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CRITICAL TEMPERATURE EVALUATION FOR A STEEL FRAME WITH JOINT STIFFNESS DECREASING IN FIRE

A procedure to determine the critical temperature of a selected steel frame bearing structure is presented and discussed. This temperature, in case of fully developed fire, when the temperature of the exhaust gasses enveloping the structural members is equalized within the whole fire zone, may be considered as an impartial measure of safety. The obtained result does not depend on the heating progress but only on the static scheme and the load level in the considered structure. The quantitative and qualitative evaluation of the influence the joint stiffness decreasing in fire exerts on the resultant critical temperature constitutes the basic objective of the authors. It has been shown, that proceeding according to the recommended computational procedure does not necessarily result in an estimate fully unambiguous in interpretation. The critical temperature specified in a global mode, for the whole considered frame, is usually associated with a specific component of such frame, interpreted as the so called “weakest link”. Thus local loss of bearing capacity in such element is in this approach equivalent to the total destruction of the whole bearing structure. Indication, which of the components present in the considered frame should be treated as the critical one, because of its behaviour under fire conditions, seems to be a key to the forecast safety level warranted to the users of the structure. The authors show, that this association changes depending on the selected computational method, and this in turn substantially limits the reliability of the obtained estimate.

Keywords: steel bearing frame, fire, critical temperature, joint stiffness, moment – rotation relationship, weakest link

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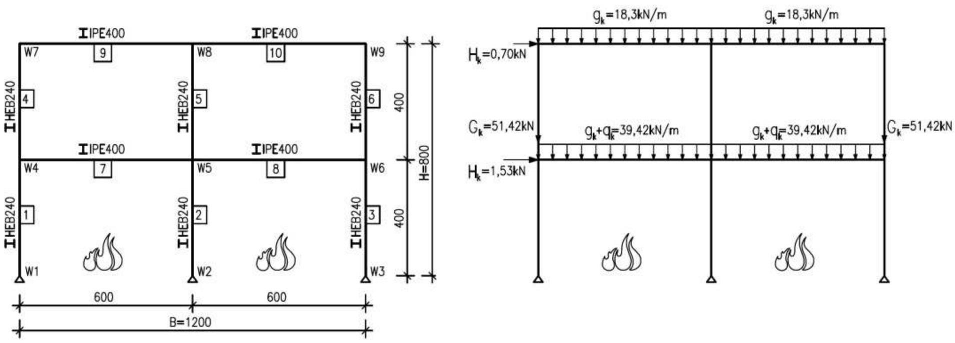
1. Introduction

In the classical computational approach the time during which in fire conditions, and counting from the moment of initiation to fire exposure, the structure is capable of safely resisting the loads applied to it, including the internal forces induced in the structure by the restrained capability to freely yield to thermally induced deformations, is treated as the measure of fire resistance. Unfortunately, a measure of this type can hardly be treated as an impartial measure of safety. Its value determined for given frame will change with changed fire scenario, this means, that it may not be uniquely assigned to considered structure and be interpreted as one of its characteristics. Thus the authors of this paper recommend for use in its place the critical temperature calculated globally for the whole bearing structure and associated with this structure reaching the ultimate limit state in fire. However, this temperature will be reached locally in fire, in the weakest part of the structure, treated as the "weakest link". Unequivocal indication of such "weakest link" in the analyzed frame, when subjected to the forecast fire action, constitutes the basic task of an expert performing fire safety appraisal for the users of the considered building. The qualitative and quantitative verification of this influence, taking into account decreasing joint rigidity in fire exerts on the estimated critical temperature of the selected steel frame, is the basic purpose of this paper. The authors intend to show, that in the specific case selection of a single critical structural element, authoritative for the specification of critical temperature depends on the selected calculation method, and this in turn significantly undermines credibility of the obtained estimate.

2. Description of the frame analyzed in this case

Let us consider in detail a two-storey two-aisle steel sway frame having the dimensions and static scheme as depicted in Fig. 1a. All the structural components of this frame are made of the low carbon steel S235, with HEB 240 wide- flange I sections used for columns and IPE 400 I sections used for beams. These sections have been selected so, as to in the persistent design situation, excluding the influence of a fire, assure the safe bearing of applied loads. Both the ultimate and serviceability limit states of the considered structure have been checked. The distributions of dead and selected live loads (applied to the floors of intermediate level) are depicted in Fig. 1b. The dead weight of structural members has been accounted for automatically in the computer program. The equivalent horizontal forces modeling the influence of global sway imperfection having the magnitude prescribed by the code (without amplification) are depicted in the same figure [1].



a) b)

Fig. 1. Frame considered here, including: a) dimensions, static scheme and sections used, b) distribution of dead and selected live loads (combination with live load applied to both floors at the intermediate level – i.e. combination 20, authoritative for consideration of exceptional design scenario – see Table 1) and horizontal equivalent forces modeling the influence of initial sway imperfection (the force values depicted are preset for combination 20)

The static analysis and dimensioning of cross sections in the considered frame have been performed using Robot Structural Analysis computer code [2]. The following loads indicated by consecutive numbers have been declared for the purpose of this analysis:

1 – dead load of structural members,
2 – other dead loads (weight of roofing materials, floor slabs, curtain walls),
3 – live load applied to the flat roof (category H, top left beam),
4 – live load applied to the flat roof (category H, top right beam),
5 – live load applied to the floor (category C, left intermediate floor beam),
6 – live load applied to the floor (category C, right intermediate floor beam),
7 – snow load on the roof (determined directly in Robot Structural Analysis for selected location),
8 to 15 – consecutive wind load schemes (determined directly in Robot Structural Analysis for selected location).

The structural steel used to make all structural components of the considered frame has been modeled in the considered scenario as elastic perfectly plastic material. It has been also assumed, that all the joints in the frame have the same configuration depicted in detail in Fig. 2. As there are no ribs stiffening the column web at the levels of beam flanges, these joints are undoubtedly flexible. However, in the considerations pertaining to the persistent design scenario, these joints have been treated, with a certain level of simplification, as nominally rigid.

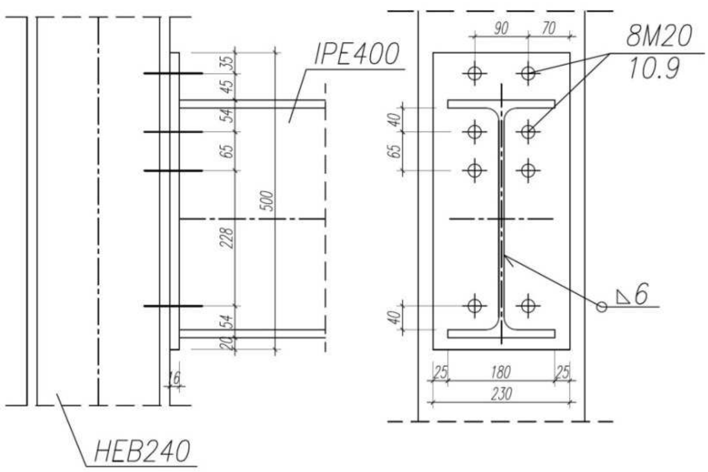


Fig. 2. Scheme of the beam to column joint used in the frame considered here

Detailed analysis of the frame described above, performed for the persistent design scenario resulted in the following selection of sections based on the "weakest links" scenario: beam denoted with No 7 and column denoted with No 3 in Fig. 1a.

3. Analyzed frame in the exceptional design scenario of a fully developed fire

The essential part of analysis was related to the exceptional design scenario of a fully developed fire. It was assumed, that the fire was initiated and developed in both aisles at the ground level of the considered building. This resulted in the heating of only lower columns and lower beams in the bearing structure due to the fire action (Fig. 1). It was also assumed, that top columns and top beams were perfectly insulated from the fire action and were not heated. The fire itself was modeled by the increasing temperature of structural elements indicated above, evenly distributed in their cross sections. The dependence of yield limit and longitudinal modulus of elasticity in steel used to make the considered frame on temperature has been accounted for. The standard formulae, $f_{y,\Theta} = k_{y,\Theta} \cdot f_y$ and $E_{a,\Theta} = k_{a,\Theta} \cdot E_a$ were used, where f_y and E_a represent the respective values specified at the room temperature, while $k_{y,\Theta}$ and $k_{a,\Theta}$ represent the respective reduction factors listed in the code EN 1993-1-2 [3] for different temperature values of steel. The uniform heating of columns 1, 2 and 3 as well as beams 7 and 8 is treated as additional exceptional loading scheme, and assigned the number 16.

Further considerations have been performed according to the rules specified for exceptional combination of actions [4]. The following combinations proved to be the most adverse (Table 1).

Table 1. Combination of actions, authoritative for verification of the ultimate limit state in fire for frame beams and columns, respectively (an excerpt of calculations protocol generated by Robot Structural Analysis [2])

Combination No.	The worst case combinations - top and bottom beams	
20	19(K)	(1+2+16)*1.00+(5+6)*0.70
24	56(K)	(1+2+16)*1.00+5*0.60+7*0.20
25	57(K)	(1+2+16)*1.00+6*0.60+7*0.20
Combination No.	The worst case combinations - bottom columns	
21	20(K)	(1+2+16)*1.00+5*0.70
22	39(K)	(1+2+16)*1.00+5*0.60+9*0.20
23	47(K)	(1+2+16)*1.00+6*0.60+9*0.20

The behaviour of the frame considered here, when subjected to fire action developing on the ground floor as described above, for comparative reasons has been analysed in detail using two approaches to model joints. In the first approach it was assumed conventionally, that all joints remain nominally rigid during the whole time of fire. In the second approach the flexibility of joints, increasing with fire development, has been accounted for (Fig. 3), and the characteristics of this flexibility have been described by a set of curves linking bending moment with increasing angle of rotation at the joint (the so called $M-\varphi$ curves), developed based on the classical component method [5–7] generalised to the case of fire. In the case of node depicted in Fig. 2, and considered in this example, these curves had the shape depicted in detail in Fig. 4.

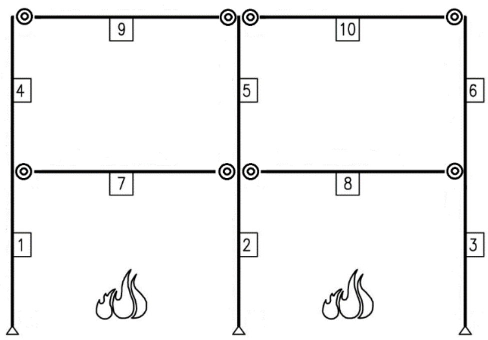


Fig. 3. Scheme of the frame considered in this example, in the case of joint flexibility increasing with the development of a fire

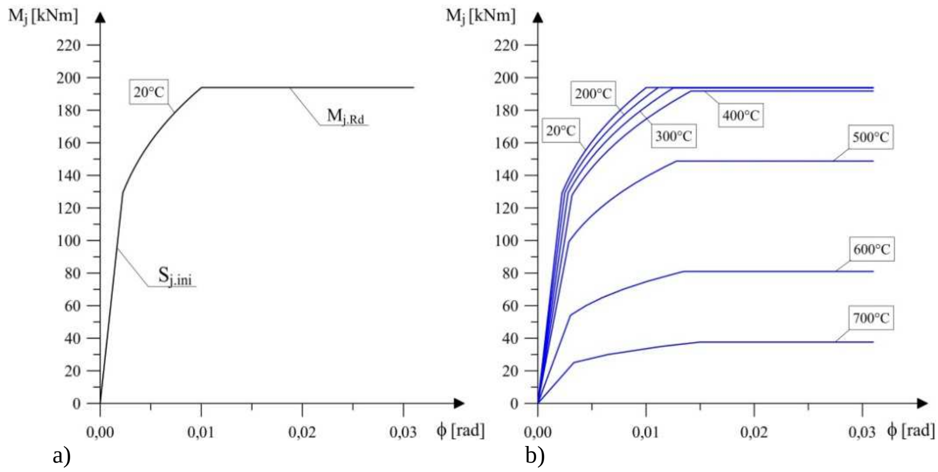


Fig. 4. Bending moment – rotation angle relationship for the joint considered in the example (based on [6]), including: a) single curve determined for room temperature, b) a set of curves developed for fire conditions

4. Alternative methods of analysis

Verification of ultimate limit state for fire conditions has been performed based on two alternative computational approaches [8]. In the first approach the first order analysis using buckling length concept has been applied. For the in plane buckling the multiplier λ_{cr} and after that the critical load $N_{cr,y}$ have been determined (for the first sway vibration eigenform with respect to the column, for the first symmetrical vibration eigenform with respect to the beam). For the out of plane buckling case the critical load $N_{cr,z}$ has been determined under the assumption, that the out of plane buckling length of an element is equal to its theoretical length. In the next step the relative slendernesses $\overline{\lambda_y}$ and $\overline{\lambda_z}$ have been determined, and after that the buckling coefficients χ_y and χ_z. Independently the lateral – torsional buckling coefficient χ_{LT} had been found. In the second computational approach the second order analysis has been applied. This analysis has been performed within the Robot Structural Analysis computational environment [2] taking into account the nonlinear phenomena specified both globally for the whole frame (of the $P-\Delta$ type), and locally for its components (of the $P-\delta$ type). After the internal forces had been found, the critical forces $N_{cr,y}$ and $N_{cr,z}$ were determined, subject to the assumption, that both in plane and out of plane buckling lengths of the structural components were equal to the theoretical lengths of these components. The remaining steps were identical to the steps taken earlier in the first approach.

After the detailed analysis it was found, that the ground floor column No 3 (cf. Fig. 1a) and the beam No7 supporting the ceiling over this floor are authoritative for the determination of critical temperature. These elements are bent and compressed at the same time, thus the sought temperature θ_{cr} is determined by the more restrictive constraint of the two listed below (the upper index Θ denotes in these formulae the dependence of so indexed quantity on steel temperature θ_a):

$$\rho_1 = \rho(\theta_{cr}) = \frac{N^{\Theta}_{fi,Ed}}{\chi^{\Theta}_{\min,\,fi} \cdot A \cdot k^{\Theta}_{y,\theta} \cdot \dfrac{f_y}{\gamma_{M,\,fi}}} + \frac{k^{\Theta}_{y} \cdot M^{\Theta}_{y,\,fi,Ed}}{W_y \cdot k^{\Theta}_{y,\theta} \cdot \dfrac{f_y}{\gamma_{M,\,fi}}} = 1 \tag{1}$$

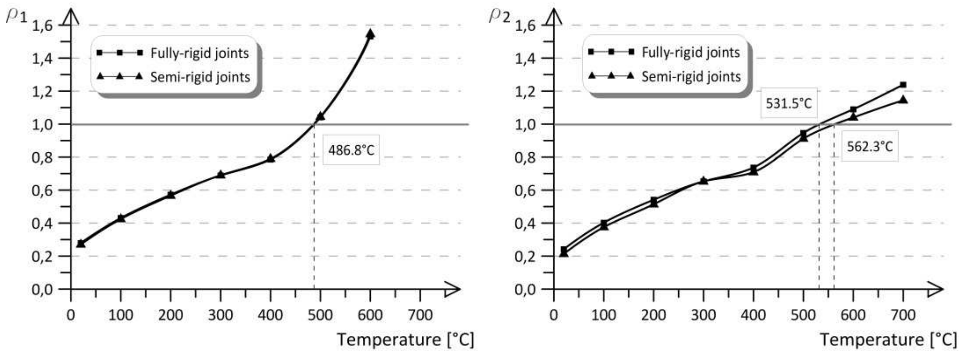
$$\rho_2 = \rho(\theta_{cr}) = \frac{N^{\Theta}_{fi,Ed}}{\chi^{\Theta}_{z,\,fi} \cdot A \cdot k^{\Theta}_{y,\theta} \cdot \dfrac{f_y}{\gamma_{M,\,fi}}} + \frac{k^{\Theta}_{LT} \cdot M^{\Theta}_{y,\,fi,Ed}}{\chi^{\Theta}_{LT,\,fi} \cdot W_y \cdot k^{\Theta}_{y,\theta} \cdot \dfrac{f_y}{\gamma_{M,\,fi}}} = 1 \tag{2}$$

In these relationships $N^{\Theta}_{fi,Ed}$ represents the longitudinal force identified in the structural component for the design scenario related to the developed fire, while $M^{\Theta}_{y,\,fi,Ed}$ represents the bending moment correlated with this force and determined with respect to the so called strong axis of the cross section. In addition, $\chi^{\Theta}_{\min,\,fi}$ is the smaller of $\chi^{\Theta}_{y,\,fi}$ and $\chi^{\Theta}_{z,\,fi}$, which in turn represent the buckling coefficients determined for the fire scenario and computed with respect to strong and weak axes in the considered frame cross section, k^{Θ}_{y} and k^{Θ}_{LT} quantify the nonlinear phenomena, specific to flexural and lateral – torsional buckling, while $\gamma_{M,\,fi}$ represents the partial safety factor covering the uncertainties in modeling material properties appropriate for fire scenario.

5. Detailed analysis of obtained results

5.1. Results obtained for the column No 3

The critical temperature estimates obtained by various methods for the column No 3 are depicted in Fig. 5, 6 and 7. One may easily observe, that these results are not completely unequivocal. If, for instance, the first order analysis is applied to determine the sought temperature, then the analysis yields the authoritative value of 486,8°C determined by the effort ρ_1 (Fig. 5a). Interestingly, there is no difference in the graph depicted in Fig. 5a due to the changing real joint stiffness, decreasing with increasing temperature of frame components. Such difference, however relatively small, is visible on the graph depicted in Fig. 5b, where more precise model of the joint behaviour allowed for the demonstration of an additional safety margin having the magnitude of approximately 30 degrees Centigrade. This safety margin is conditioned by the effort ρ_2.

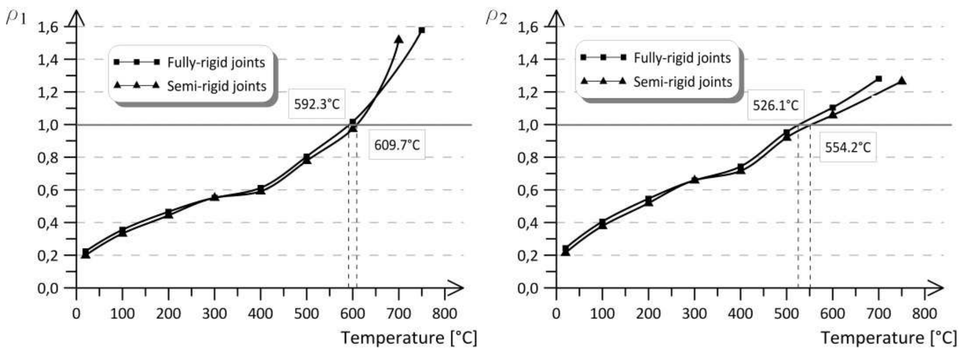


a) b)

Fig. 5. Determination of the critical temperature in the case of column No 3 according to the first order theory, including: a) based on effort ρ_1, b) based on effort ρ_2

Qualitatively different result has been obtained for the same column when the second order analysis has been applied. This time the effort ρ_2 proved to be authoritative for the determination of critical temperature. Here, with joint flexibility increasing with temperature the critical temperature of 554.2°C has been obtained, and when this phenomenon was disregarded a more cautious value of 526.1°C was delivered (Fig. 6b). Both those estimates are significantly less restrictive, than the estimate obtained based on the Fig. 5a after application of simpler first order analysis.

Juxtaposition of the results obtained for the column No 3 after application of first and second order theories and taking into account the joint flexibility changing with the progressing fire is depicted in Fig. 7. It is clearly visible there, that equation (1) used to determine the effort ρ_1 proved to be very sensitive to the type of analysis performed, as the difference in obtained estimates exceeds 120°C (Fig. 7a). Such sensitivity is not observed on the graphs depicted in Fig. 7b, related to the effort ρ_2.



a) b)

Fig. 6. Determination of the critical temperature in the case of column No 3 according to the second order theory, including: a) based on effort ρ_1, b) based on effort ρ_2

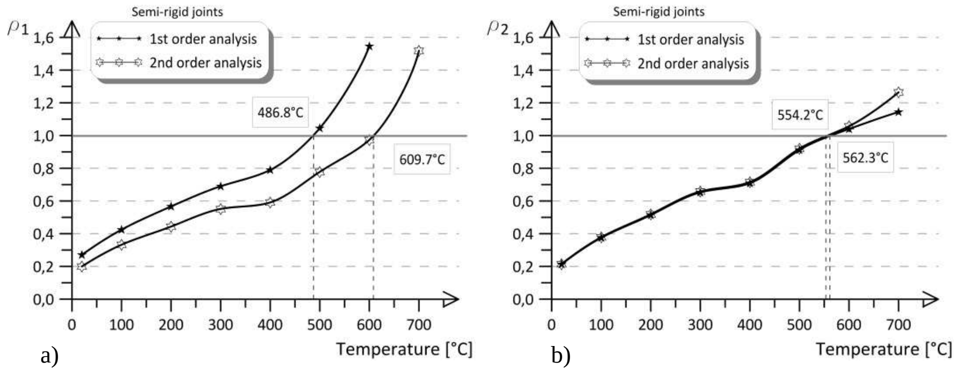


Fig. 7. Determination of the critical temperature in the case of column No 3 according to the first and second order theories, taking into account the joint flexibility changing with temperature, including: a) based on effort ρ_1, b) based on effort ρ_2

5.2. Results obtained for the beam No 7

Analogous results obtained for the beam No 7 are depicted in Fig. 8, 9 and 10.

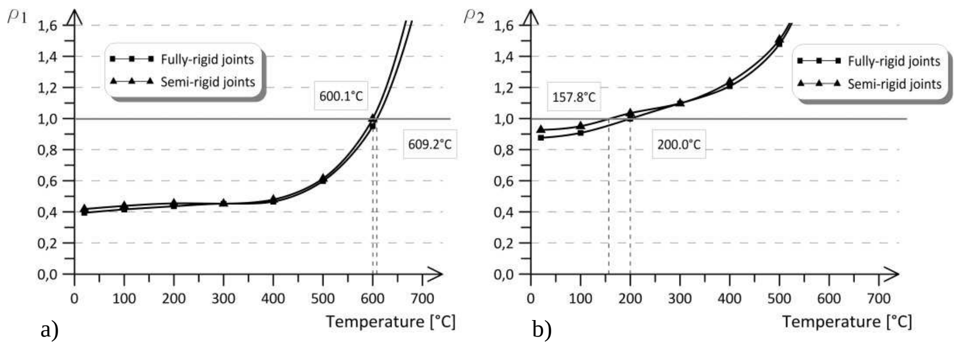


Fig. 8. Determination of critical temperature in the case of beam No 7, according to the first order theory, including: a) based on effort ρ_1, b) based on effort ρ_2

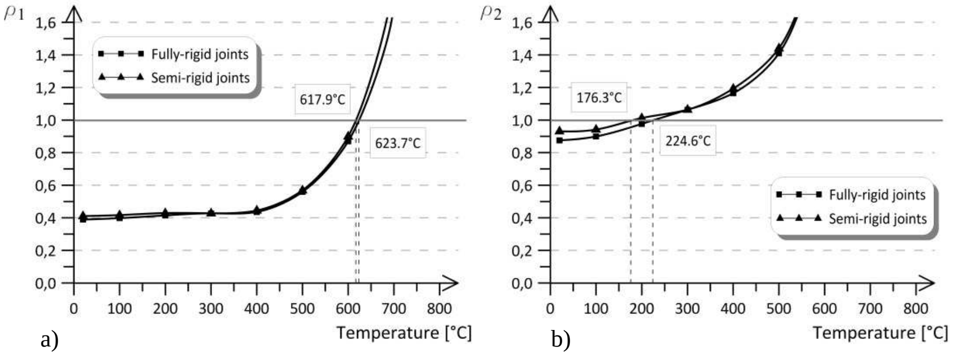


Fig. 9. Determination of critical temperature in the case of beam No 7, according to the second order theory, including: a) based on effort ρ_1, b) based on effort ρ_2

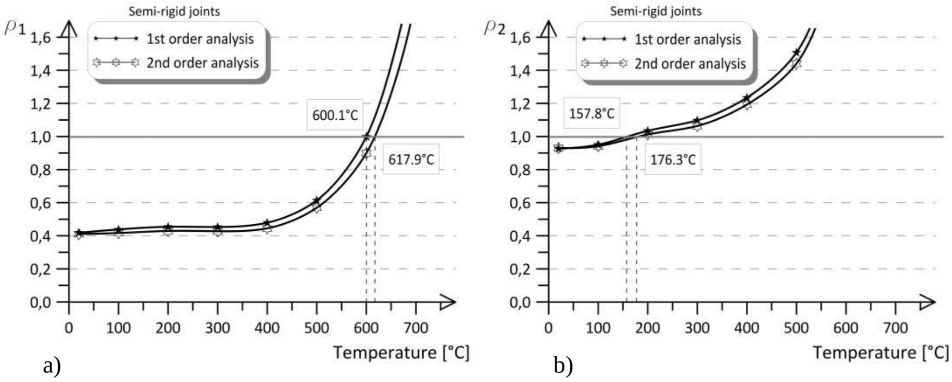


Fig. 10. Determination of critical temperature in the case of beam No 7, according to the first and second order theory, taking into account the joint stiffness changing with temperature, including: a) based on effort ρ_1 , b) based on effort ρ_2

This time, the far more restrictive estimates have been obtained for material effort ρ_2. However, it seems surprising, that a lower value of critical temperature is forecast in the case when the joint rigidity is decreasing with the developing fire (157.8ºC related to 200.0ºC when the first order theory is applied – Fig. 8a, and respectively 176.3ºC related to 224.6ºC when the second order theory is applied – Fig. 8b). Nevertheless, the estimates obtained with the second order theory seem to be much less restrictive than those obtained when the first order theory is applied (Fig. 10), this is analogous to the results obtained for column No 3. In addition, in the case of beam No 7 the sensitivity of the estimated effort ρ_1 on the applied method of analysis is not very pronounced (Fig. 10a), this is in opposition to the phenomenon observed in the case of column No 3. This difference seems to be attributable to the fact that in the case of column compression plays the leading role in the interaction between bending moment and compressive axial force, while in the beam bending plays the dominant role.

6. Concluding remarks

Based on the performed analysis one may clearly foresee, that under the conditions of fully developed fire initiated at the ground floor of the considered frame, the beam denoted as the No. 7 in Fig. 1a would constitute the weakest link. The critical temperature assigned to this element, i.e. the temperature after reaching which the whole frame would lose the capability to safely support the applied loads, however, does depend on the way the calculations have been performed. It is not always true, that the application of a formal model more precisely describing the behavior of joints, i.e. taking into account the joint rigidity decreasing with increasing temperature would reveal an additional reserve of bearing capacity. In the example considered here the obtained critical temperature estimates proved to be even more restrictive than the analogous estimates obtained earlier, with application of a simpler computational model. The estimated critical temperature also does depend on whether the first or second order theory has been used for calculations. This difference is especially pronounced in the case when the axial force starts to play a dominant role in the interaction of bending and compression.

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