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Effective width method to account for the local buckling of steel thin plates at elevated temperatures

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ABSTRACT

The local buckling of thin steel plates exposed to fire is investigated using a finite element model. The reduction of strength and stiffness that occurs at elevated temperatures needs to be taken into account in the design, as it increases the susceptibility to local buckling of the plates thus affecting their load carrying capacity. The obtained results show that the current existing design method of Eurocode 3 to take into consideration the local buckling in the calculation of the ultimate strength of steel thin plates at elevated temperatures needs to be improved. These methods are based on the same principles as for normal temperature but using for the design yield strength of steel, at elevated temperatures, the 0.2% proof strength of the steel instead of its strength at 2% total strain as for the cases where the local buckling is not limiting the ultimate strength of the plates. This consideration, however, leads to an inconsistency if cross-sections are composed simultaneous of plates susceptible and not to local buckling. To address this issue, new expressions for calculating the effective width of internal compressed elements (webs) and outstand elements (flanges) are proposed, which have been derived from the actual expressions of the Part 1.5 of the Eurocode 3 and validated against numerical results. It is also demonstrated that it is not necessary to use for the yield stress at elevated temperatures the 0.2% proof strength of the steel instead of the yield stress at 2% total strain, given that the necessary allowances are considered in these new expressions, thus leading to a more economic design.

1. Introduction

In steel construction, the engineering challenge of having the most economic structural elements demands the use of slender cross-sections. Moreover cross-sections can be considered as an assembly of plates which are often referred as internal (webs) and outstand (flange) elements and if these plates are thin, with an high width-to-thickness ratio, they may buckle when submitted to compression preventing the attainment of the yield stress in one or more parts of the cross-section, thus affecting the ultimate load bearing capacity of the structural members. According to the Eurocode 3 [1], this type of cross-sections where local buckling governs the ultimate limit state are classified as being of Class 4 and their design rules at normal temperature is well established. Under fire conditions, however, the recent investigations of Fontana and Knobloch [2], Renaud and Zhao [3] and Quiel and Garlock [4] have shown that the existing design rules are too conservative for Class 4 cross-sections and hence the need to have more realistic formulae to account for the local buckling at elevated temperatures, mainly because consistency between the rules at normal temperature and in fire situation has prevailed.

At normal temperature, the Eurocode 3 gives in its Part 1.5 [5] two methods to account for the effects of local buckling in the design, namely the effective width method and the reduced stress method. At elevated temperatures, the same concepts are used, and in the informative Annex E of the Part 1.2 of the Eurocode 3 [6] some recommendations are given for the fire design of steel members with Class 4 cross-sections. In this annex, it is suggested to use the simple calculation methods with the design value for the steel yield strength as the 0.2% proof strength instead of the strength at 2% total strain as normally done in the fire design of the other cross-sectional classes. In addition, it is stated that the effective cross-section can be determined with the effective width method as for normal temperature, i.e. the effective widths of the different elements that constitute the cross-section are determined on the basis of the material properties at normal

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temperature. The early work of Ranby [7] has demonstrated that this methodology is safe to consider, and leads to accurate results for determining the ultimate load of plates susceptible to local buckling at elevated temperatures. Nevertheless, it must be noted that, if a cross-section is built up of plates of which some are with and others are without local buckling, using the 0.2% proof strength of steel as the design strength underestimates the cross-sectional resistance. Take, for example, an element submitted to pure bending about the major-axis with a regular I-shaped cross-section with Class 1 or 2 flanges and Class 4 web, and therefore the overall cross-section classification is Class 4. Since the cross-section is Class 4, the designer is forced to use the 0.2% proof stress as the design yield strength, even for the flanges that are classified as Class 1 (or 2). This is very much conservative because in these type of cross-sections it is usual that around 80% of the bending resistance is provided by the flanges that will have no local buckling problems, in that case.

The limits of the width-to-thickness ratio from which the plates are susceptible to local buckling at normal temperature are defined in the Eurocode 3 Part 1.1. Under fire conditions these limits are the same as for normal temperature but the value $\varepsilon = 0.85\sqrt{235/f_y}$ is used instead of $\varepsilon = \sqrt{235/f_y}$. Using a reduced value of ε in fire situation can lead to a higher classification but prevents that classification changes for each temperature, as it will be justified in the next section. It worth be mentioned that on this subject, Renaud and Zhao [3] point out that would be more consistent to classify the cross-sections as for normal temperature, instead of those recommendations, since the square root of the reduction factors for the steel strength $k_{0.2p,\theta}$ and for the *Young modulus* $k_{E,\theta}$ at elevated temperatures is close to one, in accordance with the recommendation of calculating the effective cross-section to be used at elevated temperatures on the basis of the material properties at normal temperature. On the other hand, the experimental results of Ala-Outinen and Myllymäki [8] and Yang et al. [9–11], show that local buckling occurs even for cross-sections with plates that are classified as Class 1 or Class 2 for temperatures above 500 °C.

Quiel and Garlock [4] calculated the buckling strength of steel plates exposed to fire and proposed new expressions for calculating the effective widths and account for the local buckling, however, these expressions are codified in a more similar manner to North American standards than to the Eurocodes and the effect of the steel grade has not been taken into account. Fontana and Knobloch [2] have developed a strain-based approach for calculating ultimate strength of steel plates at elevated temperatures and expressions to calculate the effective cross-section, however those expressions vary for various ranges of strain at each increment of temperature and are only for outstand elements. Both works show that an improved method is needed in order to obtain a more realistic cross-sectional resistance at elevated temperatures taking into account the local buckling.

In this paper, a parametric study with the help of the finite element method (FEM) software SAFIR [12] has been performed to assess the ultimate strength of steel plates with different support conditions and load patterns at elevated temperatures. Comparisons of the numerical results with the existing formulae demonstrate the need of new expressions to determine the effective width of the steel plates at elevated temperatures which have been derived accordingly and are herein presented. The necessary allowances for the local buckling is taken into account in the determination of the effective width of the plates and as a result using these new expressions it is not necessary to use the 0.2% proof strength of steel at elevated temperatures and therefore the yield stress at a total strain of 2% can be used to calculate the resistance of Class 4 cross-sections.

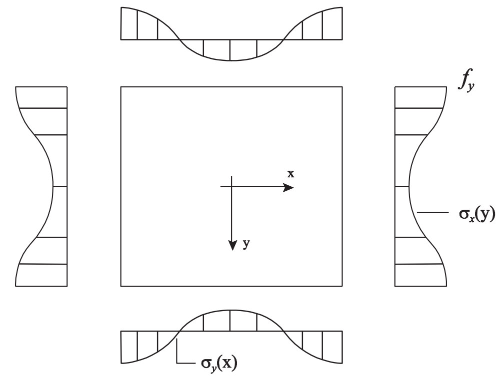

Fig. 1. Stress distribution on a rectangular plate in the post-buckling regime.

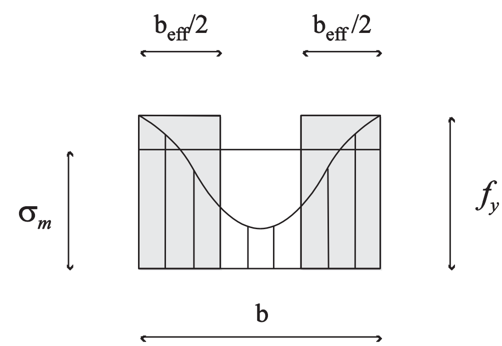

Fig. 2. Effective width b_{eff} concept.

2. Actual design provisions of Eurocode 3 to take into account local buckling

2.1. Effective width method

Thin plates when submitted to in-plane compressive stresses may buckle. The stress distribution on a plate after buckling is clearly non-linear as shown in Fig. 1, with lower values on the central part and maximum stresses at the edges of the plate equal to the yield stress f_y. The effective width method originally developed by von Karman [13] and then adapted by Winter [14] to account for the influence of the geometrical imperfections and the residual stresses, translates this concept into a “fictitious plate” with an effective width of b_{eff} and a uniform stress distribution equal to the yield stress, as shown in Fig. 2.

The effective width can then be determined as the ratio between the mean value of the stresses along the plate $\sigma_m = (1/b)\int \sigma(x)dx$ and the maximum stress on the edge of the plate f_y multiplied by the total with of the plate

$$\rho = \frac{\sigma_m}{f_y} = \frac{b_{eff}}{b} \quad (1)$$

being ρ the reduction factor for the plate buckling resistance.

According to Part 1.5 of Eurocode 3 this reduction factor is calculated by [15]

$$\begin{aligned} &\rho = 1 && \text{for } \bar{\lambda}_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ &\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^{\,2}} && \text{for } \bar{\lambda}_p > 0.5 + \sqrt{0.085 - 0.055\psi} \end{aligned} \quad (2)$$

and for outstand elements under compression by

$$\begin{aligned} &\rho = 1 && \text{for } \bar{\lambda}_p \le 0.748 \\ &\rho = \frac{\bar{\lambda}_p - 0.188}{\bar{\lambda}_p^{\,2}} && \text{for } \bar{\lambda}_p > 0.748 \end{aligned} \quad (3)$$