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Structural Design Decisions and Safety

Décisions sur les projets structuraux et sécurité

Bauliche Entwurfsentscheide und Sicherheit

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Introduction

Food and shelter are two of the most basic human needs and it is the provision of the latter which, of course, is a major concern of the structural engineering profession. The very high level of reliability expected of structures by the general public is probably the result of this basic need and of course the fact failures have in the past been few in number. A number of writers have noted this [1, 2, 3] and a summary of estimates of some typical risks is shown in Table I. It is obvious from the table that the degrees of risk associated with events about which there is much public concern are orders of magnitude greater than the risks associated with the safety of structures.

As structural analysis techniques have improved over the years so has the structural engineer been able to feel more confident about the behaviour of his structure. This has resulted in factors of safety being reduced in value. FREUDENTHAL [4] discussed the nature of the concept of factor of safety and described two basic notions. Firstly that factors of safety are intended to cover the imperfections of the intellectual concepts devised to reproduce physical phenomena (ignorance). Secondly that they are intended to cover the imperfections of human observations and actions (uncertainty). With increasing improvement of analytical techniques the elements of ignorance are being substantially reduced, however the elements of uncertainty may be changed but can never be removed. If the safety factor becomes largely a measure of uncertainty rather than ignorance the continuing trend of lowering its value must not be the result of improved design methods but the result of modified objective circumstances. This is brought about by standardising

Table I. Typical Personal Accident Risks

Risk	Occurrence per exposed hours	Typical hours exposed in lifetime	Probability of occurrence in lifetime
Fatal car accident	57 in 10^8	30,000	0.017
Fatal motor cycle accident	660 in 10^8	15,000	0.1
Fatal Hotel Fire	1 in 10^8	10,000	0.0001
Fatal House Fire	0.1 in 10^8	400,000	0.0004
Risk	**Probability of occurrence in life of structure**	**Typical hours exposed in lifetime**	**Probability of occurrence in lifetime**
Aeroplane Accident	1 in 10^5 hours	200	0.002
Aeroplane Accident due to structural failure	1 in 10^7 hours	200	2×10^{-5}
Bridge collapse	1 in 10^6 years	0.25 years	0.25×10^{-6}
House collapse	1 in 10^6 years	45 years	4×10^{-5}
Office collapse	1 in 10^6 years	15 years	1×10^{-5}

engineering materials, by introducing better quality control in production and by applying standard acceptance tests by users of such materials and stringent regulations for the control of workmanship.

A large proportion of structural accidents, whether involving partial or total collapse, occur because an error is made in the design or construction of the structure. Very few accidents occur through variations in the load and strength of the structure which are large enough to nullify the assumed factors of safety. In other words very few structures collapse if they are properly designed and constructed according to present methods. Errors in the design or construction of a structure may vary from small errors such as slightly misplaced reinforcement in a concrete slab to large errors such as an insufficient consideration of the lateral stability of a building. Small errors are in fact covered by the assumed factor of safety. The prediction of the likelihood of large errors has received recent attention [5].

Design has been defined by Asimov [6] as "Decision making under conditions of extreme uncertainty". Although every practising engineer knows this and recognises it in the execution of his work he has only up to the present indirectly acknowledged the large variations in the value of the design parameters in his calculations. In fact certain extreme values of load and strength parameters, usually as quoted in British Standard specifications or codes of practice, are taken as fixed values in the calculations and safety factors then applied. The calculations are in fact deterministic i.e. the values of the parameters are assumed to be fixed in value. An alternative to this is termed the probabilistic or stochastic approach where each of the parameters are known to vary in value and these variations are taken into account in the calculations.

Limit state design recognises the variability of certain parameters by the use of characteristic values and partial factors. The partial factors serve part of the function of the traditional safety factor in as much as they also cover non-statistical variations such as minor errors in design or construction and incomplete theoretical knowledge. However, the partial factors have to cover the fact that characteristic values of the parameters cannot be determined with any confidence because there is not sufficient information available concerning the variations of those parameters. It is encouraging that more work collecting and analysing data [7, 8, 9] is being done.

Limit state design is therefore semi-probabilistic. It is important that the designer realises that the absolute safety of a structure can never be guaranteed, a fact that is disguised by the use of factors of safety. Of course the probability of failure is extremely low and may be of the order of 10^{-4} to 10^{-10}.

Most authors discussing structural safety agree that the uncertainty involved in structural design can only be tackled using probability theory. Probability theory allows the treatment both of statistical data and of subjective assessment. This means that the practical experience that all engineers recognise as essential to ensure professional competence can be used quantitatively to separate the factors which contribute to the engineer's uncertainty about his design. The concept of a safety factor is then replaced by the concept of probability of failure fixed at a suitable low value.

The rigour involved through looking at structural design in a probabilistic way brings benefits in other directions also. Every designer likes to think he is working towards a solution which is the optimum solution under the particular circumstances. In other words he attempts to ensure that every decision that he makes is the best one under the circumstances. This is most often done intuitively and therefore depends upon the experience of the designer making the decision. A lot of attention has been given recently to formal mathematical methods of optimising the form of a structure and the sizes of the components making up the structure. Linear programming and geometric programming have been used in this way as for example in references [10, 11]. It is not easy to compare directly the results of such techniques, as the degree of safety provided by the methods are not always consistent for different structural solutions. However decision theory [12, 13] provides a theoretical framework to enable such comparisons and includes the probabilistic approach to design.

Decision Theory

Decision Theory was developed in order to help business management make better decisions. Managers are often faced with decision problems about which they have little information. They have to decide whether to seek new information (which may or may not involve extra costs) or whether to make their decision on the available information. They may also be uncertain about the consequences of their decisions. Structural engineers are faced with similar difficulties. If a particular structural solution is adopted the consequences may depend upon some factor which is not known with certainty. This factor is called "the state of nature". The factor may be the total settlement of the soil below a proposed bridge footing or the deflection of a particular beam, for example.

The decision making process is formulated as the process of choosing an action a_i from among the available alternative actions $a_1, a_2 \ldots$, the members of action set or space A. Once the decision has been made a state of nature θ_j will occur in the set of possible states θ and the consequences will be a loss (or gain) of expected value $E(u/a_i, \theta_j)$ [read as u given a_i, θ_j] a numerical measure of the benefit gained from the decision taken. The function $u(a, \theta)$ is known as a utility function. If this function is continuous it has an expected value (given a_i and θ_j) which is determined by a sample of values $x_1, x_2, x_3 \ldots$ with a probability density function $f(x/a_i \theta_j)$ of

$$E(u/a_i \theta_j) = \int_{-\infty}^{\infty} u(a, x) f(x/a_i \theta_j) dx.$$

The best decision or action a_i is however the action that produces the maximum value of the expected utility given only a_i and not θ_j. If therefore θ is discrete

$$E(u/a_i) = \sum_j E(u/a_i \theta_j) P(\theta_j/a_i).$$

This is shown diagrammatically by a decision tree (Fig. 1).

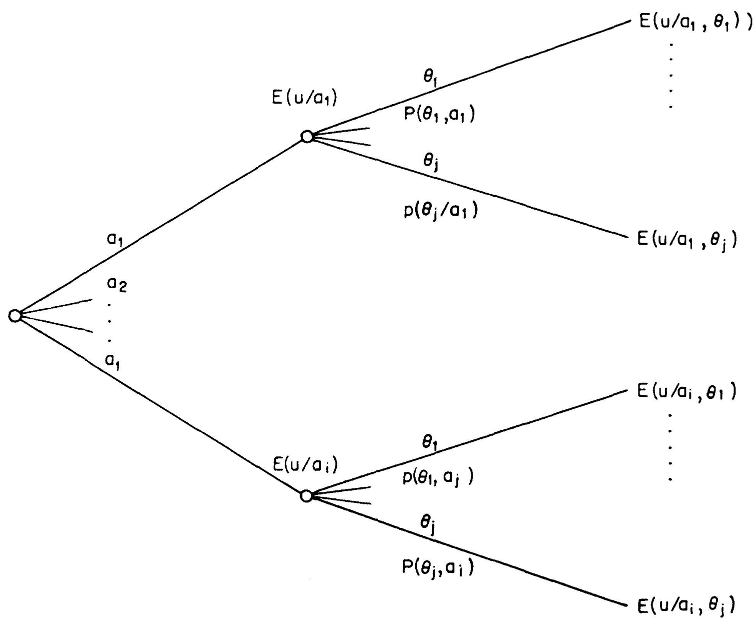


Fig. 1. Decision Tree.

Comparison of Structural Solutions

As mentioned earlier, formal mathematical methods of optimisation may be used to decide on the best structure given the type of structure. Alternatively, as is usually the case, the best structure of a type is produced by subjective assessment on the part of an experienced engineer. However a major difficulty arises when deciding on the best structural solution from a number of completely different alternatives. These alternatives may differ in form and in the type of materials used. Decision theory may be usefully applied by considering the best structural alternative to be that providing maximum expected utility.

A structure such as a building consists of a number of distinct parts each of which may be of interest to different professional disciplines. The structure itself, the foundations, the heating, ventilating and other services, the internal finish are all cost interactive and of course it is no use optimising each part if a global optimum is not considered. Consider a structure composed of M_i such parts where $E(I_j/a_i)$ is the expected initial cost of the jth part ($j = 1, 2 \ldots M_i$) given that structural alternative a_i is chosen. Assume that each of these parts have N_{ij} possible states of nature θ_{ijk} ($k = 1, 2 \ldots N_{ij}$) with a probability of occurrence p_{ijk}, then the expected utility of alternative a_i is

$$E(u/a_i) = \sum_{j=1}^{M_i} [E(I_j/a_i) + \sum_{k=1}^{N_{ij}} p_{ijk} E(u/a_i, \theta_{ijk})].$$

Structural engineers are of course interested in the behaviour of the structure and Fig. 2 shows the decision tree considering only some possible states of nature for the structure. An extra set of branches on the decision tree is included to allow for alternative optimisation methods. For simplicity of notation the expected value of the utility given the ith structural alternative, the jth optimisation technique and the kth state of nature is denoted as C_{ijk} with a probability of occurrence p_{ijk}.

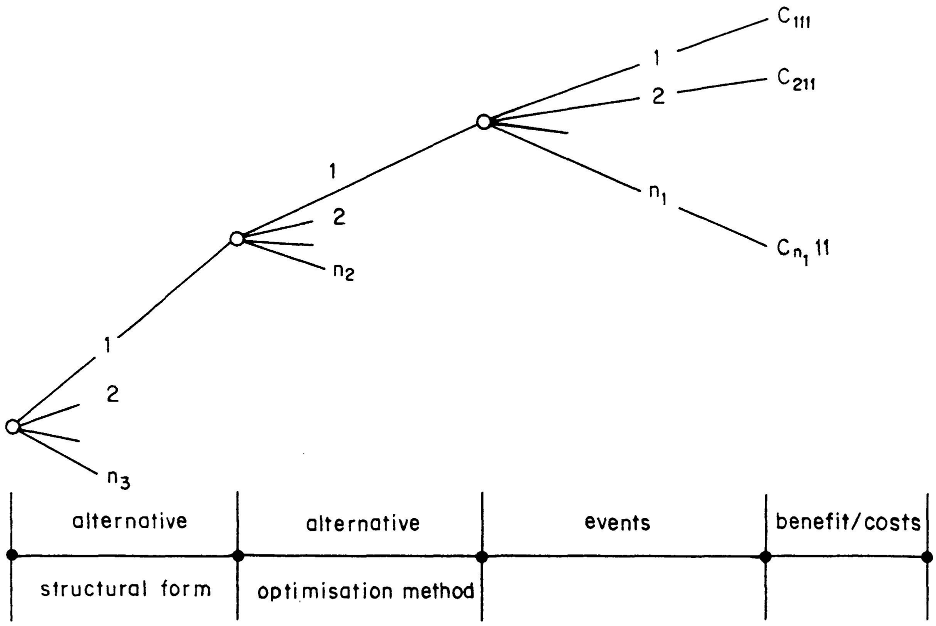
Thus if n_1 is the number of states of nature,
n_2 is the number of optimisation methods considered (if any),
n_3 is the number of alternative structural types considered,

then, if the structure alone is considered, the structural type adopted should be

$$\max_{i}^{n_3} \left\{ I_i + \max_{j}^{n_2} \left[\sum_{k=1}^{n_1} p_{ijk} C_{ijk} \right] \right\},$$

where I_i is the expected value of the initial cost of the ith alternative structural type.

Now, of course, I_i is a function of p_{ijk}. In other words, the safer the structure the more expensive it is to build. The determination of appropriate values for the probabilities is dependent upon two principal factors. Firstly, the maximum values should be those which do not bring about undue public concern



Event No.	Event	Benefit	Probability assessment
1	Satisfactory through life	Rentals – Maintenance	very close to 1
2	Accident – design error	– Cost of repair	subjective?
3	Accident – construction error	– Cost of repair	subjective?
4	Collapse – variations in load and strength > assumed	– Cost of repair	simulation
5	Partial failure as 4 i.e. exceeding various limiting states	– Cost of repair	simulation
6	Explosion	– Cost of repair	statistics
7	Fire	– Cost of repair	statistics
8	Floods	– Cost of repair	statistics
9	Earthquake	– Cost of repair	statistics

Fig. 2. Decision Tree for Comparison of Alternative Structural Designs.

over structural failures. Secondly, the values should be those which bring maximum utility. It is not possible, without much more information about the relationship between initial costs and the probabilities of limiting states being exceeded, to calculate optimum values for these probabilities.

TURKSTRA [14] however has considered the relationship between probability of failure and total cost of a structure. He showed that below certain levels of failure probability the utility loss associated with failure of the structure becomes negligible compared to the initial cost of the structure. In other words the probability of failure regarding only the economic consequences becomes effectively zero. However Turkstra does not recognise that the actual probability of failure may still be high enough to cause undue public concern and in an absolute sense cannot be regarded as zero.

Calculation of Probability of Failure

The probability of occurrence of explosions, fires, floods, etc. may perhaps best be found by studying the frequency of occurrence in available statistics. Fig. 3 shows a frequency histogram produced by the Fire Research Station [15]. However it is the calculation of the probability of failure or partial failure under various combinations of dead, live, wind and earthquake loads etc. which is of interest to structural engineers. In order to calculate probabilities of failure, the variations in the loads and the variations in the strength parameters of the structure must be known. It is surprising that so few surveys of, for instance, office floor loads have been undertaken until quite recently [8]. PEIR and CORNELL [7] have examined the use of this survey data on office buildings. Wind loading and earthquake loading have been given far greater attention, but again until quite recently not much attention had been drawn to the variations in strength parameters such as steel yield strengths and the various geometric properties [9].

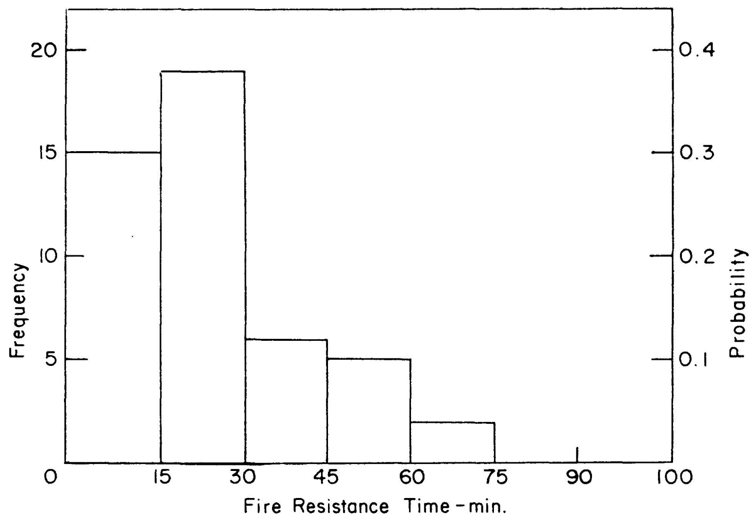


Fig. 3. Frequency Diagram for Predicted Fire Resistance Requirements.

The calculation of the probability of failure when standard probability density functions of load (Q) and strength (R) can be assumed is straightforward. There is a slight overlap of the load and strength effect distributions at the extremes, so that in this area the load effect is greater than the strength effect and failure occurs. The probability of failure occurring is given by

$$p_f = P(R < Q) = \int_{-\infty}^{\infty} f_q(x)\, F_R(x)\, dx, \tag{1}$$

where $f_q(x)$ is the probability density function of the load effect and $F_R(x)$ is the cumulative density function for the strength effect.

The major problem is that given certain data (in the form of histograms) regarding the load and strength effects the functions $f_q(x)$ and $F_R(x)$ have to be determined. Whatever curve fitting method or probabilistic analysis used it is certain that although the curves chosen may fit the data quite well in the region of the mean and mode of the data the curves will not fit well at the extremes because of the very fact that values at the extremes do not occur very often. Different assumptions for the equations of those curves may according to given statistical tests fit the data equally well but in fact give different values for the probabilities of failure. However it is likely that an order of magnitude calculation for the collapse failure probabilities is sufficient and the exponent (herein called the safety index) need only be quoted (Table II).

Table II. Simulation Results for Probability (p_f) after 7×10^5 cycles

Limit State	Mean		Variance	Coefficient of Variation
	Integration	Bernoulli		
Collapse	1.699×10^{-4}	1.457×10^{-4}	2.081×10^{-10}	0.099
Deflection	0.1082	0.1082	1.378×10^{-7}	0.003
Max. Stress	0.113	0.1064	1.358×10^{-7}	0.003

CORNELL [16] has calculated bounds on the probability of failure of a structure which may fail in one of M modes under one of N loads in terms of values of P_{mfn} the probability of failure of the mth mode under the nth load. These values are calculated in a manner similar to equation (1) but assuming that the load and strength effects are random variables having distributions which vary in time.

An alternative approach is to simulate the life history of a structure by applying loads to the structure which are generated in a random way from a load distribution function which describes the variation in load intensity over the life of the structure. Restricting attention to the class of structures which are such that the resistance or strength of the structure does not vary with time but does in fact vary over the actual structure (in space), it is possible to also generate random values of strength to compare with the random values of load. By comparing such values generated a large number of times an estimate of the probability of failure or of any limiting state being exceeded can be calculated.

Portal Frame Example

In order to illustrate how the concept of simulation of the loading history of a structure might be carried out a portal frame structure (Fig. 4) was

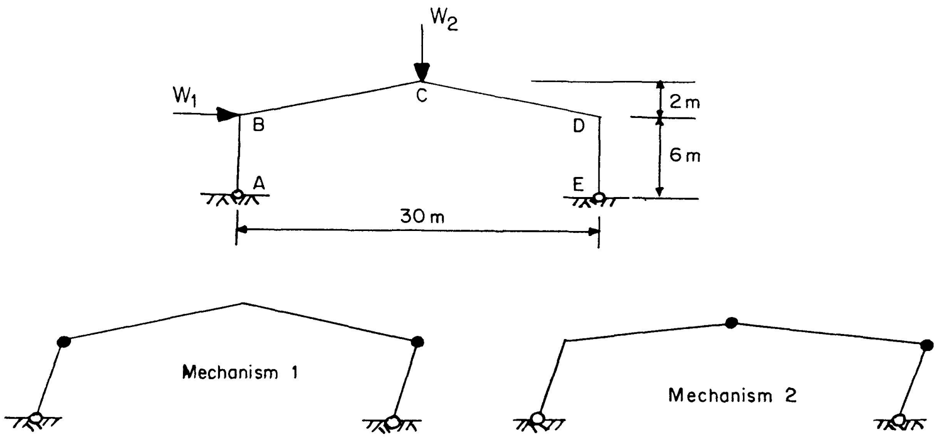


Fig. 4. Portal Frame and Collapse Mechanisms.

analysed. The probabilities of exceeding the following limit states given the cumulative frequency polygons over the life of the structure for the applied loads W_1 and W_2 and c.f.p.'s for the strength parameters yield stress (f_y) and plastic modulus (z_p) in space were calculated. The limit states were:

1. Collapse.
2. Unserviceability a) Vertical elastic deflection $\ngtr$ 120 mm.
   b) Elastic bending stress $\ngtr f_y$.

The two possible collapse mechanisms for the portal are shown in Fig. 7 and by well known methods collapse occurs if

$$\text{for mechanism 1 } M_p < 3\,W_1,$$
$$\text{for mechanism 2 } M_p < 1.28\,W_1 + 3.22\,W_2.$$

Now $M_p = f_y z_p$, i.e. collapse occurs if

$$f_y z_p < \max \begin{bmatrix} 3\,W \\ 1.28\,W_1 + 3.22\,W_2 \end{bmatrix}.$$

The vertical elastic deflection at the apex of the frame is given by

$$\delta_v = (55.2\,W_2 + 14.75\,W_1)\frac{1000}{I}\,\text{mm},$$

where I is the second moment of area and serviceability failure occurs if

$$\delta_v > 120\,\text{mm}.$$

The maximum elastic stress occurs at B or C and assuming a shape factor of 1.15 the serviceability limit state is exceeded if

$$\text{Max}\begin{bmatrix} 2.155\dfrac{W_2}{z_p} & : C \\ \dfrac{1.62\,W_2 + 4.42\,W_1}{z_p} & : B \end{bmatrix} > f_y .$$

Cumulative frequency polygons for W_1, W_2, z_p, f_y, I were assumed. Those for W_1, W_2 and f_y are shown in Figs. 5, 6.

The flow chart for the computer program which calculates the probabilities of the various limiting states being exceeded is shown in Fig. 7. Random

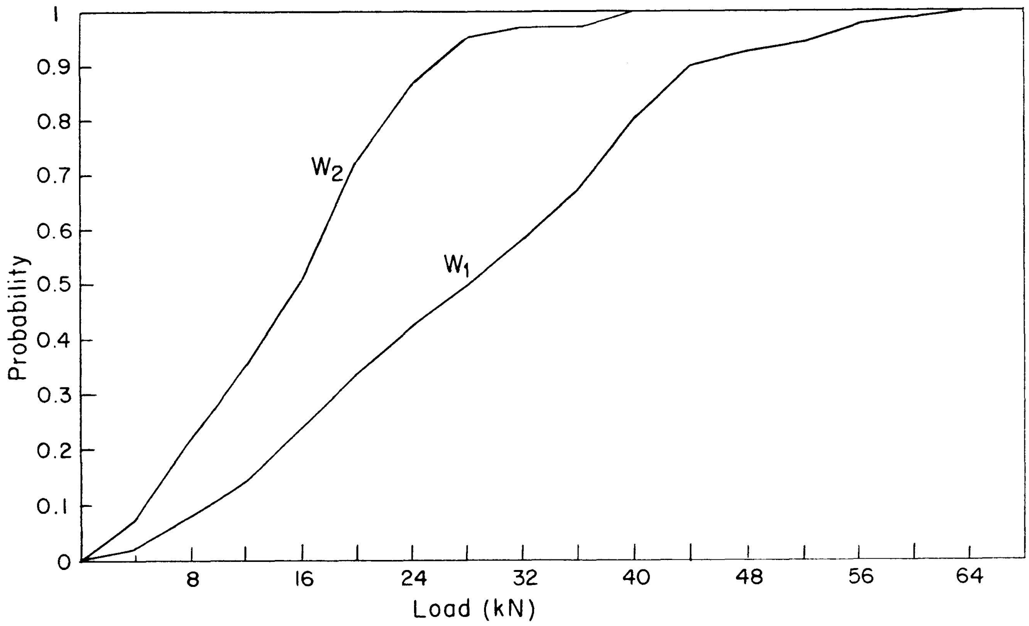


Fig. 5. Assumed Cumulative Frequency Polygon for loads W_1, W_2.

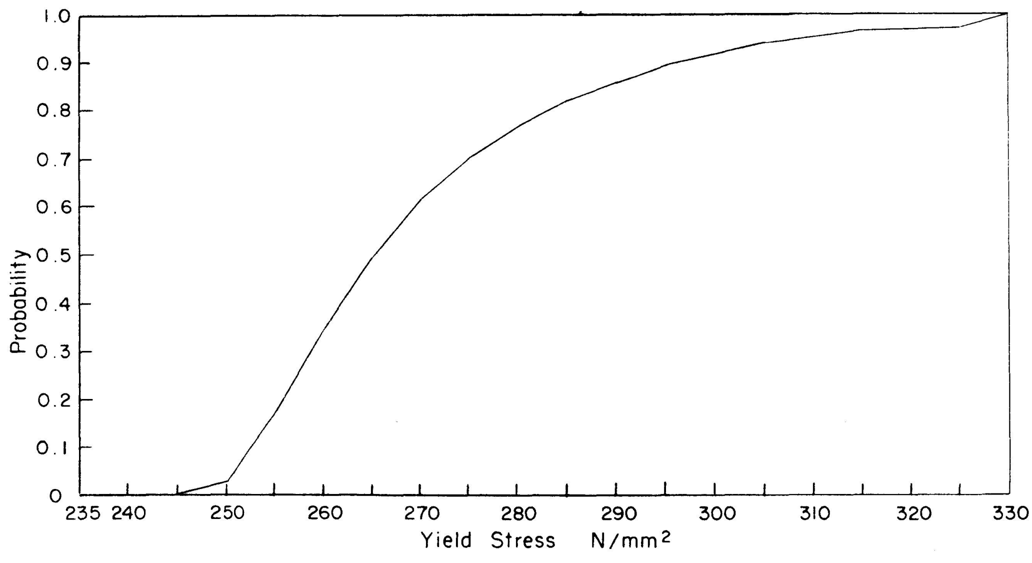


Fig. 6. Assumed Cumulative Frequency Polygon for yield stress.

values of the parameters were generated on the computer large numbers of times and the probabilities estimated. The process of estimation is a Bernoulli process and the probability values are continuous so that a β-distribution is appropriate to describe the distribution of the estimate

$$f(p_f) = \frac{p_f^{k_1}(1-p_f)^{k_2}}{B}, \qquad \text{where } B = \frac{k_1!\,k_2!}{(k_1+k_2+1)!}$$

with $$E(p_f) = \frac{k_1+1}{k_1+k_2+2}$$

variance $$\mathrm{Var}(p_f) = \frac{(k_1+1)(k_2+1)}{(k_1+k_2+2)^2(k_1+k_2+3)},$$

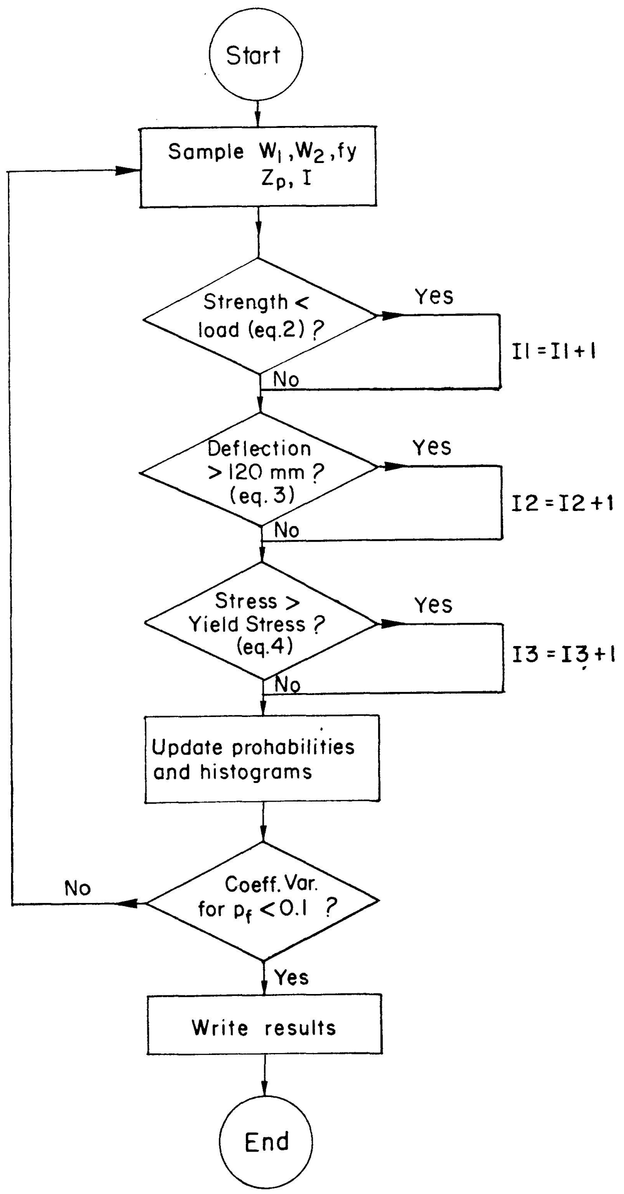


Fig. 7. Flow diagram for simulation of the portal frame example.

where p_f is the probability of failure in the limit state being examined, k_1 is the number of times a limit state is exceeded (Fig. 7: I1, I2, I3), k_2 is the number of times a limit state is not exceeded and $f(p_f)$ is the posterior distribution after (k_1+k_2) trials assuming a uniform prior distribution.

The number of trials that are needed in order to get a satisfactory estimate of the probability of failure of the structure were examined in two ways. Firstly, if after n cycles no counts of failure were obtained then

$$k_1 = 0, \qquad \begin{aligned} f(p_f) &= (1-p_f)^n (n+1), \\ F(p_f) &= 1-(1-p_f)^{n+1}, \\ p_f &= 1-\{[1-F(p_f)]^{1/(n+1)}\}. \end{aligned}$$

This is plotted for $n=10^3$ cycles in Fig. 8. In fact the mantissas of the values change only slightly for $n=10^m$ cycles $(m=3,4,5,\ldots)$ that we may conclude that to estimate a probability in the order of 10^{-m} (safety index m) with 60% confidence we need to obtain no counts in 10^m cycles and in order to obtain it with 99.99% confidence we need no counts in 10^{m+1} cycles.

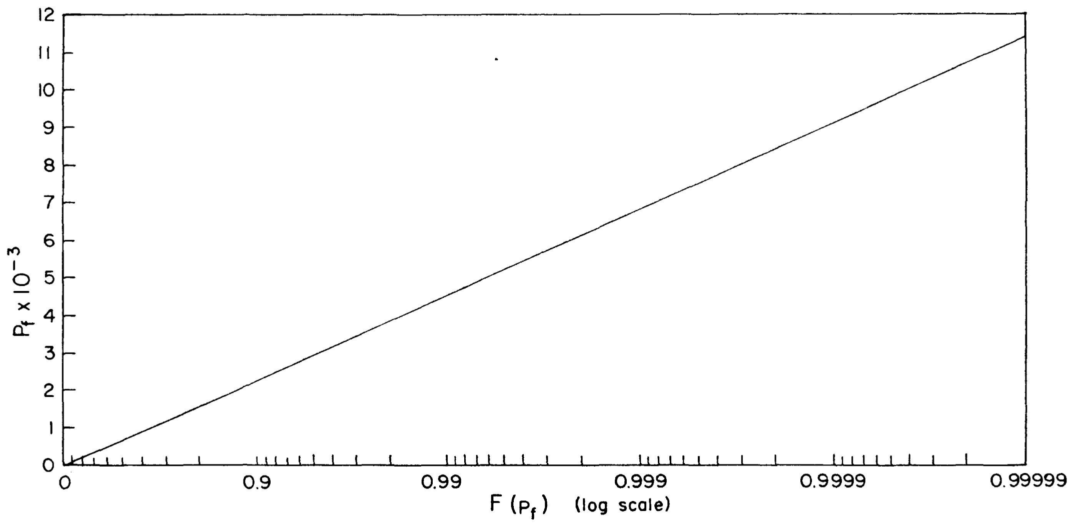


Fig. 8. C.D.F. for p_f when no counts of failure are obtained in 10^3 trials.

If however counts are obtained $(k_2 \neq 0)$ then use can be made of Chebyshev's inequality

$$P[(m-h\sigma) \leqq X \leqq (m+h\sigma)] \geqq 1-\frac{1}{h^2},$$

where m and σ are the mean and standard deviation of the set of variables $x_1, x_2 \ldots$ and h is a constant.

For 99% confidence

$$0.99 = 1-\frac{1}{h^2}, \quad \therefore\ h = 10,$$

$$\therefore\ (m = 10\,\sigma) \leqq X \leqq (m+10\,\sigma)$$

If the number of cycles are such that $\sigma \cong m/10$

or the coefficient of variation $\dfrac{\sigma}{m} \cong 0.1$.

Then $0 \leqq X \leqq 2\,m$.

For the portal frame example the value of the coefficients of variation for the limiting states considered are shown in Fig. 9. After 700,000 cycles the coefficient of variation on the probability of collapse was less than 0.1 and the values of the probabilities of failure are shown in Table II. The resulting distributions, in the form of histograms, of load effect and strength effect load factor, deflections and elastic stresses are shown in Figs. 10–13. Table II also shows the probability of collapse failure estimated by using equation (1) on Fig. 10.

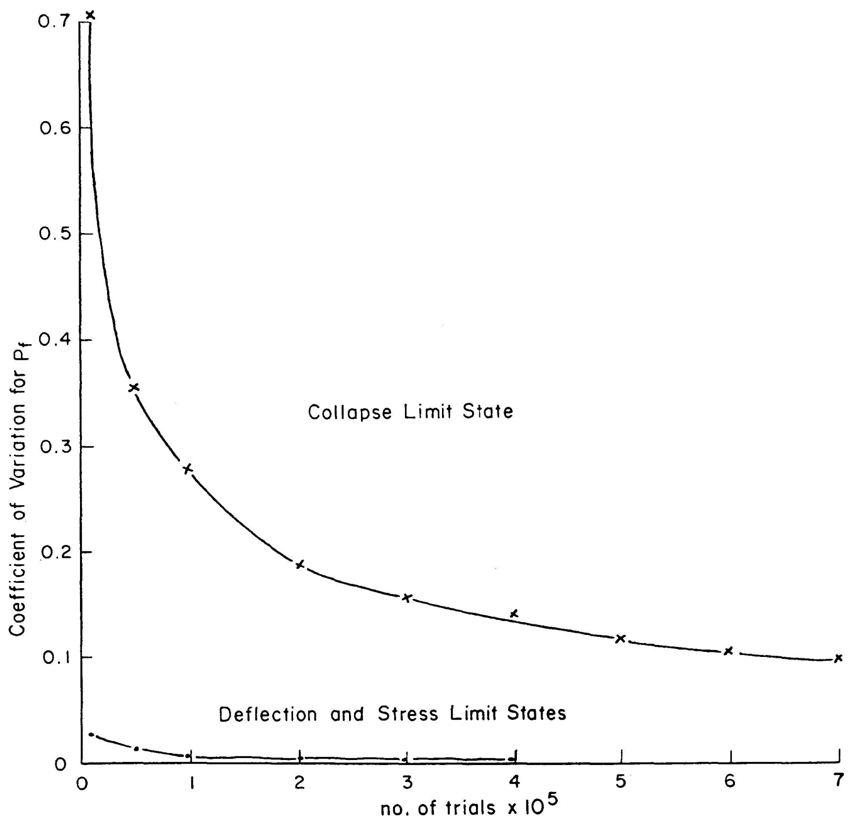


Fig. 9. Graph of the coefficient of variation of p_f against number of trials.

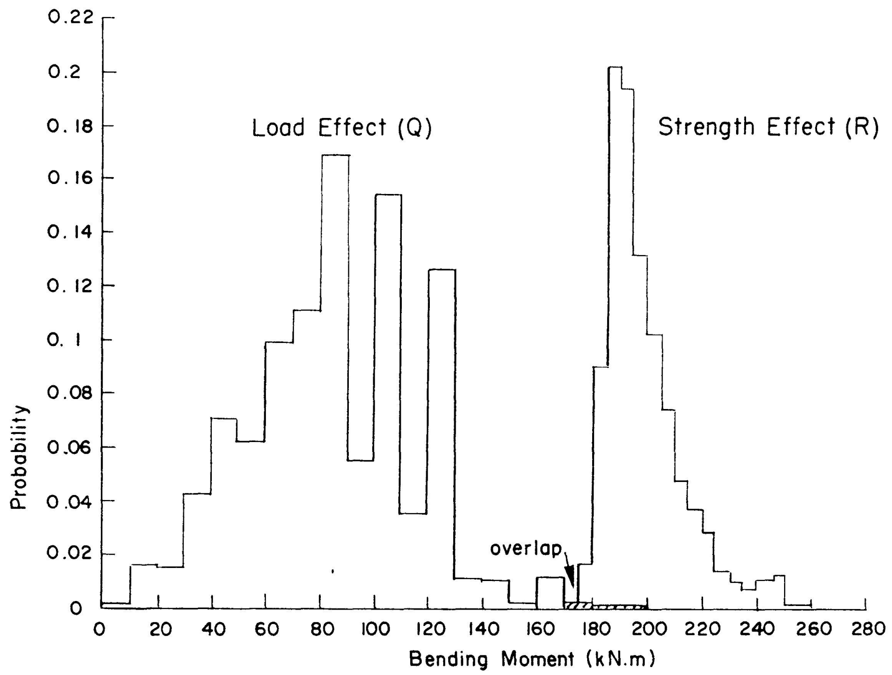


Fig. 10. Histograms of load and strength effects for the portal frame example.

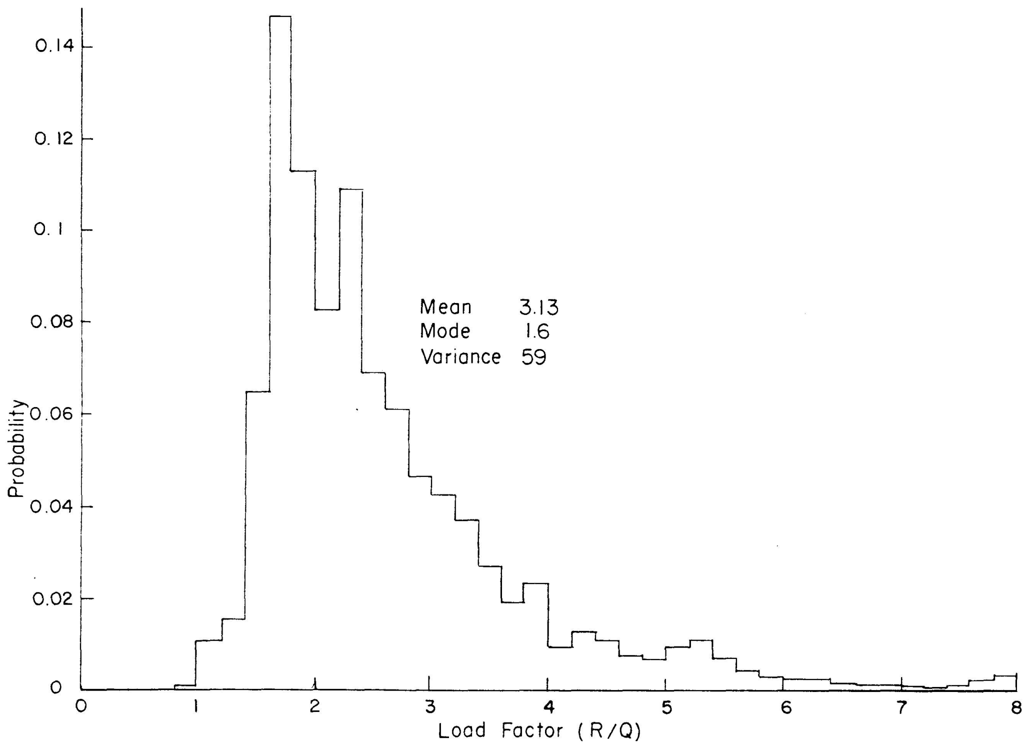


Fig. 11. Histogram of the load factor for the portal frame example.

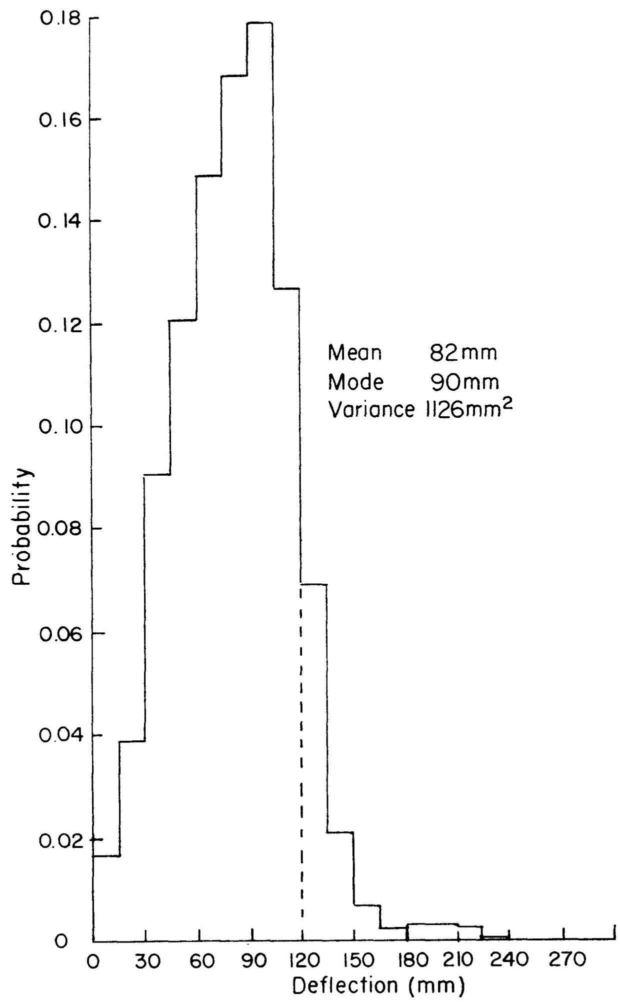


Fig. 12. Histogram of the elastic deflection for the portal frame example.

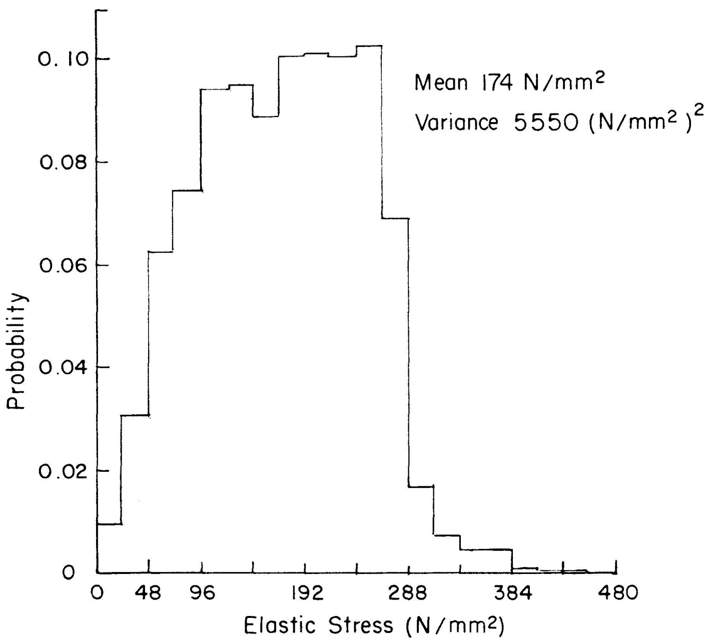


Fig. 13. Histogram of the elastic stress for the portal frame example.

Conclusions

1. Decision Theory provides a logical framework on which to base structural design decisions, such as for example a comparison of alternative structural solutions. It provides a rigour of approach which is lacking in some present methods but requires a probabilistic interpretation of structural phenomena.
2. Progress towards the establishment of probabilistic analysis of structures has been slow for two major reasons. Firstly that codes of practice must remain fairly simple. Secondly, that available data is very limited and far more effort is needed to establish the variations of the parameters considered in design calculations.
3. If information is available concerning the variations of the design parameters it is possible to estimate for a given structure a value of the probability of exceeding any given limiting state by generating large numbers of random values of the parameters and comparing load and strength effects. This is, in fact a simulation of the loading history of the structure.
4. If the product of the probability of failure and the expected cost of failure is small compared to the initial cost of the structure then adopting economic criteria, the probability of failure is effectively zero. However from a social point of view the probability of failure may be such that the frequency of failures may cause public disquiet therefore should not be considered as effectively zero.
5. Values for the probabilities of failure for existing structures may be calculated if suitable data were available. Long term measurements of changes in strain and deflection recorded over the lifetime of various types of structures should be initiated. Many structures under construction could be instrumented at relatively small extra cost. It is believed that the establishment of such data is of high priority to the structural engineering profession.

Notation

a_i	action from action set A.
C	expected value of utility for a structural state of nature.
$E(X)$	expected value of random variable X.
$f(x)$	probability density function of the continuous variable X.
$F(x)$	cumulative distribution function of the random variable $X\,[=P(X \leqq x)]$.
f_y	yield strength of steel.
I_i	initial cost.
I	second moment of area.
M_p	plastic moment.

p probability.
p_f probability of exceeding a given limiting state.
$P(B)$ probability of event B.
Q load effect.
R strength effect.
W load.
z_p plastic modulus.
δ deflection.
θ state of nature from set θ.
$\max\limits_{i}^{N} [x_i]$ maximum of values $x_1, x_2 \ldots x_N$.

Acknowledgements

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References

1. Pugsley, A. G.: The Safety of Structures. London, Edward Arnold, 1966.
2. Su, H. L.: Statistical Approach to Structural Design. Proc. I.C.E. Vol. 13, July 1959, p. 353.
3. Freudenthel, A. M.: Safety and the Probability of Structural Failure. Proc. Am. S.C.E. Structural Division, Vol. 80, Separate No. 468, August 1954, p. 468–1.
4. Freudenthel, A. M.: The Safety of Structures. Trans. Am. S.C.E. Vol. 112, 1947, p. 125.
5. Pugsley, A. G.: The Prediction of Proneness to Structural Accidents. The Structural Engineer, Vol. 51, No. 6, June 1973, p. 195.
6. Asimov, M.: Introduction to Design. Prentice-Hall, 1962.
7. Peir, J. C., Cornell, C. A.: Spatial and Temporal Variability of Live Loads. Proc. Am. S.C.E., Structural Division, Vol. 99, No. ST 5, May 1973, p. 903.
8. Mitchell, G. R., Woodgate, R. W.: Floor Loadings in Office Buildings – The Results of a Survey. Building Research Station, Current paper 3/71, Jan. 1971.
9. Baker, M. J.: Variations in the Strength of Structural Materials and Their Effect on Structural Safety. CIRIA report, Imperial College, July 1970.
10. Dorman, A. P., Flint, A. R., Clark, P. J.: Structural Steelwork for Multi-storey Building – Design for Maximum Worth. Proc. Conference Steel in Architecture, London B.C.S.A., Nov. 1969, p. 83.
11. Templeman, A. B.: Structural Design for Minimum Cost Using the Method of Geometric Programming. Proc. I.C.E. Vol. 46, August 1970, p. 459.
12. Hays, W. L., Winkler, R. L.: Statistics, Probability Inference and Decision. Volume 1, Holt, Rinehart and Winston Inc. 1970.
13. Benjamin, J. R., Cornell, C. A.: Probability, Statistics and Decision for Civil Engineers. McGraw-Hill, 1970.
14. Turkstra, C. J.: Decision Theory approach to Structural Optimisation. Final Report, Int. Assoc. Bridge Structural Engs. Eighth Congress, New York, 1968, p. 193.

15. Baldwin, R.: Some Notes on the Mathematical Analysis of Safety. Fire Research Station, Fire Research Note. No. 909, Jan. 1972.
16. Cornell, C. A.: Bounds on the Reliability of Structural Systems. Proc. Am. S.C.E., Structural Division, Vol. 93, Feb. 1967, p. 171.

Summary

Structural design is a decision making process. Decision theory is introduced and applied to the problem of choosing the best structural solution from a number of competing alternatives.

The probabilities of exceeding various limiting states were estimated for a portal frame structure using a computer simulation of the expected loads during the expected life of the structure.

Résumé

Le projet structural représente un procédé de décision. On y introduit la théorie de décision et on l'applique au problème de choisir la meilleure solution structurale parmis un nombre d'alternatives compétitives.

Les probabilités d'un dépassement de différents états limites ont été estimées pour une structure en cadre en se servant d'une simulation sur ordinateur des charges probables pendant la durée de vie de la structure.

Zusammenfassung

Der bauliche Entwurf stellt einen Entscheidungsprozess dar. Die Entscheidungstheorie wird eingeführt und auf das Problem zur Auswahl der besten baulichen Lösung unter einer Anzahl konkurrenzfähiger Alternativen angewandt.

Die Wahrscheinlichkeiten des Überschreitens verschiedener Grenzzustände wurden auf ein Rahmentragwerk unter Benutzung einer Computer-Simulation über die zu erwartenden Lasten während der voraussichtlichen Lebensdauer des Bauwerkes geschätzt.