Theme VII: Suggestions for practical recommendations

DISCUSSION PRÉPARÉE / VORBEREITETE DISKUSSION / PREPARED DISCUSSION

General Recommendations Derived From Basic Studies on Structural Safety

Règles de dimensionnement basées sur des études fondamentales de la sécurité

Allgemeine Richtlinien aufgrund von Sicherheitsbetrachtungen

JÚLIO FERRY BORGES
Associate Director
Laboratório Nacional de Engenharia Civil
Lisbon, Portugal

MÁRIO CASTANHETA
Research Officer
Laboratório Nacional de Engenharia Civil
Lisbon, Portugal

1 – INTRODUCTION

The problem of structural safety for each possible limit state can be expressed in a simple form by distinguishing two types of distributions: the distribution of loads $F_S(x)$ and the distribution of resistances $F_R(x)$. The former measures the probability of a load smaller than x. The latter measures the probability of a resistance smaller than x. These probabilities must refer to the same variable that measures both loads and resistances, and must take for reference the same interval of time, e.g. the anticipated life of the structure.

Considering loads and resistances to be independent, the probability of a limit state being surpassed, that is the probability of the load exceeding the resistance, is given by the integral

$$P_f = \int_0^{\infty} \frac{d\,F_S}{d\,x}\, F_R \; d\,x \qquad \text{1)}$$

The probability of a limit state being surpassed is called probability of failure, P_f. According to expression 1, the probability of failure depends on the parameters that define the distribution functions F_S and F_R.

Two main types of distribution functions are used for defining loads and resistances: normal and extreme distributions. For extreme distribution functions three main types must be distinguished: type I, type II and type III.

The analytical expressions of these distribution functions are:

1) - Normal $\qquad F(x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{\frac{x - \bar{x}}{\sigma}} e^{-\frac{t^2}{2}} \, dt$ ....2)

2) - Extreme type I (maxima) $\qquad F(x) = \exp(-\exp(-\alpha(x - u)))$ ..3)

3) - Extreme type II (maxima) $\qquad F(x) = \exp(-(k\,x)^{-\beta})$ ....4)

4) - Extreme type III(minima) $\qquad F(x) = 1 - \exp(-(\frac{x - \varepsilon}{k - \varepsilon})^{\beta})$.....5)
(Weibull)

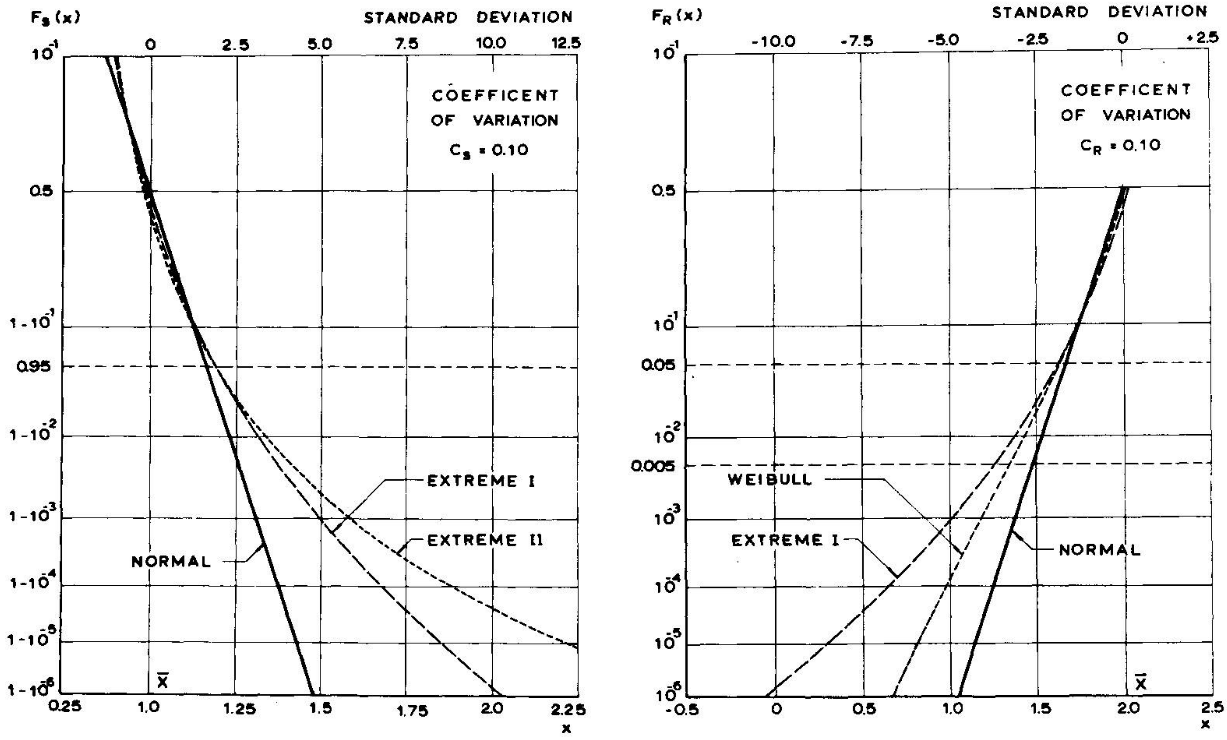


Fig. 1 – Distribution functions for loads and resistances.

Each of these distribution functions is defined by two parameters, except the last one that has three parameters. The distribution functions can also be defined in the following equivalent ways:

i) by the parameters indicated in expressions 2, 3, 4 and 5.

ii) by the mean value $\bar{x}$, the standard deviation σ, and one more parameter for Weibull distribution. When $\bar{x} \neq 0$ the standard deviation can be substituted by the coefficient of variation, $c = \frac{\sigma}{\bar{x}}$.

iii) by the parameters indicated in ii), with the mean value substituted by a fractile of chosen probability $(x_{0.95}, x_{0.05}, x_{0.005})$.

It could be argued that the indicated types of distribution functions do not cover all possible practical cases. However attention must be paid to the fact that the convolution integral of expression 1 has significant values only in the region where the two distribution functions intersect; i.e. the upper tail of F_S (x) and the lower tail of F_R (x). The theoretical distribution functions have to be adjusted to the experimental values in these regions only.

Fig. 1 represents in normal scale the different types of distribution functions, for a coefficient of variation 0.10 and mean values of 1 for loads and of 2 for resistances.

In what follows the 0.95 fractile for loads and the 0.05 fractile for resistances are called characteristic values. The 0.005 fractile of resistance is called design value of the resistance.

The factor γ_m, that transforms the characteristic into the design value of the resistances, is called minoration factor.

The ratio of the design value of the resistance $x_{R\ 0.005}$ to the characteristic value of the load $x_{S\ 0.95}$ is called factor of safety, γ,

$$\gamma = \frac{x_{R\ 0.005}}{x_{S\ 0.95}} \qquad \text{6)}$$

2 – RELATION BETWEEN PROBABILITY OF FAILURE AND FACTOR OF SAFETY

By means of expression 1 the factors of safety can be related with the probabilities of failure. These relations depend on the types and on the coefficients of variation of the distribution functions of the loads and of the resistances. Such relations are presented in (1) for the different combinations of the distribution functions indicated.

Figs. 2 and 3 express the relation between probability of failure and factor of safety when resistance is represented by a normal distribution and the loading by a normal distribution and by an extreme distribution of type I, respectively. The following values of the coefficients of variation were assumed:

a) resistances, c_R = 0.05, 0.10, 0.15 and 0.20.

b) loads, c_S = 0, 0.1, 0.2 and 0.3.

The distributions of loads and of resistances being both normal, Fig. 2 shows that a factor of safety $\gamma = 1.5$ is required to obtain a probability of failure $P_f = 10^{-5}$, if $c_R = 0.15$ and c_S ranges between 0.1 and 0.3. In the following, the above indicated situation is taken for reference.

The distribution of loads being an extreme distribution of type I, (Fig. 3), for $c_R = 0.15$ the factor of safety has to vary from 1.5 to 1.8 as c_S varies from 0.1 to 0.3 for obtaining the indicated probability of failure, $P_f = 10^{-5}$.

For maintaining convenient values of the probability of failure as the types and values of the parameters of the distributions change, the factor of safety must change also. This change is taken into account in some modern codes by means of partial factors of safety which by multiplication give the total factor of safety. As shown in (1), the multiplication rule of the partial factors of safety is not exact. Partial factors of safety are convenient if considered as correcting factors to be applied to the factor of safety that corresponds to a reference situation. Each partial factor must correspond to a well defined set of conditions.

Particular attention should be paid to the different purposes of the minoration factor γ_m and of the safety factor, γ. As indicated the minoration factor transforms the characteristic values of the mechanical properties in design values. On the other hand the factor of safety, γ, instead of applying to loads only, is an overall factor that relates resistances with loads.

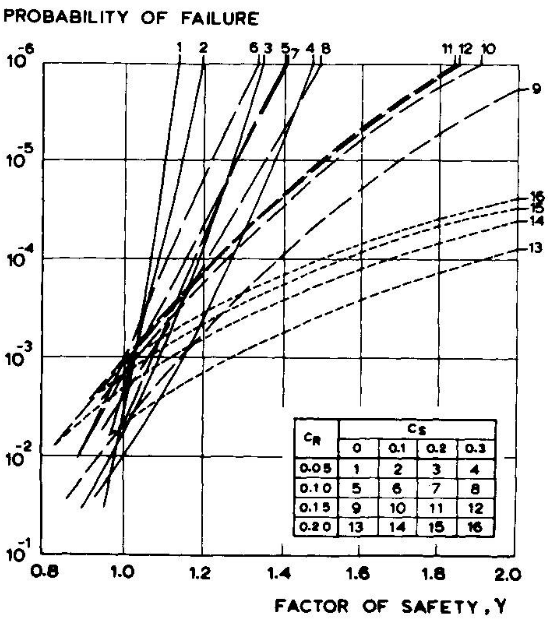


Fig. 2 – Relation between probability of failure and factor of safety for normal distributions of loads and resistances.

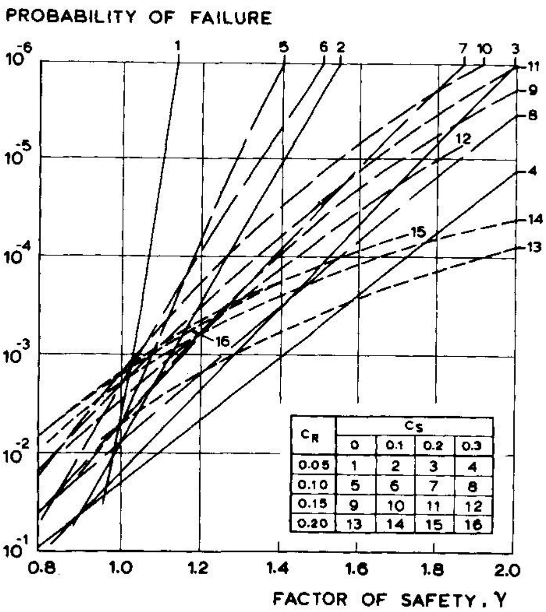


Fig. 3 – Relation between probability of failure and factor of safety for normal distribution of resistances and extreme type I distribution of loads.

3 – COMBINATION OF PERMANENT AND LIVE LOADS

The simple case of a structure acted by a total load $\bar{S}$ that can be decomposed in a permanent load, W, and a live load, L, is considered. Variables W and L are assumed independent and normally distributed. Mean values are denoted by $\bar{W}$ and $\bar{L}$, coefficients of variation by c_W and c_L, and the 0.95 fractiles (characteristic values) by W_k and L_k.

The variables being independent, the sum $S = W + L$ is also normally distributed, and has the mean value $\bar{S} = \bar{W} + \bar{L}$ and a coefficient of variation

$$c_S = \frac{\sqrt{\bar{W}^2 c_W^2 + \bar{L}^2 c_L^2}}{\bar{W} + \bar{L}} \qquad \ldots 7)$$

A simple and interesting problem consists in comparing the characteristic value of S, S_k, with the sum of the characteristic values W_k and L_k.

Putting $\alpha_k = \dfrac{L_k}{W_k}$ and

$\alpha_o = \frac{\bar{L}}{\bar{W}}$,

$$\frac{S_k}{W_k + L_k} = \frac{(1+\alpha_o)(1 + 1.645\ c_S)}{\alpha_o\ (1 + 1.645\ c_L) + (1 + 1.645\ c_W)} \quad \ldots\ldots\ldots \quad 8)$$

By means of the above expression it is possible to estimate the error due to adopting for the characteristic value of the sum, W + L, the sum of the characteristic values of W and of L. Fig. 4a) gives values of $\frac{S_k}{W_k + L_k}$ in function of α_k, c_W and c_L.

For checking the safety of the structure under different combinations of the permanent and the live loads, three solutions are considered:

i) Computing the characteristic value of the sum, S_k, and multiplying this value by the factor of safety $\gamma = 1.5$.

ii) Adding the characteristic values of permanent and live loads and multiplying the sum by the factor of safety, $\gamma = 1.5$.

iii) Multiplying the characteristic permanent load by the factor $\gamma_W = 1.4$, and the characteristic live load by the factor $\gamma_L = 1.6$ and adding the resulting values.

By computing the ratios

$$\frac{1.5\ S_k}{1.5\ (W_k + L_k)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 9)$$

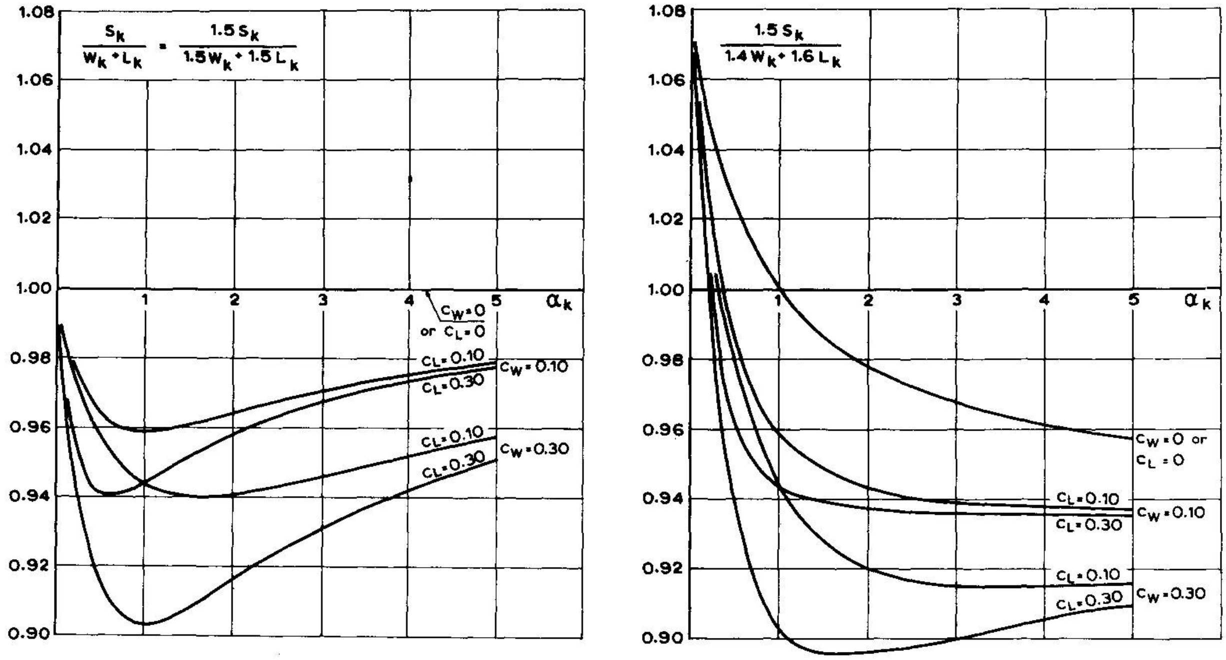
and

$$\frac{1.5\ S_k}{1.4\ W_k + 1.6\ L_k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 10)$$

solutions ii) and iii) can be compared with solution i). Expression 9 is equal to expression 8 and is presented in fig. 4a). Expression 10 is plotted in fig. 4b).

Assuming that solution i) is the correct one, fig. 4a) shows that solution ii) always corresponds to errors on the safe side. For the considered values of the coefficients of variation, the error is always less than 10%, has a maximum for $\alpha_k \cong 1$ and decreases as α_k tends to zero or infinite.

Fig. 4b) shows that for small values of α_k, solution iii) corresponds to errors on the unsafe side. For $\alpha_k = 0$, this solution corresponds to adopting a factor of safety of 1.4 instead of 1.5 . According to fig. 2, this reduction of the factor of safety approximately corresponds to duplicating the probability of failure, for the reference situation. For $\alpha_k \cong 1$ the error of solution iii) is on the safe side and of the same order of magnitude as the error of solution ii). As α_k increases the error tends to 6%.



a) b)

Fig. 4 – Comparison of different solutions for combining permanent and live loads.

The above considerations show the disadvantage of using different factors of safety for permanent and live loads, instead of a single value. The correct solution would be to compute the characteristic value of the sum of permanent and live loads instead of the sum of characteristic values, but in general the accuracy with which characteristic values are defined does not justify this refinement.

4 – EARTHQUAKE LOADS

4.1 – Statistical definition of seismicity

For studying the safety of structures under earthquake loads on statistical bases it is necessary to define the distribution function of these loads referrred to the period of life of the structure. This distribution is obtained by combining the distribution of earthquake intensities in the considered region for the anticipated period of life of the structure with the distribution of the structural response (2).

The intensity of an earthquake at a given point and for the vibration of the soil in a given direction can be defined by a single quantity (3): the mean power spectral density of acceleration for a given range of frequencies, S . It can be shown that Housner's definition of intensity (4) corresponds to a quantity proportional to $\sqrt{S}$. Also the mean maximum value of the soil acceleration, $\bar{a}_{max}$, in function of S is given by

$$\bar{a}_{max} = 7 \sqrt{S} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 11)$$

$\bar{a}_{max}$ being expressed in gal, and S in gal^2/Hz.

The seismicity of a region is thus defined by the distribution function that gives the probability of the values of S (or the equivalent values of $\bar{a}_{max}$) being attained during a given interval of time, T .

The information available on the distribution of earthquake magnitudes all over the world and within limited areas shows that these magnitudes are well represented by extreme distributions of type I. By relating accelerations with magnitudes by the usual expressions it follows that the maximum accelerations must obey an extreme distribution of type II (5). The probability of the maximum acceleration attaining the value $\bar{a}_{max}$ during the interval of time T is thus given by

$$P\,(a_{max} < \bar{a}_{max} \mid T) = F_{II}\,(\bar{a}_{max}) = \exp\,(-\,(k\,\bar{a}_{max})^{-\beta}) \;\ldots\ldots\; 12)$$

Adopting T' instead of T simply changes the value of k to

$$k' = \frac{k}{(T'/T)^{1/\beta}} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\; 13)$$

A value $\beta = 3$ is adopted in accordance with the existing data, so that the seismicity of a region is simply defined by k .

It must be noted that a distribution function of type II imposes no upper limit to the accelerations, which disagrees with physical evidence. However, to consider this limit does not much affect the final results, as will be seen below.

4.2 – Statistical definition of earthquake loads

The earthquake being assumed a stochastic process as indicated, the mean maximum value of the displacements, $\bar{\delta}_{max}$ (cm), in a one-degree-of-freedom oscillator (linear or non-linear within a convenient range of the ductility factor) is given by

$$\bar{\delta}_{max} = 0.01\ \eta^{-1/2}\ f_o^{\ -3/2}\ \ \bar{a}_{max} = \lambda\ \bar{a}_{max} \;\ldots\ldots\ldots\; 14)$$

where η – fraction of critical damping

f_o – natural frequency (Hz)

$\bar{a}_{max}$ – mean maximum acceleration (gal) related to the power spectral density of acceleration by expression 11.

For a given $\bar{a}_{max}$, the maximum displacement, δ_{max}, obeys an extreme distribution function of type I, $F_I\,(\delta_{max})$, with the indicated mean value, $\bar{\delta}_{max}$, coefficients of variation between 0.1 and 0.2 for linear behaviour and reaching about 0.4 for non-linear behaviour within the usual allowable values of the ductility factor (6).

As indicated in (2), the probability of the maximum displacements of a structure attaining a value δ_{max} during a time interval T is obtained by combining the probabilities of different values of $\bar{a}_{max}$ occurring in a given interval, with the probability of δ_{max} being attained for the different values of $\bar{a}_{max}$.

Thus, $\bar{a}_{max}$ being considered not as deterministic but random with the distribution function given by expression 12, the distribution function of δ_{max} referred to the time interval T is given by

$$P(\delta < \delta_{max} \mid T) = \int_0^{\infty} \frac{d F_{II}(\bar{a}_{max})}{d\,\bar{a}_{max}} F_I(\delta_{max} \mid \bar{a}_{max})\, d\,\bar{a}_{max} \quad ..15)$$

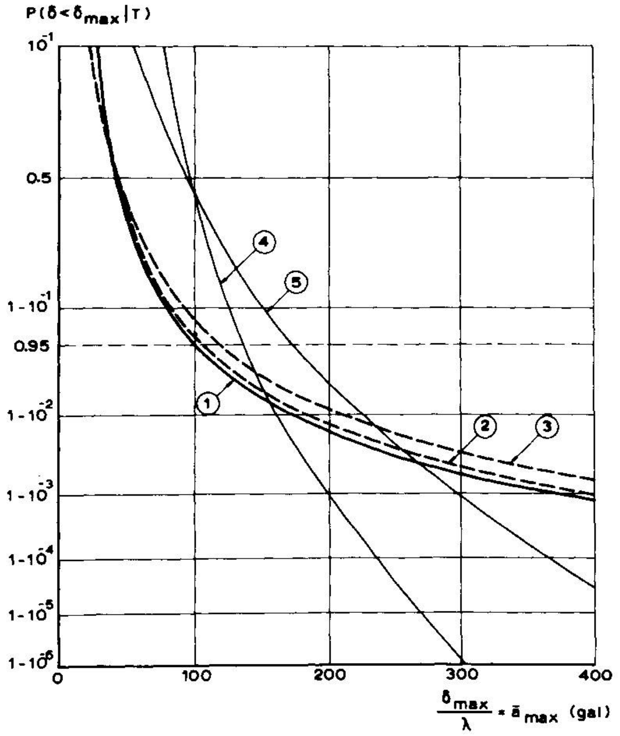


Fig. 5 – Distribution functions of maximum displacements due to earthquakes.

Fig. 5 indicates the distribution functions resulting from expresion 15. Curve 1 corresponds to the distribution functions that define the seismicity of the region and the response of the structure if this response were deterministic. Curves 2 and 3 give the distribution function of the response (expression 15) for coefficients of variation of the extreme distribution of type I equal to 0.2 and 0.4 respectively. Analysis of Fig. 5 shows that the distribution of maximum displacements is not much affected by the randomness of the structural response, even if this has high coefficients of variation. This conclusion is in accordance with results previously obtained (2).

Curves 4 and 5 of Fig. 5 indicate the distribution functions of the response, for earthquakes with a power spectral density of acceleration corresponding to $\bar{a}_{max}$ = 100 gal, and for coefficients of variation equal to 0.2 and 0.4, respectively. It is interesting to note that the maximum acceleration being deterministic even so important randomness derives from the structural response alone. Truncating the statistical distribution of seismicity does not correspond to truncating the final distribution of maximum displacement. This justifies the above assertion about the influence of not considering an upper limit of the accelerations due to earthquakes.

4.3 – Probability of failure under earthquake loads

The fact that the distribution function that defines seismicity is of type II with an exponent $\beta = 3$ has important consequences for structural safety. As shown in Fig. 5, this distribution function has a very long tail.

In accordance with the results presented in (1), the acceptance for earthquake loads of the criteria used for other types of loads, i.e., defining the load by the 0.95 fractile (characteristic value) and adopting a factor of safety $\gamma = 1.5$, corresponds to a probability of failure of about $5\ 10^{-3}$, for the usual values of the coefficients of variation of resistances. To adopt a factor of safety $\gamma = 1.0$ corresponds to a probability of failure of about 10^{-2}.

Note that for the time interval T = 50 years, the characteristic acceleration (or power spectral density of acceleration) to be assumed is the annual maximum acceleration that has a return period of 1000 years. This characteristic acceleration is more than twice the one having a return period of 100 years.

For the same characteristic value of the load, a change in the type of load distribution has a large influence on the probability of failure. In fact for $\gamma = 1.5$, assuming the final distribution of earthquake loads to be of type I instead of type II reduces the probability of failure from the indicated value of $5\ 10^{-3}$ to about 10^{-4}. On the other hand it is very difficult to derive from experimental data the type of distribution to be adopted. In fact the differences between the several types of distributions are relevant in the region of small probabilities only, and, by definition, experimental data in this region are always scarce. Thus it is of paramount importance to increase the accuracy of the definition of the type and of the values of the parameters of the statistical distribution of seismicity, based on phenomenological and statistical data. However the presented results allow an understanding of the bounds of the problem.

As shown in a previous paper, (7), the probability of failure in cases as the present can be significantly reduced only by increasing the mean value of resistance and is not affected by changes in its coefficient of variation. For earthquake loads, the resistance of a structure is proportional to its ductility factor. The fact that the values of the ductility factors usually adopted in design are in general conservative implies that the real values of the probability of failure are smaller than those indicated above. This aspect of the problem is basic and also needs further research.

Additionally, it is of interest to determine the probability of failure that corresponds to the occurrence of an earthquake with a maximum acceleration equal to the assumed characteristic value. For a coefficient of variation of the response, $c_S = 0.2$ (Curve 4 of Fig. 5) and for $\gamma = 1.5$, a probability of failure about 10^{-3} is obtained.

5 — WIND LOADS

5.1 — Statistical definition of wind velocities

The statistical distribution of the wind loads, expressed in pressure, has to be derived by combining the statistical distribution of wind velocities with a distribution allowing velocities to be related with loads.

Due to the turbulence of wind, it is convenient to define wind velocity by distinguishing the mean wind velocity in a given interval of time (e.g. ten minutes or one hour) from the superimposed fluctuations.

For non-tropical winds, the data concerning the maxima of the mean wind velocities fit in well with an extreme distribution of type I (8). For annual maxima its coefficient of variation is about 0.15. Changes in altitude and in roughness of soil do not influence this coefficient.

The maxima of the mean wind velocities for periods of 50 years have mean values about 1.5 times the corresponding velocities for periods of one year and their coefficients of variation are about 0.10.

Velocity fluctuations within an interval of time are assumed random and defined by correlations (in space or in time) and/or spectral densities of wind velocity. These functions are assumed to be deterministic.

The knowledge of correlations and spectral densities allows the stochastic process to be defined and the probability of velocities being exceeded within given intervals of time to be computed (9).

5.2 – Statistical definition of wind loads

For the usual intensities of turbulence, wind loads (expressed in pressures) can be determined by adding the effects of the mean wind velocity with the effects of turbulence.

The maximum pressures due to the mean wind velocity are described by a type I extreme distribution with coefficients of variation twice those of wind velocities (about 0.3 for the maximum annual pressures and about 0.2 for the maximum pressures in 50 years).

The pressures due to the stochastic process that corresponds to turbulence are difficult to define because they are influenced by many parameters. Vortex shedding and aero-elastic effects are disregarded in the following and upstream turbulence is considered the only forcing mechanism.

As steady conditions are assumed (implying a mean velocity, $\overline{V}$), the response of the structure due to turbulence takes place around a mean value, $\overline{\delta}$ directly related to the mean velocity. The variability of this response is defined by a coefficient of variation c_δ that is a function of the turbulence spectrum, the aerodynamic admittance, the joint acceptance and the mechanical admittance of the structure (10).

The maximum values of the response in a given interval of time can be defined by a type I extreme distribution with a mean value

$$\overline{\delta}_{max} = (1 + k\, c_\delta)\,\overline{\delta} = \alpha\ \overline{\delta}\ , \qquad \dots\dots\dots\dots\dots\dots\ 16)$$

where k is a coefficient that principally depends on the natural frequency of the structure and on the time interval considered. For usual conditions (e.g. usual types of buildings) α takes values of about 2 or 3. The coefficient of variation of the distribution of δ_{max} amounts to about 0.05.

As indicated in 4.2, the statistical distribution of maximum wind load for the expected life of the structure (e.g. for T = 50 years) must be obtained by performing the convolution of the distribution of the maxima of the mean wind pressures (e.g. for a time interval of 10 minutes) with the distribution of the maximum response. In the present case the coefficient of variation of the maximum response is considerably smaller than the coefficient of variation of maximum wind pressure. Thus, this convolution has no pratical effect and the final distribution of maximum wind loads is of the same type and has the same coefficient of variation as the distribution of maximum pressures.

However in the above considerations it was assumed that the aerodynamic behaviour of the structure can be accurately defined in a deterministic way. In fact present knowledge is scarce and the relationship between upstream wind pressures and wind loads on the structures is based on simplifying assumptions that may lead to important errors. This last source of variability is difficult to quantify. It may well supersede the randomness corresponding to the variability of maximum wind pressures. Thus further research on the relationship between upstream wind velocities and wind loads on structures is considered of fundamental importance.

5.3 — Probability of failure under wind loads

To adopt for wind loads the same design criteria used for other types of loads, i.e. to define the load by the 0.95 fractile (characteristic value), and to adopt a factor of safety $\gamma = 1.5$, corresponds to a probability of failure of about $3\ 10^{-5}$ for the usual values of the coefficients of variation of the resistance ($c_R = 0.15$) and assuming that the wind load is expressed by a type I extreme distribution with a coefficient of variation, $c_S = 0.2$ (Fig. 4). To adopt a factor of safety $\gamma = 1.0$ corresponds to a probability of failure $2\ 10^{-3}$.

As for earthquake loads, assuming T = 50 years, the characteristic pressure to be adopted is the annual maximum pressure that has a return period of 1000 years.

Attention must be paid to the fact that in several cases the probabilities of failure corresponding to the real behaviour of structures designed according to the above criteria will exceed the indicated values due to the inaccurate knowledge on the aerodynamic behaviour.

6 — COMBINATION OF WIND AND EARTHQUAKE LOADS. THEIR ASSOCIATION WITH PERMANENT LOADS

The maximum values of both wind and earthquake loads occur during some seconds only of the expected life of the structure. Both phenomena being independent, the probability of their simultaneous occurrence is very low. In fact assuming that the characteristic earthquake load occurs during one minute of the life of the structure, the probability of the simultaneous action of a wind load (with a duration of 1 minute) exceeding the one that has the return period of two years is $\frac{1}{2 \times 365 \times 24 \times 60} \simeq 10^{-6}$. The wind pressure that corresponds to a return period of two years is only $\frac{1}{2.5}$ of the characteristic one. As the probability of exceeding the characteristic earthquake load is 0.05, the probability of the association of this load with a wind of even reduced intensity is negligible.

A further important point concerns the association of permanent loads with earthquake and wind loads and, particularly, the values of the factors of safety, γ_W, to be applied to the permanent loads. A complete discussion of this problem cannot be presented here. However attention is called to the fact that a value $\gamma_W = 1$ must be adopted. This can be demonstrated by considering the bivariate distribution due to the association of the two types of loads and its intersection with the statistical condition of failure expressed in terms of load effects.

7 — CONCLUSIONS

Basic studies on structural safety yield results that can be directly used to improve design rules. The problems dealt with in the present paper are instances of the above assertion. It must be emphasized that the use of basic results does not imply a complete statistical information. They are particularly important as a guide for a general policy of structural safety.

The main pratical conclusions of the present study are the following:
1 – A single factor of safety must be used for both permanent and live loads. A sufficiently accurate characteristic value for the sum of permanent and live loads is obtained by adding their characteristic values.
2 – When combining wind and earthquake loads with permanent loads, the latter shall be affected by a factor of safety equal to one.
3 – Characteristic values corresponding to annual maxima having a return period of about 1000 years must be adopted for defining wind and earthquake loads. These values must be estimated by fitting to the experimental data an extreme distribution of suitable type.
4 – The characteristic values of wind and earthquake loads must be multiplied by an adequate factor of safety in order to obtain a sufficiently small probability of failure, as indicated in 4.3 and 5.3 .

REFERENCES

1 – Ferry Borges, J. and Castanheta, M. – Structural Safety, Course 101, Laboratório Nacional de Engenharia Civil, Lisbon, Nov. 1968.

2 – Ferry Borges, J. – Statistical Estimate of Seismic Loading, Preliminary Publication, V Congress of the International Association for Bridge and Structural Engineering, Lisbon, 1956.

3 – Ferry Borges, J. and Ravara, A. – Seismic Design of Reinforced Concrete Buildings, International Center of Earthquake Engineering "Arturo Danusso", Milano, Bergamo, 1969.

4 – Housner, G.W. – Spectrum Intensities of Strong-Motion Earthquakes, Proceedings of the Symposium on Earthquake and Blast Effects on Structures, Earthquake Engineering Research Institute, Los Angeles, June 1952.

5 – Cornell, C.A. and Vanmarcke, E.H. – The Major Influences on Seismic Risk, Fourth World Conference on Earthquake Engineering, Santiago, Chile, Jan. 1969.

6 – Penzien, J. and Shih-Chi Liu – Nondeterministic Analysis of Nonlinear Structures Subjected to Earthquake Excitations, Fourth World Conference on Earthquake Engineering, Santiago, Chile, Jan. 1969.

7 – Ferry Borges, J. – Structural Behaviour and Safety Criteria, Seventh Congress of the International Association for Bridge and Structural Engineering, Rio de Janeiro, Aug. 1964.

8 – Davenport, A.G. – The Application of Statistical Concepts to the Wind Loading of Structures. Proc. of the Inst. of Civil Engineers, Vol. 19, Aug. 1961.

9 – Ferry Borges, J. – Dynamic Loads, General Report Theme VI, Preliminary publication, VIII Congress, International Association for Bridge and Structural Engineering, New York, Sept. 1968.

10 – Davenport, A.G. – The Buffeting of Large Superficial Structures by Atmospheric Turbulence, Annals of the New York Academy of Sciences, vol. 116, Article 2, June 1964.

SUMMARY

Probability of failure is related with the factor of safety for combinations of various types of statistical distributions of loads and resistances.

Structural safety is discussed in conncetion with the following problems:

i) combination of permanent and live loads;

ii) earthquake and wind loads;

iii) association of earthquake, wind, and permanent loads.

RESUME

La probabilité de rupture est mise en rapport avec le coefficient de sécurité pour des combinaisons de différents types de distributions statistiques des charges et des résistances.

On discute le problème de la sécurité des constructions en rapport avec les problèmes suivants:

i) combinaison des charges permanentes et des surcharges;

ii) actions dues aux tremblements de terre et au vent;

iii) combinaisons des charges permanentes avec les actions dues aux tremblements de terre et au vent.

ZUSAMMENFASSUNG

Die Bruchwahrscheinlichkeit wird auf die Sicherheitszahl für verschiedenartige statistische Festigkeits- und Beanspruchungsverteilungen bezogen.

Die Bausicherheit wird hinsichtlich folgender Probleme diskutiert:

i) Zusammenstellung ständiger Belastungen und Auflasten;

ii) Erdbeben- und Windbeanspruchungen;

iii) Zusammenstellung ständiger Belastungen, Erdbeben- und Windbeanspruchungen.

Leere Seite
Blank page
Page vide

DISCUSSION LIBRE / FREIE DISKUSSION / FREE DISCUSSION

Some Practical Rules of Up-to-date Dimensioning

E. MISTÉTH
Budapest

The fundamental principles of dimensioning can and should be deduced on the basis of probability theory. Dimensions should be selected to the effect that internal breaking forces during the planned lifetime, T exceed internal forces caused by loading by a probability given in anticipation,

$$P\left\{\underset{0<t\leqq T}{[R(t)-S(t)]}\geqq 0\right\}\geqq 1-\frac{1}{k} \qquad \ldots 1./$$

For a first step the planned lifetime of engineering structures should be introduced.

1./ Lifetime of structures and their influence on quantities in strength theory

Engineering structures should be classified with a view to their planned lifetime.

1.1 Lifetime of structures

$T = 50$ years for permanent, $T = 5$ years for temporary structures are suggested in this paper. Internal forces /stresses/ occurring within the first two years of proper use in permanent structures should be compared with internal forces prescribed for temporary structures.

1.2 Influence of lifetime on breaking stress

The strength characteristics of temporary structures /breaking stress, cross section quantity/ are, fundamentally, even in $T = 5$ years equal to the initial values as existent during the period of construction /breaking stress is, for concrete, even higher by 2o to 25 per cent, a fact which should be considered/. With permanent structures breaking stress will loose lo to 2o per cent of its initial value in $T = 50$ years due to the ageing of artificial building materials /with concrete the initial value of breaking stress should essentially be considered/. As to the rate of diminishing of strength accurate information can be provided through material testing. For steel valuable data are produced on grounds of testing 80 years old Hungarian railway bridges by T. Pap [1]. As to bauxite concrete experiments conducted at the Chair for r.-c. constructions of the Technical University of Budapest yield proper informations [2].

1.3 Influence of lifetime on the amount of useful load

The basic value of live load which is defined, for one and the same type of structure, by the average of maximum values existent during lifetime, is higher for permanent than for temporary structures. If load values for temporary structures are being calculated from the average of five years' maxima, the average of 50 years'maxima equals, properly speaking, to the value occurring with 10 per cent probability, of the distribution function osculatory to the 5 years maxima. For example, in case of normal distribution

$$\bar{p}\,(T=50)=\bar{p}\,(T=5)\left[1+1{,}282\ v_p\,(T=5)\right]$$

The relation 2./ has to be solved for $\bar{p}(T=5)=p_i$; the numerical value of p_i is, if the relative deviation of the distribution varies between v_p=0,08 and 0,20, p_i is equal to from 0,90 to 0,80 p As a matter of course, if v_p=0 /for store-buildings and containers/,

2./ The risk taken

The optimum risk taken against the ruin of structures is with a good approximation, if cost can be calculated by means of the formula $C=C_0\,(1+b_1\log k)$ [3]

$$k_{max} \cong \frac{2{,}6}{b_1}\left[\frac{Q}{C}+2\right] \qquad \ldots 3./$$

In expression 3./ Q designates the damages including profit missed, caused by the ruin, C designates the average rebuilding cost /with a risk ~ 3 per cent taken/, b_1 is the direction tangent of the cost function, increasing with increasing relative deviation / b_1 = 0,04 - 0,1, a good mean value being 0,052/.

As to the ratio of damages caused and cost of rebuilding there being available no clear values recourse should be made to hypotheses. The damages caused vary with the differing types of structures and take on a different shape with the main girder system or with its secondary girder system being concerned. Accordingly the risk taken will also assume different values. These values are registered in the Table below:

	Permanent		Temporary	
	structures			
	main	secondary	main	secondary
	girders		girders	
planned lifetime	$T=50$		$T=5$	
live load	p		$0{,}9\ p$	
permissible stresses	σ_p	$1{,}1\ \sigma_p$	$1{,}1\ \sigma_p$	$1{,}2\ \sigma_p$
$\frac{Q}{C}$	40–200	4–20	4–20	–
k	2.10^3-10^4	2.10^2-10^3	2.10^2-10^3	10^2
taken risk $\frac{1}{k}$	$5.10^{-4}-10^{-4}$	$5.10^{-3}-10^{-3}$	$5.10^{-3}-10^{-3}$	10^{-2}

References
[1] Pap, Tibor: Evaluation of Material Testing of Old Railway Bridges Made of Welded Steel. Mélyépitéstudományi Szemle, 1959. 1 issue.
[2] Bölcskey, Elemér - Szalay, Kálmán: Surplus Load Capacity of Bauxite-Concrete Constructions. Magyar Épitőipar 1969.
[3] E. Mistéth: Some Safety Problems. 8th Congress of IABSE 1968. New-York. Final Report

VII

Free Discussion / Discussion libre / Freie Diskussion

J.L. DARLISON
London

I would draw your attention to the following:-

> Army barrack buildings at Aldershot, Steel frame building in construction at Edinburgh, Staircase in multi-storey block of flats at Isleworth, Restaurant floor in Spain, Ferrybridge cooling towers, Ronan Point and many others.

Some of these disasters have been horrifying and I hope all have been disturbing to those assembled here. I am surprised that a theme was not introduced at this conference examining such failures. I ask you to consider carefully how many of these disasters would have been prevented had this symposium taken place before their occurrence. I suggest to you regretfully that the answer is very few.

The task of the practising engineer is to design structures with economy and an acceptable degree of safety. We do not always succeed - why? Perhaps we have taken insufficient account of variability of materials, workmanship, and loads (gravity, wind temperature etc.), or the inadequacy of design methods. These factors can to a greater or lesser dégree be dealt with by probabalistic methods and it is encouraging to see so much research going on in this field.

In practice however, failures are more often due to mistakes, negligence, lack of knowledge, poor communications or inadequate control and supervision of the work. We must therefore take a broader view of the question of safety than that provided by probability theories alone.

If mistakes are to be reduced our methods of design must be simple, clear and easily checked with the principles clearly stated and understood. This is true whether a computer is used or not because a computer can make mistakes and wrong information can be fed in. The trend today is towards more elaborate design procedures consuming more of the engineers time and perhaps diverting attention from the more general aspects of safety. It is vital that if the ideas put forward in this conference are to be of real value in the design office then the principles must be clearly stated in broad terms and the detailed application must be reasonably simple and capable of easy checking otherwise the effect on safety may be adverse rather than beneficial.

The question of communication is becoming increasingly important with the increase in the size and complexity of projects and the numbers of different people involved. Many failures can be traced to poor communications between Architect and Client, Engineer and Client, Designer and Fabricator, Designer and Erector, and so on and it is essential to pay proper attention to this matter.

Negligence is not easy to deal with but penalties can be imposed and control procedures adopted which will help. Lack of knowledge can only be remedied by continuing research and feed back of information but despite our best endeavours and intentions there will continue to be instances of the unforseen happening because of an inevitable degree of ignorance which will always be present.

It will be seen therefore that however much care we take it is not possible to eliminate the cause of failure entirely but we can frequently localise the affect by adopting 'fail safe' or'alternative path designs' and this aspect should be considered at an early stage in the design.

At this conference great emphasis has been laid on the use of statistics and probability theories; while recognizing the value of these in helping to make our structures safer with economy I recommend to you that at least as much attention be given at a future conference on safety to the other important questions referred to above.

Discussion libre / Freie Diskussion / Free Discussion

C. CHANON
London

Dans la contribution de Mr. Rodin et de moi - même sur le problème de sécurité dans les structures à grands panneaux prefabriqués sous l'effet de charges exceptionnelles, telles que les explosions dûes au gaz par exemple, nous avons essayé de présenter une philosophie de conception tendant à traiter ce problème. La philosophie est basée d'un côté sur l'estimation du niveau du risque et d'un autre sur l'effet de ce risque sur le comportement de la structure. Nous avons aussi présenté des exemples pratiques tendant à illustrer comment l'effondrement progressif peut être émpêché. En particulier nous avons illustré dans notre communication l'exemple d'une structure de 24 niveaux où ce problème est traité à peu de frais, d'une manière, à notre avis, plus que satisfaisante.

Depuis deux jours, nous avons discuté dans cette assemblé de beaucoup de problèmes, certains pratiques, certains théorques, tous intéressants bien sûr. Mais nous ne pouvons nous empêcher de constater que le problème de securité des structures à grands panneaux sous l'effet de charges exceptionnelles a été un peu mis de côté malgré que nous savons tous que ce problème est d'un intéret immédiat et qu'il consitutue un sujet de préocuppation à beaucoup d'ingénieurs et aux autorités aussi.

Ce problème ne doit plus être considéré comme étant d'un intêret mineur. Nous construisons de nos jours très couramment des bâtiments préfabriqués de 20 à 25 étages. Beaucoup de vies humaines dépendent de la résistance de ces bâtiments et par conséquent de la manière dont les ingénieurs approchent et résolvent les problèmes posées par elles.

D'un autre côté les structures à grands panneaux peuvent présenter des résistances intrinséques très importantes à condition de savoir mobilisir ces résistances. Et c'est à nous de chercher à le faire et de le faire. Malheureusement ceci n'a pas toujours été le cas.

Notre souhait est que cette assemblée malgré le manque de communications à ce sujet ne se sépare pas aujourd'hui sans avoir reconnu que nous avons un problème immédiat de sécurité à résoudre, que ce problème est d'un intérêt très pratique on peut même dire vital, et surtout de reconnaitre aussi qu'il existe des solutions possibles et pas très onéreuses, qu'il faut essayer d'adopter, et auquelles il faut à notre avis très sérieusement réfléchir.

VII

Load Factors in a Proposed Norwegian Standard Specification

IVAR HOLAND
Professor, Dr. techn.
The Technical University of Norway
Trondheim, Norway

So far, Norwegian standard specifications for structural design have been based on the concept of allowable stresses. An exception is the code for prestressed concrete, which includes an ultimate limit state analysis.

Most of our standard specifications for design of structures in various materials are at present under revisjon. At the same time a new code for calculation of loading [1] is under preparation. Thus the time was found suited for introduction of a unified limit state approach, and load factors have been included in a tentative version of the loading code. The load factors given are intended to allow for abnormal and unforeseen loads and reduced probability of combinations of loads. Thus, the load factors include the product of γ_{s1} and γ_{s3} described in [2], p. 17.

Two sets of load factors are given as shown in Tables 1 and 2, both of which include three different combinations of loading.

The abbreviations used in the tables are:

- D dead load (weight)
- L live load
- W water (liquid) pressure
- S earth pressure
- O ordinary loading (occurring frequently or for longer periods)
- E exceptional loading (occurring occasionally with larger intervals, or seldom occurring with the characteristic value)

Table 1 gives values for an ultimate limit state, whereas Table 2 gives values for a serviceability limit state. The values in Table 2 are also intended for use in combination with allowable stresses in the transition period until the various design specifications have been revised.

A load factor of 1.0 for earth pressure has been used for the ultimate limit state. The cause is that there is no linear relationship between the magnitude of earth pressure and the magnitude of for instance angle of friction. Thus, the whole factor of safety must be taken in the strength reduction coefficient γ_m (compare [2]) for this case. In spite of the lack of linearity, a factor of 0.8 has been introduced for earth pressure in Table 2.

If two or more exceptional loads occur simultaneously, the largest one is to be multiplied by the load factors given in the tables, whereas the remaining ones are reduced by 30 %.

REFERENCES

1. The Norwegian Council for Building Standardization. Calculation of Loading NBR F 8/69, Oslo 1969.

2. Rowe, R.E.: Safety Concepts, with Particular Emphasis on Reinforced and Prestressed Concrete. Symposium on Concepts of Safety of Structures and Methods of Design. London 1969.

TABLE 1

LOAD FACTORS FOR THE ULTIMATE LIMIT STATE

LOADING	LOAD FACTOR FOR				
	D	L	W	S	E
0	1.3	1.7	1.1	1.0	-
D+E	1.3	-	-	-	1.5
0+E	1.04	1.36	0.88	0.8	1.2

TABLE 2

LOAD FACTORS FOR THE SERVICEABILITY LIMIT STATE

LOADING	LOAD FACTOR FOR				
	D	L	W	S	E
0	1.0	1.0	1.0	0.8	-
D+E	1.0	-	-	-	1.0
0+E	0.8	0.8	0.8	0.64	0.8

Free Discussion / Discussion libre / Freie Diskussion

A.L.L. BAKER
Prof.
London

In the field of reinforced concrete, statistics of unit strength are available from laboratory tests and can be used to calculate the probability of failure of a structure made of identical material. The possible differences between site concrete and laboratory test specimens, however, are so unpredictable that the probability of failure of a structure may lie between, say, 10^{-6} and 10^{-3}, according to the reliability of the construction supervisor, and many other factors appertaining to the site. Laboratory statistics, however, are useful for calculating and comparing safety factor values for various materials, assuming appropriate statistical distributions and the same probability of failure, as a basic criterion.

From investigations of failures, it appears that the coincidence of extreme weakness and overload, according to typical statistical distributions, never seems to occur. The cause of failure is always a definite fault, such as omission of reinforcement or serious overload. Present safety factor values, used in design in conjunction with good site control, are therefore satisfactory and will continue to avoid the, say, 1 in 10^{-6} hypothetical failure, which appears at first to be statistically inevitable.
In the case of concrete, good site control is practised by limiting deviations of strength in c oncrete at the mixer and by the rejection, at critical sections, of the structure of any material weaker than, say, 85 per cent of characteristic strength.

The difference in philosophy of the laboratory engineer and site supervisor may be reconciled by recognising that safety depends on a double line of defence, viz. control within specified limits at the mixer and the rejection of weak material at critical sections. In addition, overload tests are necessary, when there is uncertainty.

There is sometimes an inconsistency in codes of practice between principles of safety defined in terms of "acceptable probability of failure" and construction requirements, to ensure the rejection of weak material.

Comparing the statistics of road accidents and their inevitability to building failures is to be deprecated. Young structural engineers are in danger of accepting failures as statistically inevitable and alleviating the contractor of his responsibility to reject weak material and apply test loads, where there is doubt.

Margins of safety, as defined by Safety Factor values, must be sufficient to result in weak material and overloading being fairly obvious. The tails of the strength and load histograms for the structure are then hypothetically cut off, unless there is incompetence or irresponsibility and the probability of failure is virtually reduced to zero.