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Influence of increasing joint flexibility on critical temperature of steel frame in fire

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Анотація. Вивчений вплив підвищеної температури на поведінку простої рамної конструкції. Зроблено припущення про застосування з'єднань балково-стоякового спряження. Наведено дві технології альтернативного проектування і дається їх порівняння. Отримані рішення приведені на діаграмах.

Аннотация. Изучено влияние повышенной температуры на поведение простой рамной конструкции. Сделано предположение о применении соединений балочно-стоечного сопряжения. Представлены две технологии альтернативного проектирования и дано их сравнение. Полученные решения приведены на диаграммах.

Abstract. The influence of elevated temperature on the behaviour of a simple framed structure is studied. The assumption of the application of end-plate beam-to-column connections is made. Two alternative design techniques are presented and quantitatively compared one to another. Obtained solutions are shown in diagrams.

Key words: steel frame, joint flexibility, critical temperature.

To precisely estimate the critical temperature of steel frame subject to fire exposure, being the objective measure of structural fire resistance, not only the suitable reduction of material properties but also the adequate joint stiffness decrease should be taken into consideration in the analysis. In the presented article the influence of elevated temperature on the behaviour of a simple framed structure is studied in detail basing on the results of the adequate numerical example. It is assumed that the end-plate beam-to-column connections are applied in considered construction. The shape of moment-rotation-temperature curve, adopted to examine and describing joint flexibility at particular fire moments, is fully consistent with the adequate results obtained experimentally. Two alternative design techniques are presented and quantitatively compared one to another. The first one deals with classical first order analysis with the specification of member buckling lengths, whereas the second with another approach connected with the amplification of horizontal actions imposed to the structure. Obtained solutions depend on the adopted evaluation methodology. They are shown in detail in diagrams enclosed to the paper.

Introduction. Steel frame is typical and economically justified load-bearing structure applied in constructions of many football stadiums and other sports facilities. To reliably select necessary cross-sections of beams and columns composing the whole structure the adequate resistance and stability analyses have to be performed. They are usually based on the simple comparison of design value of a conclusive action effect with design value of a suitable member resistance, corresponding with such effect. The global action effect is generally specified as a sum of the partial effects originating from particular loads applied to the structure, provided that those effects are summed in accordance with the rule adequate for persistent design situations. However, in many cases, the potential accidental events can be decisive if the problem how to secure the required safety level for the user of considered building is studied in detail. The internal and fully developed fire, limited to the building compartment, seems to be one of the most dangerous events in such point of view. It is important for the identified risk of fire occurrence to be as small as possible in order that random fire ignitions would be the sufficiently rare episodes. If such conclusion can be accepted, having regarded all imminent unfavourable circumstances, then the another combination rule related to the summing of all effective partial action effects is adopted to the analysis. To specify the global action effect design value only the characteristic permanent loads as well as the frequent or quasi-permanent values of the most of variable loads are now taken into consideration. As a result of such calculation technique the reliable global action effect is in consequence significantly reduced in relation to that being adequate for classical structural analysis made in persistent design situation.

The structural fire resistance is the most frequently interpreted as a time of fire duration when the load-bearing structure can safely carry all imposed external loads, together with internal forces and moments thermally generated as a result of potential strain constraints. This time period, however, cannot be adopted as the objective safety measure because its value depends on the fire characteristics. For this reason the authors suggest to calculate an alternative quantity – the critical temperature of the whole frame - being independent on the intensity of imminent fire. It is assumed that all external loads applied to the structure remain constant during the whole fire time, but on the other hand, the member temperature Θ_a is at the same time monotonically increasing. Temperature $\Theta_{a,cr}$ related to the frame collapse is generally interpreted as critical for the analysed structure and for adopted level of its loading. The fire resistance limit state occurs in the point-in-time, described as $t_{fi,d}$, when the design value of the action effect $E_{fi,t,d}\left(t_{fi,d}\right)$ reaches the level specified by the design value of member carrying capacity $R_{fi,t,d}\left(t_{fi,d}\right)$. However, the occurrence of such event is not necessary because much earlier the random value of such effect can be too high $\left(E_{fi,t} \geq E_{fi,t,d}\right)$, or random member resistance may not remain large enough $\left(R_{fi,t} \leq R_{fi,t,d}\right)$. It is significantly important that this limit state is not reached exactly at the point-in-time when the considered member fails because its destruction really takes place, but earlier, when the probability of its failure becomes too high and, in consequence, it may no longer be tolerated.

The aim of the presented paper is to give and discuss in detail the appropriate design approach how to evaluate the critical temperature of a simple steel frame exposed to fully developed fire when the temperature of exhaust gas remains uniform inside the whole building compartment. Special attention is paid to the quantitative assessment of the influence of real joint stiffness, decreasing with member temperature growth, on conclusive frame fire resistance.

Description of the considered frame. The behaviour under fire conditions of two-aisle steel sway frame presented in Fig. 1 is systematically analysed in the enclosed numerical example. Such structure is made of steel for which the characteristic yield point value is equal to $f_y = 412$ MPa. Let the spacing in building longitudinal direction between the adjoining frames be adopted as 6,0 m. As we can see all frame beams are constructed with I-sections UB356x171x51, whereas frame columns with another I-sections – UC254x254x89. On the right side in this Figure the arrangement of all external loads imposed to the frame is shown in detail. Please note that their values marked in such scheme are taken as the characteristic ones. The frame is simply supported on reinforced concrete foundation.

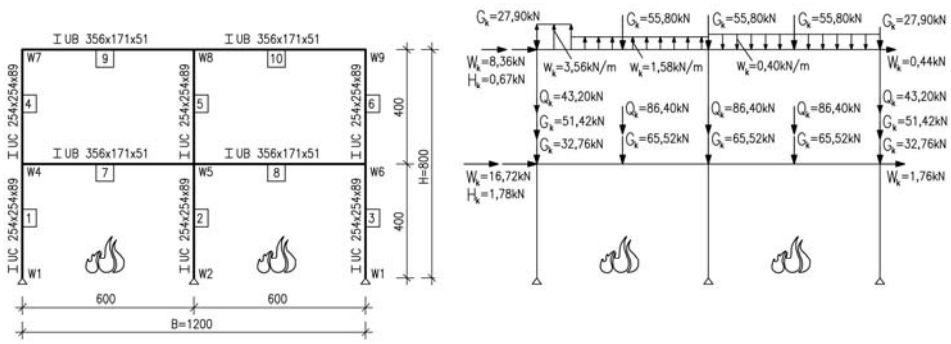

Fig. 1. Scheme of considered frame (on the left side) and external characteristic loads arrangement adopted to the analysis (on the right side).

It is accepted that in considered frame all beam-to-column joints have the same construction. However, two alternative statical schemes are quantitatively examined. In the first approach they are treated as fully-rigid ones during the whole fire time. The second approach is connected with the modelling of real joint flexibility being dependent on the actual member temperature. In fact, the steel temperature growth under fire conditions always means the simultaneous joint stiffness decrease. This influence is in general formally neglected in classical structural analysis, however, such simplification seems to be unjustified because it can lead to the assessments of the critical temperature of the whole structure being too optimistic and, in consequence, unsafe.

Let, in the second design approach, all beam-to-column joints will be adopted as the end-plate semi-rigid ones, shown in detail in Fig. 2. Such choice was determined by the authors' attainability to the experimentally obtained joint characteristics, representing by the set of the curves $M-\Theta_a-\phi$ (bending moment – member temperature – rotation) [1]. In this place it is necessary to say that at present we have only a very little number of sufficiently justified joint charactristics in which the dependence between the joint flexibility and the steel temperature is expressed, despite the fact that since many years we can use significantly rich database with comprehensive set of classical $M-\phi$ relations, specified for particular kind of joints [2].

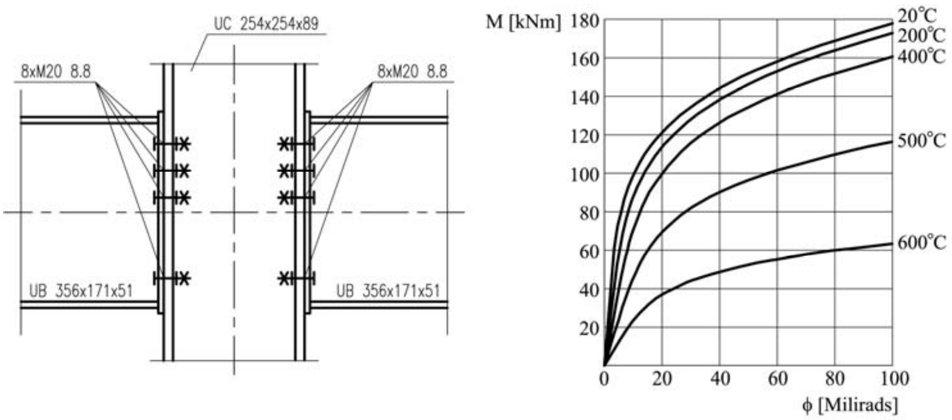

Fig. 2. Scheme of two-sided beam-to-column end-plate semi-rigid joint (on the left side) and its characteristic bending moment – member temperature – rotation, obtained basing on the experimental investigations presented in [1] (on the right side)

The statical analysis in this example was made with the computer program *Robot Structural Analysis 2010* [3]. The dependences between the member temperature and the steel yield point as well as the steel elasticity modulus were taken into consideration, in accordance with the following relations: $f_{y,\Theta} = k_{y,\Theta} f_y$ and $E_{a,\Theta} = k_{E,\Theta} E_a$, where $k_{y,\Theta}$ and $k_{E,\Theta}$ are the suitable reduction coefficients specified in the standard EN 1993-1-2 [4].

The fire limit state identification. The members of the frame structure being reliable for the assessment of its critical temperature are the first floor column, marked by the symbol «3» in Fig. 1, as well as the beam «8», supporting the floor above the first story of the building. It is easy to notice that they are simultaneously bent and compressed so, according to the standard [4], the searched temperature $\Theta_{a,cr}$ is determined by the compliance with the two following requirements:

$$\rho_1 = \rho\left(\Theta_{a,cr}\right) = \frac{N^{\Theta}_{fi,Ed}}{\chi^{\Theta}_{\min,fi} A k^{\Theta}_{y,\Theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k^{\Theta}_y M^{\Theta}_{y,fi,Ed}}{W_y k^{\Theta}_{y,\Theta} \dfrac{f_y}{\gamma_{M,fi}}} = 1 \quad (1)$$

$$\rho_2 = \rho\left(\Theta_{a,cr}\right) = \frac{N^{\Theta}_{fi,Ed}}{\chi^{\Theta}_{z,fi} A k^{\Theta}_{y,\Theta} \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k^{\Theta}_{LT} M^{\Theta}_{y,fi,Ed}}{\chi^{\Theta}_{LT,fi} W_y k^{\Theta}_{y,\Theta} \dfrac{f_y}{\gamma_{M,fi}}} = 1 \quad (2)$$

The first formula is connected with the interaction of in-plane bending (without the imminence of lateral – torsional buckling) and axial compression with potential ability of flexural buckling towards to the «weak direction» (it can be both the in-plane buckling because of the significantly large effective buckling length and also the out-of-plane buckling, linked with the smaller radius of gyration of column cross-section). On the other hand, the second equation is the expression of another interaction, between the in-plane bending not secured against the ability of lateral – torsional buckling and in-plane axial compression. The more restrictive from those two presented formulae will be conclusive in the evaluation of the temperature $\Theta_{a,cr}$. Let us pay attention to the upper index Θ applied to both discussed equations. It means that particular quantity, marked by such index, depends on the considered steel temperature and changes together with the temperature growth. It should be underlined that such quantities are not only the global instability factors (χ_y, χ_z, χ_{LT}) but also the coefficients resulting in the non-linear shape of $M - N$ interaction curve (particularly k_y and k_{LT}). However, the inevitable elastic moment redistribution (related also to other internal forces) being the result of joint stiffness decrease under fire conditions seems to be of the greatest importance in the global safety analysis.

The first order frame analysis with the application of the effective buckling length concept. Let us analyse the considered frame by means of the classical first order analysis in accordance to which the concept of the effective buckling length is applied. The most dangerous fire scenario was adopted to the study, consequently the fully developed fire is modelled in the whole building first floor area (see Fig. 1). Furthermore, it is assumed that the second-story columns as well as the beams supporting the upper frame floor are perfectly isolated against the fire exposure so only the steel members localised inside the building first floor volume are monotonically heated by fire. Because of the fact that considered fire is fully developed and the steel thermal conductivity is significantly large the steel temperature distribution can be approved as uniform not only in each member cross-section but also across the length of each frame bar at particular points-in-time of fire duration, however, this temperature is higher and higher for the succeeding fire moments. The level of frame external loading is accepted as constant during fire and independent of fire intensity. Such simplification is slightly conservative and in general leads to the safe critical temperature evaluations.

Detailed structural analysis showed that the reliable accidental combination rule of the partial action effects was the following one: *the effect of characteristic dead weight of a frame structure* $\left(G_{0,k}\right)$ *+ the effect followed from the characteristic permanent load* $\left(G_k - G_{0,k}\right)$ *+ the effect being the result of the characteristic operational load* $\left(Q_k\right)$ *x* 0,7 *+ the effect origined from the characteristic wind load* $\left(W_k\right)$ *x* 0,2 *+ the effect of the temperature action x 1,0.* Conclusively, the equivalent horizontal forces H_k, being the model of sway imperfections and specified dependently on the sum of vertical actions, are calculated according to the same summing rule.

The first step of the analysis is to find the effective buckling length for particular compressed members. It is a simple assessment in relation to the critical load level $N_{cr,z}$ and out-of-plane flexural buckling. The searched buckling length is then exactly equal to their theoretical length because all supports in this direction are adopted as non-sway and fully flexible. To identify the suitable buckling lengths related to in-plane buckling mode the critical load level $N_{cr,y}$ is previously looked for by means of the calculation of the multiplier λ_{cr} being the solution of the adequate modal analysis (the first symmetrical form of frame free vibrations is examined when the frame beam behaviour is studied in detail, whereas the first sway form of such free vibrations is appropriate in case of the column analysis). After the buckling lenghts are univocally determined the relative slenderness $\overline{\lambda_y}$ and also $\overline{\lambda_z}$ can be calculated. Finally, the flexural buckling coefficients χ_y and χ_z are identified as well as the coefficient suitable for lateral – torsional buckling mode χ_{LT}.

The stability analysis of considered steel frame was performed separately for two different design cases. The first case was connected with the situation when all beam-to-column joints were modelled as fully-rigid during the whole fire time. The second case, for which the real joint flexibility was taken into account, dependently on the actual member temperature, was studied for comparative purposes. Obtained results are shown in Tables 1 and 2 in relation to the column «3» and also in Tables 3 and 4 in relation to the beam «8». Tables 1 and 3 are linked with the fully-rigid joints, whereas Tables 2 and 4 with semi-rigid ones. As a result of such calculations the values of the coefficients $\rho_1 = \rho_1(\Theta_a)$ and $\rho_2 = \rho_2(\Theta_a)$ were estimated and marked in suitable figures (in Fig. 3 and Fig. 4 regarding to the column «3» and beam «8», respectively). The critical temperature of considered member is then evaluated as a temperature value for which the examined factor $\rho_1 = \rho_1(\Theta_a)$ or $\rho_2 = \rho_2(\Theta_a)$ reaches the level 1,0. In the case when such event will take place for steel temperature greater than $600\ ^{\circ}\mathrm{C}$ the searched temperature $\Theta_{a,cr}$ is predicted by means of the polynominal interpolation.

Table 1

Results related to the column «3», obtained by means of the application of the first order analysis for the frame with all beam-to-column joints remaining fully-rigid in the whole time of fire duration

$\Theta_a\,[^{\circ}\mathrm{C}]$	$N\,[kN]$	$M_y\,[kNm]$	λ_{cr}	$N_{cr,y}\,[kN]$	$\overline{\lambda_y}$	χ_y	$N_{cr,z}\,[kN]$	χ_z	χ_{LT}	ρ_1	ρ_2
20	226,7	44,1	11,17	2533,1	1,36	0,35	5842,3	0,56	0,65	0,23	0,21
100	220,5	77,2	11,15	2458,2	1,38	0,34	5842,3	0,56	0,65	0,30	0,30
200	214,2	110,3	10,01	2145,1	1,47	0,31	5258,1	0,53	0,62	0,38	0,41
300	209,5	135,1	8,89	1862,0	1,58	0,28	4673,8	0,50	0,60	0,46	0,50
400	206,4	151,6	7,77	1603,2	1,70	0,25	4089,6	0,47	0,57	0,52	0,58
500	204,8	159,8	6,65	1362,8	1,63	0,27	3505,4	0,49	0,59	0,68	0,73
600	213,8	112,7	3,45	737,2	1,72	0,24	1811,1	0,45	0,55	1,03	0,91

It is noteworthy that the multiplier λ_{cr} estimated for the design case when the beam-to-column joints are modelled as flexible is significantly smaller in relation to another one being the result of similar calculations in which the model with all fully-rigid joints is applied. In consequence, both the buckling length estimate for in-plane buckling mode and also the relative slenderness $\overline{\lambda_y}$ are then greater than those, previously identified (see Fig. 5). Finally, the suitable flexural buckling coefficient χ_y is much more restrictive. On the other hand, as a result of elastic moment redistribution, conclusive values of the bending moment $M_{y,fi}$ and the compressive axial force N_{fi} related to the column «3» and taken from Table 2 are distinctly smaller than those taken from Table 1 (let us notice that such rule is not satisfactory in relation to the beam «8»).

Table 2

Results related to the column «3», obtained by means of the application of the first order analysis for the frame with beam-to-column joints being semi-rigid under fire conditions with the flexibility increasing together with the steel temperature growth

$\Theta_a\left[{}^{o}\mathrm{C}\right]$	$N\left[\mathrm{kN}\right]$	$M_y\left[\mathrm{kNm}\right]$	λ_{cr}	$N_{cr,y}\left[\mathrm{kN}\right]$	$\overline{\lambda_y}$	χ_y	$N_{cr,z}\left[\mathrm{kN}\right]$	χ_z	χ_{LT}	ρ_1	ρ_2
20	217,3	33,7	7,72	1677,5	1,67	0,26	5842,3	0,56	0,65	0,25	0,18
100	211,9	63,7	6,94	1469,9	1,78	0,23	5842,3	0,56	0,65	0,33	0,26
200	208,3	95,4	6,17	1286,1	1,95	0,21	5258,1	0,53	0,62	0,43	0,36
300	205,5	118,2	5,23	1074,3	2,14	0,18	4673,8	0,50	0,60	0,52	0,45
400	203,9	133,7	4,48	914,1	2,37	0,16	4089,6	0,47	0,57	0,60	0,52
500	203,1	138,2	3,26	662,4	2,23	0,15	3505,4	0,49	0,59	0,83	0,65
600	206,2	92,2	1,58	325,5	2,41	0,12	1811,1	0,46	0,55	1,41	0,78

Table 3

Results related to the beam «8», obtained by means of the application of the first order analysis for the frame with all beam-to-column joints remaining fully-rigid in the whole time of fire duration

$\Theta_a\left[{}^{o}\mathrm{C}\right]$	$N\left[\mathrm{kN}\right]$	$M_y\left[\mathrm{kNm}\right]$	λ_{cr}	$N_{cr,y}\left[\mathrm{kN}\right]$	$\overline{\lambda_y}$	χ_y	$N_{cr,z}\left[\mathrm{kN}\right]$	χ_z	χ_{LT}	ρ_1	ρ_2
20	-5,1	94,1	74,37	376,1	2,67	0,12	517,7	0,15	0,70	0,27	0,37
100	17,7	94,9	70,01	1242,6	1,47	0,31	517,7	0,15	0,70	0,30	0,40
200	40,6	95,7	58,56	2375,2	1,06	0,47	465,9	0,14	0,69	0,37	0,47
300	57,7	96,3	48,65	2804,9	0,98	0,52	414,1	0,13	0,66	0,44	0,53
400	69,1	96,7	40,39	2789,2	0,98	0,51	362,4	0,11	0,64	0,50	0,59
500	74,7	96,9	33,65	2514,2	0,91	0,55	310,6	0,12	0,66	0,64	0,73
600	42,3	95,8	20,04	848,0	1,22	0,40	160,5	0,11	0,63	0,91	1,03

Table 4

Results related to the beam «8», obtained by means of the application of the first order analysis for the frame with beam-to-column joints being semi-rigid under fire conditions with the flexibility increasing together with the steel temperature growth

$\Theta_a\left[{}^{\circ}\mathrm{C}\right]$	$N\left[\mathrm{kN}\right]$	$M_y\left[\mathrm{kNm}\right]$	λ_{cr}	$N_{cr,y}\left[\mathrm{kN}\right]$	$\overline{\lambda_y}$	χ_y	$N_{cr,z}\left[\mathrm{kN}\right]$	χ_z	χ_{LT}	ρ_1	ρ_2
20	-1,0	100,4	64,28	61,8	6,58	0,02	517,7	0,15	0,70	0,30	0,42
100	19,4	101,9	61,20	1187,9	1,50	0,30	517,7	0,15	0,70	0,35	0,46
200	38,9	102,3	53,37	2074,0	1,14	0,44	465,9	0,14	0,69	0,41	0,52
300	53,0	102,9	45,82	2427,3	1,05	0,48	414,1	0,13	0,66	0,47	0,58
400	62,4	103,2	38,91	2426,4	1,05	0,48	362,4	0,11	0,64	0,52	0,63
500	65,5	104,4	32,56	2131,0	0,99	0,51	310,6	0,12	0,66	0,67	0,79
600	39,4	105,0	17,72	698,3	1,34	0,35	160,5	0,11	0,63	1.02	1,18

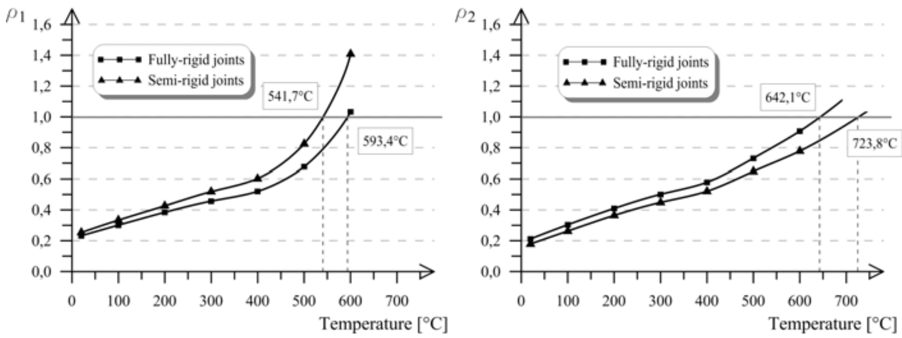

Fig. 3. Estimation of critical temperature of the column «3» when the first order frame structural analysis is made

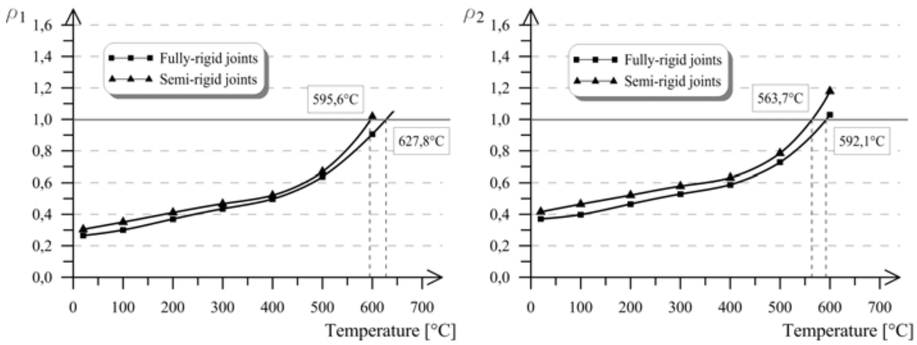

Fig. 4. Estimation of critical temperature of the beam «8» when the first order frame structural analysis is made

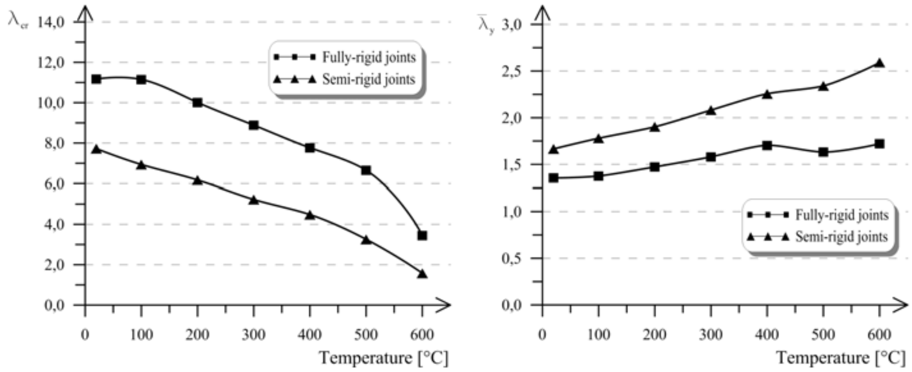

Fig. 5. The values of the multiplier λ_{cr} and the relative slenderness $\overline{\lambda_y}$ in relation to the steel temperature Θ_a

Alternative simplified second order approach with the amplification of horizontal loads. It is obvious that the first order analysis described in the previous chapter is not the only one possible to apply if we want to reliably evaluate the critical temperature of considered steel frame. Let us study in detail an alternative simplified second order approach according to which the effective buckling length is not identified (the theoretical member length is always adopted to the analysis) but all horizontal loads, both external and equivalent ones, are amplificated to take into account the real influence of sway imperfections. If the typical standard requirements are conclusive [4] then the first step of the analysis should be the verification whether the considered frame is sensitive to the potential second order effects. To do this the sensitivity factor α_{cr} is calculated separately for the first and for the second frame story. The fulfilment of the limitation $\alpha_{cr} < 10$ for only one examined story means that the whole load-bearing structure is really sensitive for such effects and at least the suitable amplification of horizontal forces should be made. This amplification can be taken into consideration by means of the multiplication of real load values by the coefficient η_{amp} being specified separately in the levels of upper and of lower frame beams. Finally, the classical static analysis is performed but for the amplificated load arrangement. As a result of such calculation technique the conclusive distribution of the bending moments as well as of the other internal forces is found. It is necessary to remember that the values of the critical axial forces $N_{cr,y}$ and $N_{cr,z}$ are now admittedly specified similarly to the methodology adequate for the first order approach described above, however, a slight difference is the fact that no effective buckling length is defined in such calculations.

The results obtained in relation to the considered frame are shown in Table 5.

Let us notice that, in relation to the frame considered in the example, the sufficient sensitivity of load-bearing structure can occur not earlier than the temperature of steel members localised inside the first frame story will reach the level ca. $600\ ^{o}C$ (i.e. when $5{,}89 < 10$ – see Table 5). It is noteworthy that such imminence was detected only in the case when the model with semi-rigid joints was analysed. Furthermore, let us underline that it is the temperature value being something like the adequate critical temperature of the whole frame. In consequence of such conclusion it is really not necessary to make the amplification of all horizontal loads imposed to the structure, however, those calculations have been made only for comparative purposes. Obtained results are presented in Fig. 6 on the example of the column «3» and in Fig. 7 in relation to the beam «8». They are quantitatively significantly different than the previous ones, identified by means of the application of classical first order analysis. The basic reason of such inconsistencies seems to be the other way of the calculation of reliable value of flexural buckling coefficient χ_y. As we can see the critical temperature evaluation related to the considered steel frame is not univocal because its value strongly depends on the applied calculation methodology. Nevertheless, it should be underlined that this type of the estimation uncertainty does not have the source in simplified modelling of fire phenomenon, but it is only the result of the lack of compatibility between various techniques currently used for structural analysis.

Table 5

Sensitivity factors α_{cr} for the influence of second order effects as well as the amplification coefficients η_{amp} obtained for considered steel frame

$\Theta_a\left[{}^{o}C\right]$	Joints fully-rigid during the whole time of fire duration.		Joints semi-rigid with the flexibility increasing together with member temperature growth.	
	α_{cr} (for upper frame story / for lower frame story)	η_{amp} (for upper beam level / for lower beam level)	α_{cr} (for upper frame story / for lower frame story)	η_{amp} (for upper beam level / for lower beam level)
20	178,21/43,03	1,01/1,02	73,25/28,32	1,01/1,04
100	178,21/43,03	1,01/1,02	59,00/25,00	1,02/1,04
200	160,05/38,72	1,01/1,03	51,91/22,21	1,02/1,05
300	141,69/34,44	1,01/1,03	41,75/18,61	1,02/1,06
400	124,36/30,10	1,01/1,03	35,28/15,96	1,03/1,07
500	106,70/25,82	1,01/1,04	23,26/11,60	1,04/1,09
600	55,05/13,34	1,02/1,08	11,70/5,89	1,09/1,20

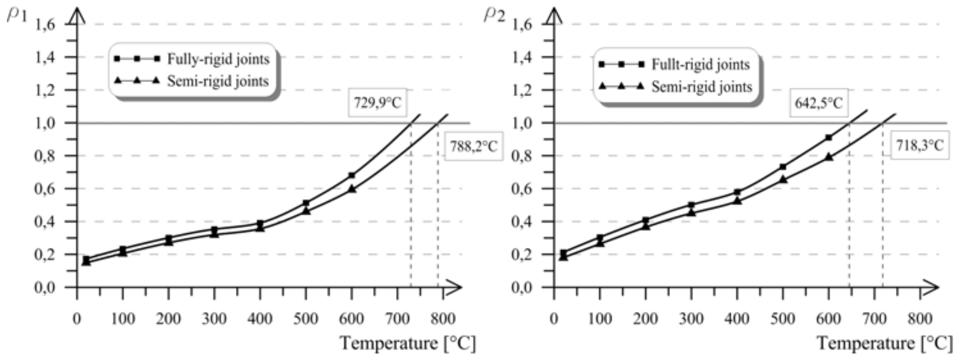

Fig. 6. Estimation of critical temperature of the column «3» when the amplification of horizontal loads was applied

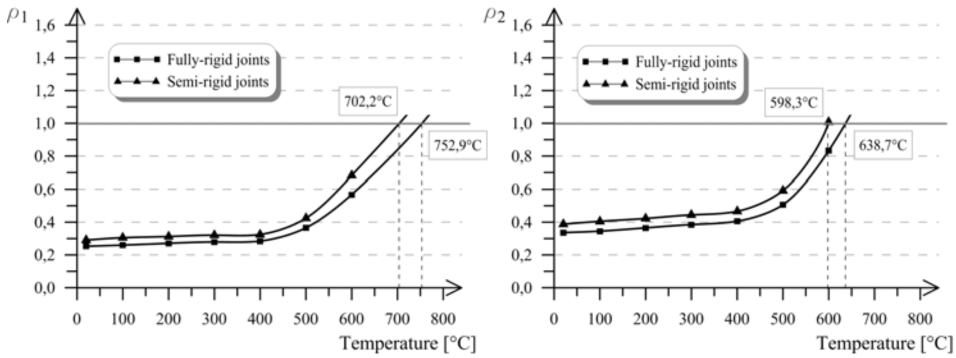

Fig. 7. Estimation of critical temperature of the beam «8» when the amplification of horizontal loads was applied

Conclusion

The detailed analysis of the behaviour of considered steel frame under fire conditions allows to formulate some conclusions which seem to be of a great importance for the qualitative assessment of a structural safety level.

1. The application of the formal model with joints remaining fully-rigid during the whole time of fire duration led to the assessment of the frame critical temperature on the level $592{,}1\,^{o}\mathrm{C}$, provided that the classical first order analysis was made. Let us notice that the beam "8" was the frame member reliable for the evaluation. On the other hand, if the real joint flexibility was taken into account, increasing together with the steel temperature growth, another and quite unexpected conclusion was met. In this case the critical temperature of the same frame was calculated to be equal to only $541{,}7\,^{o}\mathrm{C}$, furthermore, such

evaluation was connected with the column «3». Such level of the steel temperature will be reached under potential fire significantly earlier than the previous one, which means that the value of frame fire resistance as well as the predicted safety level, if they both are estimated without any consideration of the inluence of real joint stiffness, seem to be considerably overestimated.

2. If the simplified second order theory is applied to the analysis, according to which the amplification of both external and equivalent horizontal loads is implemented, then the evaluations of critical temperature of the frame, obtained as a result of such calculations, are less restrictive in relation to the other ones, resulting from using the classical first order design methodology. In the presented example they are as follows: $638{,}7^{o}C$ if all joints were adopted as fully-rigid ones and $598{,}3^{o}C$ when the real joint flexibility was considered. Let us see that the beam «8» was the reliable frame member in both cases. The reason of such general relation is the fact that in this kind of the analysis the considerable reduction of the member compression resistance was not implemented to the formal model, being the effect of the adoption of a large effective buckling length when the in-plane flexural buckling stability analysis was made.

3. If the first order frame analysis is performed then considering the real joint flexibility under fire gives, in general, the assessments of conclusive critical temperature being more careful in comparison with those resulting from the acceptance of the full joint stiffness, independent on the real steel temperature. The important exception is in this field the situation shown in detail on the right side in Fig. 3. In this case, for the evaluation of the value of ρ_2 coefficient, the interaction between the member bending and compression is examined with potential out-of-plane buckling mode (see Eq. 2). Let us pay attention to the fact that the bar effective buckling length is then assumed in this direction to be exactly equal to its theoretical dimension.

4. When the second order analysis is carried out, with the amplification of horizontal loads, then the interpretation of obtained results has to be more complex. On one hand, taking into account the real joint flexibility under fire leads to the assessments of critical temperature being less restrictive in relation to the column «3» (see Fig. 6). On the other hand, such calculation procedure gives the evaluations being more restrictive in relation to the beam «8» (see Fig. 7). In such design approach the member effective buckling length is not specified at all; whereas, its specification is of the great importance when the classical first order analysis is performed.

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