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# ANALYSIS OF STEEL STRUCTURAL MEMBERS IN FIRE WITH SLENDER CROSS-SECTIONS THROUGH BEAM FINITE ELEMENTS APPLYING AN EFFECTIVE CONSTITUTIVE LAW

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**Abstract.** *Steel profiles with slender cross-sections are characterized by their high susceptibility to instability phenomena, especially local buckling, which are intensified under fire conditions. This work presents a study on numerical modelling of the behaviour of steel structural elements in case of fire with slender cross-sections. To accurately carry out these analyses it is necessary to take into account those local instability modes, which normally is only possible with shell finite elements. However, aiming at the development of more expeditious methods, particularly important for analysing complete structures in case of fire, recent studies have proposed the use of beam finite elements considering the presence of local buckling through the implementation of a new effective steel constitutive law. The objective of this work is to develop a study to validate this methodology using the program SAFIR. Comparisons are made between the results obtained applying the referred new methodology and finite element analyses using shell elements. The studies were made to laterally restrained beams, unrestrained beams, axially compressed columns and columns subjected to bending plus compression.*

## 1 INTRODUCTION

The use in construction of steel structures with slender cross-sections has been increasing in recent years as they provide a good weight/resistance relationship. However, these structures are more susceptible to the occurrence of instability phenomena jeopardizing their stability. The existence of local buckling is due to the high slenderness of the different parts of the cross-section (web and flanges). Slender sections with high susceptibility to local buckling are classified according to the Eurocode 3 [1] as Class 4. Global instability is associated to members that are not properly restrained and can occur for example by flexural buckling or lateral-torsional buckling (LTB).

In addition, fire safety is often decisive in the design of steel structures, especially to those who are composed of thin walled sections. The reduced thickness of the profiles combined with the high thermal conductivity of steel, impose high steel temperatures when these profiles are submitted to fire, which directly affect the steel mechanical properties [1, 2]. Due to the high cost and sizes limitation of experimental fire resistance tests, in recent years, numerous studies have been conducted based on numerical simulation, especially through the finite element method (FEM) [2]. There are several programs with geometrically and materially non-linear analysis based on the FEM, as the one used in this study, SAFIR [3]. This program was developed especially for the analysis of structures subjected to fire.

The application of shell finite elements corresponds to one of the most accurate methods for the study of the behavior of structures with slender sections because they can reproduce the local buckling phenomenon. But, when they are applied, the calculating time is too high, being its use limited to small structures and isolated structural elements. In the numerical analysis of complete structures subjected to fire [4], beam finite elements are more used, however, they cannot reproduce local buckling phenomenon. Some studies have been addressing this limitation [5, 6]. The most commonly used approach for analyzing the local buckling is based on the concept of effective width [7], however applying this methodology in beam finite elements introduces some difficulties in the code formulation [5]. The studies conducted in this work are based on a recent study by Franssen and Cowez [5] who have proposed the use of an effective constitutive law to enable the study of structures with Class 4 sections subjected to fire using beam finite elements.

The main objective of this study is to evaluate the accuracy of this method in different types of structural elements. The results obtained with this methodology are compared to analysis performed using shell finite elements, which are taken as the reference values.

This work is included in the European research project FIDESC4 “Fire Design of Steel Members with Welded or Hot-rolled Class 4 Cross-section” [8, 9]. Some of the chosen case studies also correspond to benchmark cases proposed under the COST Action TU0940 - IFER “Integrated Fire Engineering and Response” [10]. Those numerical models (applying shell finite elements) were validated using experimental tests in the project FIDESC4.

## 2 EFFECTIVE STRESS-STRAIN RELATIONSHIP FOR CONSIDERING LOCAL BUCKLING IN BEAM FINITE ELEMENTS

Franssen and Cowez [5] presented a proposal for numerical modeling of steel structures subjected to fire with beam finite elements, which takes into account the local instability of structural elements with slender section through an effective constitutive law. This new approach uses the constitutive law from EN 1993-1-2 [1], and is based on the effective stresses method that has the following advantages: application of the correct value of stiffness; no need to predetermine the tensioned and compressed areas; and no need for the classification of the cross sections [5].

Because the local buckling develops only when the element is subject to compression, the stress-strain relationship is changed only with respect to compression, and remains unchanged in tension. This results in a non-symmetrical law (tension / compression), as shown in Figure 1. The effective stress-strain relationship in compression depends on the slenderness of the element, on the boundary conditions of the element (plate supported on four sides for the web, or three for the flange) and on the steel grade.

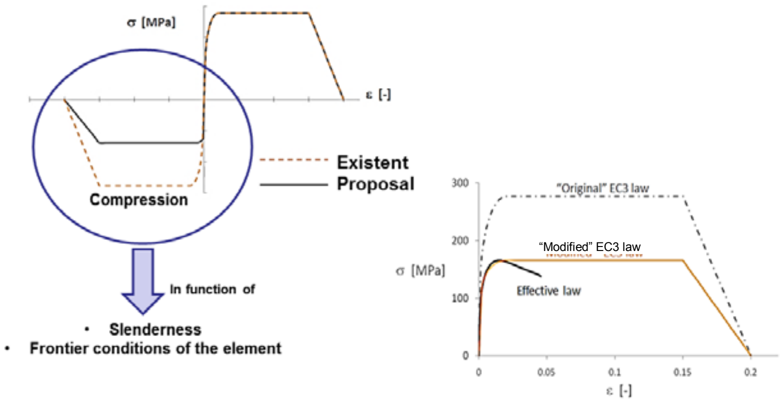


Figure 1. Effective constitutive law modifing EC3 law [5].

This effective constitutive law also depends on the temperature by the same reduction proposed in EC3 [1]. Knowing the slenderness, steel grade and support conditions of the elements, which corresponds to new material properties, the user introduces these new materials on the web and flanges (Figure 2) and the program automatically determines the amount and direction of tensile or compression stresses in each integration point [5].

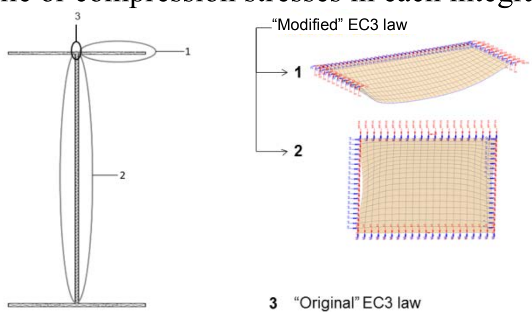


Figure 2. Materials definition: 1 - flanges; 2 - web; 3 – intersection between the flange and web.

## 3 NUMERICAL MODELS

The study cases were chosen from the following types of members [8, 9]: laterally restrained beams; unrestrained beams; axially compressed columns; and columns subjected to compression plus bending (about the strong axis). The members have different cross sections, length, loading type (bending moment diagrams) and restrained conditions (LTB restriction). The considered cross sections are I welded profiles named $h_w$x$t_w$ + bx$t_f$ ($h_w$ - height of the web, $t_w$ - web thickness, b - profile width, $t_f$ - thickness of flange). One test was made using a hot-rolled section HE340AA. The elements are of steel grade S355 at 450 °C (some tests were also performed at 500 and 650 ºC). Other members typologies were also analyzed: as for exemple tapered members designated by $h_{w,MAX}$ - $h_{w,min}$ x$t_w$ + bx$t_f$ ($h_{w,MAX}$ - maximum height of the web,

$h_{w,min}$ - minimum height of the web). These cases were chosen in order to be a reasonable representation of the common application of steel profiles with slender sections.

On the models with beam finite elements only global geometric imperfections were considered, in accordance with Annex C of Part 1-5 of EC3 [7], 80% of the geometric manufacturing tolerances described in the standard EN 1090-2 [11] were used. In the models with shell finite elements local imperfections were also considered according to the same recommendations of Part 1-5 of EC3 and EN 1090-2. Residual stresses were also introduced following the typical distributions on I welded sections [12].

## 4 RESULTS AND DISCUSSION

This section presents the obtained results using shell finite elements, beam finite elements with EC3 constitutive law and beam finite elements with the effective constitutive law.

First, the obtained results for beams subject to pure bending without LTB are presented. A case corresponding to an experimental test [8] is here detailed (first row in Table 1). This beam is subjected to two concentrated loads, being only the central span at 450 °C. Figure 3 shows the models with shell and beam finite elements and the obtained load-displacement curves.

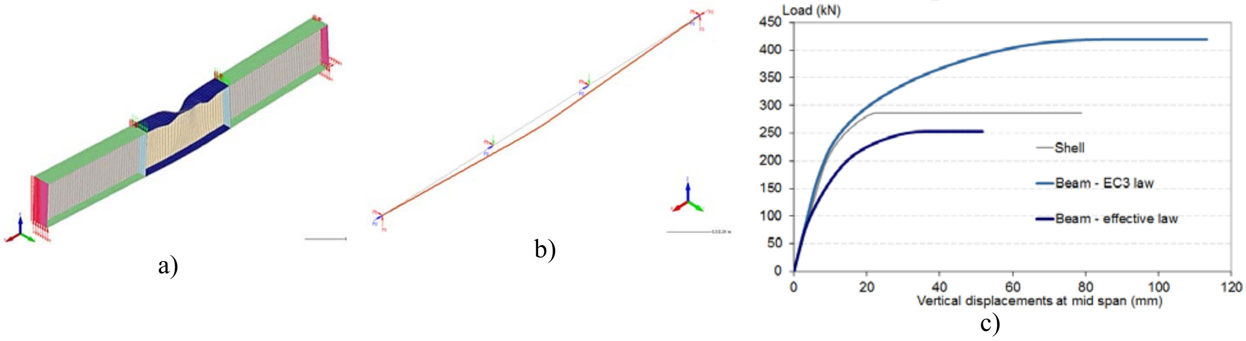


Figure 3. a) Model with Shell FE; b) Model with beams FE; c) Obtained load-displacements relations.

Table 1 summarizes all the obtained ultimate bending moments with all methodologies.

| | | Mult (kNm) | | | | |
|---|---|---|---|---|---|---|
| Sectiono | L [m] | a) Shell | b) Beam EC3 law | c) Effective law | b/a | c/a |
| 656x4+250x12 | 1.5 | 286.91 | 419.04 | 252.83 | 1.46 | 0.88 |
| 1000x14+300x22 | 10 | 2496.46 | 2936.89 | 2089.21 | 1.18 | 0.84 |
| 1000x12+300x18 | 10 | 1843.62 | 2445.86 | 1607.81 | 1.33 | 0.87 |
| 1000x12+300x14 | 10 | 1422.02 | 2074.32 | 1275.53 | 1.46 | 0.90 |
| 1000x8+300x18 | 10 | 1608.47 | 2153.67 | 1370.39 | 1.34 | 0.85 |
| 1000x6+300x13 | 10 | 943.87 | 1566.21 | 859.62 | 1.66 | 0.91 |
| 450x6+150x11 | 5 | 251.52 | 318.50 | 229.37 | 1.27 | 0.91 |
| 450x6+150x9 | 5 | 195.33 | 277.05 | 188.53 | 1.42 | 0.97 |
| 450x5+150x8 | 5 | 157.57 | 239.95 | 154.20 | 1.52 | 0.98 |
| 450x4+150x6 | 5 | 106.00 | 183.28 | 103.25 | 1.73 | 0.97 |
| 450x4+150x5 | 5 | 89.33 | 162.31 | 85.16 | 1.82 | 0.95 |

Table 1 – Obtained ultimate bending moments for restrained beams.

It can be concluded that the models with the effective law provide similar results to those obtained with the shell finite element models. As expected, the models with beam finite elements using the EC3 constitutive law provided higher results, due to the local buckling.

Table 2 presents the obtained results for laterally unrestrained beams.

| | | | Mult (kNm) | | | | |
|---|---|---|---|---|---|---|---|
| Section | L [m] | psi | a) Shell | b) Beam EC3 law | c) Effective law | b/a | c/a |
| 610-450x4+150x5 | 2.8 | 1 | 31.19 | 38.58 | 18.76 | 1.24 | 0.60 |
| 610-450x5+150x5 | 5 | 1 | 45.87 | 37.20 | 27.5 | 0.81 | 0.60 |
| 610-450x5+150x5 | 5 | 0 | 85.36 | 63.66 | 46.51 | 0.75 | 0.54 |
| 610-450x5+150x5 | 5 | -1 | 97.9 | 92.80 | 68.60 | 0.95 | 0.70 |
| 450x5+250x5 | 8 | 1 | 52.2 | 51.84 | 37.05 | 0.99 | 0.71 |
| 450x5+250x5 | 11 | 0 | 73.29 | 66.65 | 48.52 | 0.91 | 0.66 |
| 450x5+250x5 | 13 | -1 | 80.12 | 82.95 | 59.45 | 1.04 | 0.74 |
| 1000x7+300x12 | 8 | 1 | 381.42 | 378.95 | 225.00 | 0.99 | 0.59 |
| 1000x7+300x12 | 10 | 0 | 510.45 | 537.82 | 311.34 | 1.05 | 0.61 |
| 1000x7+300x12 | 12.5 | -1 | 475.51 | 595.88 | 383.50 | 1.25 | 0.81 |

Table 2 – Obtained ultimate bending moments for unrestrained beams.

The models with the effective law provide too conservative results. The models with beam finite element using the EC3 constitutive law provide results that are close to those obtained with the shell finite elements, due to the high susceptibility to LTB of these beams.

Table 3 presents the results for axially compressed columns.

| | | Nult (kN) | | | | |
|---|---|---|---|---|---|---|
| Section | L [m] | a) Shell | b) Beam EC3 law | c) Effective law | b/a | c/a |
| 500x6+250x10 | 8 | 441.61 | 419.92 | 282.27 | 0.95 | 0.64 |
| 500x4+250x6 | 6 | 328.86 | 377.75 | 191.35 | 1.15 | 0.58 |
| 500x4+250x6 | 4 | 421.26 | 575.13 | 313.82 | 1.37 | 0.74 |
| 500x4+250x12 | 6 | 677.78 | 563.07 | 392.95 | 0.83 | 0.58 |
| 500x4+250x12 | 4 | 940.59 | 945.53 | 673.88 | 1.01 | 0.72 |
| 500x10+250x6 | 6 | 405.91 | 460.07 | 381.56 | 1.13 | 0.94 |
| 500x10+250x6 | 4 | 627 | 775.56 | 528.16 | 1.24 | 0.84 |

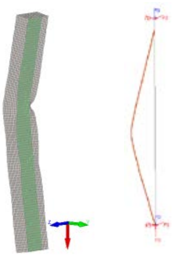

Table 3 – Obtained ultimate axial efforts for axially compressed columns.

Once again, it was observed that the models with the effective law provide too low results.

The obtained results for columns subjected to compression plus bending (about the strong axis) with and without the possibility of LTB occurrence are presented in Table 4.

| | | | | a) Shell | | b) Beam EC3 law | | c) Effective law | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Section | L [m] | psi | LTB | Nult | Mult | Nult | Mult | Nult | Mult | b/a | c/a |
| 350x4+150x5 | 2.7 | 1 | sim | 226.56 | 16.09 | 326.01 | 23.14 | 184.40 | 13.09 | 1.44 | 0.81 |
| 440-340x4+150x5 | 2.7 | 0 | sim | 227.94 | 34.42 | 314.95 | 47.56 | 157.98 | 23.85 | 1.38 | 0.69 |
| 450x4+250x6 | 10 | 1 | não | 192.66 | 62.35 | 348.89 | 113.04 | 178.94 | 57.98 | 1.81 | 0.93 |
| 450x4+250x6 | 10 | 0 | não | 238.62 | 77.22 | 454.14 | 147.14 | 225.72 | 73.13 | 1.90 | 0.95 |
| 450x4+250x6 | 10 | 1 | sim | 92.80 | 30.03 | 125.97 | 40.81 | 71.32 | 23.11 | 1.36 | 0.77 |
| 450x4+250x6 | 10 | 0 | sim | 125.72 | 40.69 | 147.04 | 47.64 | 82.09 | 26.60 | 1.17 | 0.65 |
| 450x4+250x6 | 10 | 1 | sim | 169.43 | 22.94 | 158.66 | 22.85 | 84.67 | 12.19 | 0.94 | 0.50 |
| 1000x5+300x10 | 10 | 1 | não | 486.47 | 349.98 | 1112.19 | 800.78 | 476.96 | 343.41 | 2.29 | 0.98 |
| HE340AA | 10 | 1 | sim | 312.09 | 57.48 | 385.42 | 70.15 | 317.29 | 57.75 | 1.23 | 1.02 |
| 1000-750x5+300x10 | 10 | 1 | sim | 305.49 | 165.13 | 229.43 | 124.12 | 154.04 | 83.33 | 0.75 | 0.50 |

Table 4 – Obtained ultimate bearing capacities for columns with compression plus bending.

The results obtained with the effective law are not consistent, in some cases the approximation is good but in other cases they are not satisfactory.

## 5 CONCLUSIONS

A study on the evaluation of the fire resistance of members with slender cross-sections, modeled with beam finite element with a new effective constitutive law, was presented. The obtained results were compared with those obtained through the application of shell finite elements. Laterally restrained beams, unrestrained beams, axially compressed columns and columns subjected to compression plus bending were modeled using the program SAFIR.

Reproducing the same structural element or structure with numerical models considering beam elements and shell elements, can lead to several difficulties, which may range from the reproduction of the supports and loads application to the modeling of the profile geometry (e.g. tapered members), affecting the degree of accuracy of the respective comparisons.

It can be concluded that this new methodology complies with the intended purpose for cross-section bending resistance. On the other hand, regarding the resistance of members with greater susceptibility to global buckling, the ultimate bearing capacities obtained with the effective law proposed by Franssen and Cowez [5] are much lower than the values obtained with shell finite elements. Thus, this approach can be applied to similar elements to the ones studied here, due of its conservative nature.

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