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Fire exposed steel columns with a thermal gradient over the cross-section

Oscar Delgado Ojeda [a], Johan Maljaars [b,c,*], Roland Abspoel [a]

[a] Delft University of Technology, Delft, The Netherlands
[b] Eindhoven University of Technology, Delft, The Netherlands
[c] TNO, Delft, The Netherlands

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ABSTRACT

Thermal gradients often occur in fire exposed structures. This paper considers thermal gradients over the cross-section of steel columns. By means of finite element simulations, the paper demonstrates that these gradients reduce the flexural buckling resistance of the columns. This is due to the eccentricity in the column created by the temperature gradient. Design equations in modern standards provide a gross approximation of the load bearing resistance of such columns in which the eccentricity is ignored and in order to compensate for this the yield stress and modulus of elasticity are to be determined at maximum temperature. Based on an in-depth analysis of the results of the finite element simulations, this paper provides an alternative design model which much better agrees with the actual behaviour of a fire exposed column.

1. Introduction

The temperature-dependent constitutive properties of steel cause that flexural buckling of columns at elevated temperature is different from room temperature. Instead of the distinct yield plateau that characterises (mild) steel at room temperature, a curved stress–strain relationship occurs due to which the principles behind the elastic critical (Euler) buckling load no longer holds. The 2% proof stress – i.e. the stress at a plastic strain of 2% – is usually considered as an alternative to the yield stress for steel structures exposed to fire. In a large temperature range, the modulus of elasticity reduces faster than the 2% proof stress, implying that flexural buckling at elevated temperature may be more decisive as compared to room temperature. On the other hand, residual stresses caused by rolling or welding may relax as a consequence of the action of creep. A substantial number of research activities starting in the years 80' of the previous century have been devoted to flexural buckling of steel columns at elevated temperature. An extensive experimental and numerical research by Talomana et al. [1] and Franssen et al. [2] forms the basis of the current design rules in the European standard EN 1993-1-2 [3] and the design rules in the American standard AISC 360-10 [4] provide practically identical results.

The temperature over a member or structure is always non-uniform in a real fire situation. Columns with a temperature gradient along the length of the column have been considered in more recent research, amongst others [5–7]. These studies concluded that taking the maximum temperature of the column provides a slightly conservative estimate of the failure load of individual columns. Flexural buckling of columns with a temperature gradient over the cross-section, which is the subject of the current study, has been considered before in experimental studies [8,9], finite element studies [10,11], and studies based on the principle of virtual work [12,13]. Three aspects in such columns complicate the flexural buckling behaviour:

- The variation of temperature over the cross-section causes that the modulus of elasticity and the proof stress vary over the cross-section. The colder part of the section has a higher modulus of elasticity and a higher proof stress as compared to the hotter part. This makes the calculation of the (in)elastic critical buckling load and the buckling resistance more difficult.
- The higher stiffness of the colder part implies that the neutral axis shifts towards the colder part. If the load is applied in the geometric centroid of the original section, this causes a bending moment that results in an increase of stress in the hot part and a decrease in the cold part [14,15].
- The difference in thermal expansion of the hot and the cold part of the cross-section causes the column to bend towards the fire. In a pin-ended column this so-called thermal bowing can be considered as a very large geometric imperfection or as additional loading of the column by a bending moment causing an increase of stress in the colder part and a decrease of stress in the hotter part [15].

* Correspondence to: TNO, Van Mourik Broekmanweg 6, Delft, The Netherlands. Tel.: +31 88 8663464.

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The latter two aspects are acting in opposite direction. Thermal bowing increases with column length whereas the shift of the neutral axis depends on the section type. Agarwal et al. [10] have demonstrated that a slender column bends towards the hotter side whereas a stockier column bends towards the cooler side before failure. The results of the studies are ambiguous. Where references [8,10] concluded that a thermal gradient reduces the resistance of a column in comparison with the same column with uniform temperature, [12,13] concluded that the critical temperature of a column with a thermal gradient is higher than that without a thermal gradient. Thermal bowing has been considered in all studies; however, the shift of the centroid was considered in [8,10] but not in [12,13]. Although not explicitly mentioned, it seems that the uniform temperature column used for comparison was in these studies, the average temperature of the non-uniform temperature case. Moura Correia et al. [11] have demonstrated that a column with temperature gradient gives a higher critical temperature than the same column with uniform temperature, if the maximum temperature of the non-uniform case is considered for the uniform case.

Most of the previous research into flexural buckling of columns with a temperature gradient across the cross-section was devoted to columns with an abrupt jump in temperature. Agarwal et al. [10] and Moura Correia et al. [11] considered edge columns that were partially embedded in the (insulated) wall. Dwaikat et al. [8] considered insulated columns for which a part of the insulation was removed. These conditions result in very large temperature gradients and temperature jumps. The temperature jumps result in substantial internal stresses in the column in addition to the residual stresses caused by rolling or welding. While relaxation of residual stresses caused by creep is considered in the numerical studies mentioned, relaxation of the internal stresses due to temperature jumps is not. This may result in an overestimation of the effects of the unequal temperature distribution on the buckling resistance. It may also be the cause of the fact that the numerical study in [10] shows local buckling deformations at the ultimate resistance whereas the tests used to validate their model in [9] do not show these deformations.

Numerical simulations to determine the member temperature are applied more and more often for fire exposed steel structures in order to remove conservatism that is inherently present in the more simple calculation methods. Temperature gradients occur in these structures even if the column is an internal compartment column due to unequal thickness of webs and flanges and especially in case of local fires in which the column is exposed more severely from one side. These temperature gradients are less severe as compared to gradients in edge columns and abrupt temperature jumps are only marginal in magnitude or not expected at all. It is interesting to determine if existing simple design models for uniform temperature can be applied for such cases. This paper provides a numerical study into I-shaped columns with a linear temperature variation over the cross-section.

2. Finite element simulations

2.1. Cases considered

Three sets of finite element simulation of rolled I-shaped columns were carried out:

- A set at room temperature consisting of 210 analysed columns. The simulation results are compared with the design rules in EN 1993-1-1 [3] in order to validate the finite element models.
- A set at elevated temperatures with uniform temperature consisting of 600 analysed columns. The simulation results are compared with the results in Talamona et al. [1].
- A set at elevated temperatures with a thermal gradient consisting of 120 analysed columns. The simulation results are compared with the uniform temperature results and with the design models in standards.

The analysed 7 cross-sections were considered with various dimensions according to Table 1. The sections are inspired on standard American sections. Roots between web and flanges were ignored. For each cross-section different column lengths were considered, covering a practical range of column slenderness. All columns were restrained against buckling in the strong direction and were pin-ended for buckling about the weak axis. Two steel grades were considered. Grade S275 is a steel grade in between the grades S235 and S355 that were considered in [1]. In addition grade S460 has been considered in order to determine the effects of a higher yield stress on the buckling resistance.

It is well known that geometrical imperfections and residual stresses influence the buckling resistance. The residual stresses depend on the geometry of the cross-section. In order to consider the difference in residual stresses, five column design models are considered at room temperature in EN 1993-1-1 depending on the cross-section, indicted with so-called buckling curves a0, a, b, c and d. The sections considered in this study are selected in such a way that all five buckling curves are covered.

The columns were modelled with eight-node shell elements with reduced integration using the finite element programme DIANA [16]. Each flange outstand and the web were modelled with 9 nodes (4 elements) along the cross-section and the elements were approximately square. Initial geometrical imperfections were applied with a shape matching the first Euler buckling mode and an amplitude equal to $L/1000$, where L is the column length. This imperfection has been identified as a sufficiently accurate approximation for buckling resistance calculations and has been used in many other studies at room and at elevated temperatures, amongst others in [1]. The residual stresses and stress–strain curve considered are depending on temperature and are provided in the respective sections hereafter.

Table 1
Sections considered in the finite element parameter study.

Section ID	Steel grade	h^a [mm]	b^a [mm]	$t_w{}^a$ [mm]	$t_f{}^a$[mm]	$I_z{}^a$[mm^4]	A^a[mm^2]	Buckling curve
S1	S460	492.0	199.4	18	32	4.25×10^7	2.10×10^4	a0
S2	S460	152.4	88.7	4.5	7.7	8.97×10^5	2.02×10^3	a0
S3	S460	1036.3	308.5	30	54.1	2.67×10^8	6.28×10^4	a
S4	S460	241.4	213.9	18.1	30.1	4.92×10^7	1.67×10^4	a
S5	S275	492.0	199.4	18	32	4.25×10^7	2.10×10^4	b
S6	S275	474.6	424.0	47.6	77	9.82×10^8	8.42×10^4	c
S7	S275	474.6	424.0	47.6	110	1.40×10^9	1.11×10^5	d

[a] h=section height, b=section width, t_w=web thickness, t_f=flange thickness, I_z=2nd moment of area for bending about the weak axis and A=section area.