforced concrete T-beams (ACI 2005). The procedure used in ACI, and in the AASHTO bridge design specification, has been in use for over 90 years (Chen et al. 2007). It is a crude but effective way to account for shear lag, a heuristic.

Heuristics are absolutely vital to our ability to design structures. We use them every day without thinking about them, and that is okay as long as we recognize the limits of our heuristics. When the Tacoma Narrows bridge was designed, the heuristic that was used was that wind load only needed to be examined to see how it deflected the bridge laterally (Petrosky 1994). That heuristic had reached its limit of applicability for that bridge.

Human Error

Again, consider the Tacoma Narrows bridge collapse (Petrosky 1982, 1994). The Tacoma Narrows bridge collapsed in 1940 under winds of about 68 km/h (42 mi/hr) after being open for only about 4 months. Depending on your perspective the collapse can be viewed as caused by model uncertainty, incorrect use of a heuristic, or human error. In reality it is probably a combination of these three since the model used to design the bridge was incorrect because an inappropriate heuristic was used, which was the designer's error. The designer and others believed, based on previous bridges that were doing fine, that the Tacoma Narrows bridge could be designed in the same way as the other bridges had been. This belief, based on the heuristic that if a structure is still standing then it must be safe, led the Tacoma Narrows bridge designer into trouble and can, and likely will, lead other structural engineers into trouble. Certainly it is true that the longer a building has stood the more likely it is that it is safe, but standing for a long time does not tell you how safe. The limits of a significant number of structures in the northern United States became apparent when roof insulation was added in the early 1970s to reduce heating costs, which of course increased snow loads.

Another human error is making an error during design. Many of these, particularly calculation errors, are found during design checks, which of course is the reason for design checks. But, as codes have evolved, they have become more complex so more subtle errors in code usage are more likely. For instance, wind load calculations can be relatively complex under certain conditions (ASCE 2005), but are more representative of actual wind effects when used, i.e., they reduce epistemic uncertainty. Reducing epistemic uncertainty by developing more complex load or resistance equations might possibly lead to more design errors, thus increasing uncertainty. So, it is possible that we are trading one source of uncertainty for another. There needs to be a balance between perceived code complexity and perceived code accuracy.

Last, errors can also occur during construction. Inspection is performed to help minimize this type of error. Inspection and design checking, as well as other efforts to minimize human error, are all forms of quality control or assurance. Human error cannot be handled in design equations, but must be dealt with through techniques that reduce the likelihood of error, quality control.

Inconsistent Crudeness

The principle of consistent crudeness implies that "to some extent the choice of level of detail in any part of an engineering procedure must to some extent be governed by the crudest part of that procedure" (Elms 1985). As codes evolve, certain areas of the code evolve at a faster rate than others. This varied rate of evolution leads to inconsistent levels of crudeness in the code. For instance, the calculations for stress in the steel at ultimate capacity of a reinforced concrete T-beam is much less crude than the determination of the effective flange width of the beam (ACI 2005; Chen 2007). So, although we might be fairly confident of the steel stress calculation for a rectangular beam, we will be less certain for a T-beam as the calculation of the effective flange width increases the uncertainty of our calculations. Inconsistent crudeness is almost unavoidable in both codes and in design, but its effects should be considered, and it should be reduced as much as practicable.

Contingency

Contingent means dependent on something not yet certain. In structural engineering, contingency refers to the need to visualize the structural system and perform analysis and design on the visualized system before it can be built. Simon (1996) discusses contingency and points out that it is one of the major differences between engineering and science. A scientist analyzes systems that already exist; engineers must visualize the system they are going to analyze. The visualized system is obviously not the actual system, so contingency is guaranteed to increase uncertainty. In some engineering disciplines it is possible to build prototypes of a design, e.g., a new automobile engine design can have a prototype built and tested allowing further redesign before the final engine is put on the market. Structural engineers typically do not have that luxury. Clearly nonprototype systems like most structural systems have more inherent uncertainty than do systems that have been able to be tested using one or more prototypes.

Decision Time

Uncertainty must be reduced far enough that we are willing to make decisions, but eventually we must decide; although choosing not to decide is also a decision. Paraphrasing the 19th Century Swiss writer Henri-Frederic Amiel, "The person who insists on seeing with perfect clearness before deciding, never decides."

One approach to making decisions is called risk-based decision making. Risk is often defined as the product of the probability of an event and the consequences of the event, i.e., $\text{Risk}=P[\text{event}]^{*}C[\text{event}]$. The problem with explicitly using this approach is that $P[\text{event}]$, say building collapse, is a very small number and the consequences of that event, $C[\text{event}]$, is very large. To make matters worse, we are very uncertain about the actual values of these terms. But, even though we generally do not explicitly make risk-based decisions, the manner in which our codes have evolved implicitly allows us to make risk-based decisions. Remember that the LRFD code was calibrated to the ASD code to help account for the impact of code evolution.

Uncertainty Reduction

There are a number of ways that we as structural engineers reduce uncertainty so that we can make the decisions necessary to completing a design. Following is a list of some of the ways we reduce uncertainty:

- Use load and resistance calculation techniques that have stood the test of time, but update as necessary (possibly due to failures);
- Use characteristic values (e.g., 5% exclusion values);
- Use prototypes where possible (reduces the impact of contingency);
- Check designs and inspect construction (quality control reduces human error);
- Make appropriately conservative assumptions in analysis (in complex analyses, this technique can sometimes be difficult, e.g., leaving out nonstructural elements is not always conservative);
- Check complex analyses with more simple methods where possible (reduces model uncertainty and human error);
- Use your own experience (i.e., your own heuristics); and
- Recognize that heuristics are used everywhere in design and think about their limits.

You can probably think of more, and you may have been throughout the reading of the paper. Also note, in passing, that all of the previous ways are themselves heuristics.

Conclusion

Lord Kelvin, the physicist William Thomson, said, "It's no trick to get the answers when you have all the data. The trick is to get the answers when you only have half the data and half that is wrong and you don't know which half." Although he was talking about science, Lord Kelvin's statement fairly well summarizes the problems associated with uncertainty in structural engineering. The purpose of this paper was to describe the range of uncertainties that arise in structural engineering and discuss the ways that engineers have developed to handle those uncertainties. Some of them are explicitly dealt with in design codes, some are dealt with through quality control measures, and some are dealt with in implicit ways that we often do not think much about, e.g., heuristics. Design codes can deal with uncertainties caused by randomness, statistical limits, and some portions of time uncertainties and model uncertainties. Other uncertainties such as human error must be dealt with using quality control methods, both institutional and personal.

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