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Structural Safety – a Matter of Decision and Control

La sécurité des ouvrages de génie civil – un problème de décision et de contrôle

Tragwerksicherheit – eine Frage von Entscheidung und Kontrolle

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SUMMARY

Two new tools of structural reliability are reviewed. Their potential impact on future structural codes is examined. An attempt is made to redefine certain fundamental concepts of structural safety in a more rational way.

RÉSUMÉ

Deux moyens nouveaux dans la fiabilité des structures sont discutés. Leur influence possible sur les normes futures est examinée. Une tentative est faite de redéfinir, de façon plus rationnelle, certains concepts de base de la sécurité.

ZUSAMMENFASSUNG

Zwei neue Werkzeuge der Tragwerkzuverlässigkeit werden besprochen. Ihre möglichen Auswirkungen auf künftige Tragwerksnormen werden untersucht. Es wird versucht, gewisse Grundbegriffe der Tragwerksicherheit auf eine vernünftigere Grundlage zu stellen.

0. INTRODUCTION

Since the publication of a complete literature survey on structural safety [Task Committee 1972], two milestones have been placed in the theory of this discipline: the solution of the problem of statistical uncertainty in design and the answer to the question of how reliability in design should be measured in terms of available (first and second moment) information.

This report reviews the substance of these new tools in chapters (1) and (2). It outlines how they might be used in design and in future structural codes in chapters (3) and (4). The aim is to show that second moment reliability and statistical prediction techniques cannot be "included" into an existing framework of design procedures and structural codes. Their impact is much more fundamental. It will change both the way we define the central values of our profession and the concepts we base our daily work on. What emerges at this time is not "a game for mathematicians" as a prominent French codemaker recently put it, but - for the first time ever - a theory of structural design decisions both logically consistent and accessible to empirical feedback.

The reader we hope to address with this report is a young man interested in the basic values and the foundations of structural engineering. Hopefully, he will become a member of a structural code committee soon, and he will produce the creative ideas required now if the new theoretical tools should result in more rational engineering.

Few of the concepts outlined in this report are new, but several have not been published elsewhere. If no references are given, credits should go to the working group for the new Swiss Directives for Structural Safety and Serviceability [SIA 260, 1978], which was composed of J. Schneider, R. Hauser, Th. Schneider and the writer. Mistakes should be blamed exclusively on the writer.

This report was originally planned as a broad, well balanced survey of all matters relevant to structural safety. The writer has made the experience that he cannot write such a survey at this time; he knows both too much about too little and too little about too much. In particular, the notion of feedback (Chapter 4) and the control strategy of a code are connected with a vast array of professional and practical questions not mentioned in this report. They should probably be discussed by other writers with practical experience in those areas. All we hope to show here is how the theory and the formal structure behind design decisions must be brought back to respond to that practical experience - by an appropriate definition of the designer's task, and by feedback at the code level. In developed western economies, structural safety will then probably be discovered to be a non-issue. The real issues are economy and, most important, the effect of structural engineering decision rules on competition between different types of construction and material.

1. PREDICTION

Consider mass-produced structural elements from a well controlled stationary, but to some extent random production process. The strength of these elements may then be modelled by independent random variables

$$R_1,\ R_2,\ \ldots R_n,\ R_{n+1} \qquad (1.1)$$

with a common distribution

$$F_R(r) = P\{R \leq r\} \qquad (1.2)$$

which we assume normal with unknown mean μ and unknown variance σ^2:

$$F_R(r) = \Phi(\frac{r-\mu}{\sigma}) \qquad (1.3)$$

By destructive testing, we obtain the actual values of R_1 through R_n, a sample of size n:

$$r_1,\ r_2,\ \ldots r_n \qquad (1.4)$$

The next, (n+1)st element is used for construction. Let us assume for the sake of simplicity that the designer may choose the load s to be applied to the element, and that he is able to control that value with precision (Fig. 1). In his decision, he is constrained only by a safety requirement: on average, the proportion of failures from his decision rule shall be no larger than p_f.

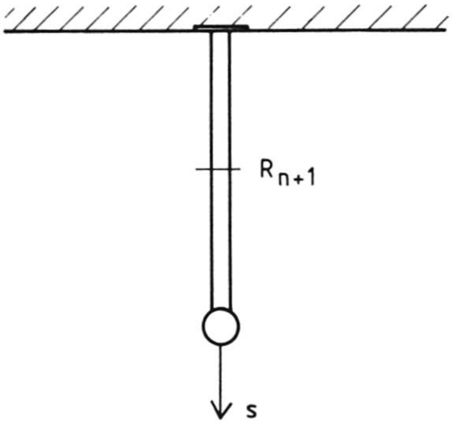

Fig. 1

How should the designer make his decision, given

- the distributional assumption (1.3) with unknown μ and σ
- the past experience (1.4) from destructive testing
- the safety constraint ?

A frequently used tactic is the following:

- compute the sample mean and variance from (1.4):

$$\hat{R} = \frac{1}{n}\sum_1^n r_i \qquad S^2 = \frac{1}{n+1}\sum_1^n (r_i - \hat{R})^2 \qquad (1.5)$$

- define a "design value" r* by the requirement

$$\Phi(\frac{r^* - \hat{R}}{S}) = p_f \qquad (1.6)$$

or

$$r^* = \hat{R} - \beta_\infty S$$

with

$$\beta_\infty = -\Phi^{-1}(p_f) \qquad (1.7)$$

For very large n, $\hat{R} \to \mu$ and $S \to \sigma$. The design value approaches the p_f-fractile of the distribution of R, and the designer may indeed apply the load

$$s = r^* \qquad (1.8)$$

For limited n, this tactic may be dangerous. To see this, we compute the expected probability contents of the failure domain $(-\infty, r^*)$. The random variable

$$E = \frac{R_{n+1} - \hat{R}}{S\sqrt{1+\frac{1}{n}}} \qquad (1.9)$$

has a Student t-distribution with (n-1) degrees of freedom [Mood, Graybill, Boes 1974]. Thus

$$P\{R_{n+1} \leq r^*\} = P\{R_{n+1} \leq \hat{R}-\beta_\infty S\}$$

$$= P\{\frac{R_{n+1}-\hat{R}}{S} \leq -\beta_\infty\} = F_{t,n-1}\left(\frac{-\beta_\infty}{\sqrt{1+1/n}}\right) \qquad (1.10)$$

For two typical cases, this becomes:

n	$P\{R_{n+1} \leq r^*\}$ $p_f = 0.05$	$p_f = 10^{-5}$
2	0.20	0.08884
5	0.10	0.00877
10	0.08	0.00141
30	0.06	0.00024
120	0.05	0.00001
∞	0.05	10^{-5}

(1.11)

This shows that the decision rule (1.5, 1.6, 1.8) violates the safety constraint for small samples. If p_f is small, the effect of limited information (statistical uncertainty) is far from negligible. For target $p_f = 10^{-5}$ and samples smaller than 100, it may well be the dominant uncertainty.

The correct solution of the statistical prediction problem posed is straightforward. From (1.10),

$$P\{R_{n+1} \leq r^*\} = F_{t,n-1}\left(-\frac{\beta_n}{\sqrt{1+\frac{1}{n}}}\right) = p_f \qquad (1.12)$$

or $$\beta_n = -\sqrt{1+\frac{1}{n}}\; F^{-1}_{t,n-1}(p_f) \qquad (1.13)$$

The values of β_n for the two cases in(1.11)are:

n	β_n $p_f = 0.05$	β_n $p_f = 10^{-5}$
2	7.76	
5	2.34	
10	1.93	8.3
30	1.73	5.1
120	1.67	4.3
∞	1.65	4.27

The safe domain (r^*,∞) with

$$r^* = \hat{R} - \beta_n S \qquad (1.7')$$

is called a prediction interval of expected content $(1-p_f)$ for R. β_n is seen to depend strongly on the sample size. The penalty ratio

$$\frac{\beta_n}{\beta_\infty} \qquad (1.14)$$

for limited information takes values around two for n = 10 and $p_f = 10^{-5}$.

All these results are conditional on the distribution assumption (1.3). Droping it and retaining only the much weaker assumption that the density of R, $f_R(r)$, be continuous in some range, but otherwise unknown, we may still construct prediction intervals for R_{n+1} from the sample (1.4). Such nonparametric prediction intervals of prescribed expected contents are based on the order statistics

$$r_{(1)},\ r_{(2)},\ \ldots r_{(n)} \qquad (1.4')$$

of the sample - a rearrangement of the sample in ascending order. It can be shown that each of the n+1 intervals

$$(-\infty,\ r_{(1)}],\ (r_{(1)},\ r_{(2)}],\ \ldots.,\ (r_{(n)},\infty) \qquad (1.15)$$

has expected contents 1/(n+1). Without a distribution assumption, the designer would have to use the following tactic:

- Test a sample of

$$n = \text{entier}\ \frac{1}{p_f} \qquad (1.16)$$

  items

- Put $s = r^* = r_{(1)}$ (1.8')

  with $r_{(1)}$ the smallest value from the sample.

Note how large the sample size must be: with $p_f = 0.05$, $n = 20$, and with $p_f = 10^{-5}$, $n = 10^5$.

The problem of statistical uncertainty in design has been identified and well posed by Cornell, Anderson and Veneziano. In Veneziano's monumental thesis [1974], all relevant results from statistical theory are collected, including simple frequentist and Bayesian prediction for univariate and multivariate independent sequences, simultaneous predition for univariate sequences, and prediction for first-order autoregressive models. A good monograph on the subject without particular reference to structural safety is Guttman [1970].

Statistical prediction models quantify the effect of limited information on design decisions. From the simple example above, it is clear how such models could be used for the definition of design loads. Such direct applications are not the only benefit from the study of prediction, however.

"The main role of models is often not so much to explain and to predict, as to polarize thinking and to pose sharp questions" (Mark Kac). Statistical prediction models give structure to the learning process usually labeled "experience". They spotlight the fact that all our experience is conditional on some observation model. In particular, experience related to a decision rule for design is possible only if this decision rule and its domain of application are well defined. Most of today's structural codes are very deficient in this respect, so that no feedback of experience can take place.

The model of a sequence of independent, identically distributed random variables in our example draws the attention on the importance of control. Most systematic or gross errors in a production process will destroy this "white sequence" property in the first place, substituting trend, dependence, unpredictable instationarity for it.

Rackwitz [1977] has tried to account for human error and negligence by superposing such errors as additional random effects on the models of structural reliability. In the opinion of the writer, gross errors and human negligence cannot be reduced to such inoffensive "in the model" patterns. Most human errors of real importance will not only affect the mean and variance of some design variable, but the very basis of our models for structural behaviour and decision.

Structural reliability is based on the mechanical description of structural behaviour. A reliability model including human errors would necessarily stand on a much broader foundation. The very minimum required would be the inclusion of the procedures of communication and control in design and execution of structures. Such models are not impossible: see Drury and Fox [1975] for an example from quality control.

However, the complexity of design and construction procedures and their low degree of formalization and uniformity make them an unlikely subject for such reliability studies.

Last but not least, even a brief look at simple nonparametric prediction problems will convince any open-minded reader of the hopelessness of structural reliability as a tool for the prediction of the rate of failure of structures. If anythinkg like a "probability of failure" of structures could be unambiguously defined (which is not the case, as we will show), then this quantity would have to be several orders of magnitude smaller than the resolution power of the statistical methods and the data available to us. This insight is not new, but it is constantly forgotten.

2. SECOND MOMENT RELIABILITY

Proper reliability statements about structures are not possible. But design decisions have to be made, and most procedures used now for this purpose are unsatisfactory.

Most structural designers will agree on the following qualitative statements:

- Many variables in the mechanical design equation of structures cannot be predicted and controlled with precision. Design decisions must be based on conservative design values of such variables.
- Variables with larger prediction uncertainty and less control must be assigned more conservative design values.
- If several independent uncertain variables enter a design decision simultaneously, then their design values may be chosen with less conservativism. It is very unlikely that several independent variables take on extremely unsafe values simultaneously.

Second moment reliability gives a quantitative form to these postulates.

2.1 Safety indices: The basic idea [Basler 1960], [Cornell 1969]

Consider a design problem of the simple type of Fig. 2 with the resistance R and the load S modelled by independent random variables. Failures occurs if

$$Z = R - S < 0 \qquad (2.11)$$

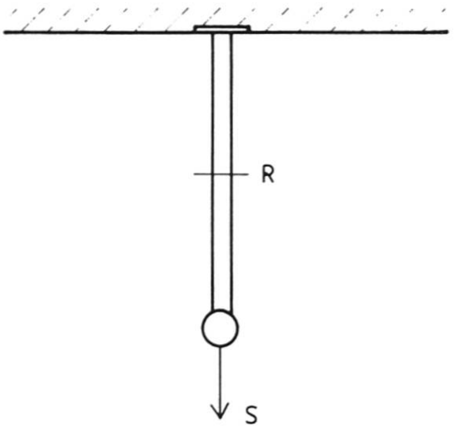

Fig. 2

Let us assume no other knowledge on the probability laws of R and S other than first and second moments:

$$E(R) = \overline{R} \qquad E(S) = \overline{S}$$
$$Var(R) = \sigma_R^2 \qquad Var(S) = \sigma_S^2 \tag{2.12}$$
$$Cov(R,S) = 0$$

The safety zone Z has mean

$$\overline{Z} = \overline{R} - \overline{S} \tag{2.13}$$

and variance

$$\sigma_Z^2 = \sigma_R^2 + \sigma_S^2 \tag{2.14}$$

A safety index is then defined by

$$\beta = \frac{\overline{Z}}{\sigma_Z} \tag{2.15}$$

It measures the distance from the mean of Z to failure in units of standard deviation.

2.2 The general time-independent design problem

Consider next a design problem of the more general nature of Fig. (3), with random loads S_1, S_2, random bending resistances R_1, R_2, ... R_5 and several possible modes of failure.

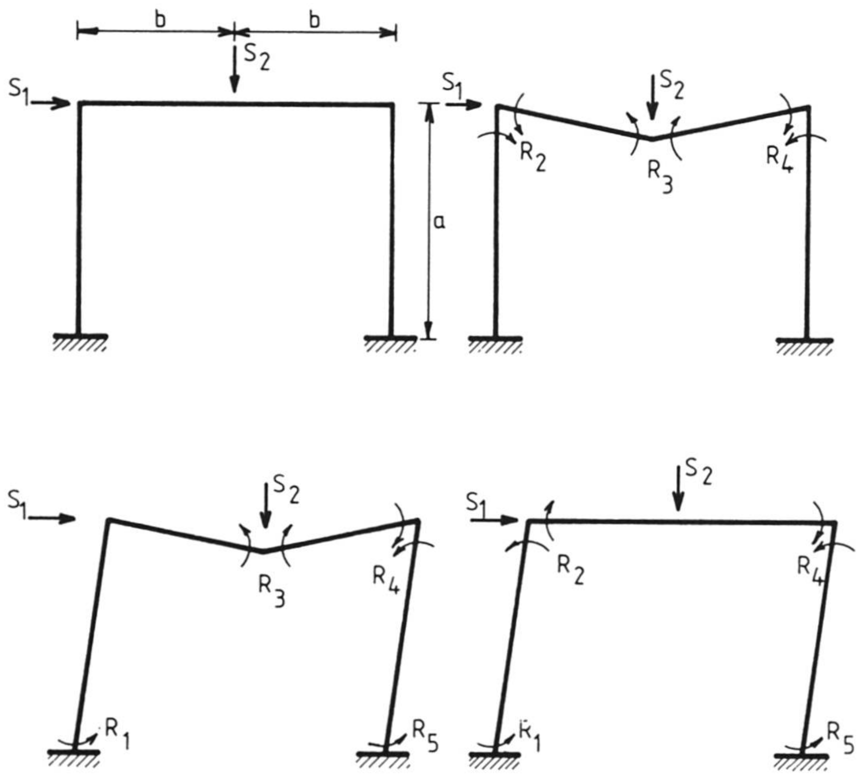

Fig. 3

The failure surface in the space of the 7 design variables is defined by the hyperplanes

$$Z_1 = R_1 + 2R_3 + 2R_4 + R_5 - S_1 a - S_2 b = 0$$

$$Z_2 = R_2 + 2R_3 + R_4 - S_2 b = 0 \qquad (2.21)$$

$$Z_3 = R_1 + R_2 + R_4 + R_5 - S_1 a = 0$$

The safe domain is given by

$$(Z_1 > 0) \text{ and } (Z_2 > 0) \text{ and } (Z_3 > 0) \qquad (2.22)$$

In general, a safe domain is written in the form

$$G(X_1, \dots X_n) > 0 \qquad (2.23)$$

in the n-space of random design variables $X_1 \dots X_n$. We assume no knowledge on these other than their range of values and their second moments.

The range of values of a variable is of interest only on the dangerous side of its mean. There are two cases:

- Variables of the type S_1, S_2 in Fig. (3). Their range is unlimited to the dangerous side of their mean
- Variables of the type of $R_1, \dots R_5$ in Fig. (3). Their range is bounded by the origin on the dangerous side of their mean. They are replaced by variables of unlimited range on the dangerous side by using the new, transformed variables

$$T_i = \ln R_i \qquad (2.24)$$

Their second moments are obtained from [Ditlevsen 1978]:

$$E(T) = E[\ln R] = \ln E[R] - \frac{1}{2} \ln (1+V_R{}^2)$$

$$\mathrm{Var}(T) = \mathrm{Var}[\ln R] = \ln (1+V_R{}^2)$$

$$\mathrm{Cov}(T,S) = \mathrm{Cov}(\ln R,S) = \frac{\mathrm{Cov}(R,S)}{E(R)}$$

$$\mathrm{Cov}(T_1,T_2) = \mathrm{Cov}(\ln R_1, \ln R_2) = \ln \left(1+\frac{\mathrm{Cov}(R_1, R_2)}{E(R_1)\,E(R_2)}\right) \qquad (2.25)$$

After this range transformation, we are left with an n-space of (in general) correlated design variables

$$\mathbf{U} = \{S_1 \dots, T_1, T_2 .. T_k\} = \{U_1 \dots U_n\} \qquad (2.26)$$

with a vector of mean values

$$\mathbf{E} = \{\bar{S}_1 \ \dots \ \bar{T}_1, \ \bar{T}_2 \dots\} \tag{2.27}$$

and a covariance matrix

$$\mathbf{C} = \begin{bmatrix} \mathrm{Var}(U_1) & \mathrm{Cov}\,(U_1, \ U_2) \dots & \mathrm{Cov}(U_1, \ U_n) \\ & \mathrm{Var}\,(U_2) \dots\dots & \mathrm{Cov}(U_2, \ U_n) \\ (\text{Symmetrical}) & & \vdots \\ & & \dots \mathrm{Var}(U_n) \end{bmatrix} \tag{2.28}$$

2.3 Normalization

By a linear transformation

$$\mathbf{U} = \mathbf{MY} + \mathbf{E} \tag{2.31}$$

we normalize the n-space $\mathbf{U}$ to standard form with zero means and unit diagonal covariance matrix. The transformation matrix is obtained from

$$\begin{aligned} \mathrm{Cov}(\mathbf{U}) &= E\left((\mathbf{U}-\mathbf{E}) \quad (\mathbf{U}-\mathbf{E})^t\right) \\ &= \mathbf{M}\ E(\mathbf{YY}^t)\mathbf{M}^t \\ &= \mathbf{MM}^t = \mathbf{C} \end{aligned} \tag{2.32}$$

The safe domain in the normalized space $\mathbf{Y}$ is written in the form

$$\begin{aligned} G(S_1 \dots R_1 \dots) &= G\ (S_1 \ \dots \ e^{T1} \ \dots) \\ &= G_T(\mathbf{U}) = G_T(\mathbf{MY}+\mathbf{E}) \\ &= G_N(\mathbf{Y}) = G_N(Y_1 \ \dots Y_n) \end{aligned} \tag{2.33}$$

$\mathbf{Y}$ is the standardized error vector of the design problem.

2.4 Standard error space [Ditlevsen 1978]

The basic postulates of second moment reliability are the following properties of the joint distribution of the standardized error vector

$$\mathbf{Y} = \{Y_1 \ \dots \ Y_n\} \tag{2.41}$$

Continuity The marginal density functions $f_{Yi}\ (y_i)$ are continuous.

Independence The random variables $Y_1 \dots Y_n$ are mutually independent.

Isotropy The joint distribution

$$f_{Y_1 \dots Y_n}\ (y_1 \dots y_n) = f_{Y_1}\ (y_1) \cdot \ \dots \cdot f_{Y_n}\ (y_n)$$

is a function of the distance

$$r = \sqrt{y_1^2 + \ldots + y_n^2} \qquad (2.42)$$

from the origin only.

Surprisingly enough, these general qualitative properties determine the probability law of the error vector uniquely: each component has a (Gaussian) standard normal distribution with density

$$\varphi_{Yi}(y_i) = \frac{1}{\sqrt{2\pi}} e^{-y_i^2/2} \qquad (2.43)$$

For an elementary proof, see Mathai and Pederzoli [1977] or Breiman [1969].

En engineering decision is well founded if it uses the information accessible to the decision maker. In the case of error space, this implies compliance with the conditions (2.42) within the limits of accessible information. In particular:

- Avoid discontinuous distributions. Discrete probability events in a model must be handled outside of error space. Ditlevsen [1978b] has shown how this is possible by use of the total probability theorem.
- Avoid nonlinear functional and stochastic dependence between variables through suitable formulation of the problem. Only linear regression type dependence (correlation) can be properly handled.
- Avoid anisotropy by suitable transformation of variables. Most important sources of anisotropy in error space are limitations of the range of variables (see 2.24) and extreme value selection: the variable

  $$W = \max [X_1, X_2 \ldots X_k] \qquad (2.44)$$

  has a distribution defined by an integral over a cuboidal domain of the space $X_1 \ldots X_k$ (Fig. 4). If the individual X_i are legitimate components of error space, W is not. If it is used, it must be subject to a normalizing transformation, at least in the region of interest for safety. Technical details and procedures for this may be found in Fiessler, Hawranek and Rackwitz [1976].

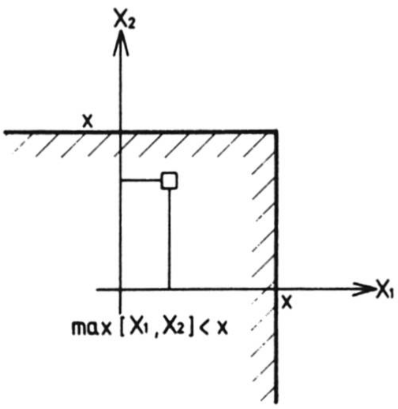

Fig. 4

Apart from these considerations, any speculation on the "true" probability law of design variables is pointless. Second moment reliability handles uncertain quantities with the properties (2.42) after normalization. They are normally distributed by these very properties.

2.5 The general second moment safety index

Historically, safety indices have now a decade-long history of trial and error. In their original definition (2.1), they were soon discovered to be seriously flawed. The reason was the anisotropy introduced into error space by using the variable R without an appropriate range transformation. This problem was overcome by the Esteva/Rosenblueth safety index

$$\beta = \frac{\ln \bar{R} - \ln \bar{S}}{\sqrt{V_R^2 + V_S^2}} \tag{2.51}$$

Next, Ditlevsen discovered the problem of manipulative invariance [Ditlevsen, 1973]. The answer was the now well-known Hasofer-Lind safety index β_{HL} [Hasofer, Lind 1974]. In standard error space, β_{HL} is the minimum distance from the original to the failure surface G_N. For smooth G_N (Fig. 5), the "design point" is located at

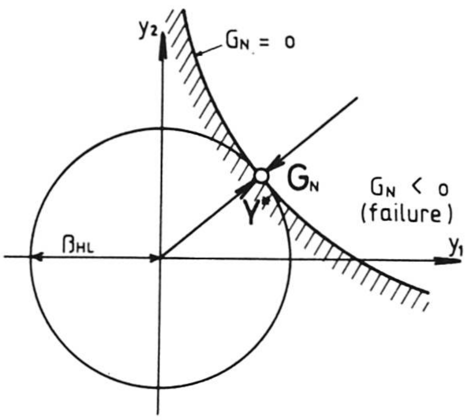

Fig. 5

$$Y^* = -\beta_{HL} \cdot \frac{\mathbf{G}_N}{|\mathbf{G}_N|} \tag{2.52}$$

where $\mathbf{G}_N$ is the gradient vector of G_N at Y^*:

$$\mathbf{G}_N = \{\frac{G_N}{y_1} \cdots \frac{G_N}{y_n}\}|_* \tag{2.53}$$

From (2.31)

$$\frac{\partial G_N}{\partial y_i} = \sum_j \frac{\partial G_T}{\partial u_j} \frac{u_j}{y_i}$$

So that

$$\mathbf{G}_N = M^t \mathbf{G}_T$$

and the design point before normalization is

$$U^* = E - \beta_{HL} \cdot \frac{MM^t G_T}{\sqrt{G_T^t MM^t G_T}}$$

or

$$U^* = E - \beta_{HL} \cdot \frac{CG_T}{\sqrt{G_T^t CG_T}} \tag{2.56}$$

For uncorrelated U_i, this simplifies to

$$u_i^* = \overline{U}_i - \beta_{HL} \frac{\sigma_i \frac{\partial G_T}{\partial u_i}|_*}{\sqrt{\sum_j (\sigma_j \frac{\partial G_T}{\partial u_j}|_*)^2}} \sigma_i$$

or

$$u_i^* = \overline{U}_i - \beta_{HL}\, \alpha_i\, \sigma_i \quad \text{with} \quad \alpha_i = \frac{\sigma_i \frac{\partial G_T}{\partial u_i}|_*}{\sqrt{\sum_j (\sigma_j \frac{\partial G_T}{\partial u_j}|_*)^2}} \qquad (2.57)$$

This equation is the point of departure for many approximations to the Hasofer-Lind model in design codes. Note that

$$\Sigma \alpha_i^2 = 1 \qquad (2.58)$$

In the special case where G is a hyperplane, the coefficients α_i are constants. From the definition of β_{HL} and the properties of error space, the probability contents of the safe domain in error space is

$$r = \phi(\beta) \qquad (2.59)$$

This "second moment reliability" is an operational quantity without practical meaning. To avoid confusion with relative-frequency reliability statements, it should not be used in any other than an operational, technical context. In particular, any use of (2.59) for estimates of risk from structural failures should be openly denounced as unscientific and fraudulent.

If G is not close to a hyperplane, the Hasofer-Lind safety index has certain weaknesses. In particular, it may be unable to differentiate between cases of clearly unequal second moment reliability when the safe domain of one case is a true subset of the safe domain of another case (Fig. 6)

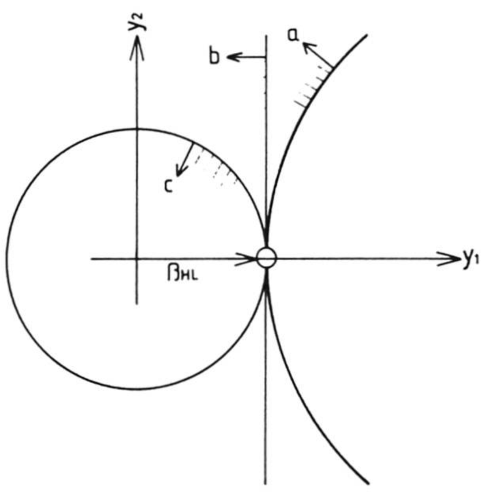

Fig. 6

$\beta_{HL}(a) = \beta_{HL}(b) = \beta_{HL}(c)$

but

$r_a > r_b > r_c$

The best generalization of the Hasofer-Lind safety index to such cases is [Ditlevsen 1978]

$$\beta = \phi^{-1}(r) \qquad (2.59)$$

$$\text{with } r = \int_{\text{Safe Domain}} \varphi(y_1) \ldots \varphi(y_n)\, dy_1 \ldots dy_n$$

In particular, such a definition will allow proper treatment of extreme value expressions like (2.44) without special normalizing transformations.

3. THE HAZARD SCENARIO CONCEPT

3.1 A short history of design codes

Decision making in design becomes more difficult as the amount of information available to the designer increases [Tribus 1969]. The history of codes for structural design is a good illustration of this.

The background of traditional codes from the first half of this century was individual experience of eminent designers and communication between small groups of such professionals. The sample size accessible in such an "interactive" setting for the validation of a decision rule is 100 or smaller. Events with probabilities of occurrence below 10^{-2} or 10^{-3} may go unnoticed and are then subjectively defined as "impossible". The associate subjective notion of "absolute" safety is an excellent basis for decision making. In particular, the joint occurrence of actions is not a conceptual problem: if "absolute" safety is the goal, then any set of given code actions must be combined into a "worst case" scenario for the purpose of design decisions. This notion is deeply ingrained in classical linear statics; the importance given to influence lines in that context tells it all. The second important element of traditional codes was the use of admissible stresses as an unspecific allowance for uncertainties.

The conceptual frame of traditional codes broke down under the impact of limit state analysis. Limit state analysis was introduced into structural engineering by experimental researchers, not by structural designers. In the transition, very little care was taken to replace the traditional decision rules by new ones of similar generality and logical consistency. In particular, the clear-cut, but not very explicit notion of worst-case scenario design was lost. Outside of the elastic regime, it is not practical to insist on worst-case "combination" of given code actions. Only the best codemakers realized the full scope of definition problems so generated.

An important step toward a design philosophy adapted to the potential of limit state analysis was made by Danish soil mechanicians in the fifties [Brinch Hansen/Lundgren 1960]. They realized that the unspecific allowance for uncertainties in the form of an admissible stress or a load factor was unsatisfactory. Instead, they proposed application of an individual safety element to each uncertain design quantity, a "partial factor". The discussion on invariance in second moment reliability has later shown that the essential point is not the partial factor (which can be replaced by another rule for definition of a design value), but the one-to-one association of a design variable with its design value. No other format is free of invariance problems. In particular, any design format with factors between aggregates of variables suffers from invariance problems.

Second moment reliability can be used directly for the quantification of a "partial factor" format. The nominal limit state postulated by Brinch Hansen is the design point in Fig.5; design values of variables have the form (2.57). The first implementation of a code on this basis also came from the nordic countries [Nordic Code 1974].

The present state-of-art of general principles for design codes is probably best seen in [Joint Committee 1978]. The paper is, in a general form, a descendant of the Nordic Code. In particular, all logical inconsistencies from the Joint Committees "semiprobabilistic era" have been eliminated. The weaknesses of the paper are probably not in the text, but in what is missing from it:

- there is no strong, unified concept for the definition of what the designer should design for. Clearly, the construction of a "worst case scenario" from given code actions is not implied. In a section labelled "combinations of actions", a somewhat synthethic rule is given, including reduction factors on variable actions. The writer doubts whether any meaningful engineering design work will ever be possible under such a rule.

- there is no indication whatsoever on how practical experience should be fed back into a code. This is not a trivial question given the obvious necessity for internationalization of codes and the large amounts of strategically gathered and statistically evaluated data accessible to codemakers now. The days of interactive feedback are gone, but what replaces those simple learning patterns ?

In the sequel, we concentrate on these two cardinal issues. We have no ready answer, but we hope to propose first steps in the right directions.

3.2 The morphology of design life

From the viewpoint of the designer, the life of a structure begins with the first construction steps and ends with the intervention of another planner or designer when the structure is rebuilt, adapted to a new purpose or torn down. Between these endpoints, a considerable number of constructions states, service states, and special repair and revision states may require attention.

Over the duration of design life, the structure or its parts already in place are threatened by certain hazards. These hazards fall in two broad classes:

- Man-made-hazards: Errors, mistakes and blunders in planning-, design-, and construction procedures. Overload by loss of control over service loads, accidents in service, fire, vehicular collisions with structural elements. Fatigue and deterioration in conjunction with deficiencies of maintenance procedures.

  Man-made geotechnic hazards and other hazards from the built environment.

- Natural hazards: Wind, water, snow, ice, earthquakes, avalanches landslides ...

Completeness is essential for planning and design with respect to safety. It makes sense to use formal morphological methods for identification of relevant safety problems [Zwicky, 1966]. Their form depends on the particular problem. A simple example with a two-way morphological matrix is shown in Fig. (7). Each state-hazard intersection in the table identifies a possible hazard

		Man-made hazards					Natural hazards			
		Fire	Mistakes Blunders	Overload in Service	Explosion Collision	....	Wind	Water Snow Ice	Earthquake	..
Construction States	Excavation									
	Foundations									
	Transport of Elements									
	Erection of Elements									
	Concrete Phase 1									
	1st Prestressing									
	....									
Service States	....									
	Ordinary								☒	
	Special									
	Revision									

Fig. 7

scenario. In the definition of "The Advanced Learner's Dictionary of Current English", a scenario is "the outline of a film, giving the story, scenes, directions for actors, etc.". This is exactly what the planners and designers of a structure should make of it. It is well known that the best designers have always based their work on a clear vision of distinct future situations. The quality of design and planning depends on the clarity of this vision. Codes with synthetic "combination rules" for actions are a disaster for such needed creative immagination. In the opinion of the writer, the word "load combination" should be deleted from all structural codes forever.

The two-way hazard-scenario table may be insufficient. Frequently, catastrophic events seem to follow the pattern of Fig. (8).

Planners of very sensitive systems with large consequences of failure should probably be required to go through the formidable task of qualitative evaluation of a three-way hazard scenario table.

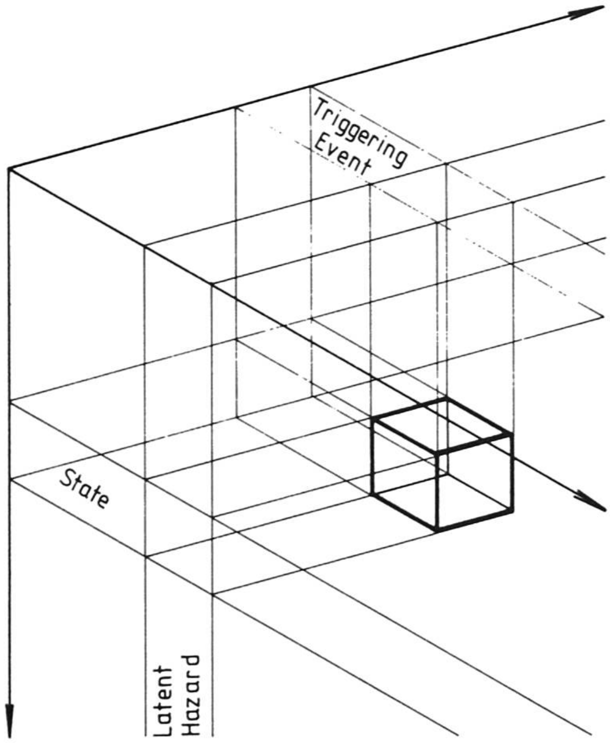

Fig. 8

3.3 The definition of the designer's task

Contrary to the apparent belief of certain "international" code-makers, the safety of structures is not the exclusive domain of the structural designer. The most consequential decisions for structural safety are made early in the planning stage. Ideally, a careful qualitative evaluation of a hazard scenario table including an effort of immagination on the implications of the (usually small) set of relevant scenarios should take place at that time. The full scope of available actions is large: Hazards can be

- avoided by changes in the overall design or the location of a building or structure
- controlled by measures of management, supervision, or by auxiliary technical devices
- overpowered by structural design and general robustness provisions
- accepted as inevitable,

Im most cases, the best policy will be a well balanced combination of actions from all these categories. Such a package of coordinated actions in various areas is itself a planning task. We label it the safety plan.

While it is clear that the structural designer will play an important role in the elaboration of such a plan, it is just as clear that its success will depend mainly on communication with others: managers, specialists from other disciplines, supervisors, and the owner. The basis of communication is a common language. One of the most dangerous flaws of today's structural codes lies right here:

it is difficult to explain to an outsider what our design criteria really are. It is very common to find architects, owners, managers, or construction workers shrug their shoulders at our preoccupation with a safety problem and ask: "Don't you over-design it anyway ?". We should probably see this as an indication of insufficient clarity and intellectual quality in our codes. Ideas which are difficult to communicate are frequently not worth communicating ...

Calling for a safety plan may be overly perfectionistic for most ordinary run structures. After all, everybody in the business of planning, designing and construction takes his precautions. In such everyday problems, the notion of a safety plan may just serve as a reminder: There is a fundamental need for communication and coordination in any planning, design and construction process. If this need is neglected, safety or economy (or both) will be impaired.

The "General Principles on Reliability..." [Joint Committee 1978] begin with the following sentence:

"Structures or structural elements should be designed such that, with appropriate degrees of reliability, they sustain actions liable to occur during construction and use ..."

In our view, this should read otherwise:

"Structures or structural elements should be designed to meet the requirements of the safety plan for the building or the overall structure."

3.4 Modelling a hazard scenario

The infrastructure of design decisions in a modern code is shown in Fig. 9. A model for a hazard scenario at the level of code theory integrates elements from the five sources outlined in the picture:

- a qualitative description of a standard scenario from the morphology of design life

- simplified models for actions of importance in that scenario. Modern scientific models for actions are, in general, stochastic both in space and in time. A typical example is the live load model for buildings by Peir and Cornell [1973].

  For direct use by code committees or individual designers, random fields are too complex. The nature of simplified hazard description in engineering has been discussed by Cornell [1973]. Most hazards may be characterized, for design purposes, by a small number of parameters which are, in the worst case, dependent random functions in time. Each member of the class of simplified models for actions on structures is, in other words, a vector-valued stochastic process. The components of the vector are intensities or geometric parameters. They may depend on each other, but they are independent of other actions.

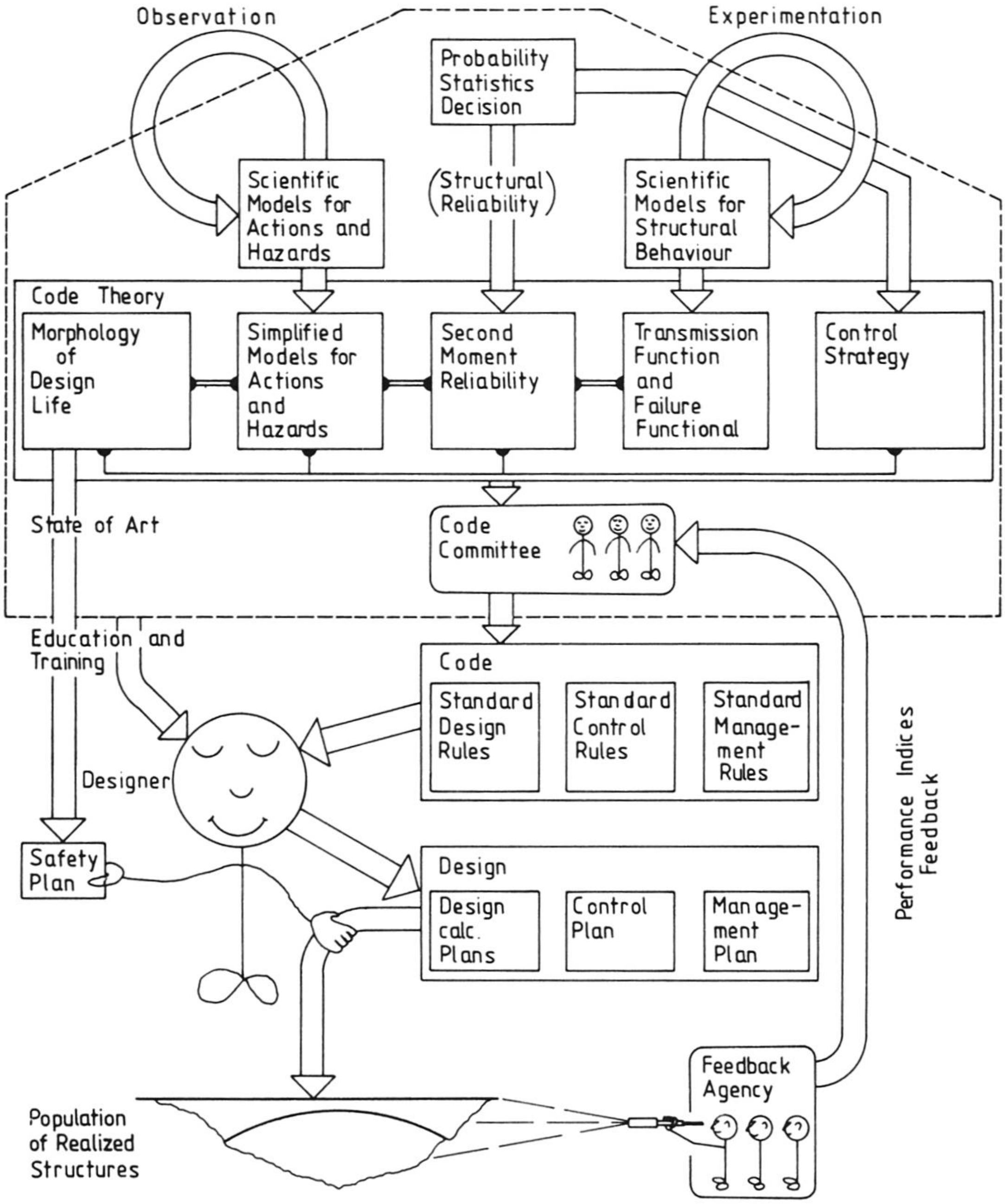

Fig. 9

- a deterministic transmission function and a failure functional for the structure or the structural element considered. In contrast to most such relations now in use, they should give an unbiased prediction of the elements behaviour in the range of interests. See (3.2) and (3.3) below.

- certain elements prescribed by the control strategy of the code. Typically, this could be the global safety index β and some sectorial model uncertainties.

- the second moment reliability model (2). Some additional formal structure is required to make simplified models for actions compatible with the reliability model:

For each hazard scenario, there is a <u>leading action</u> $\vec{A}(t)$, a vector valued stochastic process. Other actions simultaneously present are

$$\vec{B}_1(t), \ \ldots\ldots, \ \vec{B}_k(t) \qquad (3.1)$$

We label them <u>companion actions.</u> The distinction between the leading action and the companion actions will always be clear from the genesis of the hazard scenario in the hazard scenario table.

In general, we assume all actions to be stochasticallly independent from each other. This may be brought about by proper identification of joint phenomena in load modelling. As a didactical example, consider the design of a small footbridge

(a) in a public botanical garden

(b) over an alpine down-hill race track

for crowd loads and snow loads; crowd loads is the hazard identified in the scenario.

The proper definition of actions is

(a) $A_1(t)$ = crowd load
$A_2(t)$ = 0 (no snow load)

(b) $A_1(t)$ = crowd load
$A_2(t)$ = snow load correlated with crowd load.

In both cases, snow is not a companion action, because its occurrence is correlated with crowd loads. The correlation is negative in case (a) and positive in case (b). The example may be farfetched, but it highlights an important point: the morphology specific to the particular project has a decisive influence on the correlation structure of actions. This is another reason why concepts like "load combination" and "load combination factors" should be deleted from the vocabulary of structural codes. The codemaker provides the elements of the scenery; he does not write the scenario of the piece.

For each hazard scenario, there is a well defined state of the structure and a <u>transmission function</u>

$$\mathcal{F}\ \{\vec{A}(t),\ \vec{B}_1(t)\ \ldots\ \vec{B}_k(t)\} \rightarrow \mathcal{S}(t,\ \vec{s}) \qquad (3.2)$$

which maps the set of actions into the history of stresses and/or deformations of the structure. t denotes time and $\vec{s}$ location.

For each hazard scenario, there is a <u>failure functional</u>

$$\mathcal{G}\ (\mathcal{S}(t),\ \vec{s}) \rightarrow G(X_1,\ \ldots\ X_n) \qquad (3.3)$$

which maps the history of stresses and/or deformations into a single scalar variable. In line with (2.33), G is an input to the second moment reliability model. Its parameters are simple random variables.

Two classes of failure functionals are of practical importance for design: first outcrossing functionals and damage accumulation functionals. Common examples are overload and fatigue, respectively.

The essential feature of a failure functional is that it eliminates randomness in time from the problem. In the case of first outcrossing failures, functionals are best formulated with the help of Turkstra's principle [Turkstra 1970]. In our setting, it may be stated as follows:

> Failure happens when one of the components of the leading action takes on its maximum over a reference period τ.

Consider a case where A(t) has two dependent components $A_1(t)$, $A_2(t)$ both stochastic in time, and the only companion action B(t) is a scalar process in time. The meaning of Turkstra's principle may then be seen from Fig. 10. Note that each function plotted is just one sample from the stochastic process. With the random variables

A_1, A_2, B	Random-point-in-time values of the processes
$\max_{0 \ldots \tau} A_1, \max_{0 \ldots \tau} A_2$	Maximum values of the processes
$A_2 \mid \max_{0 \ldots \tau} A_1$	Values of one component conditional on the maximum of the other
$A_1 \mid \max_{0 \ldots \tau} A_2$	(3.4)

we may formulate the safe domain in the form

$$G_1 \left(\ldots, \max_{0 \ldots \tau} A_1, A_2 \mid \max_{0 \ldots \tau} A_1, B, \ldots\right) > 0$$

and

$$G_2 \left(\ldots, A_1 \mid \max_{0 \ldots \tau} A_2, \max_{0 \ldots \tau} A_2, B, \ldots\right) > 0 \qquad (3.5)$$

Turkstra's principle is not "true". It is easy to construct counter-examples where failure happens at a point in time where none of the component processes takes on a maximum. However, it has been checked and found satisfactory for a variety of special situations [Waugh and Cornell 1975, Madsen 1978]. Most important, Turkstra's principle is the only procedure available for reduction of stochastic outcrossing problems to a tractable form with a clear qualitatative meaning. This is very important to the code infrastructure shown in Fig. 9; unless the designer understands the model for

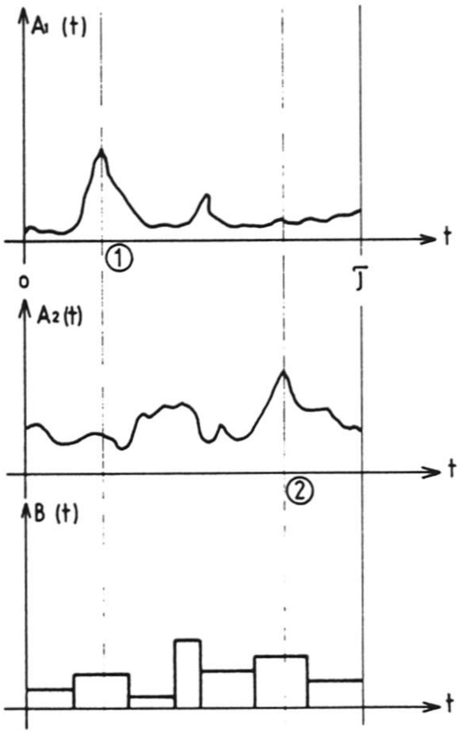

Fig. 10

joint actions, he will not be able to apply the code rules deduced from it properly. Note that the hazard scenario concept mirrors Turkstra's principle at a qualitative level.

The point is that we cannot cope with the complexity and randomness of the world around us in design decisions unless we limit our attention to a small finite subset of wisely chosen states. In the hazard scenario concept, this choice is made in three steps:

- the first step is the qualitative identification of dangerous situations with the help of systematic search in a morphological table

- the second step eliminates randomness in time with the help of Turkstra's principle

- the third step eliminates randomness altogether. The auxiliary device is the Hasofer-Lind definition of a safety index and, in particular, the design point of Fig. 5.

Codemakers are faced with the necessity of one discretization step more. The entire formalism outlined above is problem-specific. In particular, the factors α_i in (2.57) depend on the parameters of the special case considered. For codes, simpler and more explicit formulations are required. A theoretical frame for this step is discussed in chapter 22 of Appendix I of [CEB 1976]. See also Skov [1976] and the Nordic Code [1974] for more information. A problem with a damage accumulation functional in (3.3) has been discussed by Rackwitz [Rackwitz and Fiessler 1977].

4. FEEDBACK

Three different sources of structural engineering experience are indicated in Fig. 9:

- Experiments in the laboratory on the basis of mechanical models for structural behaviour
- Observation and sample statistics based on stochastic models for actions and hazards
- Feedback of practical experience from a population of realized structures.

Like the first two sources of experience, feedback is conditional on an "observation model": the code, with its design-, control-and management rules. Some kind of feedback has existed ever since technical rules, regulations and codes were made. In fact, such rules were usually the reaction to inacceptable, dangerous or uneconomic situations. This traditional type of feedback, which we label <u>interactive,</u> is based on personal experience and direct communication between professionals. It will remain one of the most important contributions to structural engineering experience.

The type of feedback we have in mind here is of a different nature. It may be labelled <u>strategical</u> feedback. Its constituent elements are

- <u>Explicit,</u> quantitative objectives for the population of structures under the code.
- <u>Objective</u> observation of performance indices related to these objectives by a <u>feedback agency</u>
- A <u>control strategy</u> to guide code evolution (and other measures) in order to attain the objectives.

Feedback and control are obvious and necessary ingredients of any rational, directed human activity. As soon as such an activity outgrows the traditional context of natural, interactive feedback, strategic feedback and control procedures must be created. Otherwise, control will be lost, and stated objectives will become empty declarations.

Two examples may illustrate that loss of control is a very real possibility for structural codes and the industry supposed to operate under its directives:

If the forces of a market economy dominate those of the legal system, the public administration and the professional engineering establishment, the price tag on human life may drop to very low levels. A dreadful record of structural failure has been accumulated by some third world countries since 1970. The risk run by workers at construction sites - most of them from under-priviledged classes - is so high that it became a symbol for the dehumanizing forces of unchecked capitalism; listen to Chico Buarque de Hollanda's Song "Construção" [Hollanda 1971].

If, on the other hand, the industrial and professional interests are well established, and if certain sectors are exempt from market forces, a very strong risk aversion tendency may develop at the expense of the owner. In fact, everybody except him is interested in higher formal safety requirements. Most developed countries are probably closer to this second situation than to the first.

4.1 Objectives

The central concern of structural engineering is structural adequacy: the fitness of the structure to serve the purpose for which it is required [The Institution of Structural Engineers, 1976].

The pursuit of structural adequacy is not unconditional. Nobody aims at planning, designing and building the safest possible or the most durable structures, but structures which are safe enough and otherwise as economic as possible, in terms of initial investment, utility in use, costs for maintenance and repair.

Quantitative safety objectives for a code in a developed western country could take the form of limitations on risk to life and limb from structural failures. In the future, such limitations could be part of a national hazard control policy. It is anticipated that human safety is not now and will not be an acute problem.

The annual risk of death of any person in the UK due to collapse of a completed structure from any cause is estimated at 1 in 7 million [Ciria 1976]. This risk should be compared with the 1 in 10'000 risk of the same person to die from a traffic accident, the 5 in 10'000 risk of a construction worker to die at work in any given year, and the general mortality of the population, which is in the range of 1 in 1000 per year at age 30 and 6 in 1000 at age 60.

In a market economy, most of the owners economic interests are well protected by that market itself. The control strategy of the code should aim mainly at the proper balance of safety constraints and control requirements to achieve safety objectives and serviceability with minimal total cost.

4.2 Feedback agencies and performance indices

Performance indices for strategical feedback must be accessible to objective statistical evaluation. This means that

- there must be a well defined target population
- the sampled population must correspond to the target population
- the property or variable sampled must be observable.

Not that the "rate of failures" of structures does not qualify as a performance index. There is no well defined target population: "Structures" are for the most part, models, not physical realities. Many "failures" of structures in the sense of the theory are observable in the laboratory only; others go unnoticed due to secondary elements of the building.

A straightforward performance index for structural safety is the number of fatalities from structural collapse. No particular, new feedback agency need to be created to monitor this quantity: most countries have a national statistic of fatalities already in operation. Classification of causes has been unified in the International Classification of Diseases (ICD). Death from structural collapse is not an entry in that 134 pages long list [Eidgenössisches Statistisches Amt 1970]. Minor subclassifications would bring out the social toll of structural collapse at no extra administrative cost.

The ideal feedback agency for performance indices of economic nature is a public building insurance agency. In many countries, such institutions already process much of the relevant information.

Little extra paperwork would be created if well defined, small sets of indices were recovered from their records. In contrast to fatalities, most of these indices could be estimated from random samples.

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