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Critical temperatures of class 4 cross-sections

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ABSTRACT

This paper describes an investigation on the critical temperatures of slender cross-sections according to Eurocode 3 design. Slender cross-sections are prone to local buckling, which, as the name implies, is characterized by a local failure that occurs in the presence of compressive stresses and prevents the cross-sections from developing their full plastic bending resistance or axial compression resistance, which leads to the reduction of the load bearing capacity. Eurocode 3 classifies these cross-sections as Class 4, the highest class, and suggests a default critical temperature of 350 °C irrespective of the load level conditions. This study demonstrates that this temperature is too conservative especially if different degrees of utilization of the cross-section are taken into account. The establishment of a default critical temperature is invaluable from a practical standpoint and, therefore, it is the purpose of this work to recommend and validate new default critical temperatures for Class 4 cross-sections subject to compression and bending about the major axis for different reduction factors for the design load level under fire conditions, based on an extensive numerical investigation. From this study, it is observed that the critical temperatures for such cross-sections are within the range of 400 °C and 600 °C for the usual load levels and, based on that, new default critical temperatures are suggested.

1. Introduction

Unprotected steel members tend to perform poorly when subjected to fire meaning that protection measures need to be accounted for in order to comply with national fire regulations. Eurocode 3 Part 1–2 (EN 1993-1-2) [1] provides several simplified methodologies for the stability verification of cross-sections. Annex E of the EN 1993-1-2 covers the calculation of the effective properties and the corresponding capacity of slender (Class 4) cross-sections and suggests that the stability check of members with slender cross-sections should be made using that reduced cross-sectional resistance simultaneous with the design procedures for Class 3 cross-sections.

Specifically, the stability check of Class 4 members is assessed by replacing the cross-section area with the effective area and the section modulus with the effective section modulus (both obtained at 20 °C) into the Class 3 formulae. Moreover, the norm suggests the use of the design yield strength corresponding to the 0.2% proof strength as opposed to the stress for 2% total strain used for other cross-section classes. However, studies made by Renaud and Zhao [2] have shown that the aforementioned methods are too conservative and investigation in the scope of the project FIDESC4 [3] led to more accurate and economical design methodologies for members with Class 4 cross-sections — more precisely, the development of procedures to account for the influence of local buckling and assess the capacity of Class 4 cross-sections [4,5] for the case of fire.

Furthermore, if no safety check is performed, EN 1993-1-2 suggests the use of 350 °C as the critical temperature for Class 4 cross-sections irrespective of the loading conditions and degree of utilization, which has a considerable influence on the behaviour of members subjected to elevated temperatures [6]. Despite the undeniable practical value of using a default critical temperature, it is still unclear whether the definition of a single default value for a wide range of loading scenarios is appropriate.

Recent research activities on this matter have focused mainly on the mechanical response and load-bearing capacity of members based on experimental programmes [3,7–9] or numerical investigations [4,5, 10–16]. Yet, none of the mentioned studies aimed specifically at establishing and/or validating different default critical temperatures for slender cross-sections.

In this context, the present work first establishes the boundaries for the maximum load-level for which a member is subjected for a case of fire based on the load combination methodology of the Eurocode [17]. For the sake of comparison, a description of the current simple design methodologies for the verification of Class 4 cross-sections present in Eurocode 3 as well as the one developed in the scope of the project FIDESC4 [5] are presented. Then, based on an extensive parametric study with FEM software SAFIR [18] new values for the default critical temperature of Class 4 cross-sections subject to compression and bending about the major axis are proposed.

Finally, the numerical results are compared to those obtained by the simple design procedures.

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2. Simple design methods

2.1. Load combinations and reduction factor for the case of fire

At normal temperature, during the design and verification process of a structure, different load combinations must be considered in order to account for the effect of the several actions. In case of fire, however, distinct load combinations must be taken into account for the fire limit state and specific design values of actions have to be considered. These are lower than the values for normal temperature design due to the fact that fire is a rare event and thus a higher probability of failure is allowed in that case.

The fire situation is classified as an accidental situation in EN 1990 [17]. The design effect of actions for the fire situation, $E_{fi,d,t}$, can be obtained using the combination of actions for accidental situation given by expressions (1) and (2).

$$\sum_{j\geq 1} G_{k,j} + P + A_d + \psi_{1,1} Q_{k,1} + \sum_{i\geq 2} \psi_{2,i} Q_{k,i} \tag{1}$$

$$\sum_{j\geq 1} G_{k,j} + P + A_d + \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \tag{2}$$

In these expressions, the symbol "+" means that the different loads have to be combined, as opposed to added in a mathematical sense.

It is observed that all permanent actions $G_{k,j}$ must be taken into account with their characteristic value and that, in case of fire, significantly lower imposed loads are considered relative to those at normal temperature. This results in permanent loads being relatively more important in fire than under normal conditions.

The characteristic values of variable actions are denoted by $Q_{k,i}$ in expressions (1) and (2). Additionally, the prestressing load P must be also considered if present and relevant.

Term A_d represents the indirect fire actions. These are the variations of effects of actions induced by restrained thermal expansion.

G and Q represent "external" loads applied on the structure while A_d represents "internal" effects of actions that appear in the elements because of the temperature increase, such as axial forces, shear forces and bending moments.

The difference between expressions (1) and (2) lies in the coefficient, applied to the leading variable action, $Q_{k,1}$. The coefficient for the frequent value ψ_1 is considered for the leading action in expression (1) and ψ_2 for expression (2). Table 1 gives the recommended values for the coefficients ψ_1 and ψ_2. It is a decision of Each Member State to choose which one to use in the relevant National Annex and, it is noteworthy, that the adoption of expression (1) is consistent with the accidental design for the seismic situation. Moreover, some countries have decided to take expression (2) except when the leading action is wind, in which case expression (1) should be used.

Table 1
Recommended values of coefficients ψ for buildings.

Action	ψ_1	ψ_2
Live loads in buildings, category		
• category A: domestic, residential	0.5	0.3
• category B: offices	0.5	0.3
• category C: congregation areas	0.7	0.6
• category D: shopping	0.7	0.6
• category E: storage	0.9	0.8
• category F: traffic, vehicles ≤ 30 *kN*	0.7	0.6
• category G: traffic, vehicles ≤ 160 *kN*	0.5	0.3
• category H: roofs	0.0	0.0
Snow loads		
• Finland, Iceland, Norway, Sweden	0.5	0.2
• Other countries, altitude $H > 1000$ m	0.5	0.2
• Other countries, altitude $H \leq 1000$ m	0.2	0.0
Wind loads	0.2	0.0

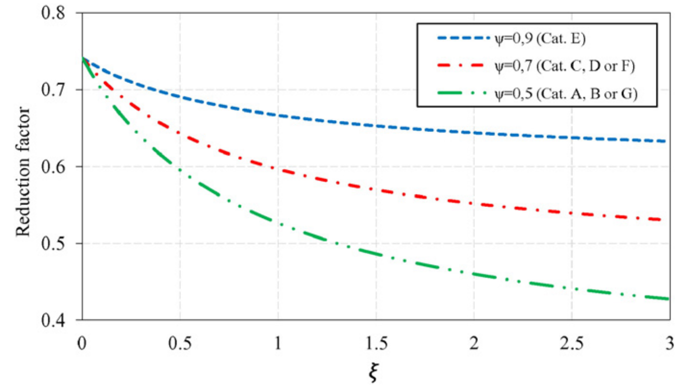

Fig. 1. Evolution of the reduction factor η_{fi}.

The design of the structure is often performed at normal temperature before the fire situation is considered. The effects of actions in the normal situation E_d have, therefore, been previously determined for the various load combinations that must be considered at normal temperature. Eurocode 1 Part 1–2 [19] allows the use of $E_{fi,d}$ (effects of actions in fire situation), which is obtained, for each load combination, from the multiplication of the effects determined at normal temperature E_d by a scalar reduction factor η_{fi}, as shown in Eq. (3).

$$E_{fi,d,t} = E_{fi,d} = \eta_{fi} E_d \tag{3}$$

According to Part 1–2 of Eurocode 3 [1], the reduction factor can be determined from Eq. (4).

$$\eta_{fi} = \frac{G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{4}$$

where

γ_G is the partial safety factor used for the permanent action at normal temperature, $\gamma_G = 1.35$;

$\gamma_{Q,1}$ is the partial safety factor used for the leading variable action at normal temperature, $\gamma_{Q,1} = 1.5$.

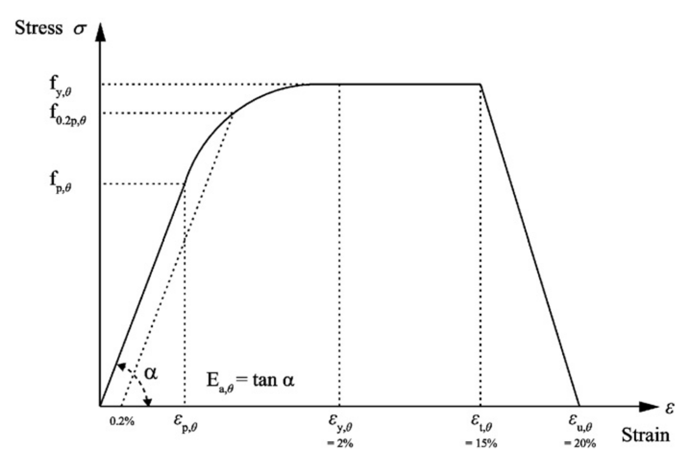

Fig. 2. Stress–strain model at elevated temperatures according to EN 1993-1-2.