BUCKLING LENGTH OF STEEL COLUMNS IN FRAME STRUCTURES STRESSED BY FIRE

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Abstract

Fire resistance is crucial for load-bearing structures. However, current rules in structural Eurocodes may not cover all fire design situations. In EN 1993-1-2, fire design rules exist only for braced steel frames, treating each floor as a separate fire section. Unbraced frames lack specific fire design rules and use regular temperature design buckling lengths. This article presents an analysis of the behaviour of columns in braced and unbraced frames and compares it with the procedures of some other authors.

Keywords

Steel, columns, frames, buckling, fire

1 INTRODUCTION

When analysing structures affected by fire, it is necessary to consider their actual behaviour as faithfully as possible. According to the set of STN EN standards, one of three calculation methods can be used for analysis. The first method is the local analysis of isolated elements of the structure. It is the simplest and, in most cases, most sufficient procedure. As a second procedure, it is possible to use the analysis of a part of the structure. The third method, i.e., the global calculation of the entire structure, is suitable to be used if the analysed element also depends on the stiffness of adjacent elements.

This article is devoted to braced and un-braced frames, where the buckling length is also influenced by the stiffnesses of the other columns and thus they are not isolated elements.

Behaviour of columns in frame constructions stressed by fire

To determine the buckling resistance of the column at normal temperature as well as in a fire situation, it is necessary to choose the right buckling length of the rod. The procedure for determining the buckling length of a steel rod stressed by fire according to art. 4.2.3.2 of the standard STN EN 1993-1-2 [1], is similar to the procedure at normal temperature according to art. 6.3.1.1 of STN EN 1993-1-1 [2].

According to the standard STN EN 1993-1-2 [1], it can be assumed that the fire affects one floor and the cold elements in the floors above and below the fire have much greater stiffness than the heated elements. Due to this fact, the buckling lengths of heated columns are smaller than the buckling lengths of cold columns.

In the case of braced frames, each floor is considered as a separate fire section and hot column is considered as "fixed" into cold columns in the floor above and below. In addition, the beam connections in the braced frames are pinned or semi-rigid, which does not have a significant effect on the rotational stiffness, which makes it possible to consider a buckling length of $0.5 \times L$ for the intermediate floors and $0.7 \times L$ for the first and last floors. However, in reality, the buckling lengths are different from those which are specified in the European standard and approach the specified values from above, i.e. from the dangerous side. Several authors have addressed the issue. The buckling lengths can be determined by Wood's method [3], or according to the informative extension E of the pre-standard ENV 1993-1-2 [4]. The buckling length can also be determined by a linear approximation between the buckling length at 20 °C and at 1200 °C with the reduction factor $k_{E,\theta}$ of the modulus of elasticity of steel at elevated temperatures, which was proposed by Gomes et al. [5]. In the national annex of UK BS EN 1994-1-2 [6], these values have been increased for the case hybrid constructions. It is recommended to use the value $l_{fi} = 0.7 \times L$ for intermediate floors or $l_{fi} = 0.85 \times L$ for last above-ground floors.

However, in the case of un-braced frames, the Eurocodes do not specify rules for determining the buckling lengths of fire-stressed columns. Authors Couto et al. [7] showed in their research that the critical load of frames decreases with increasing temperature, because the bending stiffness of the elements decreases with increasing temperature. They also showed that the critical column of a multi-story frame, which determines the buckling of

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the frame, can change with an increase in temperature. For un-braced frames, they recommended values of $l_{fi} = 1.0 \times L$ for all floors and $l_{fi} = 2.0 \times L$ for the column on the first floor if it is pinned.

The article shows simple examples of the behaviour of columns in braced and un-braced frames with the influence of temperatures. The results are compared with the procedures for determining buckling coefficients according to the authors mentioned in the text.

2 CALCULATIONS OF BUCKLING LENGTHS

Unprotected steel supporting structures must be adequately protected in the event of a fire, given that their fire resistance is low. The protection of the steel supporting structure must be chosen so that its design is safe and economical. For protection, fire protective elements such as sprays and element linings can be used, the thickness of which depends on the critical temperature of the heated element.

The critical temperature of the member was determined as a temperature at which the design buckling resistance of the member is equal to the axial force in the member induced by the combination of fire loads. In this case it is $N_{b,fi,RD} = N_{fi,ED}$. This is not a critical temperature method, which is given in EN 1993-1-2 [1]. The critical temperature depends on the degree of use of the element at the beginning of the fire as well as on the buckling lengths during the fire.

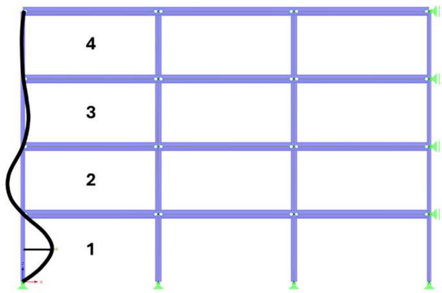
To demonstrate the behaviour of frame under fire stress, a 2D structure of a frame with three fields and four floors was created. Field spans are 6 m long, floor heights are 3 m. The load on the frame consists of the own weight of the structure, the useful load for administrative buildings, snow and wind. The cross-sections of the columns are constant in height and thus have different degrees of use.

Braced frame

The braced frame structure was designed for normal temperature with HEB 200 as an external column and HEB 280 as an internal column cross-sections. All beams were designed as HEA 400. All load-bearing members, i.e. the columns and beams were attached as pinned.

The calculation of the buckling lengths of the rods during fire was carried out using the calculation programme Dlubal RFEM [8], specifically using the stability calculation in the RF-STABILITY module.

Considering that the critical temperature depends on the buckling length, which is not known at the beginning, the buckling length of the rod at normal temperature was used as the initial buckling length (Fig. 1a). Subsequently, the calculation of the critical temperature of the rod was calculated out in several iterations, while at the beginning of the next iteration was used the buckling length from the previous iteration. The own buckling shape of column no. 2, when the second floor is affected by fire, it is in Fig. 1b.

a)


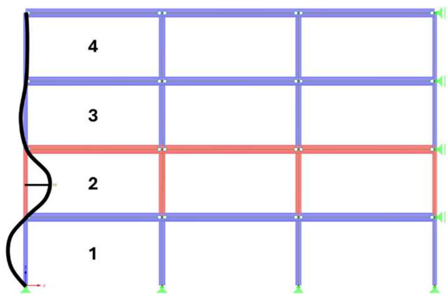
b)


Fig. 1 Buckling shapes of the outer column, a) buckling shape of column at a normal temperature of 20 °C, b) buckling shape of column no. 2 stressed by fire (the 2nd floor is a fire section).

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The calculation was finished when the critical temperature of the rod was the same as in the previous iteration. Calculation of critical temperatures and buckling lengths for columns 1 to 4 and comparison of results can be found in Tab. 1.

The calculation given in Tab. 1 starts with the critical length of the rod at a temperature of 20 °C, which was taken from the model in the RFEM programme. For this buckling length, the calculation of buckling resistance of the rod at normal temperature was carried out according to art. 6.3.1.1 of STN EN 1993-1-1 [2]. Subsequently, the calculation of buckling resistance during fire was carried out for the given rod according to art. 4.2.3.2 of STN EN 1993-1-2 [1].

A critical temperature was sought, at which the design buckling resistance of the bar was equal to the axial force in the element induced by the combination of loads during a fire. With the determined value of the critical temperature, the reduced values of the yield strength and the modulus of elasticity were calculated. Subsequently, for the reduced modulus of elasticity were calculated the critical load factor, critical force, buckling length coefficient and buckling length of the heated rod in the RFEM programme.

It follows from the calculation methodology that the correct buckling length can only be obtained for the critical element, which always becomes a hot rod in the ultimate limit state of load-bearing capacity during a fire.

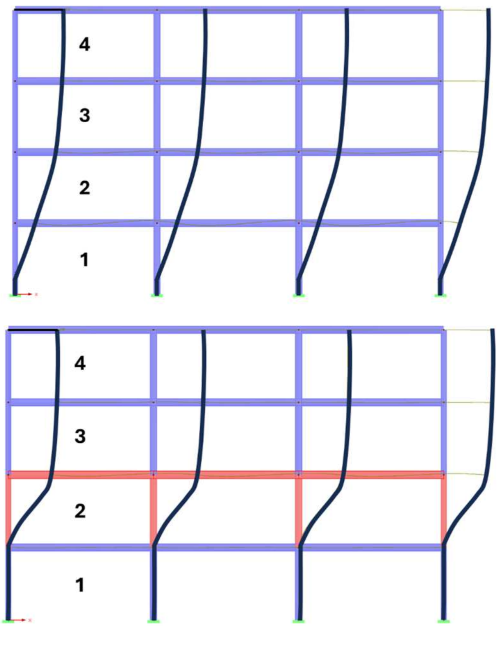
Changes in the buckling length coefficient β depending on the temperature change for the braced frame are shown in Tab. 2.

Un-braced frame

In the case un-braced frame, the structure was also designed for normal temperature with cross-sections of external columns as HEB 240 and internal columns as HEB 300. The cross-sections of all beams were designed as HEA 340. All columns were fixed into the foundation structure and the frame beams were rigidly connected to the columns.

The calculation of the buckling lengths of the rods during fire was also carried out using the calculation programme Dlubal RFEM [8].

Fig. 2 shows the actual buckling shapes, in case a) at normal temperature, in case b) during a fire when the second floor was heated.



a) b)

Fig. 2 Buckling shapes of the outer column, a) buckling shape of column at a normal temperature of 20 °C, b) buckling shape of column no. 2 stressed by fire (the 2nd floor is a fire section).

Calculation of critical temperatures and buckling lengths for columns 1 to 4 can be found in Tab. 3. Results interpreted in Tab. 3 were compared with the recommendations of Couto et al. [7] and with the procedure for calculating the buckling length of the rod at normal temperature according to STN EN 1993-1-1 [2].

The calculation took place in an analogous way as for braced frames.

3 RESULTS

Braced frame

Tab. 1 Calculation of critical temperatures and buckling lengths for a braced frame.

Col. 1	**$N_{fi,Ed}$ [kN]** 721.945	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 0.358									**Comparison**		
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [1]	[5]	[6]
0	*2.868*	*20*	*1.000*	*235*	*1.000*	*210000*	*12.708*	*14362.68*	*0.956*	*2.868*	*0.358*			
1	*2.868*	*581.2*	*0.528*	*124.1*	*0.365*	*76546*	*9.102*	*6562.74*	*0.854*	*2.562*	*0.384*			
2	*2.562*	*588.1*	*0.507*	*119.1*	*0.344*	*72334*	*8.677*	*6256.11*	***0.850***	*2.550*	*0.386*	*0.7*		*0.85*
3	*2.550*	*588.1*												
Col. 2	**$N_{fi,Ed}$ [kN]** 523.151	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 0.301									**Comparison**		
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [1]	[5]	[6]
0	*2.418*	*20*	*1.000*	*235*	*1.000*	*210000*	*24.927*	*20206.04*	*0.806*	*2.418*	*0.301*			
1	*2.418*	*643.9*	*0.365*	*85.7*	*0.231*	*48502*	*11.920*	*6225.15*	*0.698*	*2.094*	*0.328*			
2	*2.094*	*648.5*	*0.354*	*83.1*	*0.223*	*46768*	*11.667*	*6093.04*	***0.693***	*2.078*	*0.326*	*0.5*	*0.61*	*0.7*
3	*2.078*	*648.5*												
Col. 3	**$N_{fi,Ed}$ [kN]** 324.367	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 0.321									**Comparison**		
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [1]	[5]	[6]
0	*2.574*	*20*	*1.000*	*235*	*1.000*	*210000*	*35.975*	*17831.05*	*0.858*	*2.574*	*0.321*			
1	*2.574*	*698.0*	*0.235*	*55.2*	*0.134*	*28038*	*13.100*	*4237.19*	*0.643*	*1.929*	*0.319*			
2	*1.929*	*711.5*	*0.216*	*50.8*	*0.125*	*26335*	*12.468*	*4032.63*	***0.639***	*1.917*	*0.314*	*0.5*	*0.56*	*0.7*
3	*1.917*	*711.5*												
Col. 4	**$N_{fi,Ed}$ [kN]** 125.584	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 0.432									**Comparison**		
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [1]	[5]	[6]
0	*3.465*	*20*	*1.000*	*235*	*1.000*	*210000*	*55.508*	*9839.826*	*1.155*	*3.465*	*0.432*			
1	*3.465*	*834.6*	*0.093*	*21.8*	*0.082*	*17266*	*13.649*	*1701.56*	*0.796*	*2.389*	*0.316*			
2	*2.389*	*853.0*	*0.084*	*19.6*	*0.078*	*16397*	*13.004*	*1621.12*	*0.795*	*2.385*	*0.307*			
3	*2.385*	*853.4*	*0.083*	*20*	*0.078*	*16379*	*12.991*	*1619.45*	***0.795***	*2.385*	*0.307*	*0.7*	*0.81*	*0.85*

The calculated buckling lengths of all columns are slightly greater than the values recommended in STN EN 1993-1-2 [1]. The results are comparable with the relationships for the calculation of the buckling length based on [5] and with the recommendations from BS EN 1994-1-2/NA [6].

Changes in the buckling length coefficient β depending on the temperature change for the braced frame are shown in Tab. 2 and Fig.3.

Tab. 2 Changes in the buckling length coefficient β depending on the temperature change for Column 1 and 2 in braced frame.

θ_a [°C]	**β [-]**			
	RFEM [8]		**EN 1993-1-2**	
	Col. 1	**Col. 2**	**0.5**	**0.7**
20	0.956	0.805	0.5	0.7
100	0.951	0.801	0.5	0.7
200	0.942	0.781	0.5	0.7
300	0.927	0.754	0.5	0.7
400	0.912	0.727	0.5	0.7
500	0.895	0.698	0.5	0.7
600	0.844	0.661	0.5	0.7
700	0.809	0.641	0.5	0.7
800	0.801	0.624	0.5	0.7
900	0.796	0.615	0.5	0.7
1000	0.791	0.607	0.5	0.7
1100	0.786	0.599	0.5	0.7

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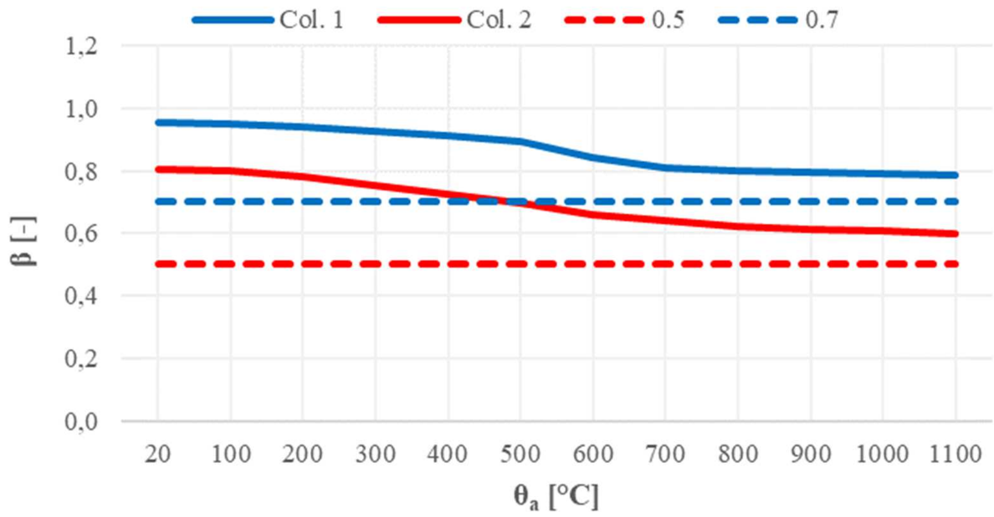

Fig. 3 Changes in the buckling length coefficient β depending on the temperature change for Column 1 and 2.

Un-braced frame

Tab. 3 Calculation of critical temperatures and buckling lengths for a un-braced frame.

Col. 1	**$N_{fi,Ed}$ [kN]** 671.885	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 0.437									**Comparison**	
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [2]	[7]
0	*3.984*	*20*	*1.000*	*235*	*1.000*	*210000*	*14.163*	*13058.11*	*1.328*	*3.984*	*0.437*		
1	*3.984*	*632.6*	*0.392*	*92.0*	*0.251*	*52767*	*6.591*	*4524.49*	*1.200*	*3.600*	*0.464*		
2	*3.600*	*639.2*	*0.376*	*88.4*	*0.239*	*50291*	*6.307*	*4334.34*	***1.197***	*3.591*	*0.465*	*1.328*	*1.000*
3	*3.591*	*639.2*											
Col. 2	**$N_{fi,Ed}$ [kN]** 487.819	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 0.514									**Comparison**	
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [2]	[7]
0	*4.686*	*20*	*1.000*	*235*	*1.000*	*210000*	*14.163*	*9438.75*	*1.562*	*4.686*	*0.514*		
1	*4.686*	*665.6*	*0.313*	*73.5*	*0.192*	*40319*	*6.852*	*3416.91*	*1.207*	*3.621*	*0.477*		
2	*3.621*	*679.7*	*0.279*	*65.5*	*0.167*	*34990*	*6.049*	*3024.91*	*1.195*	*3.585*	*0.479*		
3	*3.585*	*680.9*	*0.276*	*65*	*0.164*	*34530*	*5.978*	*2990.57*	***1.194***	*3.582*	*0.479*	*1.562*	*1.000*
4	*3.582*	*680.9*											
Col. 3	**$N_{fi,Ed}$ [kN]** 301.146	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 0.658									**Comparison**	
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [2]	[7]
0	*6.006*	*20*	*1.000*	*235*	*1.000*	*210000*	*14.163*	*5745.77*	*2.002*	*6.006*	*0.658*		
1	*6.006*	*698.0*	*0.235*	*55.2*	*0.134*	*28038*	*8.065*	*2503.44*	*1.176*	*3.528*	*0.483*		
2	*3.528*	*752.7*	*0.167*	*39.2*	*0.109*	*22871*	*6.713*	*2089.51*	*1.163*	*3.488*	*0.446*		
3	*3.488*	*753.5*	*0.166*	*39*	*0.109*	*22808*	*6.696*	*2084.37*	***1.162***	*3.487*	*0.445*	*2.002*	*1.000*
4	*3.487*	*753.5*											
Col. 4	**$N_{fi,Ed}$ [kN]** 114.402	**L [m]** 3	**$\lambda_{y,20°}$ [-]** 1.108									**Comparison**	
Iteration	**$L_{cr,fi}$ [m]**	**θ_{cr} [°C]**	**$k_{y,\theta}$ [-]**	**$f_{y,fi}$ [MPa]**	**$k_{E,\theta}$ [-]**	**$E_{s,fi}$ [MPa]**	**α_{cr} [-]**	**$N_{cr,fi}$ [kN]**	**β [-]**	**$L_{cr,fi}$ [m]**	**$\lambda_{y,fi}$ [-]**	**β [-]** [2]	[7]
0	*10.11*	*20*	*1.000*	*235*	*1.000*	*210000*	*14.163*	*2027.76*	*3.37*	*10.11*	*1.108*		
1	*10.110*	*777.6*	*0.137*	*32.2*	*0.099*	*20783*	*14.035*	*1539.85*	*1.291*	*3.873*	*0.471*		
2	*3.873*	*902.1*	*0.060*	*14.0*	*0.067*	*14078*	*9.591*	*1050.81*	*1.286*	*3.859*	*0.376*		
3	*3.859*	*902.3*	*0.060*	*14*	*0.067*	*14064*	*9.582*	*1049.78*	***1.286***	*3.859*	*0.376*	*3.370*	*1.000*

The calculated values of buckling lengths from the stability calculation are slightly greater than according to Couto et al. [7]. However, compared to the procedure for calculating buckling lengths for normal temperature according to STN EN 1993-1-1 [2], the calculated values are lower.

Given that columns 3 and 4 are not critical at normal temperature and at the same time have a low degree of utilization, their critical length at normal temperature is unrealistically large. However, in case of fire, they become critical in the ultimate limit state of bearing capacity.

Changes in the buckling length coefficient β depending on the temperature change for the un-braced frame are shown in Tab. 4 and Fig. 4.

Tab. 4 Changes in the buckling length coefficient β depending on the temperature change for Column 1 to 4 in un-braced frame.

θ_a [°C]	β [-]			
	RFEM [8]			
	Col. 1	**Col. 2**	**Col. 3**	**Col. 4**
20	1.328	1.562	2.002	3.370
100	1.321	1.551	1.976	3.217
200	1.304	1.496	1.876	3.056
300	1.288	1.443	1.770	2.886
400	1.273	1.397	1.657	2.703
500	1.259	1.356	1.538	2.507
600	1.214	1.254	1.256	1.328
700	1.158	1.176	1.174	1.297
800	1.135	1.150	1.151	1.290
900	1.118	1.132	1.136	1.286
1000	1.094	1.109	1.120	1.283
1100	1.058	1.074	1.101	1.280

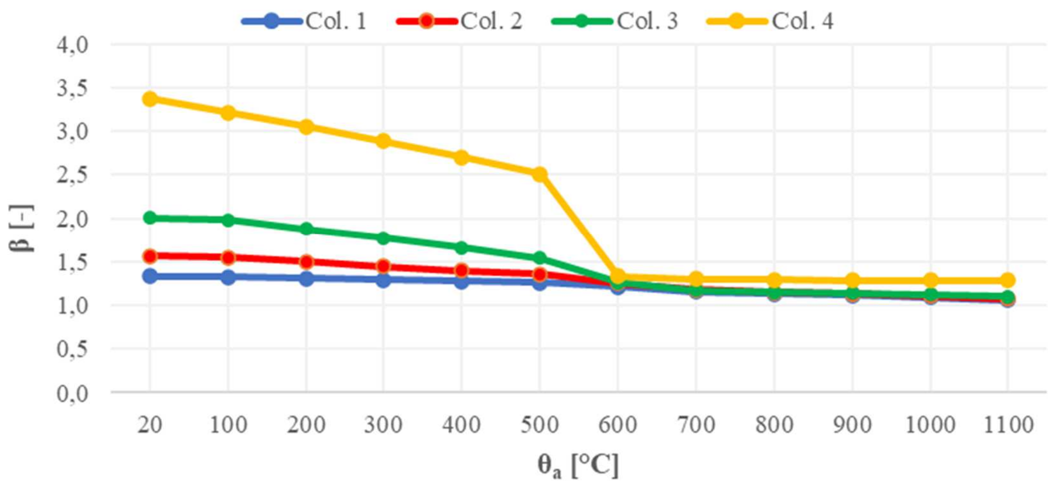

Fig. 4 Changes in the buckling length coefficient β depending on the temperature change for Column 1 to 4.

4 DISCUSSION

The buckling lengths of heated columns are smaller than at normal temperature, this is due to the fact that in a fire situation there are big differences in the bending stiffness of cold and hot parts of the structure. Based on the analysis in this article, but also according to several authors, it was found that the buckling lengths in the case of a braced frame for intermediate/end columns will be approximately *0.7 × L / 0.85 × L* and in the case of un-braced frames approximately *1.15 × L to 1.3 × L.*

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5 CONCLUSION

The protection of the steel supporting structure must be chosen so that its design is safe and economical. Therefore, it is necessary to choose the right method of analysis of the structure stressed by fire for a sufficiently accurate determination of the buckling lengths of the columns. According to STN EN 1993-1-2, it is possible to analyse the elements individually. However, from the works of several authors as well as from the results presented in this article, it is clear that the analysis of the entire structure is more appropriate not only in the case of braced frames, but also in the case of un-braced frames.

Based on the results of the performed calculation, it is clear that the rules for the design of steel structures according to the set of STN EN standards do not always take into account all design situations. Therefore, it is necessary to further investigate this issue and consider the influence of other factors, such as the temperature gradient at the ends of the columns and/or the prevented thermal elongation of the columns.

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