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Some Practical Rules of Up-to-date Dimensioning

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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,

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

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 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 0,80p As a matter of course, if v_p=0 /for store-buildings and containers/,

2./ <u>The risk taken</u>

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 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

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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.