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# 3. Discussion of IABSE PROCEEDINGS

## 3.1 Formal and Real Structural Safety Influence of Gross Errors

Ove Ditlevsen, P-36/80, published November 1980 in IABSE PERIODICA 4/1980.

### A discussion by D.I. Blockley, University of Bristol, England

Professor Ditlevsen is to be congratulated on his discussion of this extremely difficult problem and on his use of fuzzy sets. I welcome his paper and the work he describes. However, if I understand his terms $p_{th}$ and $p_{gr}$ correctly, I think it is an oversimplification to assert that they are independent. Certainly if $p_{th}$ is to include what I call system uncertainty (1) and which is covered in standard reliability theory by a multiplying random variable, the value of $p_{th}$ would be affected by some of my check list questions such as:

1 (a) the loads assumed in the design are a good (accurate) and/or safe representation of the loads the structure will actually experience;

or

4(b) there are no possible effects which could occur in the material which have not been adequately catered for.

Although I agree that $p_{th}$ will be independent of others such as

7(a) contractual arrangements are perfectly normal.

I have criticised the treatment of system uncertainty by probabilisitic reliability theory (1,2). The matching between an idealised theoretical model of the behaviour of a structure and the structure as built is obviously highly complex and I believe the use of a simple factor is inadequate. I believe therefore that uncertainty due to gross errors can be separated into two kinds, system uncertainty and human based uncertainty. It seems therefore that $p_{th}$ could be fuzzified by the former whilst $p_{gr}$ can be treated independently.

Recently I have used this idea with a method of fuzzy logic developed by Baldwin (1). The measure of safety used is a fuzzy proposition "The structure is perfectly safe" measured as a fuzzy restriction on the probability space and modified by a fuzzy truth or dependability restriction. Using fuzzy logical operators a hierarchy of necessary conditions on this initial proposition can be written down.

References

1. Blockley, D.I. "The Nature of Structural Design and Safety", Ellis Horwood, Chichester, England, 1980
2. Blockley, D.I. "A Probabilistic Paradox", Technical Note ASCE Eng. Mech. Div., December 1980

### Comments on D.I. Blockley's discussion by Ove Ditlevsen

Dr. Blockley's discussion was most welcome since it gave me the possibility to clarify my thoughts once more. It is important to note that a linguistic statement is generally not just imprecise by its nature, but it is very often also ambiguous. This ambiguity is what can get a party discussion running for hours. In professional matters it is necessary through detailed discussion and explanation to eliminate the ambiguity but not necessarily the impreciseness of the statement. This very imprecision may most often be the lubricant that makes the statement operational in practical applications.

My interpretation of the checklist questions is that they should not be taken literally but merely as tools for detecting the potentials of gross error producing circumstances. The inclusion of system or model uncertainty elements in the calculation of the theoretical failure probability $p_{th}$ does not take care of gross errors at the design stage concerning inadequate formulation of the structural analysis model, say. Model uncertainty is unavoidable just as is the random uncertainty of parameters that show up explicitly in the model. The model idealization error is not a result of incompetent engineering decision but, on the contrary, a result of careful consideration of the magnitude of error caused by a simplifying idealization of the real world behavior. For example, in question 4 (b), I interpret "no possible effects" as "no possible gross effects" and I consider in principle all the unavoidable minor effects to be consciously evaluated and represented as model uncertainty. The minor modeling errors are made by purpose, the gross errors are unconsciously made. By this interpretation of question 4 (b) it will have no influence on the calculation of $p_{th}$.

It is important for a proper understanding of the concepts of $p_{th}$ and $p_{gr}$ to view these as *decision variables* in the design and construction process.

When the structure is taken into operation there is no interest in the values of $p_{th}$ and $p_{gr}$ except if some decisions have to be made about structural changes or about changes of use of the structure. An analysis at this stage will, if rational, be based on a careful inspection of the existing structure and, perhaps, the file of drawings, descriptions and calculations. The result will most likely be quite other values of $p_{th}$ and $p_{gr}$ than those considered at the design stage. If the inspection reveals gross errors, they will, naturally, affect $p_{th}$ (it may increase or decrease) while $p_{gr}$ is likely to decrease simply because of the removal by the inspection of some potential gross error sources. After the reevaluation of $p_{th}$ and $p_{gr}$ some process follows of designing the remodeling of the structure or other preventions concerning the future use of the structure.

Design decisions based directly or indirectly (through the use of generally optimized code specifications) on some optimization considerations are made by use of a *selected* value of $p_{th}$ (or, in practice, selected safety factors). The situation is in principle the same as at the design stage of a new structure. The recalculated values of $p_{th}$ and $p_{gr}$ (or, rather, the model leading to these values) are just input values for a calculation where the new $p_{th}$ can be controlled to have any selected value. The fact that supports the operationally very important

**FUNDAMENTAL POSTULATE:**

**Minimization of total expected costs is a two variable problem with variables ($p_{th}$, $p_{gr}$).**

**$p_{th}$: theoretical failure probability**

**$p_{gr}$: proneness to failure due to gross errors**

is that for everything given except the detailed specifications of the structural dimensions and material qualities the value of $p_{th}$ can be varied in a wide range without significant influence on the value of $p_{gr}$. This is an idealization, but it is certainly not an oversimplification. I admit that there are types of gross errors that are amenable to safety factor treatment (i.e. to more or less drastic decrease of $p_{th}$), but it is of economical reasons most often not a reasonable practice to decrease $p_{th}$ intentionally in order to eliminate gross error potentials.

The interpretation of the fundamental postulate is that any pair of values of ($p_{th}$, $p_{gr}$) in principle can be selected as those values that control the decisions of the entire design and construction process. A wise choice is then the pair of values that corresponds to the smallest expected costs (in a generalized sense). In practice, however, $p_{gr}$ appears on a discrete scale and is determined by the selection of a specific team of designing engineers, their choice of the specific structural arrangement, the contractor, etc., i.e. what I call the total lay-out of the structure. The theoretical failure probability, however, varies on a continuous scale for each total lay-out. Current philosophy of using the minimum cost value of $p_{th}$ as design value within each total lay-out is healthy at least for structures where the generalized costs associated with failure are not dominated by the construction costs due to material consumption. In fact, these costs are the smaller the larger values of $p_{th}$ are selected. This, on the other hand, causes the costs of failure due to gross errors to be dependent on $p_{th}$ in the same rate. If this is taken into account, the optimal target value of $p_{th}$ for the given total lay-out shifts towards a larger value. Thus it is on the safe side in the reliability sense to neglect this influence of $p_{gr}$ on the optimal value of $p_{th}$, i.e. to follow current philosophy.

I cannot see any philosophical difficulties in adopting the mathematical apparatus of probability theory as a framework for defining $p_{gr}$. In that case $p_{gr}$ is the probability of failure caused by gross errors. Then the total probability of failure is simply $p_{th}+p_{gr}$ (or, at least, this sum is a close upper bound on the total failure probability). The sum will in practice in most cases be dominated by $p_{gr}$. Nevertheless, it is the variation of $p_{th}$ that for a given total lay-out results in variation of the structural dimensions which, on the other hand, leaves $p_{gr}$ almost unaffected.

Difficulties of formulating a practicable probabilistic definition of $p_{gr}$ makes it worth-while, perhaps, to look for other simpler possibilities. One of these is the theory of fuzzy sets or fuzzy logics. However, the gain in simplicity in comparison with probability theory is counteracted by the loss of a canonical definition of expected loss. This is unfortunate because the selection among alternative total lay-outs should in general be guided by minimization of some measure of costs.

## 3.2 Nonlinear Analysis of Cable-Stayed Bridges

A. Rajaraman, K. Loganathan, N.V. Raman, P-37/80 published November 1980 in IABSE PERIODICA 4/1980

**Discussion by J. Schlaich, University of Stuttgart, Fed. Rep. of Germany**

Analytical work in structural engineering becomes a useless or even misleading exercise, if its mechanical model and its input data are not related to the real structure. This means with respect to bridges in general and to cable-stayed bridges especially, that their real load- and system-history, as a consequence of their method of construction, has to be followed in analysing their forces.

The statical system and the usual free cantilevering erection method of cable-stayed bridges permit the completely *free choice of the moment distribution and of the profile of the main girder under dead loads.* For steel main girders the moments are chosen in such a way that after superposition with the live load effects, the most favourable use of the sections is made. For concrete main girders, in order to eliminate the effect of creep, usually the moments corresponding to those of a beam on rigid supports at the cable anchorages are chosen. By predetermining and prefabricating the stress-free workshop geometry of the main girder elements and of the cable-lengths accordingly, the desired moment distribution and profile of the main girder are automatically achieved during erection, if required by additional stepwise stressing of the cables. With respect to this profile, the deflections of a cable-stayed bridge under dead load are those of a rigidly supported bridge i.e. nearly nil and there is no nonlinear or second order effect whatsoever! Obviously these moments under dead load are very small and the main girder or decking is dominated by axial forces.

The authors, however, come out with just the opposite and therefore totally misleading statement on the characteristics and load-bearing behaviour of cable-stayed bridges. Nobody would want to build a cable-stayed bridge according to what they have analysed: They apply the total load, i.e. not only live load and the secondary dead load (wearing coat, railings, etc.) but also the predominant structural dead load to the completed final system. This would not only correspond to a totally unpractical erection procedure, i.e. to build the whole bridge, including its central span on trestles, but also leads to by far too high and unrealistic moments and deformations which the main girder could never withstand.

Unfortunately the authors give only the results of their analysis and the overall geometry of their prototype bridge, but no drawing of its cross-section, no sectional values and no loads. Therefore, it is impossible for the reader to calibrate their findings. On the other hand it is easy for the writer to identify Fig. 1 of the article with the Hooghly River Bridge at Calcutta/India, at present the largest cable-stayed bridge under construction, since he happens to be involved in its design. He is therefore able to compare approximately the dead load moments of its main beam at midspan due to the authors (+6341 tm) with the real ones (– 220 tm at the cable supports and +110 tm in between) and their deflections (2,57 m) as compared to nil.

Consequently also the authors' second order or nonlinear analysis findings due to live load–since they apply them wrongly to the heavily predeformed bridge–are not only far beyond reality but mainly totally confusing: In fact the nonlinear live load effects on the axial forces of cables and main girders of cable-stayed bridges are usually negligeable, whereas the live load moments increase commonly by about 10%, sometimes up to 20%. The effect of this live load moment increase on the total stresses in the main girder, which include dead load, is of course much less.

**Replies to J. Schlaich's discussion by A. Rajaraman**

The paper highlights mainly the influences of different types of nonlinearities, namely cable sag, beam-column and geometry changes in the response of cable-stayed bridges, using an analytical model of two dimensional plane frame members. Even though results are given for a typical example bridge under dead and live loads, the applicability of the model and programme is general in that it can handle erection sequence and variations in loading conditions. The paper further elaborates the influences of the nonlinear effects–individually and in combination–for both dead and live loads on deformations, moments, axial forces and cycles needed for convergence, an indirect measure of computer effort. It seems that the discussor has completely overlooked these and as the paper is not intended to cover various stages of construction including erection, the remaining comments by the discussor in this regard are not relevant. But it is to be emphasized here that erection sequence can be handled using the model and programme, once the details are given. Secondly, the discussor points out the nonlinear effects are not significant under live loads, and this will in effect mean that axial forces in deck and tower do not affect the moments in them and that sag in cables is insignificant. But this is not so in any cable-stayed bridge where axial forces play a dominant role in the resistance of the loads, and hence nonlinear effects should be considered even on the adjusted profile of the bridge. The details of bridge chosen for analysis are taken from available literature and since complete details are not given in terms of erection sequence, loadings and pretensions, the idealisation given in Fig. 1 has been chosen for assessing the influences of nonlinearities in the response of the bridge, which is the main theme of the paper. The influences of dead load in erection sequence, bridge profile and pretensions to maintain zero deformations at centre are entirely different studies which do not come under the scope of the present paper. But it is emphasized again that even in these studies, the present model can be used and nonlinear effect as mentioned, should be considered.

## 3.3 Remark

Looking at the discussion by Prof. Schlaich and the response by Mr. Rajaraman I do not feel that any arrangement has been reached. Prof. Schlaich states what should be introduced into an analysis of cable stayed bridges, Mr. Rajaraman on the other hand presents what could be analysed with respect to nonlinear effects using analytical models of two dimensional plane frame type. Nevertheless I think, that the discussion should be brought forward to the readers of IABSE PERIODICA as early as possible. I leave it to the author of the questioned paper and Prof. Schlaich and possibly others to continue the discussion in a further issue of the Bulletin.

In a more general way I am happy to introduce here the very first discussions of papers published in IABSE PERIODICA. I think discussion is a good thing and we should emphazise it, because it helps for better understanding between authors and readers.

Discussions of papers are welcome!

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