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Elevated Temperature Effect on the Dynamic Characteristics of Steel Columns and Frames

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Abstract

Fire performance of load bearing elements in steel buildings such as columns and frames has major importance for structural designers. This study aims to assess the elevated temperature effect on dynamic characteristics of steel columns and frames by conducting sequential heat transfer and modal analyses. A series of finite element analyses including heat transfer analysis aere performed on 62 different steel columns and frames. Three-hour part of ISO 834 fire curve is taken into consideration in the analysis. Modal analyses are conducted for the purpose of providing a more comprehensive image of dynamic characteristics in specific duration of elevated temperature. The presented study accommodates the effect of various steel profile types, cross-sectional dimensions and exposure durations on changes in dynamic characteristics. The outcomes of the performed parametric study revealed the decrease in natural frequencies with the temperature growth. The research has also shown the existence of a correlation between temperature variations and changes in mode shapes. The change in mode shapes depends on the profile type. Mode shapes do not change depending on the temperature in some profile types, while in some profile types they change after a certain temperature. The obtained results of the performed sequential analyses are presented in forms of tabulated data and approximate formulas. They can facilitate the damage detection process and contribute in required structural health monitoring measurements while enhance the accuracy of damage assessment for fire exposed steel structures.

Keywords Column · Dynamic characteristic · Elevated temperature · Finite element method · Frame · Steel

Abbreviations

α	Thermal diffusivity (m^2/s)
λ	Thermal conductivity (W/mK)
ρ	Density (kg/m^3)
c	Specific heat (J/kg K)
θ	Steel temperature (°C)
E	Modulus of elasticity (Pa)
$k_{E,\theta}$	Reduction factor at elevated temperature θ relative to the value of E at 20 °C for the slope of the linear elastic range (–)
$\dot{h}_{net}$	Net heat flux to unit surface area (W/m^2)
α_c	Coefficient of heat transfer by convection (W/m^2 K)
θ_g	Gas temperature in the fire compartment or vicinity of the fire exposed member (°C)
θ_m	Surface temperature of the member (°C)
Φ	Configuration factor (–)
ε_m	Surface emissivity of the member (–)
ε_f	Emissivity of fire (–)
σ^{sb}	Stephan Boltzmann constant (5.67×10^{-8}) (W/m^2 K^4)
θ_r	Effective radiation temperature of the fire environment (°C)
f_i	I. Natural frequency (Hz)

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X Section size specification for nominal dimensions according to profile type identification (mm)
Y Thickness of square hollow section (mm)
T Duration of exposure (s)

1 Introduction

Structural analysis under fire action is highly important topic due to fire damages incurred to both occupants and structures. For this reason, many private and governmental organizations have focused on the topic of structural fire resistance using report cases, damage assessments, manuals, and associated standards. Consequently, numbers of fire provisions have been published with the main aim of saving people's live and in order to secure the structural fire safety of buildings. One of the important measurements in this regard has been conducted by US national fire protection association (NFPA), publishing reports on annual fire cases with details of their damage to the people life and structures. Based on NFPA's data, recorded between 2013 and 2017, the average numbers of 354,400 residential fires have been reported per year. These fires cases caused an annual average of 2620 human casualties and 11,220 fire injuries, as well as annual average of \$6900 million economic loss (Ahrens, 2019). In the same way, fire analysis published by Ontario of Canada indicates that most of the fire fatalities in Canada are taking place in residential structures (Statistics Canada, 2020).

Due to high impact of fire damage to both people lives and buildings, it is crucial to assess structural carrier members using elaborate structural models which enable design solutions with higher structural performance under the elevated temperature. In another words, the specific fire behavior of each structural member and pattern of temperature development throughout those members are required to be studied. Since, at each fire developmental phase, the associated member response at each stage would vary from other heat development stages. So, the accurate assessment of structural members' behavior at each stage enables more efficient structure solution which offers higher structural durability under the fire load. Therefore, a structural design based on the result of this kind of analysis can have a significant result in structural member design as well as in the efficiency of overall building structural design, in terms of its durability and structural resistance under the fire.

To achieve this kind of structure model, it is necessary to recognize the fire progression phenomena at different points of the structural member. Then, based on the model response, the subsequent load effects on structure can be predicted. In order to model the heat development phases in the structural member, five fundamental fire development phases must be considered. These stages are:

- Ignition, which is initial stage of fire initiation
- Growth, where speed of fire growth is slow
- Flashover, which starts with compartment temperature of approximately 600°C. Sudden transition from developing fire to fully developed fire occurs
- Fully developed or post-flashover, with compartment temperature of generally more than 1000°C, and maximum amount of fire load is applied
- Decay or cooling stage, where intensity and amount of fire load starts to decrease

Among the five mentioned phases of fire development, the post-flashover stage is the sate that maximum heat has fully developed, and the exposed members are under maximum fire load. So, the appropriate structural design against the fire load at this fully developed stage has a crucial importance. Hence, the fire structural designs are mostly conducted based on this stage of fire development. At this point, finding the best structural design, which can prevent the collapse of the structure or at least increase the overall durability of structural member, and therefore provides enough evacuation time for occupants and operations of firefighters, is a challenge for structural engineers.

In response to the above demand for an accurate evaluation of heat development stages, number of fire codes and standards such as ASTM, AISC, ISO have been issued. One of the well-developed fire standards in this regard, is EN 1991-1-2 (2002). These fire codes are offering variety of fire representations for modelling the fire scenario based on different applications. One of the major representations offered by EN 1991-1-2 is natural and nominal fire representation. In this context, natural fire models consideres all five stages of fire developments and can be used for calculation application and is based on explicit physical parameters of the building. Contrariwise, nominal fire models just cover the post-flashover stage and does not requires specific details such as characteristics of fire load, specific thermal properties of building's compartment linings and ventilation condition as an input (Lennon et al., 2007). According to the guideline provided by EN 1991-1-2, the following steps need to be considered in order to perform fire design analysis for structural elements (Fig. 1).

Based on the above graph, following the Eurocode guideline, for selection of appropriate design fire for the fully developed fire stage, when the building experiences the maximum fire load at the post-flashover stage, the code offers the choice between nominal and natural fire models. This study selects the nominal fire model, which have wider applications and enables the classification and comparison of generated study models. Additionally, the appropriate fire curve illustrating the rate of temperature development over time is provided by the fire provisions such as EN 1991-1-2. According to the selected fire scenario, the associated

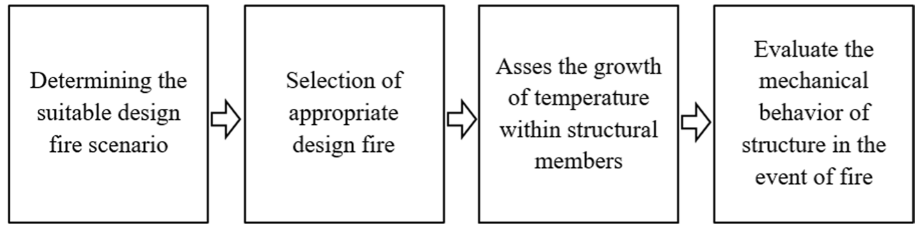


Fig. 1 Structural fire design procedure

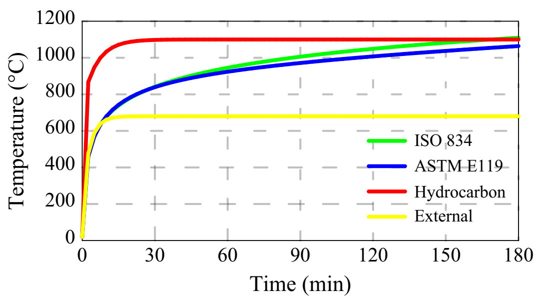


Fig. 2 Nominal temperature–time curves (ASTM E119, 2012; EN 1991-1-2, 2002; ISO834-1, 1999)

nominal fire curve is also taken from the standard fire code in order to illustrate a simplified model of fire using time–temperature relationship. So, based on the curve representation, the time dependent temperature of fully developed fire can be derived for the purpose of analysis of structural member at the associated temperatures over the time.

In the present study, the conventionally used ISO 834 (1999) nominal fire curve has been adopted to evaluate the fire resistance of structural elements. Figure 2 illustrates the various types of nominal temperature–time curves provided in different standards. According to this curve, after an initial sudden jump in temperature, by increasing the fire exposure time, the temperature continues with a constant rate. Besides, since no cooling stage is described in nominal fire curves, it is to say that the decay stage of fire is also neglected in these fire curves.

The other important aspect of structural modelling under of the fire load, is the type of structural material used as the main determinant factors in prediction of the fire behavior of the structure. One of the commonly used materials in construction industry is steel which is considered as a readily available structural material. Steel with various applications in beam, column, framework, etc. plays an important role in structural behavior. This material is well known for good seismic behavior and having natural properties such as durability, high strength and being 100% recyclable. Despite having many advantages, the drawback of steel is its high thermal conductivity which changes its thermal resistance in high temperature so that steel members will have lower thermal resistance in the event of fire (Agustini et al. 2017). The fact of being vulnerable in fire makes restriction in use of steel in construction. The melting point of steel is around 1500 °C and due to its non-combustible nature, it is categorized as A1 material in combustibility classification. However, the elevated temperature causes some losses in strength and stiffness of steel which have effect on buckling capacity. It is also noteworthy that variables such as test method, material properties and rate of heating have all effect on the amount of loss in strength of steel and its reduction factor. From all methods, it can be seen that in temperature between 400 °C to 700 °C steel experience a significant loss in its strength and stiffness (Smith, 1987). According to EN 1993-1-2 (2005), at the time which temperature exceeds 600 °C, steel experiences a reduction of 70% in its elasticity modulus and more than 50% reduction in its yield strength. Steel retains 20% strength and stiffness when it reaches to 700 °C. In traditional fire design method, temperature of 550 °C is generally considered as critical temperature. However, there is no specific failure temperature for steel members. The shape of stress–strain curve of steel is also affected by elevated temperature and becomes highly non-linear. These changes in mechanical properties of steel can result in excessive deformation and collapse of the structure (Ho, 2010).

So, in order to evaluate these changes and predict the amount of losses in strength and stiffness of steel structure, obtain the optimum design and minimize the cost of failure, finite element (FE) method is generally used. FE analysis with advantage of faster computation of results with good level of accuracy, give beneficial insights into thermal behavior of steel element and facilitates the procedures of evaluation, design and retrofitting of structures against fire actions. As dynamic characteristics (natural frequency, mode shape, and damping ratio) of structure are directly linked to its mass and stiffness, it is required to study the temperature dependent variations in dynamic properties of structure to make a proper evaluation.

How steel structures are behaving under the elevated temperature, and what is the characteristics of damaged structure, has introduced noteworthy challenges to engineers and researchers. Evaluation of structural systems under fire has been subject of many studies during the past decades. Accordingly, there exist many studies on investigation of responses of steel structures in the event of fire.

Many of these studies have considered various aspects of their mechanical properties and structural behavior under elevated temperature. The following table highlights few notable literatures on the present topic (Table 1).

The studied literatures, including those mentioned in the above table, not only were successful in developing the behavioral analysis of various steel structural members under the elevated temperature, but they also have considered the other factors which would have high contribution to the results of associated analyses. Zhang et al. (2015) have discussed the non-uniform nature of the fire and its associated load on the building structure and raising the question of credibility of Eurocode provision which specifically consider uniform distribution of fire. Furthermore, the analysis results of Zhang's study indicate that using of uniformly distributed load according to Eurocode guidelines offers designs which lay on a more conservative spectrum in compare to actual non-uniform behavior of fire. Thus, in the present study while using the Eurocode provision, the actual non-uniform behavior of fire has been regarded as uniformly distributed fire.

Due to load bearing importance of frame system as well column and beam configurations in overall durability of steel structures, the majority of the literatures have narrowed their study on frame, column and beam structural components. Jiang and Li (2017) have outlined the importance of disproportionate collapse behavior of steel frame due to compartment fire using a software generated 3D model. Additionally, the studies have also explored the influences of various factors such as potential locations of fire load. Subsequently, the results of the studies note that the location of fire load has an important effect on the resistance duration of steel frame under the fire. The perimeter fire load causes more structural damage on steel frame when compares to the localized fire scenario. Due to high hazard impact of the perimeter fire on collapse behavior of steel structure, the present study considers perimeter fire as part of its performed analysis. Similarly, Lou et al. (2018) have numerically and experimentally explored the collapse pattern of steel portal frame under the fire load and validated their numerical model using experimental results. Also results display good agreement with EN 1993-1-2, which also has been used in this study.

Delgado Ojeda et al. (2016) have narrowed down their study on flexural behavior of steel columns due to thermal gradient phenomena under elevated temperature using numerical analysis. The analysis results have been used as the basis of new refined design model which covers the effect of thermal gradient on column eccentricity. Likewise, Fan et al. (2018) have carried out the fire test on eccentrically loaded steel columns by performing experimental and analytical analysis. Important part of their results indicates the good agreement between temperature–time curve of fire furnace and ISO 834 standard fire curve. Yang et al. (2020)

Table 1 Literature reviews and motivations

Authors	Year	Method	Type of structure
Heidarpour and Bradford (2009)	2009	Numerical	Steel beam
Kodur and Dwaikat (2009)	2009	Numerical	Steel beam and column
Lee et al. (2011)	2011	Numerical	Steel beam-to-column connection
Iqbal and Harichandran (2011)	2011	Numerical	Steel column
Ahn et al. (2013)	2013	Numerical	Steel beam
Sun et al. (2014)	2014	Numerical	Steel frame
Wang et al. (2015)	2015	Experimental	HS Q460 steel
Piroglua et al. (2017)	2017	Experimental	Steel members in ındustrial building
Rackauskaite et al. (2017)	2017	Numerical	Steel frame
Parthasarathi et al. (2018)	2018	Numerical	Steel frame
Shakil et al. (2018)	2018	Numerical	Steel beam and frame
Winful et al. (2018)	2018	Numerical	Steel column
Huang and Young (2019)	2019	Numerical	Steel column
Li and Young (2019)	2019	Numerical	Steel beam
Pournaghshband et al. (2019a)	2019	Numerical	Steel column
Pournaghshband et al. (2019b)	2019	Numerical	Steel beam
Kucukler (2020)	2020	Numerical	Steel column
Kucukler et al. (2020)	2020	Numerical	Steel I column
Laím et al. (2020)	2020	Experimental	Steel column
Ren et al. (2020)	2020	Experimental	Cold-formed Q235 steel
Segura et al. (2021)	2021	Numerical	Steel frame
Shi et al. (2021)	2021	Numerical	Steel column

have concentrated on failure mode of steel columns under the fire and found out that this failure mode is combination of local and global flexural buckling. Additionally, their study has highlighted some of the factors affecting the overall failure time of steel columns such as the fire load ratio. Nonetheless, the majority of literatures have deliberate analysis on structural and mechanical behavior of fire exposed steel structures. But relatively fewer amounts of studies have performed about the modal analysis when evaluating the fire behavior of structure.

Although it has been observed that many studies have been carried out by many authors regarding the fire behavior of steel members, it seems that the structural behavior of steel members is still not adequately investigated when it comes to modal analysis. Ma et al. (2017) have proposed a new analytical method which comprises the modal analysis of a steel beam having multiple transverse open cracks under various temperature cases. The author has verified the results using FE method, and further argued that consideration of temperature load as part of the modal analysis provides results which are closer to actual fire situation.

Currently, the researches compromising both heat transfer analysis and dynamic response of structural members are still limited and only one literature have found focusing on numerically investigation of high temperature effect along with assessment of change in of dynamic characteristics of steel structures. In this context, Patil and Ramgir (2016) have conducted a thorough experimental and theoretical analysis about the behavior of a structurally loaded steel member subjected to elevated temperature. In their study, the authors have investigated the effect of varying cross-section and boundary condition of a loaded steel member under the fire load during both laboratory and numerical analysis. Moreover, the result of their numerical analysis is based on performing staged heat transfer and modal analysis on the four steel member models.

As mentioned above, to the knowledge of this paper no previous study has clearly examined the results of fire effects on steel column and steel frame using both heat transfer and modal analysis. Therefore, considering the direction of the current literature on the topic of thermal behavior of steel members, this paper addresses the topic of steel structural performance under the fire load. In addition, columns and frame systems have an important role in overall load bearing of most types of building structures. Besides, due to importance of these elements, the local failure of them can lead to serious damages to structure and the whole structure might collapse (Chandrasekaran & Nagavinothini, 2020). Therefore, this paper has focused on structural behavior of columns and frame systems as two arguably most crucial building members with the special attention on effect of elevated temperature on changes in their dynamic characteristics. The staged heat transfer and modal analysis have been conducted using ABAQUS software (ABAQUS, 2016). Additionally, the effect of varying steel sections, cross-sectional dimensions and exposure durations on changes in dynamic characteristics have been evaluated by using 62 different FE models.

2 Variable Parameters

In order to have a thorough evaluation on effect of elevated temperature on dynamic characteristics of steel columns and steel frames, variable parameters have been considered in this study. In this context, different steel profile types, cross-sectional dimensions and exposure durations have been considered. Hence, steel H-section and square hollow section (SHS) have been adopted as profile types. Moreover, columns and frames with various cross-sectional dimensions have been considered which is detailed in Table 2. All members of each frame configuration are assumed to have same profile type and cross-sectional dimensions.

As it is shown in Table 2, in order to have a precise evaluation on effects of changes in cross-sectional dimensions, a total of fifteen HEA and sixteen SHS steel profiles with different cross-section dimensions have been considered. Thus, a total of 62 different FE models, including 31 column models consisting of fifteen HEA and sixteen SHS, as well as 31 frame configurations have been constituted.

As another important parameter, fire exposure duration, which is defined as the time period which structural elements are exposed to fire, have been considered. In the heat transfer analysis, generated models have been exposed to fire according to ISO 834 temperature–time curve. In order to evaluate the temperature development, heat transfer analysis

Table 2 Profile types and dimensions

HEA	Square hollow section
100	100 × 100 × 10
120	120 × 120 × 10
140	140 × 140 × 10
160	160 × 160 × 10
180	180 × 180 × 10
200	200 × 200 × 20
220	220 × 220 × 20
240	240 × 240 × 20
260	260 × 260 × 20
280	280 × 280 × 20
300	300 × 300 × 30
320	320 × 320 × 30
340	340 × 340 × 30
360	360 × 360 × 30
—	380 × 380 × 30
400	400 × 400 × 40

with total three hours of fire exposure have been performed for each model.

Afterwards, the results of these analyses were used to perform the modal analyses with certain exposure durations. Accordingly, for modal analysis of each model, a total of thirteen different exposure durations 0 (initial condition, there is no fire), 15, 30, 45, 60, 75, 90, 105, 120, 135, 150, 165 and 180 min have been considered. Furthermore, according to the results obtained from each step of modal analyses, changes in dynamic characteristics affected by exposure durations have been evaluated.

3 Material Properties of Steel at Elevated Temperatures

Nowadays, by taking advantages of FE softwares, it is possible to consider the temperature dependent material properties in the performed analysis which lead to have analysis with high precisions and more realistic results. In this context, effect of temperature on material properties of steel are detailed in EN 1993-1-2 which are considered in the performed FE analysis in this study. Density of steel is not significantly affected by increasing of temperature and just shows minor decrease in its value comparing to ambient temperature (The Institution of Structural Engineers, 2003). Therefore, as it is suggested by EN 1993-1-2 and EN 1994-1-2 (European Committee for Standardisation (CEN) 2005), the constant value of 7850 kg/m^3 have been considered in all stages of the performed analyses. Similarly, poisson's ratio has minor changes in high temperature and is generally recognized to be ineffective from fire. So, poisson's ratio with constant value of 0.3 has been used in the present study (Phan et al. 2010).

The thermal properties of steel might consider independent from quality of steel (Twilt & Both, 1994). In order to assess the effect of temperature on specific heat and thermal conductivity of steel, expressions provided by EN 1993-1-2 can be used. By using these expressions, which are applicable for both structural and reinforcement steel classes, two related behavioral models have been generated which are illustrated in Fig. 3.

As it is illustrated in the Fig. 3, specific heat of steel reaches to its peak value at temperature 735 °C, which is due to occurance of metallurgical phase change inside the steel. At the same time, up to temperature of 800 °C, increase in temperature causes some losses in thermal conductivity of steel, and as temperature exceeds 800 °C, thermal conductivity continues with a constant value.

Thermal diffusivity is defined as the ability of material to transmit the heat from regions with higher temperature to regions with lower temperature and is directly related to the rate of temperature growth in a material. The higher thermal diffusivity results in more rapid temperature growth in certain depth in the material (Hurley et al. 2016). Thermal conductivity, specific heat and density all have correlation with thermal diffusivity. So, since thermal conductivity and specific used in this study are temperature dependent, thermal diffusivity is also temperature dependent. To calculate the value of thermal diffusivity, the following equation can be used.

$$\alpha = \frac{\lambda}{\rho c}\left(\mathrm{m}^2/\mathrm{s}\right) \tag{1}$$

Likewise, mechanical properties of steel are also affected by high temperature. Some of these properties such as modulus of elasticity and strength play an important role in structural behavior. When steel is exposed to fire, the values of strength and modulus of elasticity start to decrease as the results of rise in temperature. It is noteworthy that there is slight difference between the proposed approaches by codes and the available actual test data. Even though modulus of elasticity and yield strength are both decreasing with similar incline, there will be complications in calculation of the cases which they won't reach to zero value at same temperature. So, for ease of calculations, EN 1993-1-2 provided a range of reduction factors which can be used for nominal strength and stiffness of steel and is applicable for temperatures upto 1200 °C (Buchanan & Abu, 2017).

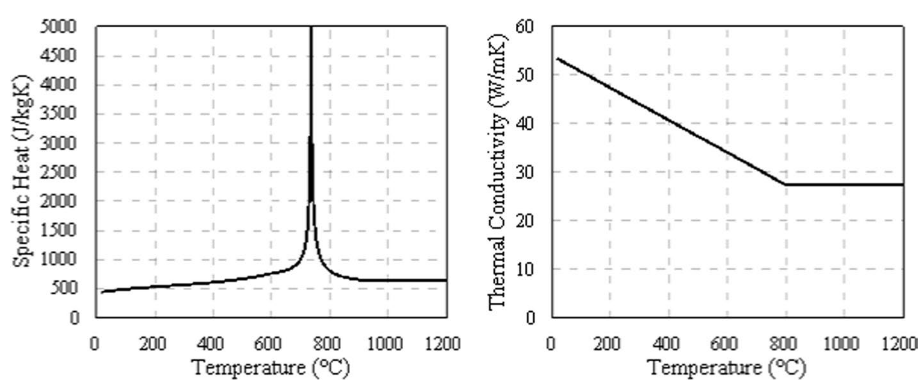


Fig. 3 Specific heat and thermal conductivity of steel at elevated temperatures (EN 1993-1-2, 2005)

In this study, to calculate the losses in modulus of elasticity which occures due to growth in temperature, the reduction factors provided by EN 1993-1-2 have been used. Accordingly, by multiplying the modulus of elasticity at ambient temperature (2.10E11Pa) to the reduction factor of the respective temperature, the modulus of elasticity at the certain temperature can be obtained. These reduction factors along with their corresponding temperatures are provided in Table 3.

4 Finite Element (FE) Analyses

The present study aims to numerically analyze the changes in dynamic characteristics of steel columns and frames as one of the most important load bearing elements under elevated temperatures. ABAQUS FE analysis software has been used to generate the models and conduct the required analyses. The details of the all considered variable parameters are mentioned in Sect. 2. According to the variation in profile types and cross-sectional dimensions, a total number of 62 different FE models are required to be generated. To carry out the numerical analyses, FE models with constant height of 3 m have been constituted. Moreover, all frame systems have been modeled as plane frame with single-story and single bay with a constant width of 3 m. As mentioned earlier, same profile type and cross-sectional dimension have been considered for all members of the modeled frame systems.

Based on the objective of this study, sequential analyses consisting of heat transfer analysis and modal analysis have been conducted. Each of the 62 different generated models has undergone a separate heat transfer analysis. Each heat transfer analysis includes fire exposure duration of three hours, which are divided into time steps with fixed increment size permited by ABAQUS. The number and sizes of these increments are selected according to various exposure durations considered another variable parameter. These various exposure durations have effect on the number of required modal analysis. Consequently, each model has undergone a total number of thirteen modal analyses with basis of these exposure durations. In this context, based on the considered parameters, the total numbers of 868 analyses consisting of 62 heat transfer analyses and 806 modal analyses have been performed. In all performed analyses, material properties detailed in Sect. 3 have been used.

4.1 Heat Transfer Analyses

As the first stage of the mentioned sequential analysis, all generated models have undergone transient heat transfer analysis procedures. There need three thermophysical properties, namely, density, thermal conductivity and specific heat to be defined as analysis input in order to perform the required FE analyses. The temperature dependent variations in value of these properties, along with their corresponding temperatures are listed in the table below (Tables 4, 5).

In the performed heat transfer analyses, column models have been analyzed according to a scenario which all exterior sides of the columns are being exposed to fire. In the same way, all frame systems have been analyzed pivoting one scenario which all exterior surfaces of frames are being exposed to fire. In all heat transfer analyses, the ISO 834 standard fire curve has been used as the representative of fire to illustrate the fire action on surfaces of the models. So, these analyses have been performed based on a three hours segment of this temperature–time curve. Accordingly, the following specifications have been adopted to perform the heat transfer analysis using ABAQUS FE software.

All FE analyses have been conducted according to three main mechanism of heat transfer, namely, conduction, convection and radiation. Through conduction mechanism, heat is being diffused within solid body. Additionally, by combination of convection and radiation, heat is transmitted from exterior environment to surfaces of the element (Buchanan & Abu, 2017). In this regard, to determine the net heat flux on the exposed surfaces of the models, the following equation can be used. Since the radiation portion of this equation is defined based on temperature unit of K, therefore, the absolute zero temperature (−273.15 °C) and Stephan Boltzmann constant of 5.67×10^{-8} W/m^2K^4 were defined in ABAQUS to resolve the unit compatibility issue.

$$\dot{h}_{net} = \underbrace{\alpha_c \cdot (\theta_g - \theta_m)}_{\text{Convection}} + \underbrace{\Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma^{sb} \cdot \left[(\theta_r + 273)^4 - (\theta_m + 273)^4 \right]}_{\text{Radiation}} \tag{2}$$

According to the various exposure durations which considered in this study, the exposure duration of three hours has been divided into twelve identical steps with duration of fifteen minutes each. Consequently, the nodal temperatures have been obtained for each step. Moreover, considering the results obtained from each step of various exposure durations, the changes in surface temperature as well as temperature development within the model have been evaluated (Figs. 4, 5).

Heat transfer is defined as transfer of energy which is caused by temperature difference. When there is temperature difference between two locations in a solid, heat flows by conduction (Forsberg, 2020). In the exposure scenario which is considered in performed heat transfer analyses for column models, all exterior sides of the columns uniformly exposed to fire. Accordingly, there were no temperature difference between the nodes which are positioned on axes parallel to height of the column and heat conduction has been acted in two dimensions. Therefore, heat has been flowing across the cross-sections which are perpendicular to the column

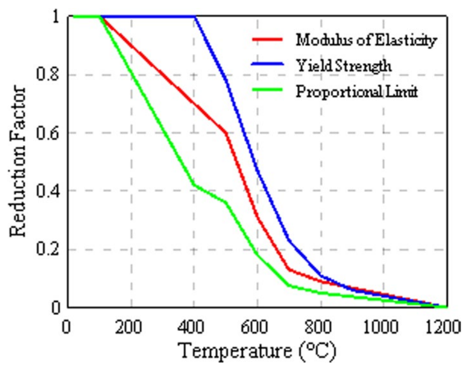


Fig. 4 Reduction factors for stress–strain relationship of steel at elevated temperatures (EN 1993-1-2, 2005)

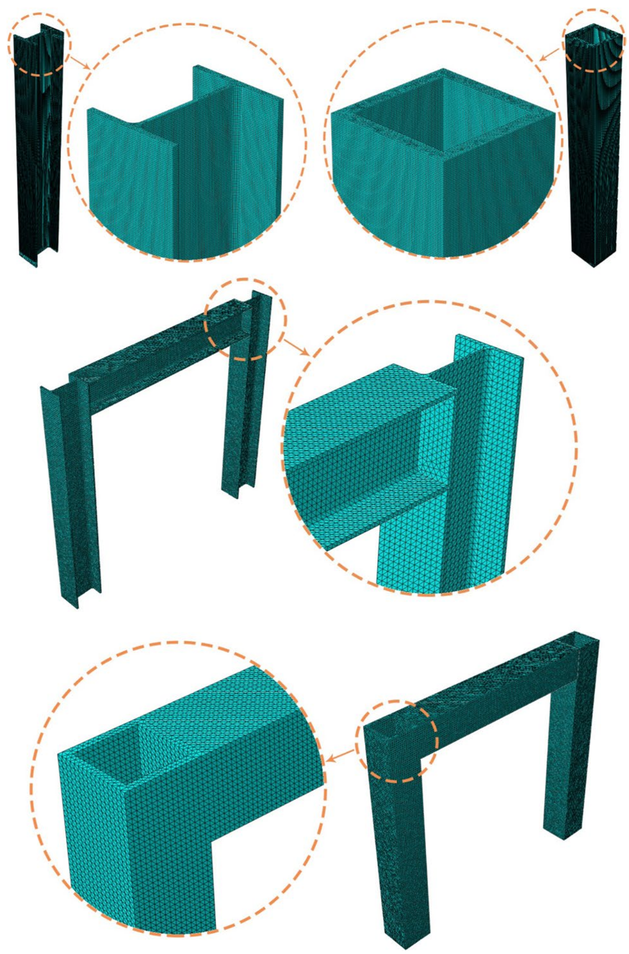


Fig. 5 Columns and frame systems layout

height. Consequently, all cross-sections along height of each columns experience same temperature distributions.

Since the transient heat conduction has been considered in heat transfer analyses, each node experiences different temperatures according to their distance from the exposed surfaces. Therefore, as a result of performing transient

Table 3 Temperature values and reduction factors related to elasticity modulus of steel (EN 1993-1-2, 2005)

Steel temperature, θ (°C)	Reduction factor for elasticity modulus, $k_{E,\theta}$	Steel temperature, θ (°C)	Reduction factor for elasticity modulus, $k_{E,\theta}$
20	1.000	700	0.130
100	1.000	800	0.090
200	0.900	900	0.0675
300	0.800	1000	0.0450
400	0.700	1100	0.0225
500	0.600	1200	0.0000
600	0.310		

Table 4 Material properties of steel used in heat transfer analyses

Temperature (°C)	Material properties		
	Density (kg/m^3)	Thermal conductivity (W/mK)	Specific heat (J/kg K)
0	7850	54	425
20	7850	53.334	439.80
100	7850	50.67	487.62
200	7850	47.34	529.76
300	7850	44.01	564.74
400	7850	40.68	605.88
500	7850	37.35	666.50
600	7850	34.02	760.22
700	7850	30.69	1008.16
735[a]	7850	29.525	5000[a]
800	7850	27.3	803.26
900	7850	27.3	650
1000	7850	27.3	650
1100	7850	27.3	650
1200	7850	27.3	650

[a]See Fig. 3

Table 5 Specifications defined in ABAQUS for heat transfer analysis

Type of model	Element type	Approximate global mesh size (mm)	Initial temperature (°C)	Convection heat transfer coefficient (W/m^{2}K)	Emissivity of the steel surface
Column	DC3D8	5	20	25[a]	0.7[b]
Frame	DC3D10	25			

[a]Adopted from EN 1991-1-2

[b]Compatible with ISO 834 fire curve

heat transfer analyses, non-uniform temperature distributions have been occurred within the cross-sections of the elements.

As mentioned above, in this study, detailed heat transfer analysis is conducted on each of the 62 generated FE models. So, in order to better illustrate the results of analysis, two different cases of column model, with largest cross-sectional area of each profile type, have been selected as an example. For this purpose, the temperature distributions within cross-sections at the end of 30 min intervals are presented in Figs. 6 and 7.

As it is depicted in above figures, temperature is distributed symmetrically within cross-sections of these columns, which is due to uniform exposure of fire within all exposure

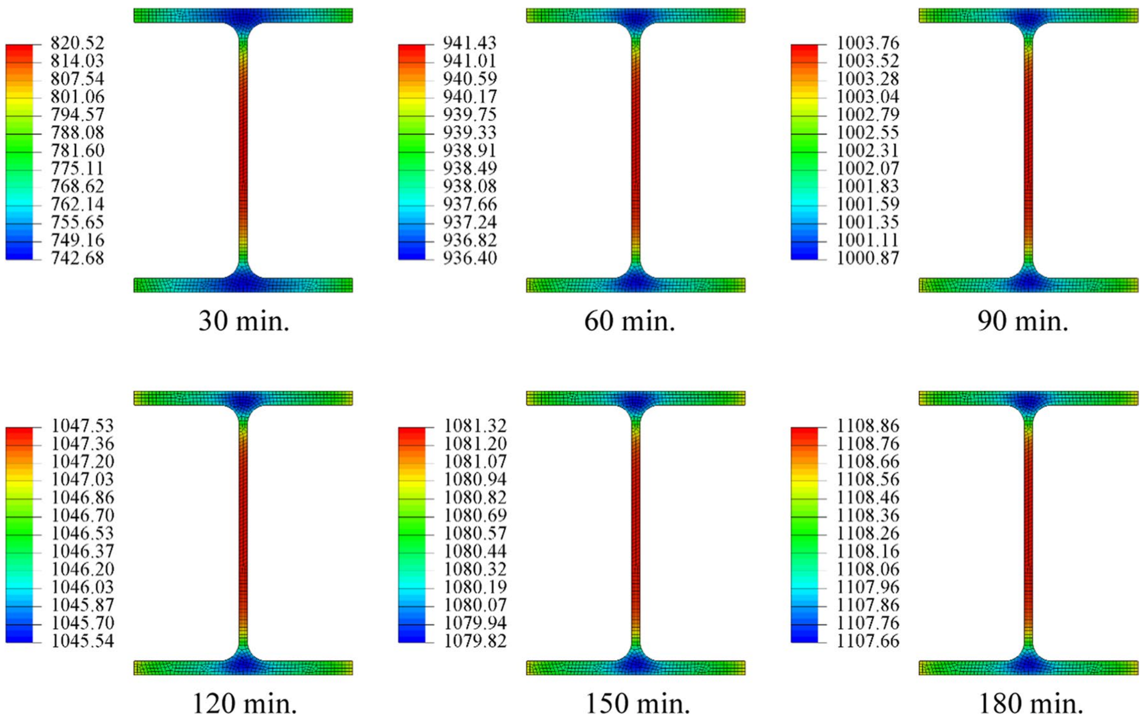


Fig. 6 Temperature distributions within cross-section of HEA400 column model

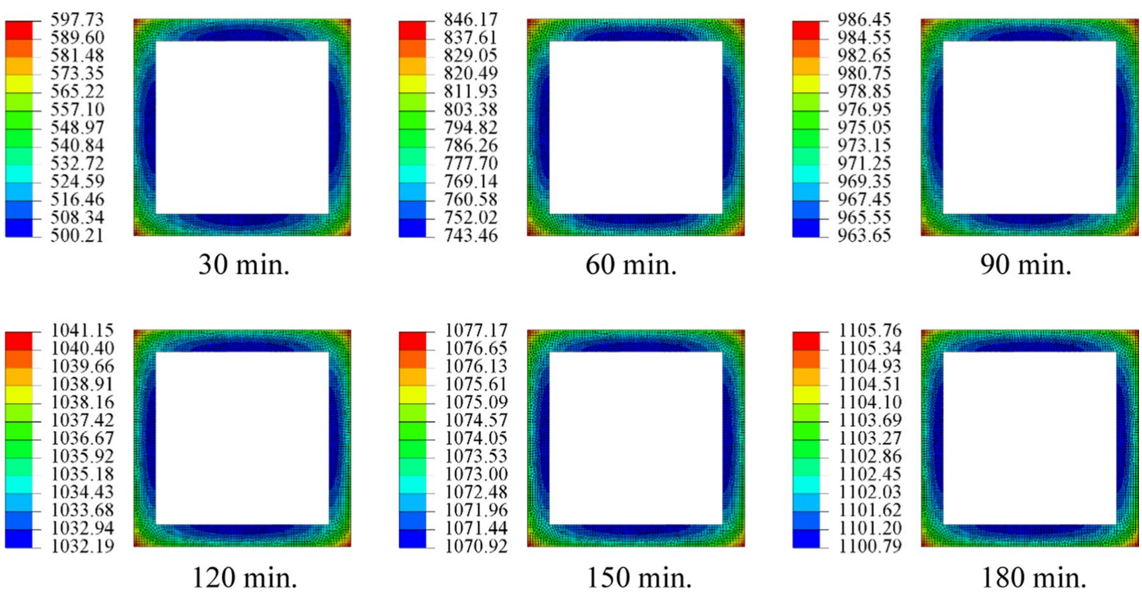


Fig. 7 Temperature distributions within cross-section of SHS 400 × 400 × 40 column model

period. Moreover, to have better evaluation on nodal temperature growth, some nodes at various depths of cross-sections have been selected from both models. These selected nodes and the growth in their temperature are presented in Fig. 8. Since the temperature is distributed symmetrically within cross-sections, all nodes have been selected from a quarter part of each cross-section.

Although columns models which have relatively larger cross-sectional area have been considered, yet it can be seen that both columns are showing similar pattern of growth in their nodal temperature. This result can be explained by the fact that steel is a good thermal conductor and is sensitive to high temperature. Therefore, in cases which unprotected steel members are being exposed to fire, significant structural damages might occur. Attention should be paid to the importance of applying damage protection methods to prevent these possible damages.

4.2 Modal Analyses

Following completion of heat transfer analysis stage, a new similar set of 62 models have been generated for modal analysis. Consequently, each of these models has undergone modal analyses on basis of various exposure durations which have been considered in this study. In the analysis, fixed boundary condition is taken into consideration at the base of the columns.

Modal analysis has been performed based on the results obtained from heat transfer analysis. In order to use the results of the heat transfer analyses as basis of modal analyses, each model should adopt a mesh size and element type which is compatible with heat transfer analysis stage. Thus, to perform the modal analyses, a mesh size equal to heat transfer analysis has been adopted. Similarly, C3D8R and C3D10 element types have been used for columns and frames to generate the required models, respectively. Thus, selection of the material properties considered in modal analyses has been done according to the nodal temperatures obtained from heat transfer analyses.

The required material properties to perform modal analysis are density, poisson's ratio and modulus of elasticity. In this study, all of the material properties used in modal analyses is constant except modulus of elasticity which is considered to be temperature dependent. In order to define the temperature dependent values of elasticity modulus, Table 3 has been used. To obtain the value for modulus of elasticity at certain temperatures, the values at ambient temperature is multiplies by reduction coefficient of these temperatures. The input parameters for material properties with their relative temperatures are listed in Table 6.

After performing 13 modal analyses on each model, the dynamic characteristics have been obtained at every 15 mins interval of 3 h fire exposure. Based on these results, the effects of fire (elevated temperature) on changes in dynamic characteristics have been determined. Moreover, the effects of the considered variable parameters on these changes have been also evaluated. In order to demonstrate the results obtained from modal analyses on 62 FE models,

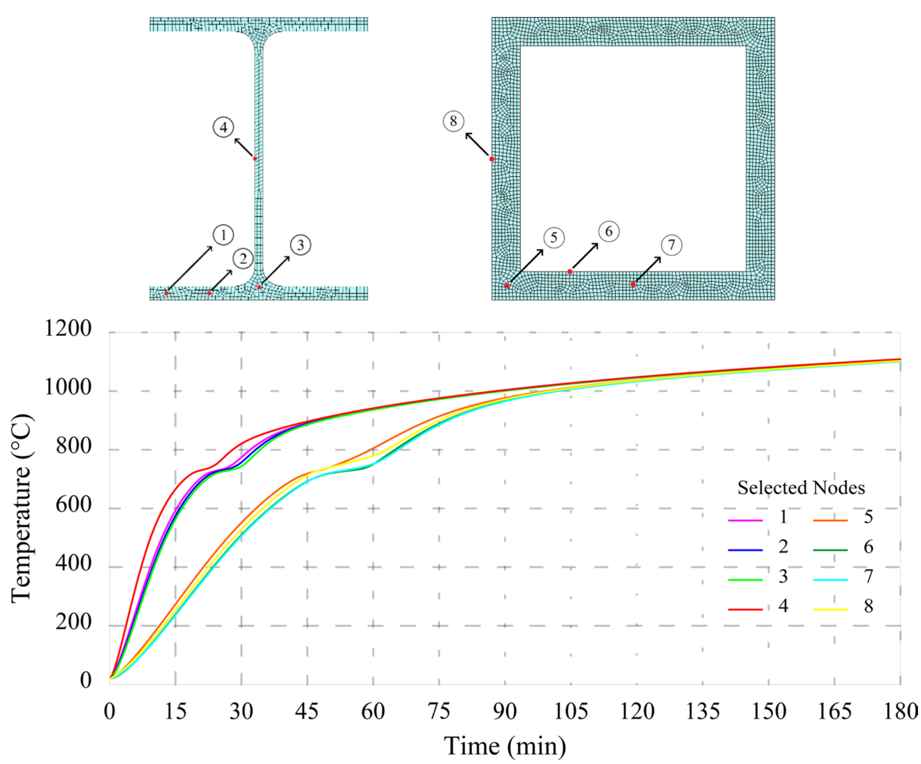


Fig. 8 Selected nodes and growth in their nodal temperature within total fire duration

Table 6 Material properties of steel used in modal analyses depending on temperature

Temperature (°C)	Material properties		
	Density (kg/m^3)	Poisson's ratio (–)	Modulus of elasticity (Pa)
0	7850	0.3	2.10E+11
20	7850	0.3	2.10E+11
100	7850	0.3	2.10E+11
200	7850	0.3	1.89E+11
300	7850	0.3	1.68E+11
400	7850	0.3	1.47E+11
500	7850	0.3	1.26E+11
600	7850	0.3	6.51E+10
700	7850	0.3	2.73E+10
800	7850	0.3	1.89E+10
900	7850	0.3	1.42E+10
1000	7850	0.3	9.450E+9
1100	7850	0.3	4.725E+9
1200	7850	0.3	1[a]

[a]Value for the modulus of elasticity at 1200 °C is equal to zero. However, since ABAQUS does not accept 0 as the input value of the modulus of elasticity, a close to zero value has been assigned

two columns and two frame models are selected as an example. As mentioned earlier, the situation when there is no fire, as well as twelve steps incorporating of 15, 30, 45, 60, 75, 90, 105, 120, 135, 150, 165 and 180 min of fire exposure according to ISO 834 fire curve have been considered in modal analysis. Moreover, the first three natural frequencies and corresponding mode shapes have been obtained from each step (Table 7).

The results of the analyses reveal that increases in temperature cause decreases in natural frequencies. The research has also shown that the change in mode shapes depends on the profile type. Mode shapes do not change depending on the temperature in some profile types, while in some profile types they change after a certain temperature. Therefore, in this study, a specific model to characterize this correlation could not be defined. More research needs to be done to further evaluate the effect of temperature on modal shapes.

4.3 Analyses Outputs

Following the performed analyses based on specified variable parameters, a relatively large size database, as a direct outcome of analyses results, have been generated. A set of tabulated data and approximate formulas have been created by using this database. In these outputs, steel columns and frame systems have been considered, separately. Here, these associated formulas are derived using linear regression analysis on the natural frequency values and specified variable parameters within the generated dataset. Table 8 presents these approximate formulas. The unknown variables in these formulas are related to cross-sectional dimensions of each profile types and exposure durations. By using these formulas, the values of first natural frequencies can be obtained.

In addition to approximate formulas, the natural frequency values obtained from the performed analyses have been classified and tabulated. Based on the profile types, a total of four data set including two tables for steel columns and two tables for steel frames have been generated. These tables incorporate of natural frequency values associated with first two translational modes related to each exposure durations, as well as the percentage of decrease in these natural frequencies. Due to large size of the mentioned datasets, these tables are presented in the appendix of the present paper (see Online Appendix Tables 1–4). Additionally, the themes obtained from these responses are set out in graphs which are presented in Figs. 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22.

The outputs of this study can be used to practically determine the changes in dynamic characteristics occur due to of fire or similar high temperature effects. Moreover, these outputs can contribute for damage identification to evaluate the serviceability limit state and performance evaluation of the structure as the result of changes in stiffness. The generated formulas and tables are suitable for cases with following specifications:

- ISO 834 fire curve or similar form of temperature–time curves is used
- Steel columns having 3 m height and fire exposure on all exterior sides
- Steel frame configurations having elements with same profile type and same cross-sectional dimensions. Besides, all exterior surfaces of the frame are being exposed to fire

Dynamic characteristics are related to mass and stiffness. Nonetheless, the mentioned analyses have been performed based on the assumption of having constant mass during the whole analysis. Additionally, since during the fire, the dimensions of elements and the geometry of models have not changed, the loss in stiffness has been represented by the changes in the modulus of elasticity. Therefore, the changes in dynamic characteristics resulted by fire have been determined based on changes in stiffness of the models.

5 Conclusions

The aim of the present research is to numerically evaluate the elevated temperature effects on dynamic characteristic changes of steel column and frames under different fire

Table 7 Temperature effects on natural frequencies and mode shapes of some selected models

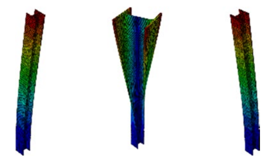
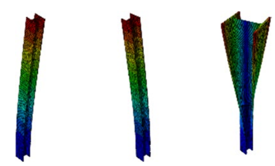
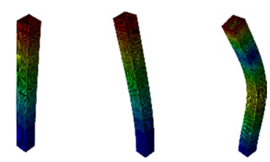
Model	Exposure duration (minute)	Natural frequencies (Hz)			Mode shapes
		1st mode	2nd mode	3rd mode	
Column HEA 200	0	15.958	24.489	26.091	
	15	6.415	10.092	10.694	
	30	4.625	7.243	7.581	
	45	4.166	6.544	6.816	
	60	3.846	6.065	6.291	
	75	3.581	5.672	5.858	
	90	3.351	5.331	5.482	
	105	3.145	5.026	5.143	
	120	2.955	4.747	4.832	
	135	2.776	4.487	4.541	
	150	2.607	4.241	4.264	
	165	2.444	3.997	4.005	
	180	2.285	3.736	3.778	
Column SHS 260×260×20	0	31.160		178.460	
	15	25.954		148.670	
	30	11.343		64.996	
	45	9.229		52.785	
	60	7.794		44.647	
	75	7.138		40.887	
	90	6.655		38.124	
	105	6.235		35.716	
	120	5.852		33.525	
	135	5.496		31.486	
	150	5.160		29.558	
	165	4.837		27.708	
	180	4.523		25.911	

Table 7 (continued)

Model	Exposure duration (minute)	Natural frequencies (Hz)			Mode shapes
		1st mode	2nd mode	3rd mode	
Frame HEA 240	0	11.404	15.823	26.524	
	15	5.034	6.992	11.699	
	30	3.341	4.637	7.792	
	45	2.982	4.138	6.939	
	60	2.752	3.818	6.402	
	75	2.562	3.555	5.960	
	90	2.397	3.326	5.576	
	105	2.249	3.120	5.232	
	120	2.113	2.931	4.915	
	135	1.985	2.755	4.618	
	150	1.864	2.586	4.336	
	165	1.747	2.424	4.065	
	180	1.634	2.266	3.800	
Frame SHS 180×180×10	0	13.265	20.253	25.462	
	15	8.188	12.696	15.853	
	30	4.326	6.633	8.329	
	45	3.507	5.373	6.746	
	60	3.220	4.922	6.186	
	75	2.995	4.575	5.751	
	90	2.800	4.278	5.377	
	105	2.627	4.012	5.043	
	120	2.467	3.768	4.737	
	135	2.318	3.540	4.450	
	150	2.176	3.324	4.178	
	165	2.040	3.116	3.917	
	180	1.907	2.913	3.662	

exposure durations. The major findings to attain from this study are listed below.

- A rapid growth in temperature is observed within the cross-section of element as a result of heat transfer analysis. It is essential to apply fire protection methods for load bearing members in steel structures under high temperate loads.
- The temperature is directly proportional to the exposure time. The increasing of temperature causes decreases in natural frequencies.
- The size effect of varied cross-sectional dimensions on decrease percentage in natural frequency of the models having HEA and SHS profile types could be observed at initial 30 and 90 minutes of fire exposure duration, respectively. It is observed that during these exposure durations, the effect of elevated temperature on natural frequency de-escalate as the cross section increases in size. As exposure duration exceeds these periods, this size effect fades. After these periods, as exposure duration increases, the percentages of decrease in natural

Table 8 Developed formulas for steel columns and frames

Profile type	Formulas for steel columns
HEA	$f_1 = 5.363 + 0.018X - 0.00084T$ $f_2 = 7.655 + 0.038X - 0.00147T$
SHS	$f_{1,2} = 8.923 + 0.04X + 0.049Y - 0.00186T$
Profile type	**Formula for steel frames**
HEA	$f_1 = 3.141 + 0.011X - 0.0005T$ $f_2 = 6.635 + 0.032X - 0.00124T$
SHS	$f_1 = 5.157 + 0.026X + 0.036Y - 0.00114T$ $f_2 = 8.163 + 0.036X + 0.063Y - 0.00173T$

X: Section size identification for nominal dimensions (mm)

Y: Thickness of square hollow section (mm)

T: Duration of exposure (s)

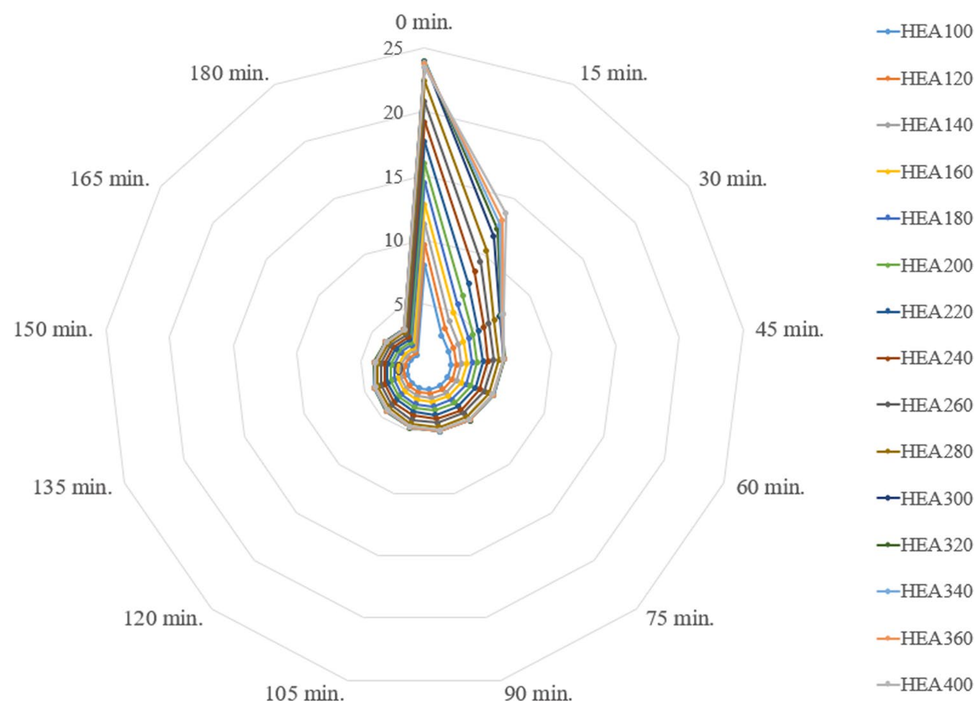


Fig. 9 First natural frequencies of steel columns (HEA profile) for exposure durations

frequencies at each segment of exposure have taken to nearly same values.

- The decrease in the natural frequency values of the profiles having both symmetrical (SHS) and asymmetrical (HEA) geometry with the increase of the exposure duration to fire is almost equal.
- After the first 30 and 60 minutes, there is an average decrease of 70% in the natural frequency values of the HEA and SHS models. The results display the high temperature role in significant strength and stiffness loss of steel structures.
- It is also shown that the change in mode shapes depends on the profile type. Mode shapes do not change depending on the temperature in some profile types, while in some profile types they change after a certain temperature. Therefore, in this study, a specific model to characterize this correlation could not be defined. More research needs to be done to further evaluate the effect of temperature on modal shapes.

The outputs of this study can contribute in performing post fire performance analysis and risk assessment on steel structures. According to the exposure scenario, the required reductions in stiffness of load-bearing elements can be applied. Based on the performance evaluation, the

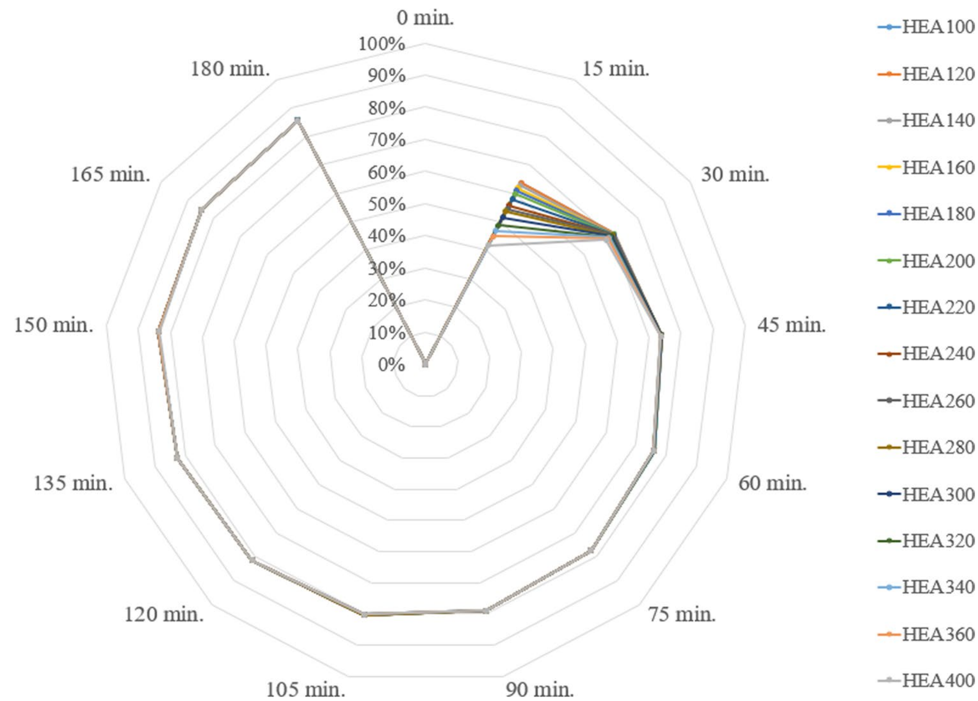


Fig. 10 Percentage of decrease in first natural frequencies of steel columns (HEA profile) for exposure durations

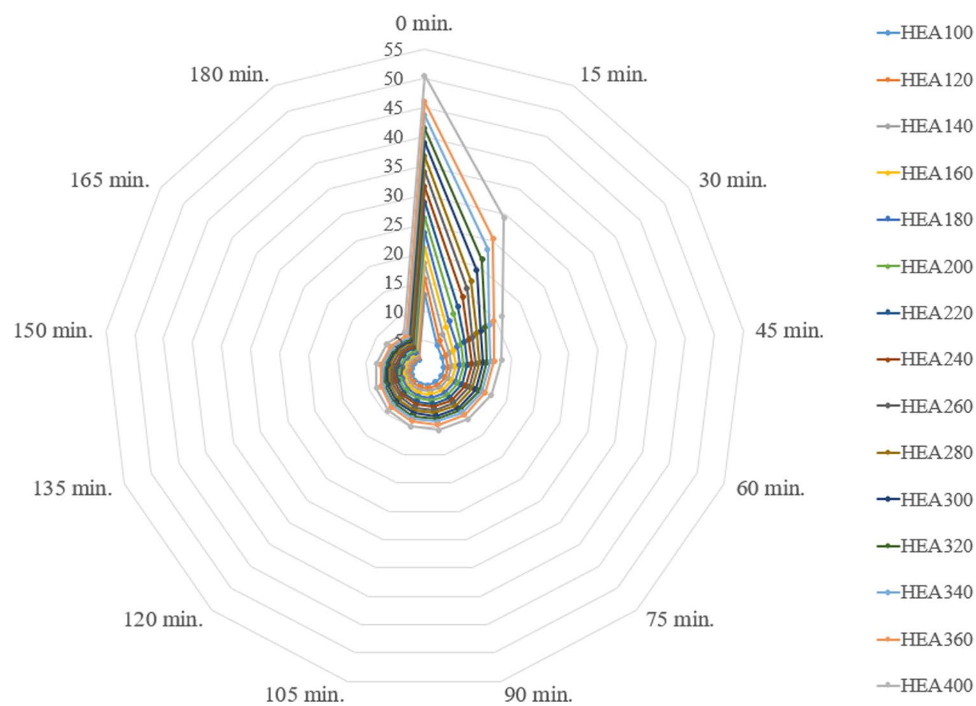


Fig. 11 Second natural frequencies of steel columns (HEA profile) for exposure durations

serviceability of the structure can be evaluated. The structural retrofitting practices can be accurately designed.

Due to novelty of this topic, the present work can be expanded from various perspectives. Different cross-section types, fire exposure scenarios, model heights and fire curves can be considered. Also, the effectiveness of connection types and various steel protection methods can be examined. The formulation created in this way can be expanded and the error rate can be reduced. An innovative approach can be generated for mode shapes.

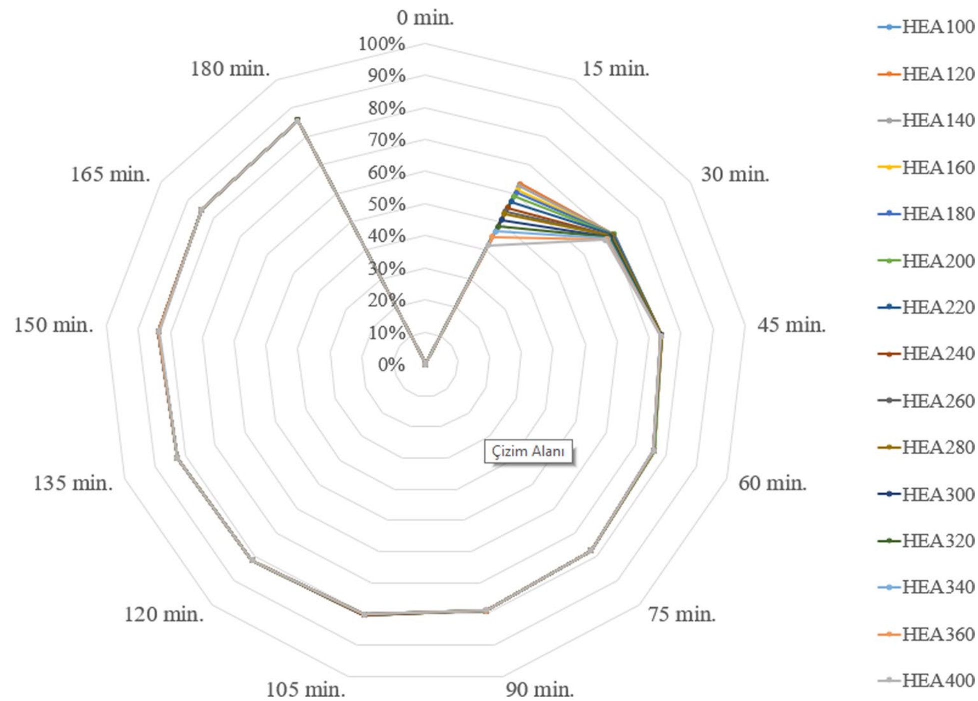


Fig. 12 Percentage of decrease in second natural frequencies of steel columns (HEA profile) for exposure durations

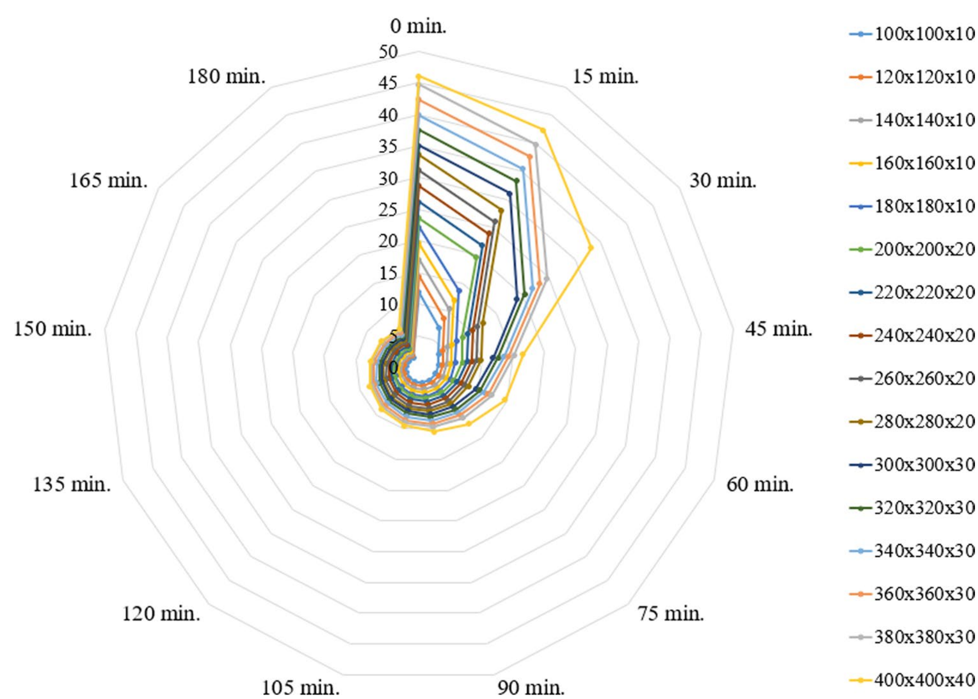


Fig. 13 First two natural frequencies of steel columns (SHS profile) for exposure durations

Fig. 14 Percentage of decrease in first two natural frequencies of steel columns (SHS profile) for exposure durations

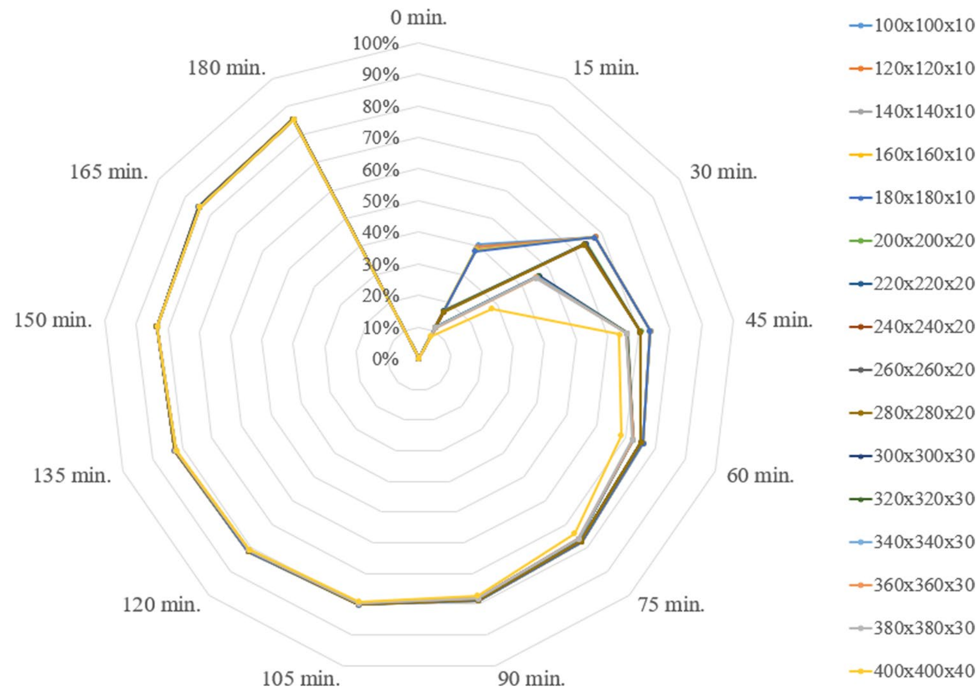
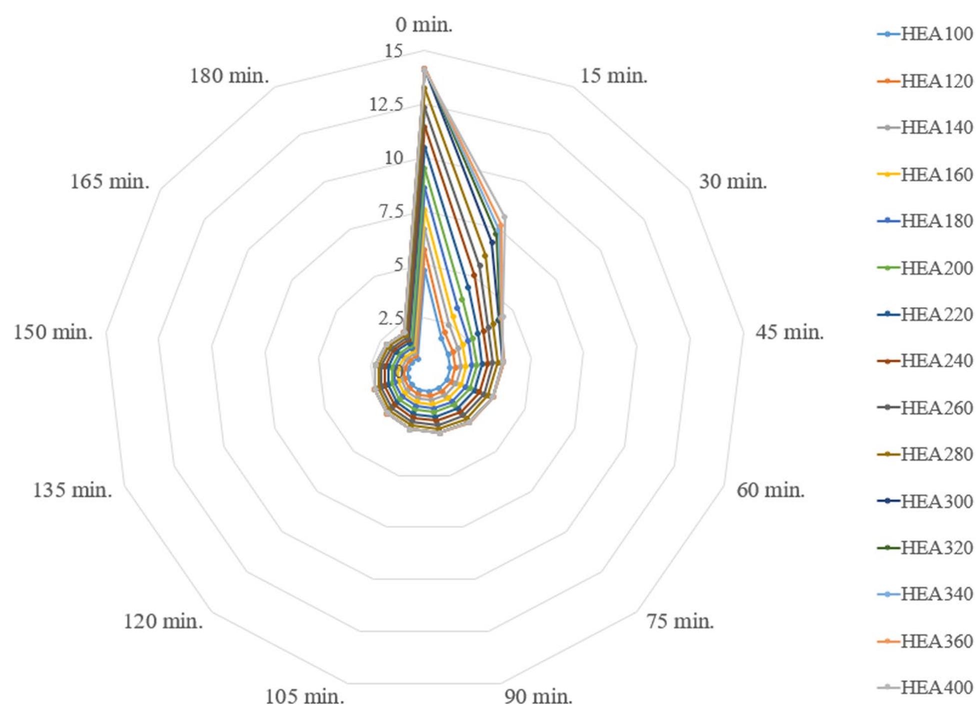


Fig. 15 First natural frequencies of steel frames (HEA profile) for exposure durations



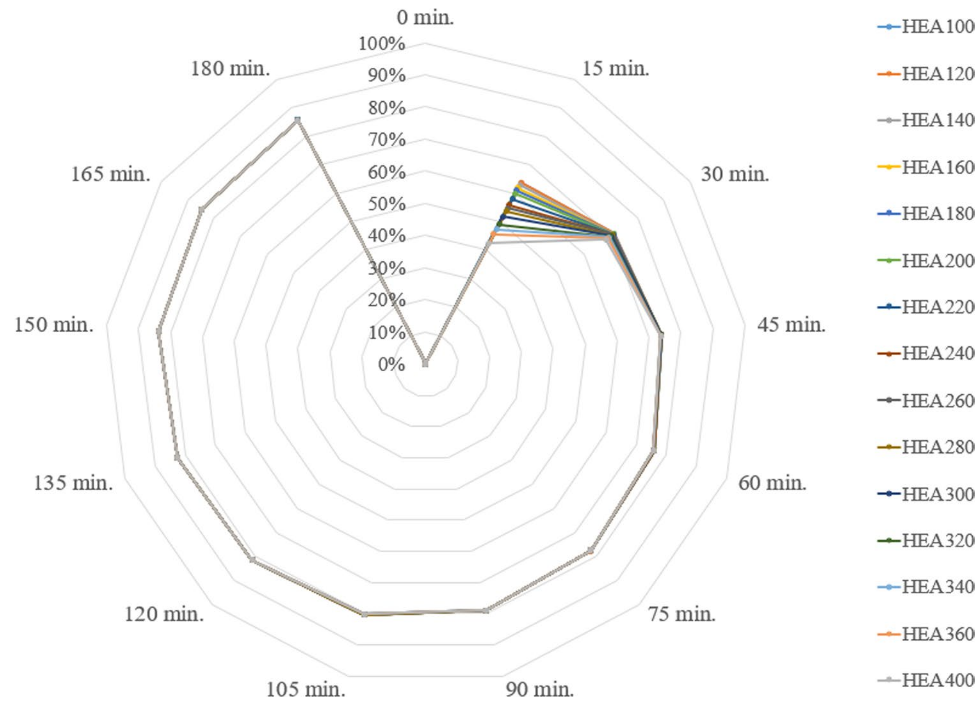


Fig. 16 Percentage of decrease in first natural frequencies of steel frames (HEA profile) for exposure durations

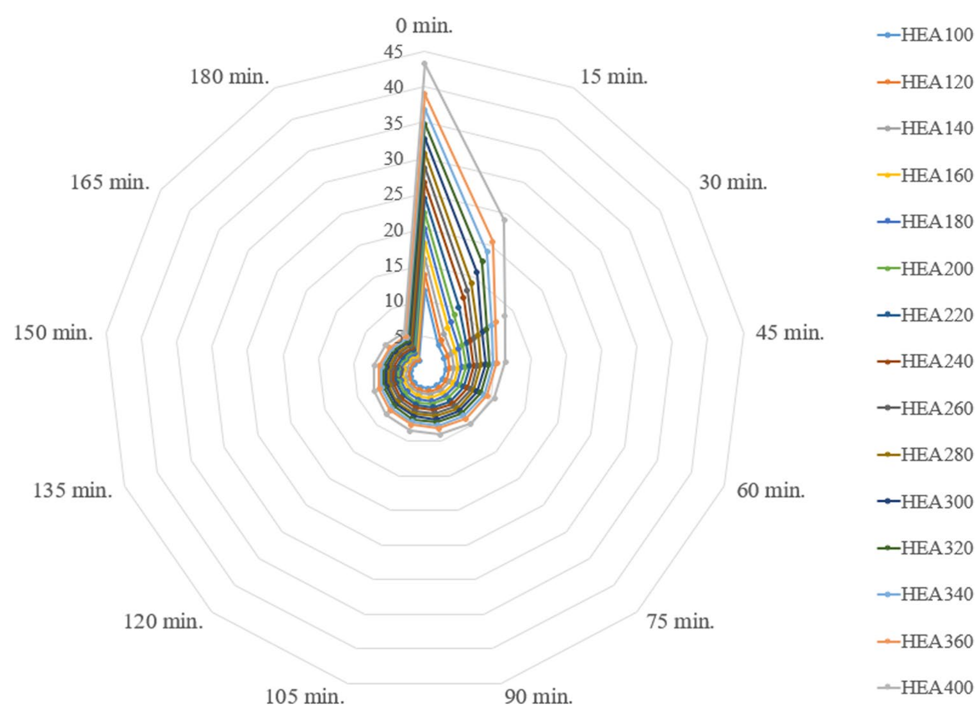


Fig. 17 Second natural frequencies of steel frames (HEA profile) for exposure durations

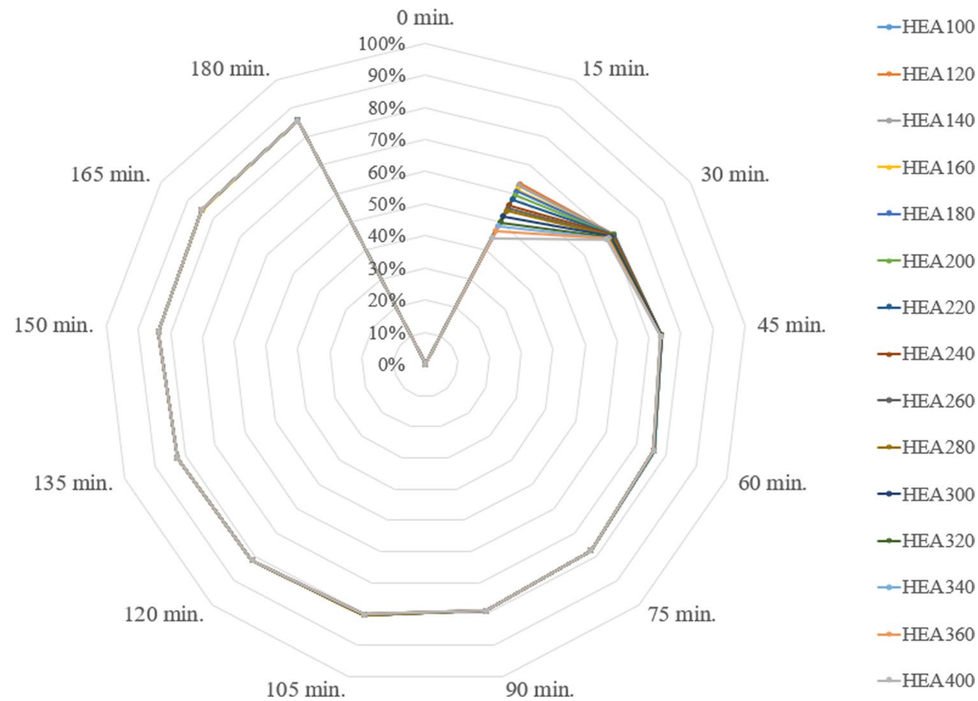


Fig. 18 Percentage of decrease in second natural frequencies of steel frames (HEA profile) for exposure durations

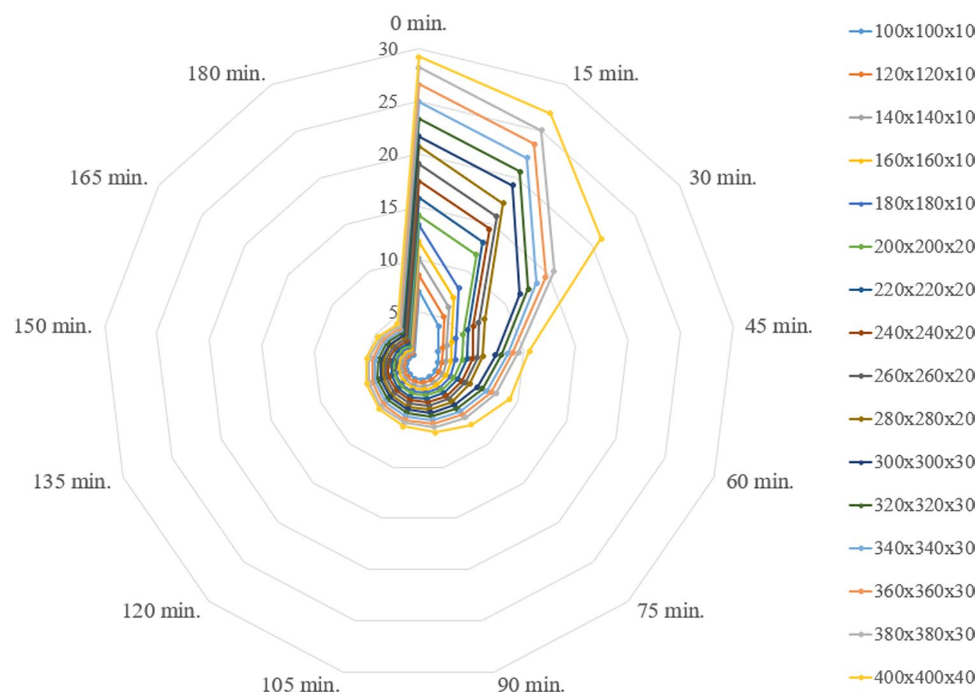


Fig. 19 First natural frequencies of steel frames (SHS profile) for exposure durations

Fig. 20 Percentage of decrease in first natural frequencies of steel frames (SHS profile) for exposure durations

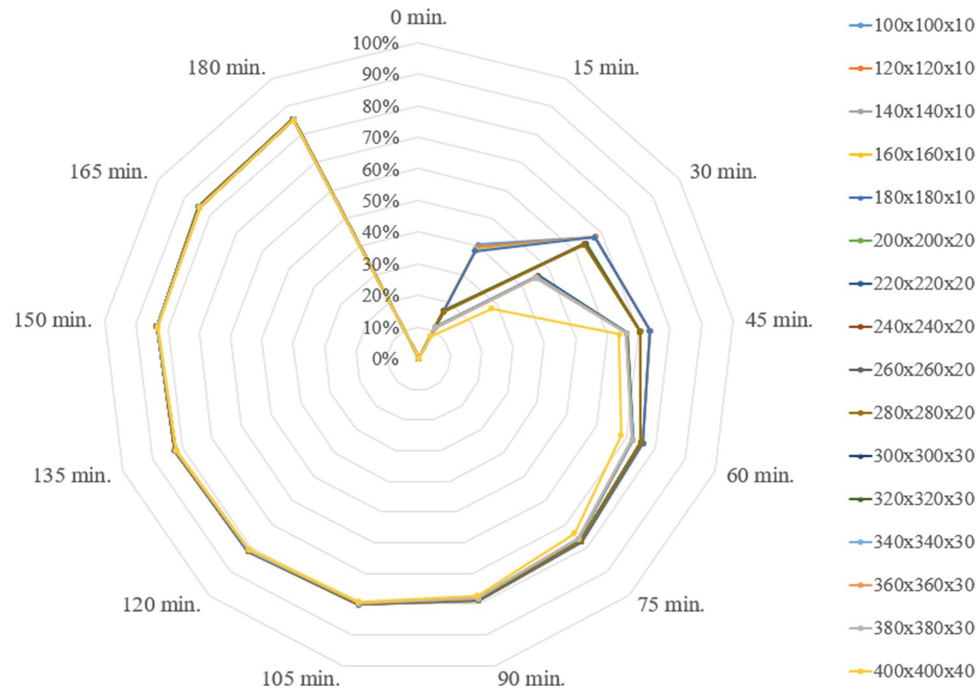
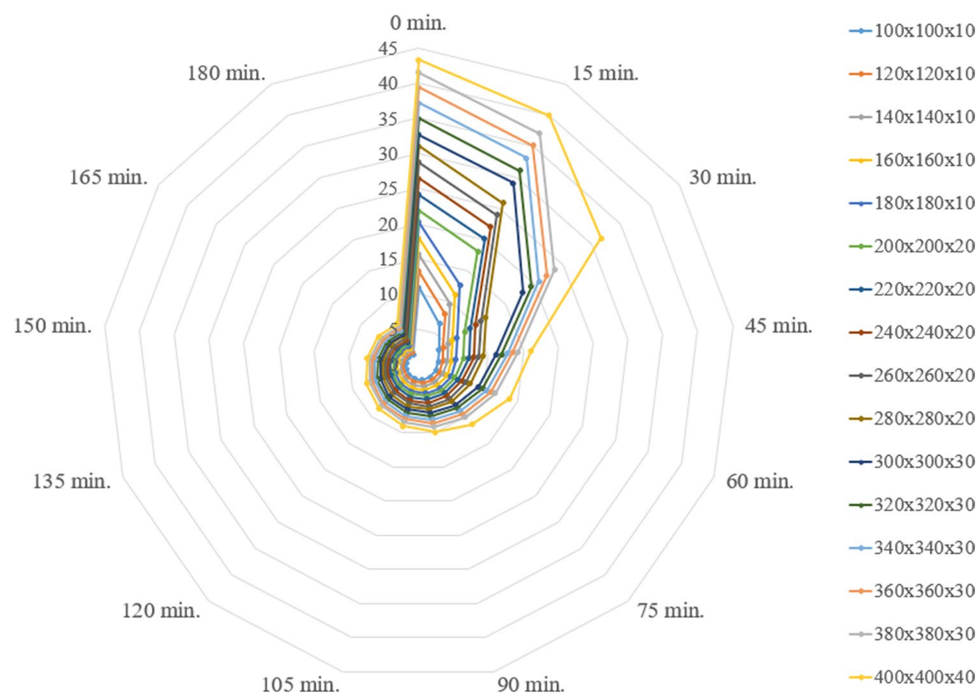


Fig. 21 Second natural frequencies of steel frames (SHS profile) for exposure durations



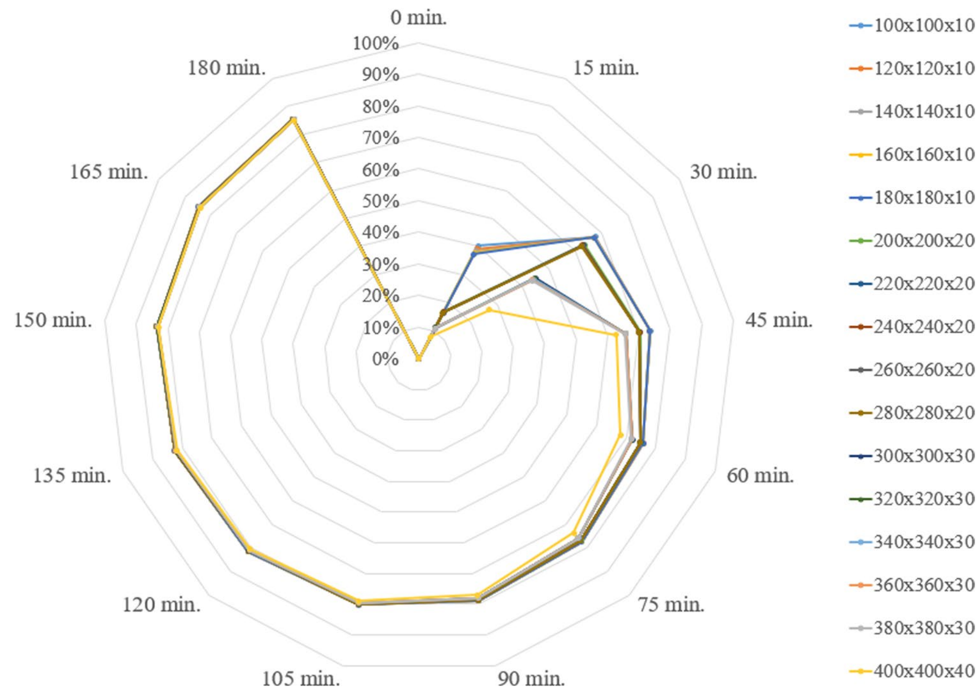


Fig. 22 Percentage of decrease in second natural frequencies of steel frames (SHS profile) for exposure durations

Supplementary Information The online version contains supplementary material available at https://doi.org/10.1007/s13296-021-00479-w.

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