**Contribution to the Free Discussion Regarding the Paper "Interaction of Postcritical Plate Buckling with Overall Column Buckling of Thin-Walled Members" by J. De Wolf, T. Pekoz and G. Winter**

Contribution à la discussion libre de l'article "Interaction of Postcritical Plate Buckling with Overall Column Buckling of Thin-Walled Members" par J. De.Wolf, T. Pekoz et G. Winter

Diskussion bezüglich des Beitrages "Interaction of Postcritical Plate Buckling with Overall Column Buckling of Thin-Walled Members" von J. De Wolf, T. Pekoz und G. Winter

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To begin with, I would like to congratulate Professor Winter and his co-workers on obtaining very interesting and valuable results regarding the limiting state of thin-walled columns.

I would also like to take this opportunity to mention that our team at the Institute of Theoretical and Applied Mechanics in Prague has been concerned with the interaction of overall column buckling with plate buckling for some 14 years. Several investigations, both theoretical and experimental, have been carried out. For example, a few years ago, twenty eight thin-walled columns were tested, with the slenderness ratio of the column and the width-to-thickness ratio of its plate elements being varied in a way that both the column and plate buckling could be studied.

It is beyond the scope of this contribution to the Free Discussion to describe all our results and observations; therefore, I have to limit myself to a few conclusions, which may be of some interest in connection with Professor Winter's paper.

To start with, it is, perhaps, worth mentioning that in all our tests a pronounced interaction of column buckling with the buckling of its plate elements was observed. It follows from this observation that it is not possible to separate (as is frequently done when following the currently held design concept) the behaviour of the column as a whole from that of its plate elements. A steel column is always a system of plates, the overall deformation of which and the local one (i.e. buckling of plate elements) are interconnected.

Further, I would like to draw attention to the fact that the performance of thin-walled columns is considerably affected by unavoidable initial irregularities (like an initial curvature, residual stresses, etc.).

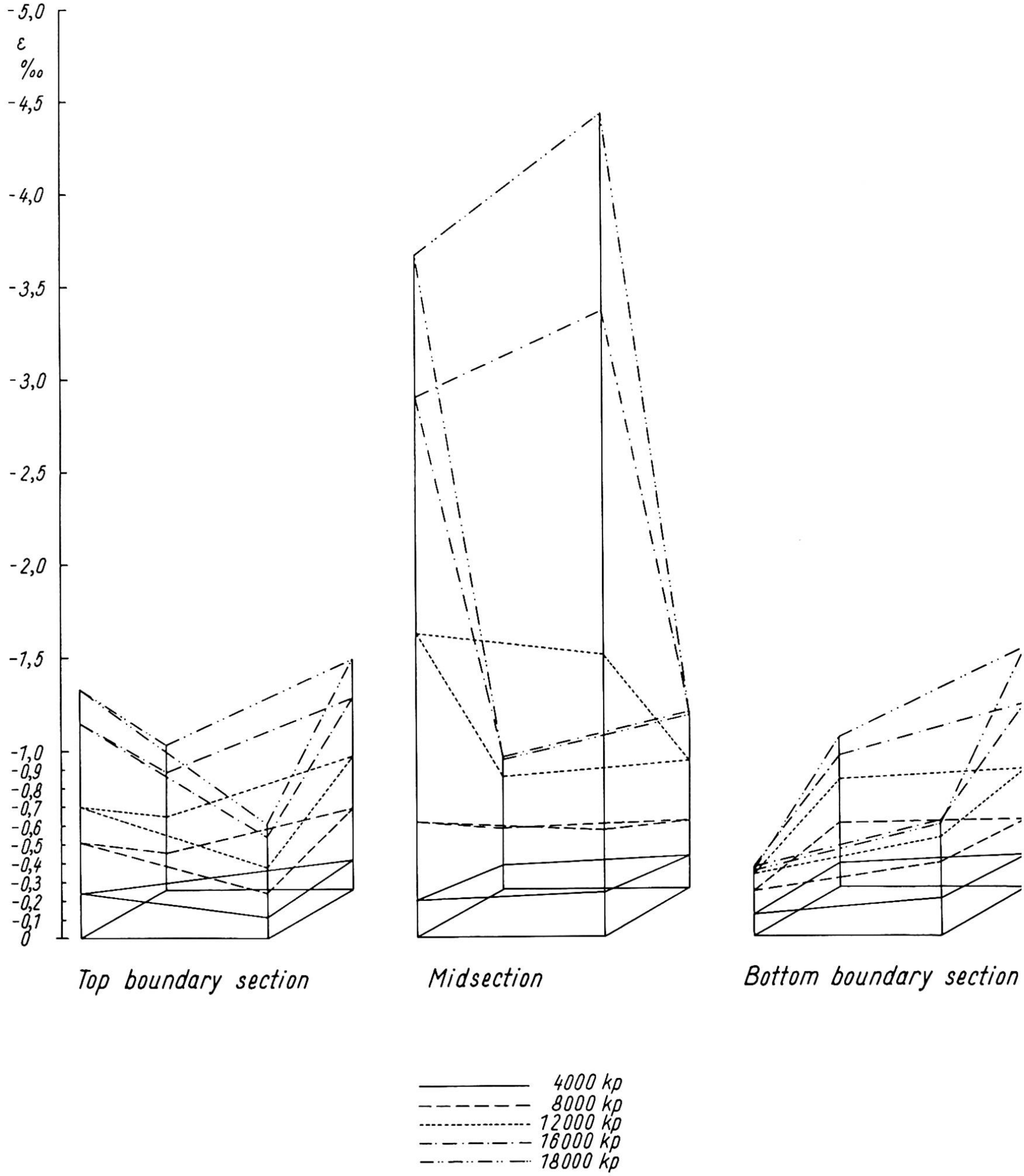
For example, in the case of a centrally loaded column, the initial irregularities make the column deflect from the very beginning of loading. As a result of the flexure of the column as a whole, the loads of the plates on the concave side of the deflected column are increased, whereas those acting on the plates on the convex side are reduced. That is why, in the most loaded section of the bar, the load acting on the concave side plate can be substantially (in the case of very slender bars even several times) larger than that which acts on the plate element on the convex side of the deflected column (Fig. 1). Furthermore, it is of importance that this increased load of the plate on the concave side is, at the most stressed section of the column, frequently substantially greater than the avarage value $\sigma = P/A$, which is considered in the design if an "ideal" column without initial deviations is assumed.

The influence of the initial irregularities upon the loading of the plate elements is reflected in the character of the waving of these elements (Fig. 2). That is why plate buckling is more pronounced on the concave side of the deflected bar than it is on the convex one. Moreover, in view of the fact that in the case of the plate on the concave side the load acting on it is larger in the central section than at the boundaries of the bar, the buckled pattern is more pronounced in the middle of the column than at its ends. On the other hand, the plate on the convex side is less loaded in the central part of the bar than at the boundary sections; therefore, the wave pattern tends to be less pronounced in the middle of the column than in the boundary zones. The aforesaid analysis shows that the currently held model of behaviour, according to which the plate elements of a centrally compressed bar are uniformly loaded and, consequently, uniformly waved, is not compatible with the behaviour of ordinary thin-walled steel columns.

The effect of initial deviations upon the limiting state and the ultimate load is shown in Fig. 3, where the experimental load-carrying capacities ($\sigma_{ult}^{exp}$) of one test series are plotted in comparison with a/ the critical load $\sigma_{cr}'$ of the bar as a whole, evaluated regardless of plate buckling, b/ the critical load $\sigma_{cr}^{p}$ for the buckling of the weakest plate element, and c/ the critical load $\sigma_{cr}^{e}$ determined for the stability of the column as a whole with due regard to plate buckling and, therefore, for an effective cross section. In this case, the effective widths of the plate elements were determined by using Winter's formula.

An inspection of the figure indicates that there is no definite relation between the experimental ultimate load on one side and a/ the critical load $\sigma_{cr}^{p}$ of the weakest plate element and b/ the critical load $Q'_{cr}$ of the column as a whole, on the other. The curve $Q'^{e}_{cr}$ calculated for the effective cross section is closer to the experimental results than the two aforementioned quantities; however, it does not seem to follow the experimental results either. As it disregards the effect of initial irregularities, it frequently gives values higher than is the actual limiting state of the thin-walled column.

I would like to conclude by making a suggestion to the Working Comission II. I think that the above discussed problems of interaction belong to the most important lines of the present research on the behaviour of steel structures. This problem is being dealt with at several places; for ex. at Cornell, in Cambridge, in Liège, in Darmstadt and in Prague, It may, perhaps, be time to give thought whether it would not be profitable to organize a colloquium with this line of work. This colloquium, which could be organized in a way similar to last year's London colloquium on plate girders, and in which all researchers concerned would take part, could significantly contribute to further progress in the aforementioned field.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Top boundary section
Midsection
Bottom boundary section
4000 kp
8000 kp
12000 kp
16000 kp
18000 kp

Fig. 1

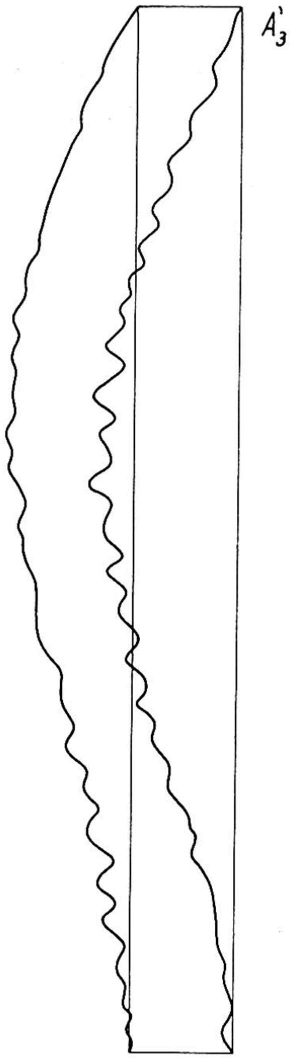


Fig. 2

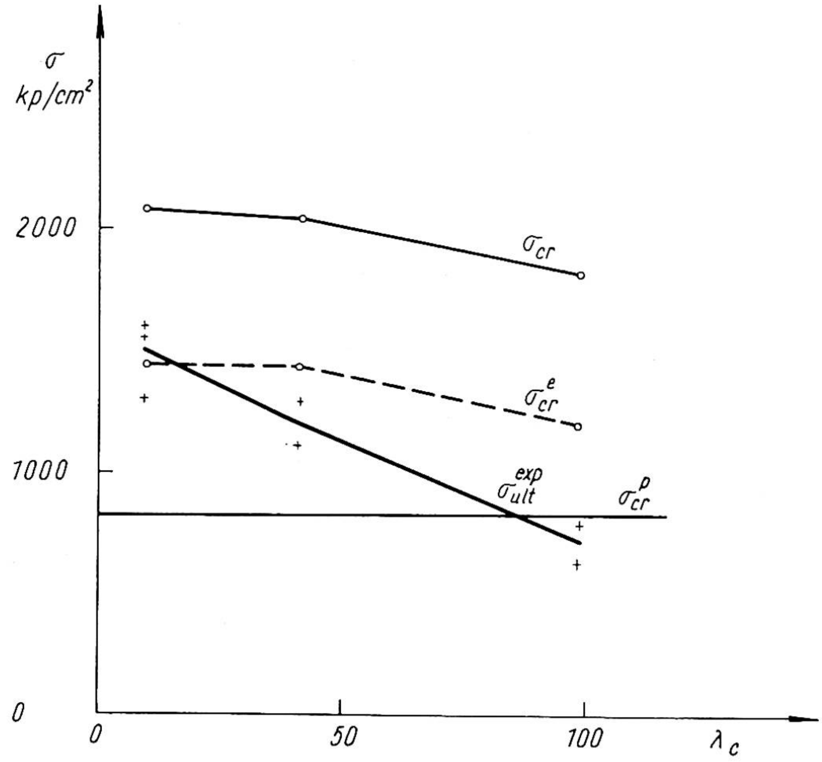


Fig. 3