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Objekttyp: **Group**

Zeitschrift: **IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen**

Band (Jahr): **4 (1969)**

PDF erstellt am: **02.05.2024**

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Design of Prestressed Concrete Bridges, based on the Fatigue Life for the expected Load Spectrum

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The author together with Professor E.I. Brown discussed this question in Chigaco this year (Ref. 1). There are two problems of probability, one relating to the random loading at highway bridges and the other to the fatigue resistance of prestressed concrete. The daily load frequency at 21 bridges over the Ohio river according to Lynch (Ref. 2) has shown that approx. 30,000 vehicles passed the bridge daily of which 85 % were cars and approx. 95 % had max. axle loads less than 9 kips (4.1 Mp). Only 0.1849 % had a max. axle load greater than 19 k. (8.6 Mp) and 0.032 % had a max. axle load greater than 21 k. (9.5 Mp). The last mentioned loading corresponds to about 9 vehicles per day. In the USA, the greatest specified axle load is 32 k. (14.5 Mp). However, infrequent "abnormal" loading may occur and there are overload provisions in the regulations by which the permissible design load may be increased by 100 % in a single lane, without loading in another lane, provided that the combined dead & live load and impact stresses are not greater than 150 % of the allowable stresses. However, with prestressed concrete only very limited tensile stresses are allowed for this abnormal loading.

In the United Kingdom higher loadings are specified than elsewhere. The abnormal HB loading relates to a max. axle load of 45 long tons (100.8 k. or 45.8 Mp)with a vehicle having 4 such axles, two pairs being 6 ft. apart (1.83 m) and the distance of the two inner axles being 20 ft. (5.18 m). However, these loads occur very rarely. In a Technical Memorandum of the Ministry of Transport provisional requirements are given for steel bridges. Whereas annually $2x10^6$ commercial vehicles should be considered at dual carriage ways and half of it at a single carriage way, the entire number of abnormal annual loadings is given as 4500. Of these only 30 are of the maximum abnormal load, whereas 70, 200 and 700 loadings are at 80 %, 60 % and 40 % respectively of the max. load and the bulk of 3500 loadings is of a weight of 20 % of the max. load. Based on these data it is possible to evaluate the number of loading cycles for different load intensities during the expected life time of the bridge and to design the bridge to withstand the cumulative effect of all these loadings. Increased stresses are permitted for steel and reinforced concrete, but this can not be applied to prestressed concrete.

In the paper (Ref. 1), five different design methods for prestressed concrete bridges are discussed. Method 1 relates to a design at which only compressive stresses occur at the maximum load; this ensures a safe but uneconomical solution. By method 2 limited tensile stresses are permitted under the maximum load, as employed by British Railways, Eastern Region 1948-62 during the author's activity (Ref. 3) and by A.A.S.H.O. in the United States (Ref. 4). Method 3 is based on a proposal by Professor C.E. Ekberg (Ref. 5) relating to the fatigue resistance of a definite stress range according to the Goodman diagram. Method 4 was suggested by the author in 1956 (Ref. 6)and relates to a design at which similarly to method 1 only compressive stresses occur under the normal loading, whereas temporary cracks may occur under the "abnormal" loading. Method 5 suggested in the paper (Ref. 1) is based on the cumulative fatigue resistance to the entire load spectrum during the bridge life, bearing in mind at random loadings.

In a paper (Ref. 7) tests have been reported which were carried out at DUKE University which showed that prestressed concrete can withstand a relatively large number of load cycles over a high loading range. For example it was found that about 200,000 cycles can be withstood over a range between 30 and 70 % of the static failure load. In this case the lower limit corresponds to the dead load and the upper to 1.82 times the maximum design load. Similarly 30,000 cycles were withstood to a loading range between 30 and 80 % of the static failure load, where the upper limit of the range corresponds to 2.27 times the max. design load. Even for a loading range between 30 and 90 % of the static failure load a few thousand cycles were recorded in which case the upper limit corresponds to 2.78 times the max. design load. The cumulative effect can be obtained from Miner's hypothesis (Ref. 8) according to which $\Sigma n_i/N_i = 1$ (where n_i is the number of cycles applied for a certain range, whereas N_i is the number of cycles which the construction can withstand). This results in a damage line which presents a curve of lesser capacity than the s-n curve or load-number of cycle curve. The applicability of Miner's hypothesis to steel construction, particularly to air craft, has been queried, but it can be assumed, based on the tests at DUKE University, that it is in good agreement with prestressed concrete with well bonded tendons.

Figs. 1 and 2 illustrate the design methods 4 and 5 respectively. In Fig. 1 two probable limits for the load-number curve are plotted from which the damage line is obtained. For example, on the assumption that 200,000 cycles between 30 and 70 % of the static failure load can be withstood (corresponding to an upper limit of 1.82 times the max. design load), but only a quarter of the number i.e. 50,000 cycles apply, a definite point of the damage line is obtained. This Fig. 1 is based on 500 million vehicle loadings and it is assumed that at 499 millions of these loadings only compressive stresses occur at which there are no open cracks, whereas under one million of abnormal loadings cracks may occur. It is further assumed that only 50,000 loadings are as high as 1.82 times the design load, the other abnormal loadings being less. Fig. 2 relating to method 5 is also based on 500 million loadings and a damage line is plotted which is based on the cumulative effect of loading.

References:

(1) P.W. Abeles & E.I. Brown, "The Expected Fatigue Life of Prestressed Concrete Highway Bridges as related to the expected Load Spectrum", 2nd International

Symposium on Concrete Bridge Design, ACI, April 1969;

(2) R.L. Lynch, "Research of Traffic Loads on Bridges", Kentucky Dept. of Highways, Division of Research, Lexington, Feb. 1968;

(3) P.W. Abeles, "Static and Fatigue Tests", ACI Journal, Title No. 51-19, Dec. 1954;

(4) American Association of State Highway Officials, Standard Specification;

(5) C.E. Ekberg etc. "Fatigue Resistance of Prestressed Concrete Beams in Bending", paper No. 130@, ASCE Journal of Struct. Division, July 1957;

(6) P.W. Abeles, "The Safety against Cracking", IABSE, Lisbon 1956, Final Report;

(7) P.W. Abeles, E.I. Brown & J.M. Slepetz, "Fatigue Resistance of Partially Prestressed Concrete Beams to Large Range Loading", IABSE, New York, 1968;

(8) M.A. Miner, "Cumulative Damage in Fatigue", Transact. 67, ASME, 1945.

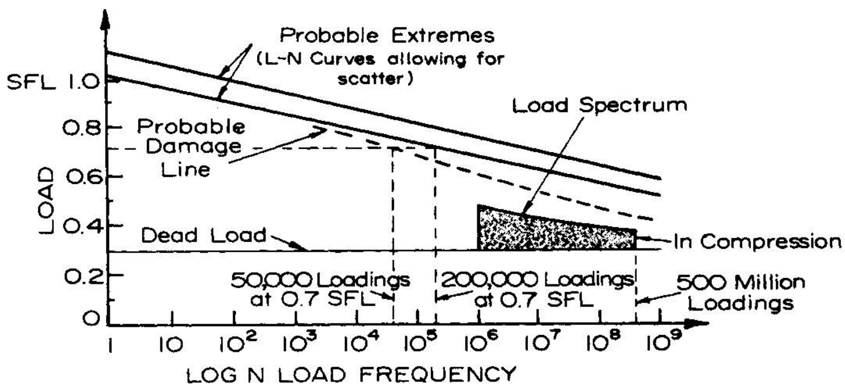

Fig. 1

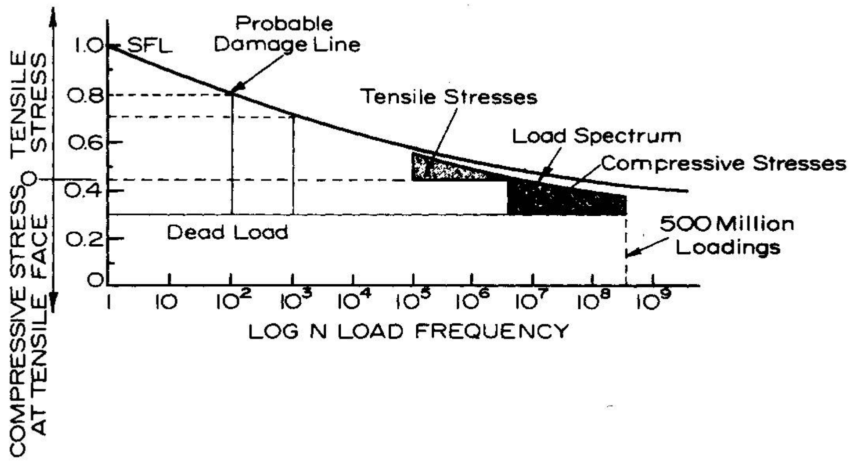

Fig. 2

V

Structural Behaviour of Members and Connections in Large Panel Buildings

Comportement statique des éléments et des moyens de liaisons dans les constructions en panneaux assemblés de grand format

Statische Wirkung von Bauteilen und Verbindungen in Großtafelbauten

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(1) Specific Factors in Assessing the Safety of Structures of Panel Buildings

My contribution deals with some specific problems arising from the structural behaviour of members and connexions in panel apartment buildings. Structures of this kind have become very frequent lately. The methods of design in such panel structures have achieved a considerable degree of development, e.g. [5]. Still, many problems in this field remain unanswered. The resulting situation has been largely conditioned by the specific structural qualities of panel buildings, as follows :

(a) Load-bearing structures in panel buildings are spacial, and considerably rigid. Consequently, they are noticeably responsive to the effects of some subsidiary influences, such as various volumen changes of the concrete - either caused by long-term loading (creep), or by changes in humidity (shrinkage) and temperature.

(b) Load-bearing structures in panel buildings consist of floor slabs and wall panels. The behaviour of the structure depends on the bearing capacity and rigidity of the structural connections.

In addition to these, there are further problems common also to all cast-in-situ structures that are formed of the same kind of material, such as heavy concrete, light-weight concrete, structural tiles, etc. In assessing the safety of any building structure (including panel building structures) one must start, then, from taking into consideration the variability of each specific factor that might influence the behaviour of the structure in question.

In the first place we are confronted with factors which, from a statistical point of view, are random. They are, e.g., the loads, the bearing capacity of the members and connections, as well as the quality of their making. The possibilities of considerable deviations or errors in workmanship cannot be statistically registered, but they must not be left out of our calculations.

Answering the needs of the building practice, some semi-probabilistic methods have been developed. These methods investigate only the variability of loading in structural members and their connections, as well as the variability of their bearing capacity. The variability of other phenomena is to be ascertained by systems of subsidiary coefficients. Ideally, the amount of such coefficients should be as high as possible, in order to assess the behaviour of the structure with a maximum of precision. For practical reasons, however, the amount of coefficients should be limited to a usable extent. In the present state of our information on structural behaviour, particularly in large panel buildings, many coefficients are often assessed, not on the basis of statistical investigations or other theoretical considerations, but rather on the basis of experiments - or even according to the individual opinion and empiric experience of the designer.

(2) Loading of Structural Members and Connections in Panel Buildings

In panel buildings, various kinds of loads and other influences are acting on the strustural members and connections. The effects of horizontal and vertical loads are assessed by well-established methods. The variability here does not substantially differ from the variability observed in other kinds of buildings. Frequent damages have recently been found in the load-bearing structures of panel buildings, mortly cracks in the inter-panel connections. Owing to such occurences, greater attention was called to the effects of the subsidiary influences (see above) exercised on panels and their connections.

The effects of shrinkage can appear in panels some time after their production. For that reason, the assembly of panels of insufficient age should be avoided. Especially noticeable are the effects of the shrinkage of concrete in panels and of infill-concrete in vertical connections (joints) between respective wall-panels. The values of shrinkage present an extensive scatter, as they are dependent on the quality and composition of the concrete, as well as the humidity and temperature of the surroundings, which may be vary considerably. The calculation cannot embrace the total amount of the scatter. Consequently, necessary precautions must be taken during the production, and particularly during the assembling.

The effects of the creep may appear favourable in the horizontal sections of panel walls, where they contribute to the redistribution of the compressive stresses. Unfavourable they may appear in the vertical connections between wall panels, where their long-term compressive strains are different.

The joints of the exterior structures register noticeable effects of the changes in the temperatures of the respective structures. Usually the exterior panel undergoes a change of temperature in the direction of its thickness. The changes of temperature cause the deformations of panels. If they are prevented, thermal stresses appear in the panels and their joints. The magnitude of such effects varies considerably, because - even with identical material in the panels - many different conditions are operating, such as the local meteorological influences, the orientation on the cardinal points of the horizont, the intensity of sunshine, the make and colour of exterior surface.

These observations have been confirmed by the results of measuring an actual eight-floor panel building, which was being undertaken in the course of three years [4] .

In view of the character of these effects, and in view of the amount of contemporary knowledge concerning those effects, the procedure in calculating the structures is, for the present, as follows : either empirical coefficients are being introduced, which reduce the bearing capacity of connections, or fictitious forces are being employed, which determine,e.g., the dimensions of transversal reinforcements.

(3) Bearing Capacity of Members and Connections

We have pointed out that the actual bearing capacities of structure members and their connections practically show some deviations from the design bearing capacities. The causes of such deviations have their origins either in the character of the structure itself, or in the method of calculation. Let us mention the most important factors in this line :

a) The strength of the concrete in actual structures differs from the strength ascertained by means of test cubes. This happens most frequently with the light-weight concrete.

b) The strength of the concrete in actual structures is lower than is required, which may be caused by insufficient control during the production of the concrete, or by special conditions during the hardening of the concrete (this applies mostly to the infill-concrete in the precast structures). Also, local reduction in the strength of the concrete may occur, e.g. in wall panels. Such events are dangerous in plain concrete structures.

c) The dimensions of the sections differ from the design ones. This may have various causes ; in the first place causes due to production or assembling.

d) Quite often one finds deviations from the design eccentricity of loading. It is well known that in calculating the wall panels and their horizontal joints a certain assembling eccentricity is being introduced, the magnitude of which is reconned mostly on the basis of various considerations. It is desirable to arrive at a greater accuracy of eccentricities, on the basis of measuring actual structures, which of course would be very laborious.

e) Another source of deviations may be found in the methods of calculation. Either the calculation itself is merely approximate, or our present data of information are insufficient. This applies particularly to the bearing capacities of the various joints of panels. Moreover, in calculating, one may consider a different, less complicated fixing of wall panels than is employed in actual structure.

f) In all types of pre-cast structures, and the more so in panel building structures, deviations may occur in the bearing capacities of panels and their joints, owing to differing methods of assembling and their differing precision.

(4) An Example of Assessing the Safety of Joints of Panels

Since 1964 we have been engayed in theoretical and experimental research in the bearing capacity of selected joints of panels. The results of these investigations are shown in design formulae of joints.

The investigation on the vertical joints between wall panels was more fully reported on in [2, 3]. The investigation on the horizontal joints of wall and floor panels was shortly reported in [1]. On the whole, one hundred and fifteen full-scale test specimens have been examined ; they present various modes of connection.

In each kind of joint, it was found impossible to subject to experimental examination a sufficient number of models, which would yield results for statistical estimation. Consequently, eventual deviations from the design formulae of the design bearing capacity must be assessed with the aid of wider considerations based on the detailed knowledge.

An instance may be adduced from the horizontal joints of wall and floor panels. Let us consider a symetrical joint, as per Figure 1 . Both wall and floor panels are solid. The thickness of the upper and the lower wall panel is identical. Also the thickness of the left and the right floor panel is identical. The loads, acting on the connection (Figure 1) are symetrical. Usually the most dangerous section is section 2 – 2 . To assess this section, we presume a simplification of the loading, as per Figure 2 . The reasons for this procedure are more fully explained in [1] .

The actual course of the compressive stress in the section 2 - 2 is shown in Figure 3 . For the treatment of the bearing capacity of the section 2 - 2 , the course of the stress as in Figure 4 is assumed. It was ascertained experimentally that the area of the statically effective section $\bar{F}_2$ is larger than the area of the sections F_1 and F_3 respectively, and that the bearing capacity of the section 2 - 2 can be expressed by the formula

$$\bar{N}_2 = R_{c,fl} \cdot \bar{F}_{fl} + R_{c,i} \bar{F}_i , \qquad (1)$$

where $R_{c,fl}$ and $R_{c,i}$ mean the strength of the concrete in the floor slabs, and the strength of the infill-concrete, respectively,

$\bar{F}_{fl}$ and $\bar{F}_i$ mean the respective areas of the floor slab ends, and of the infill-concrete,

$\bar{F}_{fl} + \bar{F}_i = \bar{F}_2$.

For practical use we propose the following formula

$$N_2 = (k_1 R_{c,fl} F_{fl} + k_2 R_{c,i} F_i) k_3 , \qquad (2)$$

where F_{fl} and F_i mean the respective areas of the floor slab ends, and of the infill-concrete;

$F_{fl} + F_i = F_2$;

$k_1 = 0,9$ is the coefficient expressing the local deterioration of the concrete in the area F_1 ,

$k_2 = 0,7$ - the coefficient expressing the greater deviation in the strength values of the infill-concrete compared to the floor-slab concrete,

k_3 - the coefficient expressing the effectiveness of the area F_2 ,

$k_3 = 1,0$, if the floor-slabs are solid,

$k_3 = 0,9$, if the floor slabs are provided

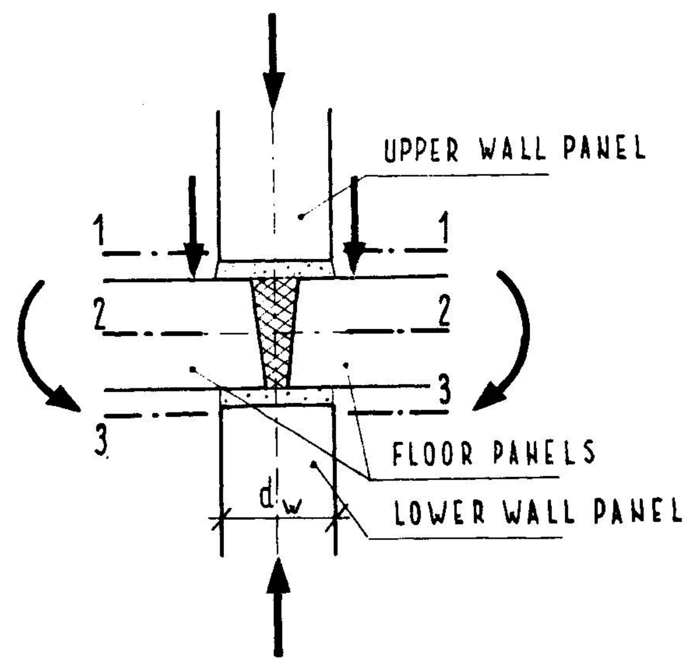

FIG. 1

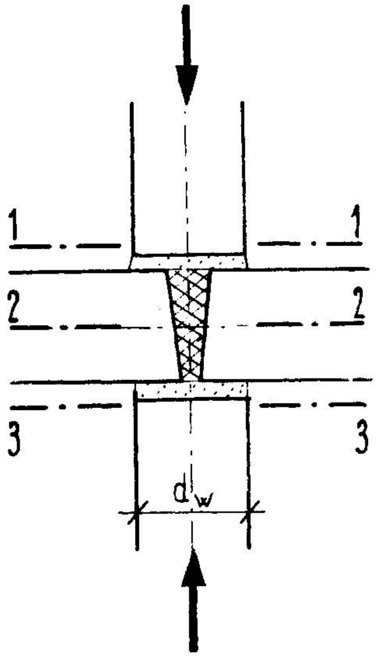

FIG. 2

SECTION 2-2

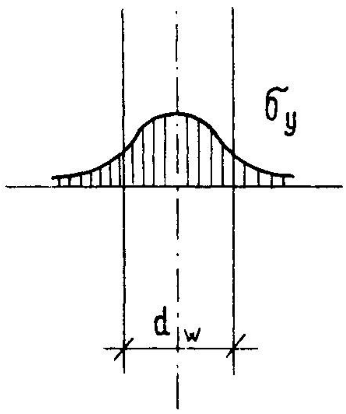

FIG. 3

SECTION 2-2

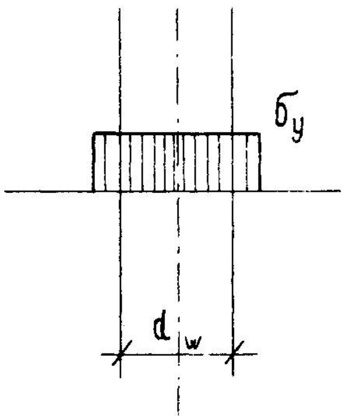

FIG. 4

with longitudinal holes.

The above formula (2) concerns a smaller area F_2 than the area $\overline{F}_2$ in the above formula (1), in order to express the unfavourable influence of some other factors acting on the joints.

References

1. Pume D. : Bearing capacity of joints between wall panels and floor slabs. International Symposium CIB on Bearing Walls. Warsaw 1969

2. Pume D. : Shear strength of vertical joints between concrete elements (in Czech). Stavebnícky časopis, 1967, No.3

3. Pume D. : Scherfestigkeit senkrechter Stösse zwischen Betonwandelementen (in German). Bauplanung-Bautechnik, 1967, No.5

4. Pume D., Laurin M. : Dimensional changes of panel building due to long-term loads (in Czech). Pozemní stavby, 1968, No.11

5. Recommandations internationales pour le calcul et l'exécution des constructions en panneaux assemblés de grand format. CEB , Paris 1967

SUMMARY

The safety of structures in large panel building must be treated taking into account special features of these structures. It concerns the loads and other subsidiary influences, as well as the bearing capacity of members (panels) and connections (joints). An example of assessing the safety of joints between wall panels and floor slabs, including the design formulae, is given.

RESUME

La sécurité des constructions en panneaux assemblés de grand format doit être traitée avec les problèmes inhérents à ce genre de constructions.

Ceci est valable tant pour les charges et les autres influences, que pour la résistance propre à chaque élément ou liaison.

On donne un exemple pour le calcul de la sécurité des liaisons entre des éléments de parois et des dalles de planchers, à l'aide de la formule indiquée.

ZUSAMMENFASSUNG

Die Sicherheit von tragenden Konstruktionen in Grosstafelbauten muss unter Berücksichtigung der speziellen Eigenschaften dieser Konstruktionen behandelt werden. Das gilt sowohl für Belastungen und andere Einflüsse als auch für die Tragfähigkeit einzelner Bauteile und Verbindungen (Knoten). Ein Beispiel zur Beurteilung der Sicherheit von Knotenpunkten aus Wand- und Deckenelementen mitsamt Berechnungsformel wird angeführt.

V

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C. ALLIN CORNELL
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I should like to comment on a theme, or perhaps better a conflict, running throughout the discussion in this symposium--namely deterministic vs. probabilistic, physics vs. statistics, or even science vs. engineering. When put to an ultimate test, it is impossible to distinguish between "random" ("inherently variable") factors and "non-random" factors in which only professional ignorance or human volition are involved.

In design we are all apparently prepared to treat as stochastic variables such factors as yield strength of steel, the maximum annual wind velocity, and the maximum lifetime earthquake intensity at a site. Yet, upon closer analysis each can be separated into identifiable, systematic, predictable compoments. Several here have discussed these components of steel strength. I have heard a meteorologist claim confidently that his science knows enough about the mechanics of their system that, given enough data and a big enough computer, they could predict every aspect of the weather including local wind velocities for years in advance. Tectonic earthquakes are caused by mechanical processes that are, at the moment, very incompletely understood, but hardly beyond the competence of man to know better and eventually predict with some accuracy.

If this is true, can we still treat these variables as random variables? Of course we can, simply because it is useful to do so. It will lead to better engineering design in the form of a better, more economical allocation of material and resources. In short, any probability assignment is an intellectual concept, not a physical attribute, such as length or weight. All probabilities are convenient fictions, simply very useful, quantitative measures of the ever present uncertainty of someone or some profession.

No coin has an inherent probability of p of coming up heads when flipped. I would be very disappointed in modern engineering mechanics and computers, if, given sufficient initial and boundary conditions, they could not predict precisely the outcome of any flip. It might, however, take several years to make the prediction. If a dollar rides on your predicting whether there will be more heads than tails in ten flips, you would be advised to base your decision on the position that there is a fixed probability of coming up heads on any flip. If your life depended on your prediction of a single flip, you would probably devote your career to the indicated mechanics and computers, not the mathematical probability theory of independent Bernoulli trials.

So, too, in engineering, probability should be used because it is the most effective way known to treat uncertainty in design. Its use should not interfere with or obscure engineering scientists in their search for better and more useful understanding of the governing physics or phenomenological evidence. Nor need confirmed engineering probabilists despair. These scientific results will reduce but never eliminate our profession's uncertainty. Nature is too complicated to be predicted by the handbook formulas necessary for conventional design practice.

V

Free Discussion / Discussion libre / Freie Diskussion

HERMANN BEER
Prof. Dr.
Graz

In their very interesting paper, Professor Tall and Dr. Alpsten mentioned that it is possible to predict the behaviour of a compression member provided that we have a thorough knowledge of the properties and geometry of the material. I would like to extend this statement by considering the entire safety problem. In his General Report, presented at the 8th IABSE-Congress, one of the leading statisticians in engineering science, Professor A.M. Freudenthal, stated that the principal theoretical problems are the existence of non-random phenomena and the impossibility of observing the relevant random phenomena within the ranges that are significant for safety analysis. Furthermore, statistical results can lead to errors if sufficient data are not available. The behaviour of a structure is somewhat complicated and there are so many different types of structural components and structures that it seems to be impossible to consider the safety problem in a purely probabilistic manner. Therefore, we have to utilise all the deterministic methods that may be helpful in predicting the behaviour of the structure until failure occurs.

V

Etude probabiliste du flambement des barres en acier

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Le Professeur Tall vient d'exposer les études sur les effets attribués aux contraintes rémanentes dans la résistance au flambement des pièces comprimées en acier. Je voudrais présenter une approche différente du problème du flambement, directement liée au thème général de ce Symposium.

L'analyse en laboratoire des contraintes rémanentes offre incontestablement des éléments du plus grand intérêt pour l'étude théorique de la résistance au flambement ; la concordance obtenue à Lehigh, entre les prédictions et les résultats expérimentaux, est frappante. Cependant, le praticien de la construction se pose des questions additionnelles, liées aux conditions industrielles dans lesquelles il réalise ses ouvrages

Ainsi, on sait que l'état de contraintes rémanentes dans une pièce donnée n'est ni indépendant ni permanent. En dressant une barre qui présente une courbure initiale, on introduit des contraintes rémanentes additionnelles. En lui appliquant ensuite une charge, on réintroduit un défaut de rectitude et on supprime des contraintes rémanentes Il en résulte que, par suite des manipulations au transport et sur parc et des opérations de mise en oeuvre, l'état de contraintes dans une barre donnée peut se trouver sensiblement modifié par rapport à celui qu'elle aurait présenté en laboratoire. De plus, on peut concevoir que cet état de contraintes subit encore une modification progressive, du fait des sollicitations de la barre dans la structure en service

C'est en partant de ces considérations que la Convention Européenne de la Construction Métallique a effectué une vaste étude expérimentale, réalisée conjointement dans sept pays, sur barres "industrielles", c'est-à-dire comportant des inperfections de toute nature, inévitables dans la pratique. Il ne s'agit donc pas d'une analyse séparée de l'influence des divers paramètres - qu'il semble d'ailleurs impossible d'isoler rigoureusement - mais d'une étude globale, menée suivant des critères statistiques et exploitée dans le sens probabiliste.

Le principal intérêt de cette méthode d'investigation, préconisée par Dutheil, réside dans le fait qu'elle permet de déterminer pour les barres comprimées de tout élancement - et, par extension, pour tous les cas d'instabilité - un critère de ruine défini avec une probabilité donnée. Il est ainsi possible d'obtenir un degré de sécurité homogène pour tous les éléments constitutifs d'une structure, quel que soit leur mode de sollicitation. Aucune des méthodes et théories classiques ne permet d'atteindre ce résultat.

L'élimination de la part d'arbitraire dans la détermination des coefficients de sécurité pour les pièces sujettes au flambement a permis à la Convention Européenne d'établir une courbe très favorable pour le dimensionnement des barres comprimées. En se référant aux résultats de ces essais, plusieurs pays ont pu améliorer considérablement les courbes de flambement dans leurs réglements L'économie ainsi réalisée apparaît notamment dans les élancements les plus fréquents.

Un programme expérimental complémentaire de la Convention Européenne est présentement en cours à Lehigh, avec la collaboration du Column Research Council. Le fait d'avoir confié son exécution au Professeur Tall permet d'entrevoir une heureuse confrontation des deux méthodes et une association, qui ne peut être que profitable, des différentes approches du problème.

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