Free discussion

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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,

$$P\left\{\left[R(t)-S(t)\right] \geq 0\right\}_{0<t\leq T} \geq 1-\frac{1}{k} \qquad \text{...1./}$$

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

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

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 1o 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 structures		Temporary structures	
	main girders	secondary girders	main girders	secondary girders
planned lifetime	T = 50		T = 5	
live load	p		0,9 p	
permissible stresses	σ_p	1,1 σ_p	1,1 σ_p	1,2 σ_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.

VII

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

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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.

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 1

LOAD FACTORS FOR THE ULTIMATE 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

TABLE 2

LOAD FACTORS FOR THE SERVICEABILITY LIMIT STATE

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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 concrete 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.