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On the use of parameter γ_z in fire situation

Sobre o uso do parâmetro γ_z em situação de incêndio



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Abstract

Herein will be presented a study on the use of parameter γ_z for reinforced concrete frames in fire situation. Currently, there are no results of similar research for concrete structures, since the subject has not received the adequate attention. In fire situation, many of the horizontal actions are no longer considered in exceptional load combination, leaving only the horizontal action due to the global geometric imperfections. Once the equivalent horizontal forces to these imperfections are obtained, the parameter is applied. One of the conclusions was that the parameter γ_z is not adequate, in particularly for high buildings, indicating that more research is necessary. Even so, using γ_z for room temperature, we conclude that fires that affect lower floors are more damaging to the building and more floors under fire also lead to worse results.

Keywords: fire, parameter γ_z, geometric non-linearity.

Resumo

A seguir será apresentado um estudo sobre o uso do parâmetro γ_z em pórticos de concreto armado em situação de incêndio. Atualmente, não se têm resultados de investigação similar em estruturas de concreto, já que o tema não tem recebido atenção adequada. Em situação de incêndio muitas das ações horizontais deixam de ser consideradas na combinação excepcional de carregamento, restando apenas a ação horizontal decorrente das imperfeições geométricas globais. Obtidas as forças horizontais equivalentes a essas imperfeições, aplica-se o parâmetro citado. Uma das conclusões foi de que o parâmetro γ_z não se mostrou adequado em especial para edifícios mais elevados, indicando a necessidade de mais pesquisa sobre esse assunto. Ainda assim, empregando-se o γ_z, concluiu-se que incêndios que afetam andares inferiores são mais prejudiciais ao edifício e mais andares atingidos também levam a piores resultados.

Palavras-chave: incêndio, parâmetro γ_z, não linearidade geométrica.

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1. Introduction

The study of structures in fire situation is a new subject when it comes to the research field and design applications. There is still much to be researched on the behaviour of the materials and structural systems.
For concrete structures, no work was found that could be related to this one. On the other hand, for steel structures, some knowledge has already been accumulated in analogous studies. Xu, Ma e Zhuang [1], for example, use a stability analysis developed in Xu [2] to understand the influence of a fire on the stability of a one-storey steel frame. Their laboratorial tests showed that fire protection on the columns is essential against generalized collapse, thus the columns are considered with fire protection, therefore, without suffering the influence of the heat transmitted by the fire. While the beams are considered without fire protection. The temperature of each beam is admitted uniform and for each one a temperature variable is attributed. This leads to an optimization problem that yields the highest and lowest scenarios of temperature, as well as the most localized and most distributed scenarios that cause instability. The fire scenario is assumed to be a single fire that spreads to adjacent compartments. The critic fire situation is that of a fire that begins at the compartment with columns that most contribute to lateral rigidity.
A stability analysis using the finite element method and with an element stiffness matrix that comprises the reduction of the flexional rigidity due to the axial force acting on the element is found in Couto et al. [3]. A load factor is applied and increased to the point when the structure reaches its limit and the axial forces acting in each column at this point are used to assess their buckling length. The global instability is so used to propose a buckling length for a local instability analysis of each column of unbraced or sway frames, which are disregarded in Eurocode 3 – Part 1-2 [4] to assess the accuracy of the values of the buckling lengths proposed it for the cases of braced or non-sway frames.
Another work analysing the behaviour of plane steel frames in fire situation is the work of Toh, Fung and Tan [5]. In it the concepts of the plasticity theory were used to consider the formation of plastic hinges in fire situation. Thereby, the structure resistance capacity is obtained when a mechanism is formed, what can also be considered loss of stability.
At last, it is worth mentioning the work of Tan e Yuan [6], where isolated steel columns stabilities are studied. In this work, springs at the ends of the columns are used to simulate the restrictions imposed by the non-heated adjacent structure. Furthermore, they are subjected to non-uniform temperature distributions along its height to achieve less conservative values for the critic load than that obtained for a uniform distribution. An analytic solution is obtained and used to demonstrate the considerable influence of the constitutive model and the temperature distribution along the height of the column in the critic external load. The results showed good agreement with experimental ones, giving credit to the achieved equations as an aid tool to engineers that need to verify the columns stability under fire without the need of any computationally expensive numerical models.
In the present study, the aim is to analyse the viability of the use of the parameter γ_z, which permits to verify whether a global geometric nonlinear analysis is needed or not for a concrete multi-storey building, in fire situation.
The parameter γ_z was proposed in the work of Franco e Vasconcelos [7], according to equation (1).

$$\gamma_z = \frac{1}{1-\left(\frac{\Delta M_{tot,d}}{M_{1,tot,d}}\right)} \tag{1}$$

Where $M_{1,tot,d}$ is the sum of all the contributions of the horizontal forces times its respective distance of each one in relation to the building base and $\Delta M_{tot,d}$ is the sum of all the vertical design loads, times the horizontal displacement of its respective point of application got from the first order analysis.
Parameter γ_z as defined in ABNT NBR 6118:2014 [8] is an exclusivity of the Brazilian code. As already observed, no similar work was found in the literature. Thus, this work constitutes an

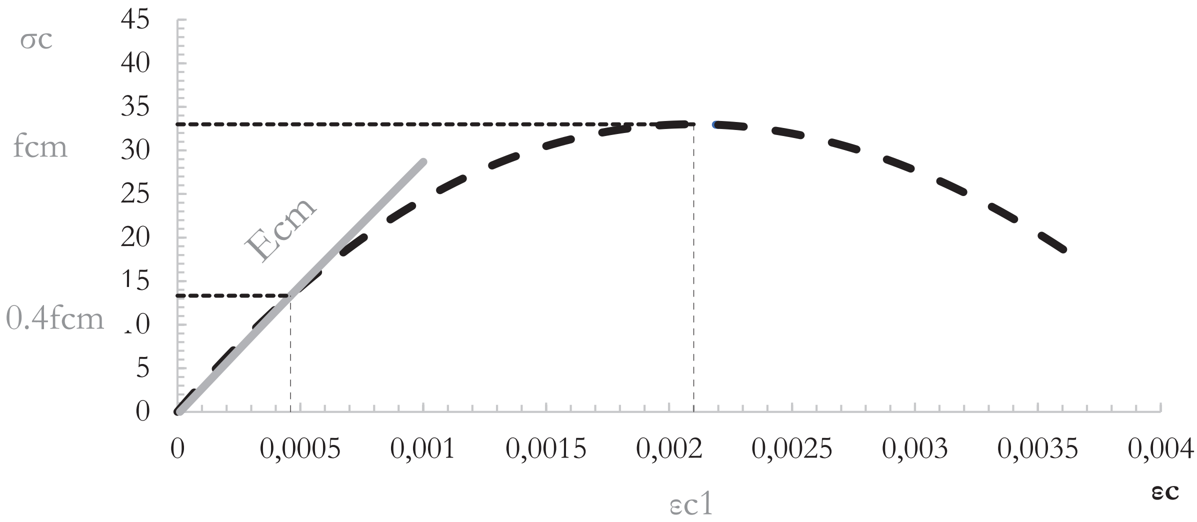

Figure 1
Stress-strain relation for structural analysis at ambient temperature. Eurocode 2 – Part 1-1 [6]

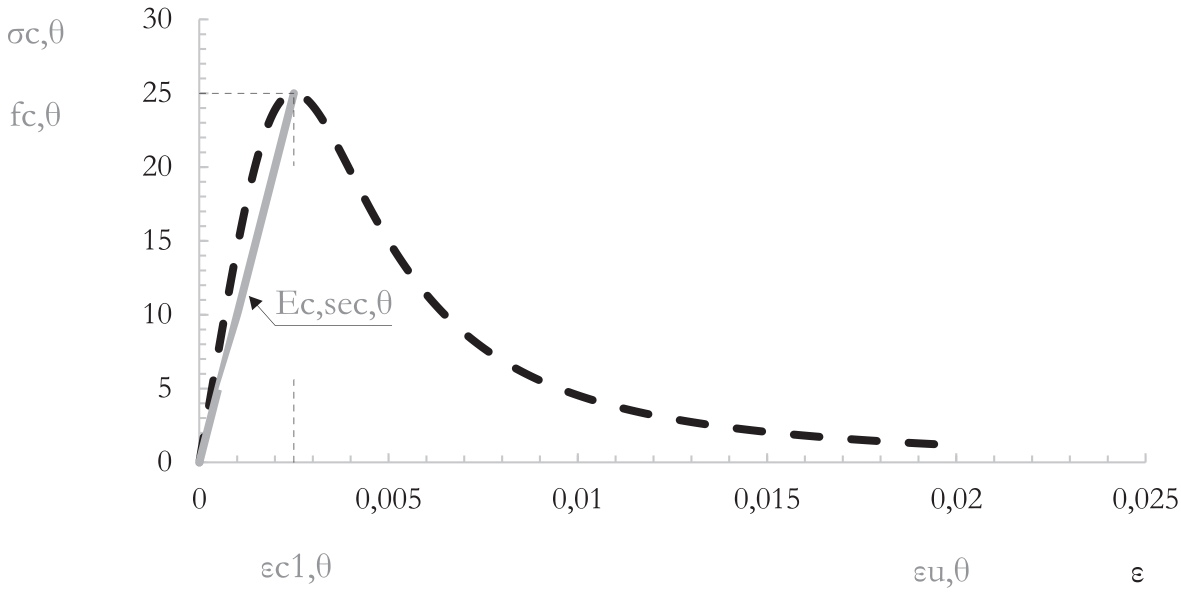

Figure 2
Stress-strain relation for structural analysis at ambient temperature. Eurocode 2 – Part 1-1 [6]

inedited approach, using known methods, but it may pave the way for new methodologies.

With the temperature increase of the structural elements during the time elapsed in fire, the modulus of elasticity of the concrete decays, increasing the horizontal displacement and, thence, increasing the γ_z. This behaviour will be analysed in this work.

For such an end, the structural analysis was carried out with the help of a software developed the first author, which only consider the geometric and material linearities [9]. Seen that the relation stress-strain has only a linear stretch defined by the chosen elastic modulus, it becomes the most important structural parameter of the analysis.

2. Elastic modulus in fire situation

In fire situation, the elastic modulus has its value reduced as the fire develops and alters the temperature of the ambient and, therefore, the temperature of the structure. A first attempt to describe these alterations can be deduced from Eurocode 4 – Part 1-2 [10], assuming that equation (2), only stated in this Eurocode is also applicable to other structures than the steel concrete composite ones, subject of Eurocode 4.

$$\boldsymbol{E_{c,sec,\theta}} = \frac{\boldsymbol{f_{c,\theta}}}{\boldsymbol{\varepsilon_{c1,\theta}}} \quad (2)$$

In equation (2), $f_{c,\theta}$ is the stress peak achieved in fire situation at θ and $\varepsilon_{c1,\theta}$ is the corresponding strain to this stress peak.

Proceeding this way, however, it arrives in an inconsistency such that the secant modulus of elasticity at 20 ºC, $E_{c,sec,20}$, calculated by Eurocode 2 -Part 1-2 [11] does not have the same value as the secant modulus at ambient temperature $E_{c,m}$, calculated by the Eurocode 2-Part 1-1 [12]. This difference is illustrated in Figure 1 and Figure 2 for a concrete with f_{ck} = 25 MPa and is based on a stress-strain relation for a structural analysis at ambient temperature and in fire situation in accordance with Eurocode 2-Part 1-1 [12] and Eurocode 2 Part 1-2 [11], respectively.

The strain at peak stress and at ambient temperature, ε_{c1}, follows equation (3).

$$\boldsymbol{\varepsilon_{c1}} = \boldsymbol{0,7(f_{ck}+8)^{0,31}} \leq \boldsymbol{0,28\%} \quad (3)$$

The secant modulus, also at ambient temperature, is determined by equation (4).

$$\boldsymbol{E_{cm}} = \boldsymbol{2200\left(\frac{f_{cm}}{10}\right)^{0,3}} \quad (4)$$

Where f_{cm}, is the average compression stress strength of the concrete. As can be observed in Figure 2, in fire situation, the secant modulus $E_{c,sec,\theta}$ is estimated for a working stress equal to the characteristic compression strength (at temperature θ) of the concrete, while at ambient temperature, one observes in Figure 1 that the secant modulus E_{cm} is estimated for a working stress equal to forty percent of the average compression strength f_{cm}, which leads to higher values. Thus, if the concrete is going to endure the heat of a fire, in accordance with Eurocode, its elastic modulus begins at lower values of what would begin in case of non-occurrence of the fire, even that the initial temperature of the fire model is the same as that considered for normal conditions.

It must be highlighted that in Eurocode 2 Part 1-2, where the theme of fire in concrete structures is treated, the elastic modulus is symbolized by $E_{c,sec,\theta}$, and in Eurocode 2 Part 1-1, which deals with the general dimensioning rules of concrete structures at ambient temperature, the elastic modulus is symbolized by E_{cm}. What is intended to do here is to show that when the temperature of 20 ºC is considered, that is, the ambient temperature of normal conditions, one would have $E_{c,sec,\theta} = E_{cm}$, which is not what happens.

The secant modulus $E_{c,sec,\theta}$ represented in Figure 2 is the average

Table 1
Flexional rigidity adjustment coefficients (Eurocode 4 – Part 1-2 [4])

Fire resistance	$\varphi_{s,\theta}$	$\varphi_{c,\theta}$
R30	1.0	0.8
R60	0.8	0.8
R90	0.9	0.8
R120	1.0	0.8

value of the tangent of the stress-strain curve inside the interval [0;ε_{c1}], which is equivalent to the indicated in equation (5).

$$E_{c,sec,\theta} = \frac{1}{\varepsilon_{c1}} \int_0^{\varepsilon_{c1}} \frac{d\sigma}{d\varepsilon} d\varepsilon \tag{5}$$

Where $f_{c,\theta}$ is given by equation (6).

$$f_{c,\theta} = k_{c,\theta} f_{ck} \tag{6}$$

The factor $k_{c,\theta}$ and the strain at peak stress ε_{c1} in fire situation depends on the kind of concrete aggregate which can be of siliceous or calcareous material and are presented in Eurocode 2 Part 1-2 [11]. Furthermore, it will be considered here an effective flexional rigidity $(EI)_{fl,eff}$ for the columns and beams in accordance with the fifth paragraph of the item 4.3.5.1. of Eurocode 4 Part 1-2 [4], here reproduced in equation (7).

$$(EI)_{fl,eff} = \varphi_{s,\theta} E_{s,\theta} I_{s,\theta} + \varphi_{c,\theta} E_{c,sec,\theta} I_{c,\theta} \tag{7}$$

The index “s” is relative to “steel” representing the steel bars of the section of reinforced concrete and which will not be considered. The index “c” is relative to concrete. The terms $\varphi_{s,\theta}$ and $\varphi_{c,\theta}$ are reduction coefficients due to stresses of thermic origin at temperature θ and provided in Table 1. The phenomena known as “spalling” will not be considered, thus, there is no losses in the concrete section $I_{c,\theta}) = I_c$. By the third column of Table 1 one sees that $\varphi_{c,\theta}$ is always equal to 0,8. With these considerations, the effective flexional rigidity becomes that of equation (8).

Table 2
Concrete elastic modulus reduction coefficient (AISC 360-16 [7])

Temp. (°C)	$k_{Ec,\theta}$
20	1
93	0.93
200	0.75
290	0.61
320	0.57
430	0.38
540	0.2
650	0.092
760	0.073
870	0.055
980	0.036
1100	0.018
1200	0

$$(EI)_{fl,eff} = 0,8 E_{c,sec,\theta} I_c \tag{8}$$

The reduction coefficient of the elastic modulus can be obtained by dividing $E_{c,sec,\theta}$ by its value at 20 °C as done in equation (9).

$$k_{c,\theta} = \frac{E_{c,sec,\theta}}{E_{c,sec,20°C}} \tag{9}$$

An alternative to Eurocode 2 is the American code AISC 360-16 [13], which provides direct recommendations for the reduction coefficient of the elastic modulus of the concrete. Its values are reproduced in Table 2.
The elastic modulus of the concrete at ambient temperature proposed by the AISC 360-16 [13], for which the reduction coefficient of Table 2 applies is given by the equation (10).

$$E_c = 5375\sqrt{f_{ck}} \tag{10}$$

It is important to note that no distinction is made between the tangent modulus and the secant modulus in the AISC 360-16 [13], even because this code focus is on steel structures. Noting too that (10) is similar to the equation for the tangent modulus proposed in ABNT NBR 6118:2014 [8], one chose to use the expressions found in the Brazilian code, being equation (11) for the tangent modulus and equation (12) for the secant modulus.

$$E_c = 5600\sqrt{f_{ck}} \tag{11}$$

$$E_{cs} = 0,85 E_c \tag{12}$$

Thus, following the recommendations of the AISC in association with equations (11) and (12) of the ABNT NBR 6118, the secant modulus for a temperature θ can be written as in equation (13).

$$E_{c,sec,\theta} = k_{E,\theta} E_{cs} \tag{13}$$

Throughout this paper, will be employed in the analysis, the moduli of elasticity and reducers, respectively, from the ABNT NBR 6118:2014 [8] and AISC 360-16 [13] and from the Eurocode 4 – Part 1-2 [10] and Eurocode 2-Part 1-1 [12].

3. Geometric imperfections

In fire situation, the structure is subjected to an exceptional combination of actions. The probability of strong winds to occur at the same time of a flashover is very low, therefore, the design value of the forces due to the wind is zero (ABNT NBR 8681:2003 [14]) and the only horizontal force that must be considered is the global geometric imperfection, which are always acting on the structure. In accordance with Eurocode 2 – Part 1-1 [12], the global imperfection can be represented by an inclination θ_1 given by equation (14).

$$\theta_1 = \theta_0 \alpha_h \alpha_m \tag{14}$$

Where α_h, is a reduction factor associated to the total height of the structure H and determined by equation (15).

$$\alpha_h = \frac{2}{\sqrt{H}} \tag{15}$$

And α_m is a reduction factor associated to the number of elements m along the height H and calculated as in equation (16).

$$\alpha_m = \sqrt{\frac{1 + 1/m}{2}} \tag{16}$$

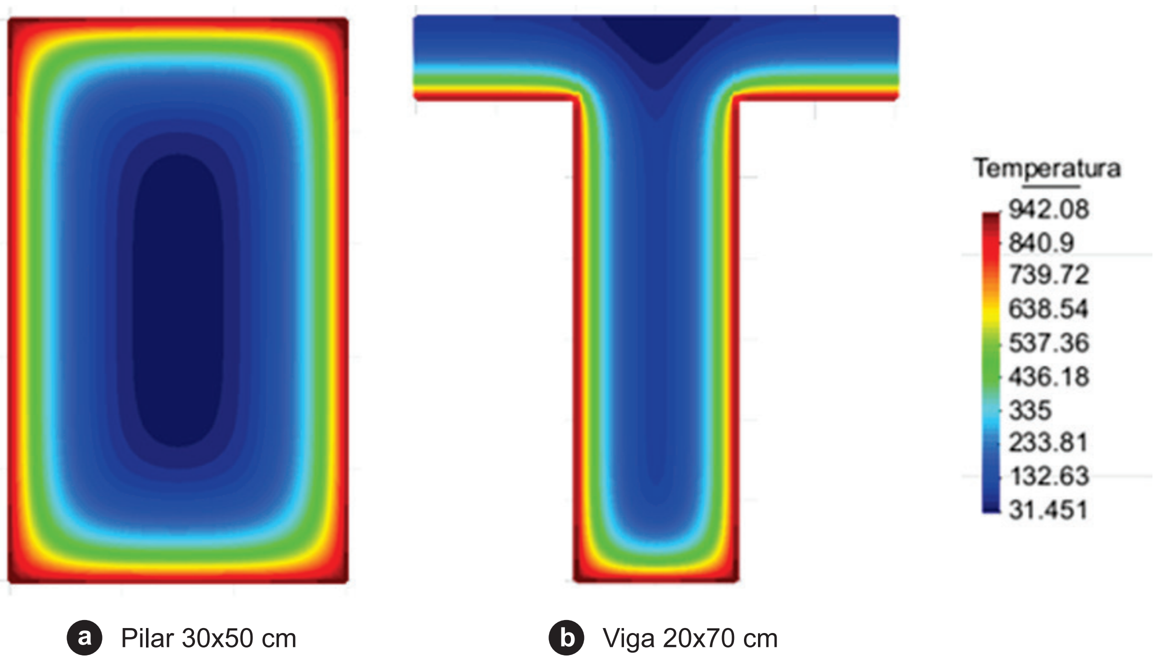

Figure 3
Examples of thermal fields determined for this work

Finally, θ_0 is a reference value to be corrected by the factors above and corresponds to 1/200. The equivalent horizontal forces are obtained from the inclination defined above and since they are known, the stability parameters can be calculated.

4. Load combinations

The applied load to the structure in fire situation is based on the exceptional combination load. This combination is called exceptional combination because it includes an action of low probability of occurrence and of short duration, as established in ABNT NBR 8681:2003 [14]. In this combination, many variable actions can be omitted or reduced by a combination factor, in accordance with equation (17).

$$F_d = \sum_{i=1}^{m} \gamma_{gi,fi} F_{gi,k} + \gamma_{q,fl} F_{q1,exc} + \sum_{j=1}^{n} \psi_{2j} \gamma_{qj,fi} F_{qj,k} \tag{17}$$

In equation (17), the combination factor ψ_2 is the combination factor for quasi permanent actions and substitutes the factor ψ_0, applied for normal combinations. As $\psi_2 < \psi_0$, this substitution reduces the design value of the variable actions, for in an exceptional situation, the group of actions has even smaller probability of occurrence than in normal situation, as already observed. In addition, it is permitted, in accordance with the ABNT NBR 8681:2003 [14], to reduce ψ_2 to $0{,}7\psi_2$.

The exceptional action $F_{q1,exc}$ can lead to forces caused by the heating of elements which are not free to expand. The values of such forces are not known very well. According with the ABNT NBR 15200:2012 [15], in general, they can be disregarded. In this work they will be.

The permanent actions $F_{gi,k}$ can be of high or low variability, which affects its respective weighting coefficients in fire situation $\gamma_{gi,fi}$ and, at last, the non-exceptional variable actions $F_{qi,k}$ that remain acting on the structure, with relevant probability, during a fire have its respective weighting coefficients all equal to 1,0, thereby, equation (17) remains valid if rewritten as in equation (18).

$$F_d = \sum_{i=1}^{m} \gamma_{gi,fi} F_{gi,k} + 0{,}7 \sum_{j=1}^{n} \psi_{2j} F_{qj,k} \tag{18}$$

5. Examples

Given the expression that relates the temperature of a concrete element with its elastic modulus at this temperature, the next step is to discover the temperature field in the element. Adopting the standard curve of fire temperature vs time ISO 834 [16], it was possible to determine the thermic field in the cross section of the columns and beams with the aid of the computer software for thermal analysis of structures by the finite element method Aterm [17], developed and validated by the second author of this paper. From the thermal field, the average temperature in each section was determined and this value was extended to the entire bar (column or beam), that is, a uniform temperature distribution along the bars was adopted herein.

The concrete used is composed of siliceous aggregate, which is the most commonly used in buildings.

The vertical loads applied to the building are those suggested by the code ABNT NBR 6120:1988 [18] when residential buildings are considered. On all the beams, the existence of brick masonry with self-weight of 18 kN/m³ was considered.

All the horizontal forces applied in the examples come from the geometric imperfections defined in item 3.

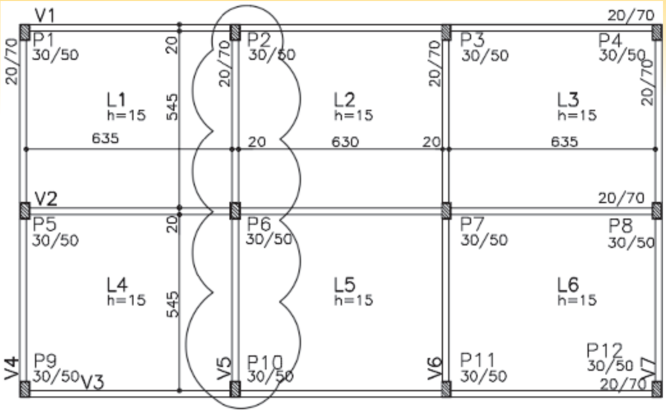

Figure 4
Nine-storey two bays frame

5.1 Nine-storey building

The first example is a nine-storey building. Its total height reaches twenty-five meters and twenty centimetres, considering a constant height for each storey. All the beams have rectangular cross sections with twenty centimetres in width and seventy centimetre in height. All the columns have rectangular cross sections with thirteen centimetres in width and fifty centimetres in length. The elastic modulus at ambient temperature is that which corresponds to a concrete with characteristic resistance of 40 MPa and applying equation (11). The plane frame analysed is that constituted by columns P2, P6 e P10, as shown in Figure 4. The parameter γ_z obtained at ambient temperature for the bidimensional structure, is very close to that obtained in case when a three-dimensional frame is considered, in accordance with the analysis done with the aid of the computer commercial program CAD\TQS (Table 3).
A first analysis at ambient temperature provides γ_z = 1,044, what is lower than the limit of 1,1. It indicates that the structure can be considered with fixed nodes and, therefore, the non-linearities of the materials do not need to be regarded in the structural analysis. However, due to the fire action, this limit will probably be exceeded during an analysis that lasts 180 minutes and considers the most critic fire scenario. Then, the non-linearity of the materials was considered since low temperatures and in a simplified way, by means of a reduction of 50% in the beam's rigidity and with a reduction of 20% in the rigidity of the columns [8].
In Figure 5, was illustrated the standard fire curve ISO 834 [16]. This curve was used to obtain the evolution of the average temperatures in the concrete beams and columns of the structure, whose resulting curves are represented together.
In a fire scenario where an ignition followed by a flashover hits only one storey, it is possible to show that the worst case happens when the first storey of the building is hit (Figure 6).
In Figure 7, diagrams γ_z vs time is presented, applying the recommendations of the AISC 360-16 [13] and in Figure 8, Eurocode 4 part 1-2 [10], for the different elastic modulus in fire situation.
When a fire scenario with more than one storey in full combustion, the consequences over the frame displaceability are more severe as illustrated in Figure 7 and in Figure 8.
By the present legislation, the vertical compartmentation is required for tall buildings, in order to prevent the vertical propagation of the fire [19]. As one sees in Figure 7 and Figure 8, more than

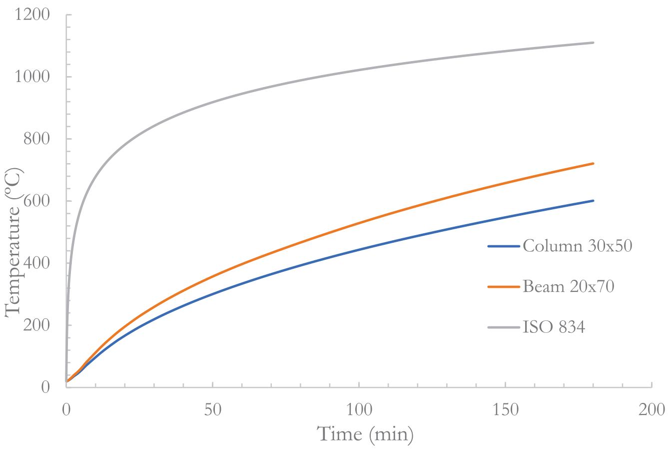

Figure 5
Mean temperatures of beam and columns sections of the first example during the fire

Table 3
Comparison of γ_z values with CAD\TQS

f_{ck} (MPa)	TQS	PORTICO2D
25	1.058	1.05676902
30	1.052	1.05156772
40	1.045	1.04435254

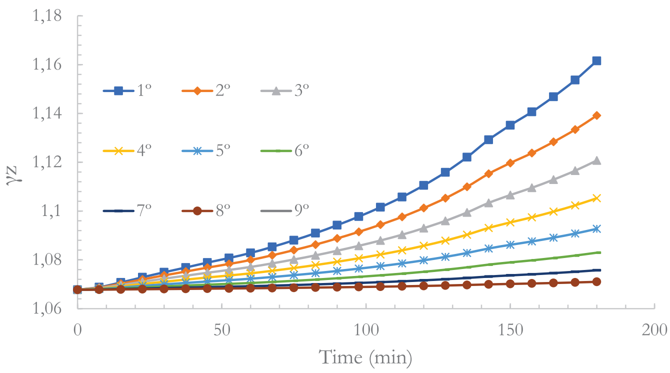

Figure 6
Parameter γ_z for different and vertically compartmentalized storeys under fire ($E_{c,sec,\theta}$) in accordance with AISC 360-16[7] and ABNT NBR 6118:2014 [2])

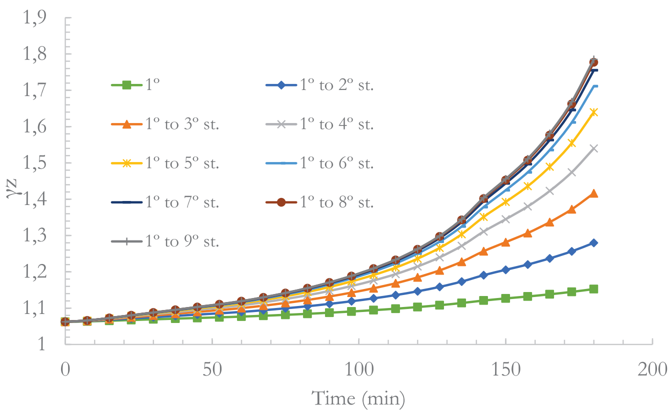

Figure 7
Parameter γ_z ($E_{c,sec,\theta}$) in accordance with AISC 360-16[7] and ABNT NBR 6118:2014 [2])

one storey in fire increase the building displaceability, providing one more argument for this requirement.

Observing Figure 7, the structure becomes very displaceable more rapidly with the use of the elastic modulus of the Eurocode, as one can conclude after comparing it with Figure 8. What exposes again the necessity of more definitive studies for the elastic modulus in fire situation.

Frames in the orthogonal direction to that analysed here, can also be studied, but the results would be very alike. Therefore, the authors made an option for omission.

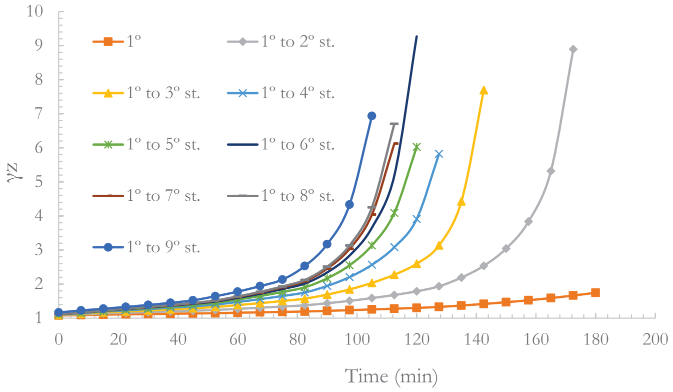

Figure 8
Parameter γ_z ($E_{c,sec,\theta}$) in accordance with Eurocode 2 Part 1-1[6] and Eurocode 2 Part 1-2[5])

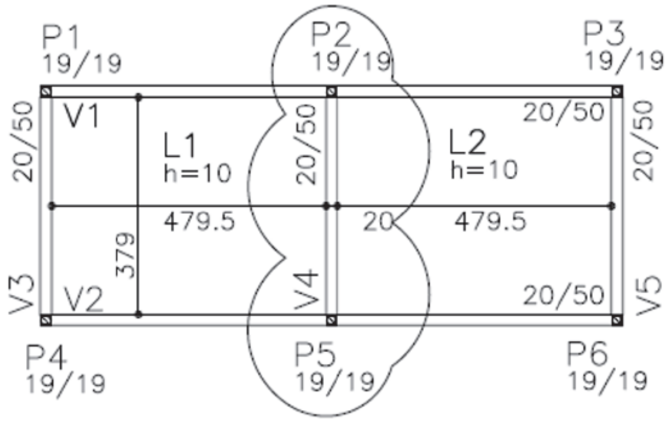

Figure 9
Four-storey one bay frame

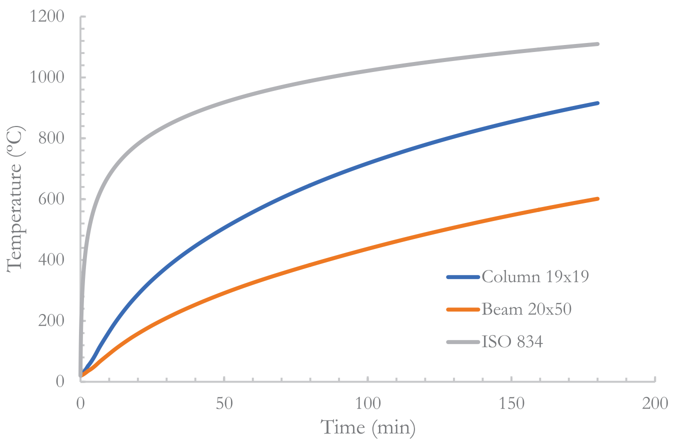

Figure 10
Mean temperatures of beam and columns sections of example 2 during fire

5.2 Four-storey building

The second example is of a small four-storey building. The total height is eleven meters and sixty centimetres. All the beams have rectangular sections with twenty centimetres in width and fifty centimetres in height. All the columns have rectangular cross sections with nineteen centimetres in width and in length. The elastic modulus at ambient temperature is that which corresponds to a concrete with characteristic resistance of 30 MPa and applying equation (11). The plane frame analysed is that constituted by the columns P2 and P5, as shown in Figure 9.

The considerations applied to this example are the same as for the former one.

As in Figure 5, in Figure 10 was illustrated the standard fire curve ISO 834 [16]. This curve was used to obtain the evolution of the average temperatures in the concrete beams and columns of the structure, whose resulting curves are represented together.

In Figure 11 the diagrams γ_z vs time is presented, applying the recommendations of AISC 360-16 [13] and in Figure 12, Eurocode 4 part 1-2 [10], for the different elastic moduli in fire situation.

With the element's temperature rising during the time elapsed on fire, the concrete elastic modulus has its value reduced, increasing the horizontal displacement and, therefore, increasing γ_z. This growth, as seen in Figure 11, is high, even for a fire in only one storey and, in Figure 12, up to the vertical asymptote. However, more studies are necessary on the determination of γ_z in buildings with abrupt rigidity variation between storeys, as is the case between

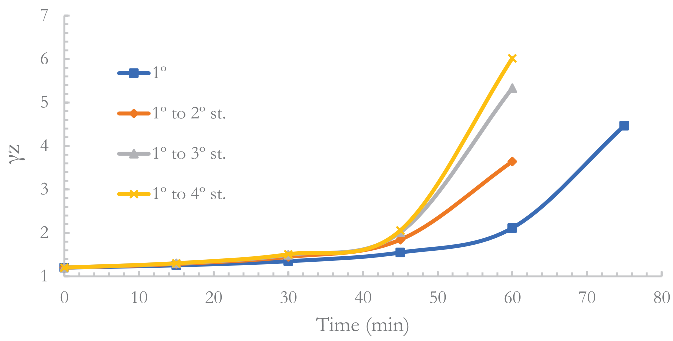

Figure 11
Parameter γ_z ($E_{c,sec,\theta}$) in accordance with AISC 360-16[7] and ABNT NBR 6118:2014 [2])

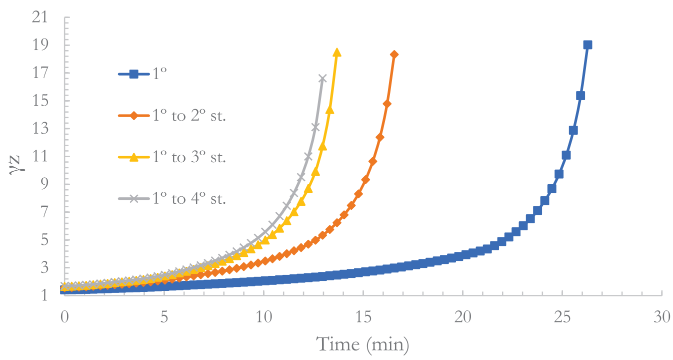

Figure 12
Parameter γ_z ($E_{c,sec,\theta}$) in accordance with Eurocode 2 Part 1-1[6] and Eurocode 2 Part 1-2[5])

storeys in fire and in ambient temperature, this is an initial study on the matter. The intense change in the inclination of curve γ_z vs time is a behaviour due to the use of a linearized equation, with its denominator nearing zero. It can be seen as an approximated answer of the instant when the loss of stability occurs.
The parameter γ_z is used too in the assessment of the global nonlinear forces of concrete buildings increasing the wind forces by 0,95 γ_z, but in accordance with the Brazilian code of concrete structures ABNT NBR 6118:2014 [8], it only can be used for this end when inside the interval $1,1 \leq \gamma_z \leq 1,3$. These limits exist because when $\gamma_z < 1,1$ the nonlinear global forces become irrelevant and when $\gamma_z > 1,3$, the estimative of these forces with the γ_z loses its precision. In relation to the structural analysis in fire situation, these limits are unlikely to be kept, which requires further studies.

6. Conclusions

The objective of this work was providing a first assessment about the use of parameter γ_z to deal with the global analysis of the concrete structures problem, considering its displaceability and, consequently, the need of a nonlinear geometric analysis.
The concrete elasticity modulus values and its reduction with the temperature in a fire is fundamental, but there is little knowledge available to achieve a reliable solution. The elastic modulus derived from Eurocodes seems reasonable, but it is not clear the extension of its applicability and, moreover, there is a difference between its value at ambient temperature by Eurocode 2 Part 1-1 [12] and its initial value, at 20 ºC, in the formulation for fire situation by Eurocode 4 Part 1-2 [10].
The equation of γ_z, in accordance with ABNT NBR 6118:2014 [8], does not have defined limits of application, however, using it for fire situation, one concludes that it is not satisfactory. However, some achieved results, in a qualitative point of view, are acceptable. It was shown that the lower the storey level hit by the fire, the larger will be the horizontal displacement of a building. It was shown too that as more storeys are simultaneously hit by the fire, worse will be the structure conditions, reinforcing the importance of the vertical compartmentation.
More realistic fire scenarios must be used for the analysis of bigger structures or in case of employing a three-dimensional frame, that because a real fire is not uniform throughout a storey neither throughout the building height. A real fire walks through the building, following the more inflammable materials and with that forming a highly non-uniform temperature field [6,20].
It is hoped that this work inspires other authors to follow the path started here, as the global analysis of concrete buildings in fire situation is of great importance to the research sequence of the structures in this area.

7. Acknowledgements

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