Human Error, Human Intervention and Structural Safety Predictions

Erreurs et interventions humaines, et prédiction de la fiabilité des structures

Menschliche Fehlhandlungen und Eingriffe und die Voraussage der Zuverlässigkeit von Tragwerken

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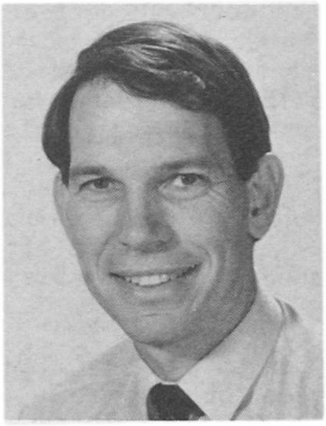

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SUMMARY

Reconciliation of served rates of failure of structures and probability theory must take account not only of human error, as commonly advocated, but also of human intervention. The manner in which human intervention affects calculated failure probabilities is explored for situations in which care has already been taken to properly define failure and failure criteria. It is suggested that current code calibration exercises have more fundamental meaning if interpreted from a serviceability rather than solely a safety point of view.

RÉSUMÉ

Afin d'expliquer la contradiction entre la fréquence des cas de ruine observés dans des structures et la théorie de la fiabilité des structures, il faut tenir compte non seulement des erreurs humaines, mais aussi de la réaction des personnes concernées tentant de corriger leurs erreurs. L'article essaie de déterminer l'influence de ces facteurs sur la théorie de la fiabilité. Il semble que la calibration des normes a plus de sens si elle se base sur les critères de l'aptitude au service plutôt que sur des considérations de la sécurité des structures.

ZUSAMMENFASSUNG

Zur Erklärung des Widerspruchs zwischen beobachteter Versagenshäufigkeit von Tragwerken und den Voraussagen der Zuverlässigkeitstheorie müssen nicht nur die Fehlhandlungen der Beteiligten, sondern auch deren korrigierende Eingriffe berücksichtigt werden. Es wird untersucht, wie korrigierende menschliche Eingriffe rechnerische Versagens-Wahrscheinlichkeiten beeinflussen. Es wird darauf hingewiesen, dass die Kalibrierung von Normen – soll sie sinnvoll sein – sich eher auf die Kriterien der Gebrauchstauglichkeit stützen sollte als auf die Betrachtung der Tragsicherheit.

1. INTRODUCTION

One of the perplexing aspects of structural reliability theory is the difficulty of relating observed rates of failure of real structures with the numbers calculated using reliability theory. The object of this paper is to consider aspects of why this is so and to question the soundness of the conventional philosophy used to relate "nominal" code based reliabilities to structural safety.

A convenient, but not wholly convincing explanation of the discrepancy is to consider the calculated values as "notional" ones, in the sense that there are factors which have not been considered in the analysis (i.e., the modelling of the real-world situation has not been perfect). In particular, the effects of human error and of human intervention are not usually included in calculated rates. It will be argued herein that both of these factors must be considered to obtained a measure of their influence on the strength of structures and structural elements and hence on the calculated failure probabilities [cf. 1].

It will further be argued that the observed failure rates should not be taken at face value but be dissected prior to comparison to calculated values. Both should be related to "predictable" human error events. It is unrealistic to compare statistics including "unforeseeable" or "unpredictable" events with calculated risks.

Finally, it is noted that in using "notional" failure probabilities to relate the reliability of one member (or material) to another, Code writing Committees effectively assume that the effects of human error and human intervention have approximately equal influence on all members (materials); an unproven and somewhat unlikely proposition. An alternate interpretation of the Code-based nominal values suggests that the discrepancy in failure rates stems largely from the differing definitions of failure in observed and calculated statistics, and that the inclusion of serviceability failures will go a long way towards bringing calculated rates more closely in line with observed rates, particularly if allowance is also made for possible (and predictable) human error consequences, as well as the generally beneficial effect of human intervention. The calculated error rates referred to here are not, however, those given in some Code calibration exercises. Such values already incorporate allowance for some human error of the predictable kind, but no allowance for human intervention.

2. OBSERVED AND NOMINAL RATES OF STRUCTURAL FAILURE

2.1 Observed Structural Failure Rates

Many statistics available do not differentiate between types of failure. There is evidence that actual collapses and cases of complete or considerable damage are quite rare, accounting for less than 10-20% of all "failures". Much more common are lesser forms of malfunction. Only "collapse" or its equivalent will be considered here as the definition of failure; the rates originally published have been interpreted and modified accordingly. These are given in Table 1 for building structures.

For bridge structures, rather less data appears to be available. Assuming that failure of bridges is well-defined as "collapse", typical rates are shown in Table 2. It is evident that these rates are rather higher than those in Table 1.

Structure Type	Data Cover	No. of Structures	Average Life (years)	Estimated Lifetime Failure Probability (p_f)
Apartment Floors	Denmark	5×10^6	30	$\sim 3 \times 10^{-7}$
Mixed (housing) (excl. fire)	Netherlands (1967-68)	2.5×10^6	45	$\sim 5 \times 10^{-4}$
Domestic Buildings	Australia (N.S.W.)			$\sim 10^4$
Controlled Domestic Housing	Australia (N.S.W.)	1.45×10^5		$\sim 3 \times 10^{-3}$
Mixed Housing	Canada	5×10^6	50	$\sim 10^{-3}$
Engineered Structures	Canada			$\sim 10^{-4}$

Table 1: Typical "Collapse" Failure Rates for Building Structures [2]

Bridge Type	Data Cover	No. of Structures	Average Life (years)	Estimated Lifetime Failure Probability
Steel Railway	USA (<1900)		40	$\sim 10^{-1}$
Large Suspension	World (1900-1940)	55	40	$\sim 5 \times 10^{-2}$
Cantilever and Suspended Span	USA			1.5×10^{-3}
Bridges	USA Australia			2×10^{-2} 3×10^{-2}

Table 2: Typical "Collapse" Failure Rates for Bridges [2]

2.2 Calculated Structural Failure Rates

There are wide differences in failure rates calculated for structures using probability theory. Apart from the problem of actually defining "failure" already referred to, there is the necessity to ensure that the failure condition used for calculation corresponds to that observed. It is also essential that the statistical models used for loading, member resistance and structural behaviour are realistic, particularly in the "tails" of the probability distributions.

For major structures, using accurate models for loadings and for structural strength, it has been suggested that the calculated rates of failure are typically one or two orders of magnitude lower than the observed rates [3] due to the neglect of human errors. However, the statistical evidence is scanty, and the calculated rates are conflicting. Calculated failure probabilities for the most likely modes of failure for single structural elements are of the order of 10^{-6} or lower [4] for the structure as built, with very much lower figures possible. This has also been found in some, but not all, bridge code calibration work [5].

Calibration exercises have more typically found existing codes to yield so-called "nominal" lifetime failure rates of around 10^{-3} ($\beta \sim 3$) for steel and concrete structural elements, and rather lower rates for masonry and timber structural elements [e.g. 6]. These appear to overestimate observed rates of failure for building structures, but underestimate it for bridges (see Tables 1 and 2).

It must be emphasized that these figures are meant to relate to structural failure in the sense of collapse, rather than serviceability. Since codes are also very much concerned with serviceability, it might be expected that some extra strength is provided merely to satisfy serviceability requirements. This makes the nominal values even more unrealistic.

3. HUMAN ERROR

Human errors may be divided into two groups; [A] those made during the performance of essentially known tasks, such as standard design or construction, and [B] those associated largely with "new" tasks or tasks not previously experienced. Systematic review of observed or analysed failure cases suggests that the former occur much more commonly than the latter [7, 8]. The latter are also largely outside the domain of conventional reliability analysis in the sense that appropriate probability density functions can not usually be deduced. If anything can be said about such errors at all, it is a subjective point estimate of probability of occurrence; however, in many cases analysed, the error itself was not even considered to be predictable or foreseeable. There is considerable evidence that such errors in particular have been responsible for major failures. They have been termed "gross errors", but it is important to recognize that not all gross errors are "unforeseeable", and may be due to carelessness or deliberate oversight [8].

Typical examples of the two types of gross errors, which will be denoted Type A and Type B respectively [9] are given in Table 3. It should be clear that the Type BII "gross" errors correspond to those involving "unforeseeability". A further category V is shown as "variability", denoting the effect of natural variability in materials, loading, workmanship, etc.

Failure Process	Mechanisms of Error	Type of Error
In a mode of behaviour against which the structure was designed	One or more errors during design, documentation, construction and/ or use of the structure	V = variability A = gross
In a mode of behaviour against which the structure was NOT	Engineer's ignorance or oversight of fundamental structural behaviour	BI = gross (foreseeable)
	Profession's ignorance of fundamental structural behaviour	BII = gross (unforeseeable)

Table 3: Conceptual Classification of Errors (Adapted from [9])

It will be recognized that the above categorization cannot be precise. It will also be recognized that interest here is strictly on the structural manifestation of human errors rather than the human errors themselves.

4. NOMINAL FAILURE PROBABILITY AND HUMAN ERROR

The inclusion of human error effects in probability calculations will introduce an extra degree of uncertainty, so that in general the failure probability estimate will increase over that given by human-error-free analysis. Any individual error, of course, may actually increase safety.

For building structures in code calibration, the nominal failure probability is, as noted, commonly around $p_{fN} \approx 10^{-3}$ for lifetime member failure probability; rather greater than the observed failure rates. Any increase in the probability as a result of considering human error will render comparison to observed failure rates even more difficult.

For bridges, in contrast, it seems that the introduction of uncertainty due to human error at least moves the calculated failure probabilities closer to those observed.

Without attempting to reconcile these quite different observations, one common argument is to note that the failure probabilities calculated for code calibration are "nominal", since rather simplified probabilistic models for resistance R and load effect S are employed. Further, it has been argued that it is valid for code calibration (and thus relative decision making), to largely ignore the effect of gross human error in the analysis [2, 9]. To see this, let the probability of failure p_f be written as

$$p_f = p_{th} + p_g \tag{1}$$

where p_{th} is the nominal failure probability, and where p_g absorbs all human error influences (as well as model simplifications). Using this expression together with the economic model

$$C_T = C_I(p_{th}) + (p_{th} + p_g)\, C_F \tag{2}$$

where C_T is the total cost; C_I is the initial cost, a function of p_{th}; and C_F is the cost of failure.

It has been argued that since $p_{th} < p_g << 1$ the minimum total cost C. is insensitive to the occurrence of gross errors, provided their rate of occurrence is realistically low [9, 10]. This means that p_{th}, and hence the corresponding partial factors for design codes, may be derived independently from a consideration of gross errors. Such a conclusion accords roughly with common sense.

The limitation of (2) is that it is not concerned with comparative judgements of uncertainty. This limitation can be seen in using (1) to compare the failure probability of two structural components (1) and (2):

$$\frac{p_{f1}}{p_{f2}} = \frac{p_{th1} + p_{g1}}{p_{th2} + p_{g2}} \tag{3}$$

which reduces to a comparable ratio p_{th1}/p_{th2} only if p_{gi}/p_{thi} is approximately the same for each component. This would not be true in general.

Even if type BII errors are ignored, it does not follow in general that the actual reliabilities can be represented by the nominal values, unless the gross error and human variability effects are essentially comparable for the components being considered [2].

5. ESTIMATING PROBABILITY OF FAILURE

The predicament noted above can be partly overcome if a different view is taken of the nominal failure probability values and if a more realistic approach is taken with regard to human error and its effects as modified by human intervention. The latter may consist of overdesign, self- and independent checking and review, inspections, etc., as well as the use of code rules which are (usually) conservative. In the discussion to follow, these latter matters will be discussed first, followed by a discussion about loading, and the effect thereby created on calculated failure probabilities. The idea of relating code work to combined serviceability and strength as "failure" criteria will be discussed in Section 7.

5.1 Modelling of Resistance

The probabilistic descriptions of strength properties of the most important manufactured structural engineering materials are reasonably well-established [2, 9]. It is important for accurate reliability analyses that probability distribution functions appropriate to the available data and the analysis or design situation be selected. However, it is not always recognized that the strength properties likely to be present in the finished structure are often poorly predicted by consideration of material-test strength and geometrical data alone. The possible influence of discretization and human error must also be considered, but this is still insufficient. The important additional factor which must be accounted for is human intervention.

The importance of human intervention may be most readily seen when examining the process of detail design. Given that the designer knows what he wants to achieve, he follows code specified design rules and certain structural analysis techniques to produce, as a result of his design effort, a set of design sizes required for the structure to be built. The probability density function of strength which might be associated with this outcome would be the result of material strength (σ), geometrical properties (A), human errors (E), (committed by the designer), and a discretization effect (D). As already noted, the last two factors result in a greater variance of resistance than obtained from material strength and geometric factors only.

Typically the various factors might combine in a multiplicative fashion (c = constant):

$$R = c.\sigma.A.E.D. \tag{4}$$

for which, in second moment terms, the mean and coefficient of variation are:

$$\mu_R = c(\mu_\sigma)(\mu_A)(\mu_E)(\mu_D) \tag{5}$$

and

$$V_R^2 = c\ V_\sigma^2\ V_A^2\ V_E^2\ V_D^2 \tag{6}$$

The resulting uncertainty in the resistance R may then be represented schematically by the probability distribution function f_R, as in Figure 1.

The designer (or his supervisor) will be aware, in general, of the reasonableness of his design; even though the "correct" result is not known to him, a designer (or his supervisor) will often be able to identify when he has made a blunder, or when the design does not "come out right" for some other reason.

Given that some level of (self-) checking is innate to the design process, the designer will rework his design, correcting some (or all) errors. The designer (or his supervisor) may not have the capacity to detect (any or all) of the errors; further, some errors are not necessarily detectable. Nevertheless, it is likely that the probability density function for the resistance will be modified to something like that shown in Figure 1.

For convenience, the resistance at which engineers discriminate errors in design or construction will be denoted R_d, and shown as a deterministic value in Figure 1. This is a useful first approximation. Similarly, the modified

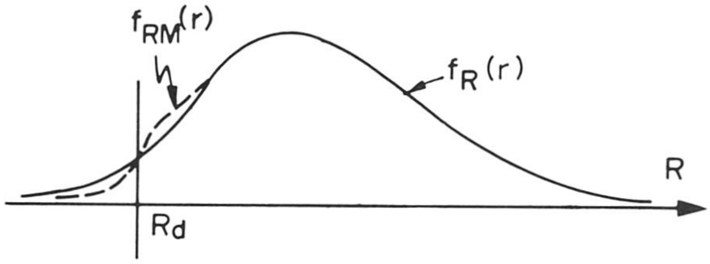

Figure 1: Modification of Probability Density Function for Design Checking Effect

distribution $f_{RM}(\ )$ below R_d will be assumed to be a fraction of $f_R(\ )$ although again a random rather than deterministic fraction is a better description. The modification of $f_R(\ )$ about R_d must be such that the total area under $f_{RM}(\ )$ is unity. The probability density function is thus described by:

$$f_{RM}(\ ) = \kappa(r)\, f_R(r) \tag{7}$$

where $f_{RM}(r)$, $R_d < r < \infty$, must satisfy

$$\int_{R_d}^{\infty} f_{RM}(r)\, dr - \int_{R_d}^{\infty} f_R(r)\, dr = \int_{-\infty}^{R_d} f_R(r)\, dr - \int_{-\infty}^{R_d} f_{RM}(r)\, dr \tag{8}$$

The variable $\kappa(r)$ denotes the failure rate in checking and intervention as a function of R; $\kappa(r) \ll 1$, $r < R_d$.

An analoguous argument could be put forward for the construction process. However, this will not be done here. Suffice it to note that construction engineers are usually capable of detecting gross errors. (An example is the highly complex construction of Lower Yarra Bridge, Melbourne, where independent design checks were commenced after misgivings voiced by construction engineers). In most situations, however, cases of such detection are not reported since the structure concerned did not then fail! Some research on this topic would be of interest [cf. 11].

Support for the distribution shown in Figure 1 is not easy to obtain directly. No known large population of structures (as distinct from materials) actually built has been tested. Indirect support comes from limited surveys conducted on design engineers [12].

5.2 Modelling of Loading

A clear distinction has to be made between the loads as modelled for use in design (which affect only the resistance provided) and the actual loadings which might act on a structure. Only the latter are of interest here.

Loadings can be divided into two classes: those due to natural phenomena, such as wind, wave, snow and earthquake loading, and those due to essentially man-imposed requirements such as live loads on floors, crane loads, bridge loads, traffic loads, and dead loads. The distribution of dead loads shows only minor uncertainty and is of little interest.

Data obtained from floor live load surveys, for example, is usually modelled by probability distributions unbounded in the upper tails. This does not mean, however, that the upper tails are not actually bounded. It would be reasonable to expect, in general, that human intervention would occur if man-imposed loading leads to signs of structural distress. In effect, an upper bound on the loading is thereby created. It may well be unlikely that man-imposed loading can ever cause structural failure unless warnings of structural distress; (i) are not heeded, or (ii) cannot be acted upon in time (such as in "brittle" structures or structures subject to buckling).

For loadings due to natural phenomena, human intervention is not normally possible, yet it is precisely this which leads to questions about the distribution to be used for rare events. Because there is no human intervention possible, it is arguable that there is greater readiness on the part of the population at large to accept a degree of structural failure under

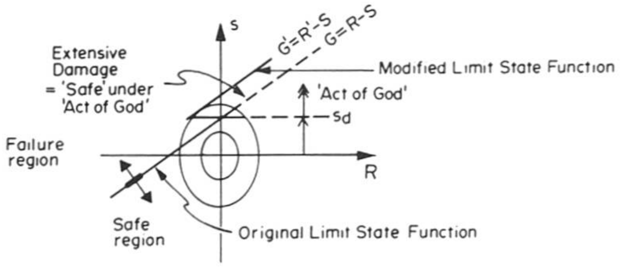

Figure 2: Modification of Limit State Function

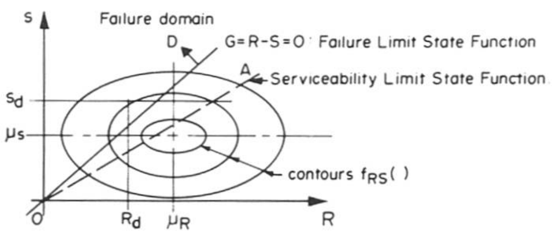

Figure 3: Joint Probability Density Function f_{RS} and Limit State Design

relatively rare (extreme) loadings (i.e., "Acts of God"). No ready empirical support for such a notion appears to be available, however if it is accepted, the limit state of structural failure is modified as in Figure 2.

Alternatively, the region of the probability density function for natural loads beyond the extreme load level (i.e.,"Act of God" level) may be considered as artificially lowered.

5.3 Prediction of Probability of Failure

The importance of human intervention on the prediction of failure probability can be illustrated with a simple example. Consider the case where gross human errors of the foreseeable type are supposed included in the distribution for resistance R and load effect S, and that unforeseeable errors are ignored. Further, suppose that R and S are the only two variables. This is the fundamental case considered in all reliability studies. It may be represented as in Figure 3. The line $G = R - S = 0$ represents the (known) limit state function, with $G < 0$ describing failure of the structure.

The joint density function of R and S is described by $f_{RS}(\ )$ and is sketched as contours on Figure 3. The discrimination levels R_d and S_d are also shown. The probability of limit state violation is then

$$p_{fM} = \iint_{D:R<S} f_{RS}(r,s)\, dr\, ds \tag{9}$$

where D is the domain in which $R < S$. Evidently the integration must be carried out in a piecewise manner. Using (7) and assuming R, S are independent:

$$p_{fM} = \iint_{D:R<S} \kappa_R(r)\, \kappa_S(s)\, f_R(r)\, f_S(s)\, dr\, ds \tag{10}$$

where $\kappa_R(r)$ and $\kappa_S(r)$ represent the functions describing the modifications to $f_R(\ )$ and $f_S(\ )$ respectively, to account for human intervention.

6. EXAMPLE

An indication of the possible implications of the model outlined above may be given by considering the simple case where R and S are each described by a Normal distribution and, for simplicity, κ_S is not considered. This corresponds to the calculation of the modified notional failure probability p_{fNM}. It will be assumed that a constant value of p_{fN} is adopted (i.e., prior to any tail modifications). This will allow a constant base for comparison of p_{fNM}.

To calculate p_{fN}, use can be made of First Order Second Moment theory (e.g., [2, 9]) to reduce the problem to the standard Normal space, $r = (R - \mu_R)/\sigma_R$, $s = (S - \mu_S)/\sigma_S$ for each variable.

The linear limit state equation $G = R - S = 0$ (from Figure 3) is transformed according to the values for V_R and V_S using well-known theory:

$$0 = (r\,\sigma_R + \mu_R) - (s\sigma_S + \mu_S) \tag{11}$$

where σ_R, σ_S represent the respective standard deviations ($\sigma_R = V_R\,\mu_R$ etc.).

The nominal failure probability p_{fN} is defined as

$$p_{fN} = \Phi(-\beta) = \Phi\left(\frac{0 - \mu_G}{\sigma_G}\right) = \Phi\left[-\frac{\mu_R - \mu_S}{(\sigma_R^2 + \sigma_S^2)^{\frac{1}{2}}}\right] \tag{12}$$

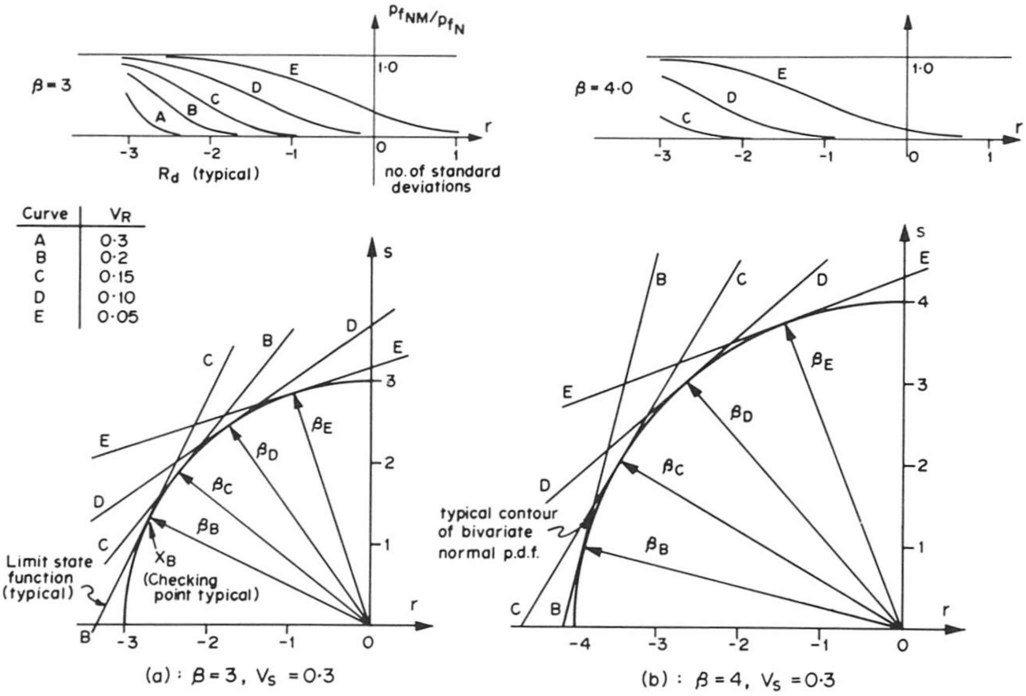

Figure 4: Limit State Functions for Various V_R Values (Lower) and Corresponding Ratio of Modified (p_{fNM}) to Nominal Failure Probability (p_{fN}) for Given Safety Index (β) and Various Levens of Error Discrimination (R_d)

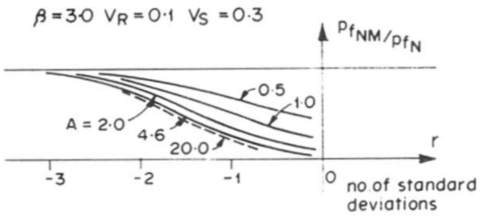

Figure 5a: Effect of Constant A in Equation (13)

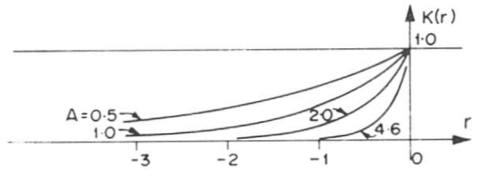

Figure 5b: Effect of Constant A on p_{fNM}/p_{fN}

where β is the usual "safety index" [16, 19]. For constant β, the limit state equation (11) becomes

$$0 = \beta(\sigma_R + \sigma_S)^{1/2} + r\sigma_R - s\sigma_S \qquad (13)$$

Figure 4 shows several limit state equations (13) corresponding to different values of V_R, with $V_S = 0.3$ and $\beta = 3$ and 4.

To calculate p_{fNM}, an appropriate distribution modification for R satisfying (7) is given by (Figure 5);

$$\kappa_R(r) = \exp[A(r - R_d)] \quad \text{for } r \leq R_d \qquad (13)$$

with the probability distribution for $r > R_d$ given by

$$f_{RM}(r) = f_R(r) + f_R(2R_d - r) \quad \{1 - \exp\{A(R_d - r)]\} \qquad r > R_d \qquad (14)$$

A typical value of A = 4.6 was chosen to give a 99% reduction of the tail one standard deviation away from R_d, the discrimination level, but other values were also tried.

Values for p_{fNM} for a range of V_R and V_S values were obtained by numerical integration, with the discrimination level R_d transformed to r_d in the standardized space of Figure 4. The upper parts of Figure 4 show the results thus obtained for p_{fNM}/p_{fN} as a function of discrimination level R_d. A typical case might be $\beta = 3$, $V_R = 0.1$ and $V_S = 0.3$ (curve D). With V_R increased to 0.15 as a result of gross human error, say, the ratio for curve C, is then $p_{fNM}/p_{fN} = 0.37$ for $R_d = -2$, that is, if detection of errors begins to occur at two standard deviations from the mean. Similarly, $p_{fNM}/p_{fN} = 0.035$ if discrimination occurs already at one standard deviation ($R_d = -1$).

It is evident that the ratio p_{fNM}/p_{fN} is quite sensitive to the judgement about R_d, the discrimination level. It is also sensitive to V_R, particularly at low failure probabilities (i.e., high nominal safety indices, β).

If the probability density function for S is also modified, but now in the upper tail region, there will be a further reduction in failure probability. This reduction will be greater for situations in which $V_S >> V_R$ since there is then some appreciable reduction in probability content to the right of the resistance discrimination values in Figure 4 which was not previously considered. In principle the calculations are in direct parallel to those above, but will not be given here. Similarly, the above calculations could be repeated for other forms of probability distributions for R and S; it is unlikely that the conclusions would be very much different. Figure 5 illustrates the effect of changing the value of the constant A in equation (13).

7. OBSERVATIONS AND IMPLICATIONS

The above example was based, for convenience, on nominal failure probabilities related to code calibration. In placing interpretation on the example, it should be noted that the nominal failure probabilities for building structures, of around 10^{-3} (β = 3), include already some allowance for design model and construction variabilities (i.e., V) and are conservative. Some measure of allowance for Type A and BI errors also appears, commonly, to be made [6]. This is not done in all code calibration work, and may account partly for the much lower nominal failure rates calculated for bridges, for example.

The checking function $\kappa_R(\ )$ has been assumed to be applicable to the design process in general, although it will very clearly be task dependent. Similarly, the checking function need not be identical for bridges and for buildings. High failure rates are common for bridges when they are in a developmental stage, during which time it is unlikely that checking can be very effective.

The effect of structural redundancy has been ignored in the present discussion. Its presence undoubtedly explains the much lower observed failure rate for slabs (see Table 1). It is also a factor (but not necessarily a consistent one) in many other structures or structural elements.

Turning now to currently practised code calibration efforts, it is important to note that in most cases only relatively little modification of probability distributions is carried out to allow for the possibility of human error and human intervention. Usually, natural variability of materials, dimensions and loads determine the shape of the relevant distribution [6]. As already noted, however, the "notional failure probability" p_{fN} calculated using such distributions is generally rationalized as a consistent measure indicative of structural failure. As also noted, a consistent value is commonly advocated in order to ensure approximately equal probabilities of failure for all structural elements [13]. Hence it is clear that p_{fN} is assumed proportional to a more accurate, perhaps "true" failure probability p_f. The validity of this proposition does not appear to have been explicitly addressed.

In view of the previous discussion and example, the proposition can only be valid if the effects of human error (Type A and BI), human variability (V) and human intervention, essentially nullify each other, perhaps with the (non-consistent) influence of system effects. This is unlikely to be the case in general. An uncomfortable conclusion about the validity of some current code calibration exercises must then follow.

An alternative and possibly more plausible view is to focus not on safety, but on serviceability as the dominating function of codes. In many cases, a safety criterion can be related to a serviceability criterion; thus allowable deflection in beams relates to strength or stress. For a simple beam, for example, a serviceability limit state might be OA in Figure 3.

Clearly, the probability p_{fS} of violation of the serviceability limit state again involves integration to obtain a probability content under the joint probability density function, f_{RS}; the content is considerably greater than that associated with p_f, p_{fN} or p_{fNM}. It is readily apparent that because human intervention modifies the probability distributions mainly in the "tail" regions, such intervention has relatively little influence on the greater probability content p_{fS} associated with serviceability violation. It would therefore be expected, if this argument is valid, that the calculated and observed rates of serviceability limit state violation are at least comparable for given structure types, provided, again, that consistency is applied to failure definitions in both data and calculations. The presently available data appear to be inadequate to support or reject this view.

None of this is to deny the importance of codes nor of code calibration as developed over the last 15 - 20 years using probability theory. Codes ensure that both the mean strength μ_R and its coefficient of variation V_R, for a particular situation, are maintained and reasonably contained, respectively. If codes are poorly written, or non-existent, it is likely that designs will be less consistent and hence V_R will be high. The discrimination level R_d will then be rather low, as professional experience will vary. Also, $\kappa_R(\ )$ is likely to be rather closer to unity in this circumstance.

8. CONCLUSION

The preliminary analysis and the example given above indicate the roles of human intervention in relating calculation failure probabilities to observations and rates of failure, provided correct definitions and classifications of failure states are used. The present discussion has indicated another interpretation of code calibration procedures, derived from focussing on human error and human intervention in modifying probability calculations.

9. ACKNOWLEDGEMENTS

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