ASIAN JOURNAL OF CIVIL ENGINEERING (BHRC) VOL. 18, NO. X(2017)
PAGES 993-1008

TIME RESISTANCE OF INDUSTRIAL STEEL PORTAL FRAMES WITH HAUNCHES UNDER FIRE

N. Benlakehal[1*], A. Kassoul[1], P.A.G. Piloto[2] and A. Bouchair[3]
[1]Laboratory of Structures, Geotechnic and Risks, Faculty of Civil Engineering and Architecture, Hassiba Benbouali University of Chlef, Route de Sendjas, Chlef 02000, Algeria
[2]Polytechnic Institute of Bragança, Campus Sta Apolónia Apartado 1134, 5300-857 Bragança, Portugal
[3]Clermont University, Blaise Pascal University, Pascal Institute, BP 10448, 63000 Clermont Ferrand, France

Received: 15 February 2017; **Accepted:** 20 May 2017

ABSTRACT

To protect human lives and prevent against failure of structures, the collapse of industrial buildings under fire should always occur inward with a minimum failure time. The objective of this paper is to investigate the behaviour of industrial steel portal frames with haunches in fire situation. The structure is studied using finite element software ANSYS with an uncoupled thermal and mechanical analysis. The inclination angle of the rafter and the haunch's length for the portal frame are taken as variables. A comparative study of the numerical simulations and the simplified method (R15) show a close agreement between the two analyses according to the failure time.

Keywords: Industrial buildings; steel portal frames; fire; time resistance; numerical simulations.

1. INTRODUCTION

Portal steel frame structures are widely used as industrial buildings for practical reasons of exploitation, durability and cost efficiency. However, steel although is a ductile material, it remains vulnerable to the effect of excessive temperatures. The recent accident, which occurred in the industrial buildings of Sonatrach (Petroleum Industry in Algeria) in 2015, remind us of the real danger and the potential risk of fire [1].

The European Code for steel structures [2] provides simplified formulas for single structural elements under fire condition. However, it does not describe the actual behaviour

*E-mail address of the corresponding author: n_blk@yahoo.fr (N. Benlakehal)

of the structure when global deformations are large and the structure presents non-linear behaviour [3-5]. Fire tests conducted by O'Connor and Martin [6] indicated that structural steel members in steelwork behave significantly better than single members with isolated restraining conditions. Research works using numerical simulations on single steel members have shown that the response of the structure is overestimated with regard to displacements, internal efforts and critical temperatures when Eurocode approach is applied [7-10].

For industrial buildings, new guidance by the European norms [11] define active and passive exigencies for the behaviour of industrial buildings under fire conditions to avoid human losses and prevent against failure of the structure. The main criterion is that failure should always occur while the structure is dragged inward to avoid outward collapse of the building frames. A few years before, Wong [12] studied experimentally and numerically the behaviour of industrial pitch portal frame under fire condition. Based on plastic theory, he developed a method to calculate the critical temperature of pitch roof portal frames. De Souza Junior *et al* [13] conducted a comparative study between the 2D and 3D analyses of a single storey industrial building under fire using SAFIR program [14]. In their work, the portal frames with simply roof truss were modelled using 2D and 3D beam elements. Results showed that for the 3D analysis, the out-of-plane instability played an important role in the structural performance. They confirmed that the 2D analysis gave higher time resistance which is unsafe for the structure.

In other research, Vassart *et al* [11] lead a comparative study on a double portal frame using both static and implicit dynamic analyses with different softwares (ABAQUS, ANSYS and SAFIR). They found that the critical time at which the collapse of the structure occurs using 3D analysis is less than that when using 2D analysis. This time is referred as the time resistance. Song [15] and Song *et al* [16] investigated the failure mechanism of a single-storey haunched portal frame with different column bases. They pointed out that the critical temperature at which run-away collapse occurs may be higher than that at which the rafter initially loses its stability when the column bases are enough stiff. Renaud and Sakji [17] issued a practical guide for engineer offices in France to design warehouses under fire conditions. It is established in this guide that the fire stability time required for frames and purlins is 15 minutes (R15). El-Heweity [18] analysed steel portal frame with hollow sections by considering several fire scenarios and different rafter inclination angles. He concluded that the studied parameters strongly affect the failure mechanism of the structure. Huang *et al* [19] developed a numerical model which combined static and dynamic solution to model snap-through behaviour of industrial steel portal frame. They suggested the use of this modelling tool for the perform-based fire safety of industrial buildings to the highly simplified design methods which are in use. Not a long ago, Rahman *et al* [20] investigated the effect of column base strength on steel portal frames. They confirmed that fire protection of columns has no effect on snap-through-buckling when column bases are pinned. However, when they are fixed, snap-through-buckling temperature was shifted to higher value witch is better for the safety of the structure. Gentili [21] analysed Vassart's portal frame using computational fluid dynamic (CFD) to develop the fire model. The outcomes showed that the use of a simplified thermal model [22] does not always lead to secure results. Recently, Kmet *et al* [23] investigated an industrial hall severely damaged by fire. Comparing the behaviour of the numerically modelled structure with the real post-fire response of the damaged construction, the results of the critical temperature and the time resistance showed a good agreement between the simulated analysis and the real characteristics of the damaged structure.

In the present paper, a three dimensional analysis is presented to simulate the behaviour of industrial steel portal frame with haunches under conventional fire ISO834 [22]. A model is developed using ANSYS program [24] to determine the critical failure time by considering particularly the effect of the rafter inclination angle and the haunch length. More attention has been given to the later in order to enhance its effect in improving time resistance in industrial structures. The parametric study was performed by both shell elements to model the portal frame in 3D and non-uniform temperature within the structural beam-columns elements.

2. FIRE MODEL AND THERMAL RESPONSE

Although the governing parameters of a real fire are numerous such as fire load density, ventilation condition and material compartment, the ISO834 standard time-temperature curve [22] is assumed testing purposes and numerical simulations. It represents the action of a fire in a confined compartment of building and the gas temperature evolution given according to the formula of the EC1 [25]:

$$\theta = 20 + 345 log_{10}(8t + 1) \tag{1}$$

where: θ is the temperature of gases in (°C) and t is time in (min).

It is known that for thermal response, the governing equation for the two-dimensional non-linear transient heat conduction within the cross section of a structural element takes the following form:

$$\frac{\partial}{\partial x}\left(\lambda_a \frac{\partial \theta_a}{\partial x}\right) + \frac{\partial}{\partial y}\left(\lambda_a \frac{\partial \theta_a}{\partial y}\right) = \rho_a C_a \frac{\partial \theta_a}{\partial t} \tag{2}$$

where λ_a is the thermal conductivity and C_a the specific heat of steel both are expressed as a function of the temperature in EC1 [25], θ_a the steel temperature, t the time and ρ_a is the density of steel.

The temperature field which satisfies Eq. (2) within the structural element must satisfy the boundary conditions like prescribed temperatures θ, the specified net heat flux $\dot{h}_{net,d}$ or heat transfer by convection and radiation given by Eq. (3) [25].

$$\dot{h}_{net,d} = \phi \sigma \varepsilon_f \varepsilon_a \left(\theta^4 - \theta_a^4\right) + \alpha_c (\theta - \theta_a) \tag{3}$$

where ϕ represents the view factor (usually 1 or less), σ is Stephan Boltzmann constant equal to 5.67×10^{-8} W/m²K^4, ε_a is the emissivity of steel equal to 0.7, ε_f represents the emissivity of the fire (in general equal to 1) and α_c is the coefficient of heat transfer by convection equal to 25 W/m²K.

Since the solution of equation (2) is non-linear, simplified solution for the temperature rise of an unprotected steel member is provided by EC3 [2] with the following Eq. (4), assuming the lumped thermal capacity model (the energy received at the surface boundaries is used to increase the assumed uniform temperature distribution of the steel section):

$$\theta_{a,t} = k_{sh} \frac{1}{\rho_a C_a} \left(\frac{A_m}{V} \right) \dot{h}_{net,d} \, \Delta t \;\; (°C) \tag{4}$$

where A_m/V is the section factor for unprotected steel elements, Δt is the time interval in seconds ($\Delta t \leq 5\ s$) and k_{sh} is the correction factor for the shadow effect.

3. FIRE RESISTANCE OF PORTAL FRAMES USING SIMPLIFIED METHOD R15

For industrial steel buildings such as warehouses, most European countries adopt in their regulation codes design requirements which include structural fire stability varying from 15 to 60 minutes depending on national code. In France, Renaud and Sakji [17] issued a guide in which simplified methods are described to verify frames and purlins for fire stability of 15 minutes (R15). However, some hypothesis made can lead to a lack in using these methods such as the slopes of the rafter which can only be 10%, or less and haunches can be used without any limitation in their length.

For frame structures using hot rolled sections of class 1 and 2, the condition to be satisfied for fire stability R15 is given by Eq. (5).

$$q_{fi,Ed} \leq q_{fi,Rd} = \frac{14 k_{y,\theta} W_{pl,y} \; f_y}{L^2} \tag{5}$$

where $q_{fi,Ed}$ is the uniformly distributed load applied to the rafter under fire condition, $q_{fi,Rd}$ the design load resistance after 15 minutes of a conventional ISO834 fire, $k_{y,\theta}$ is the reduction factor for the yield strength, $W_{pl,y}$ plastic modulus and L is the length of the rafter (outside haunches).

For European profiles such as HEA and IPE, the guide gives in tables the corresponding $k_{y,\theta}$ for a time of 15 minutes and Eq. (5) can be solved very easily.

To compare results using this method with the proposed model, Eq. (5) can be rearranged to Eq. (6) in order to calculate critical temperature using table (1).

$$k_{y,\theta} \geq \frac{q_{fi,Ed} \; L^2}{14 W_{pl,y} \; f_y} \tag{6}$$

Table (1) gives the reduction yield strength factor $k_{y,\theta}$ as function of steel temperature θ_a [2].

Table 1: Reduction factor $k_{y,\theta}$ for yield strength at elevated temperatures

θ_a (°C)	20 ÷ 400	500	600	700	800	900	1000	1100	1200
$k_{y,\theta}$	1	0.78	0.47	0.23	0.11	0.06	0.04	0.02	0

Failure time t_{cr} of the portal frame can be calculated using Eq. (4). According to R15 method, the correction factor k_{sh} (shadow effect) is taken equal to:

$$k_{sh} = \left[\frac{A_m}{V}\right]_b \bigg/ \left[\frac{A_m}{V}\right] \tag{7}$$

where $[A_m / V]_b$ is the section factor using envelop or box section.

4. PARAMETRIC STUDY

A parametric study is conducted using numerical simulations with ANSYS [24]. The industrial portal frame, shown in Fig. 1, is analysed based on the variation of the following parameters: inclination angle of the rafter and haunch length.

This structure is illustrated in the CTICM guide [17] where both frame and purlins are checked for 15 minutes of fire exposure. Load combination on the rafter take into account G (dead load, roof, industrial equipment under roof and cladding) equal to 4.16 kN/m and S (snow) equal to 4.4 kN/m. According to EC0 [26], a total uniformly distributed load q in fire situation is calculated using G and $0.2S$ which gives 5.04 kN/m. Elements are chosen using hot rolled sections with IPE400 for the rafter and IPE500 for both columns. The steel grade is taken as S235 with a density of 7850 kg/m^3 and Poisson's ratio equal to 0.3.

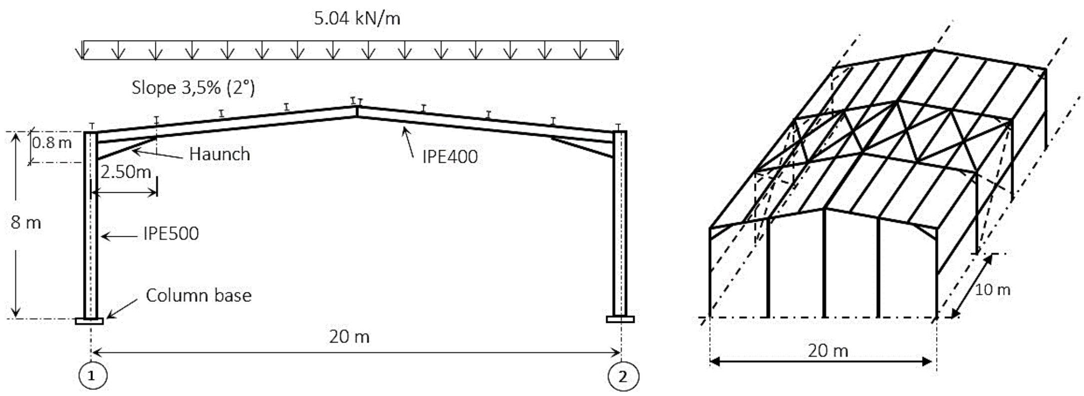


Figure 1. Portal model to be analysed

4.1 Finite element modelling

The frame models are created using 3D finite elements of SHELL131 and SHELL181. The task is solved as a combined one using material nonlinear thermal analysis and geometry and material nonlinear static analysis in the ANSYS software [24]. In the thermal analysis, the temperature distribution is obtained in the section. In the nonlinear static analysis, the corresponding displacement, internal efforts and stress-strain state of the structure caused by both applied loads and constrained thermal dilatation are solved in the steps of temperature increments.

After a convergence test, the finite element mesh is defined. The columns and beam are subdivided into 60 elements and 86 elements respectively along their lengths. Along the height, the section is subdivided into 12 elements (Fig. 2). Lateral-torsional buckling of the rafter has been prevented by adding appropriately lateral supports to the flanges: see Fig. 2.

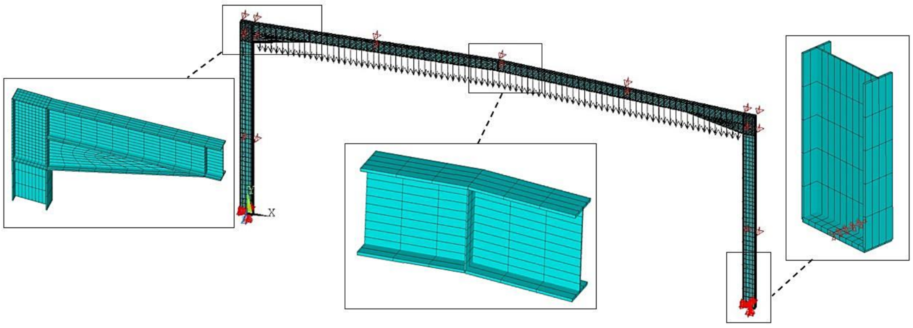


Figure 2. Numerical model, discretization of the portal frame

4.2 Thermal analysis

The nonlinear transient thermal analysis is defined with an integration time step of 60 seconds, which can decrease to 0.1 second. Thermal properties of the steel are set as temperature dependent as given in EC1 [25]. The initial temperature of the frame is set to 20 °C. The temperature field is determined for the total time of 3600 seconds. It is important to note that all the four sides of the elements are under fire load.

Figs. 3-4 show the temperature distributions in respectively the portal frame and joints (eaves, apex and column base) after 15 minutes of a standard ISO834 fire.

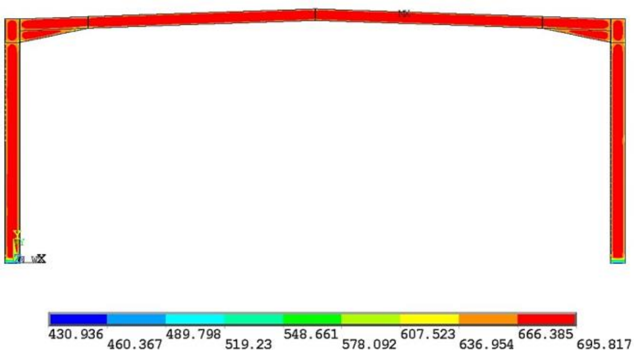
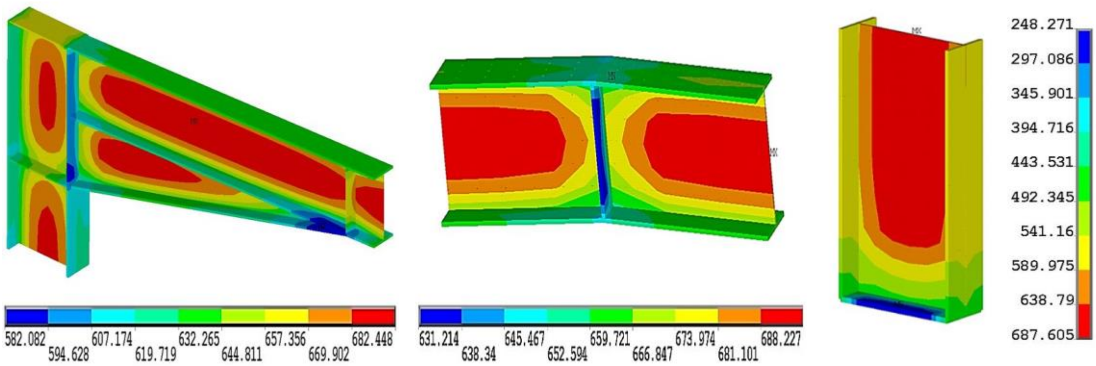


Figure 3. Thermal field in the portal frame at time t = 15 minutes



(a) Rafter eave (b) Rafter apex (c) Column base

Figure 4. Temperature distributions at different joints at time t = 15 minutes

Fig. 5 shows temperature evolutions with time in the rafter (IPE400) using thermal analysis and Eq. (4) of EC3 [2].

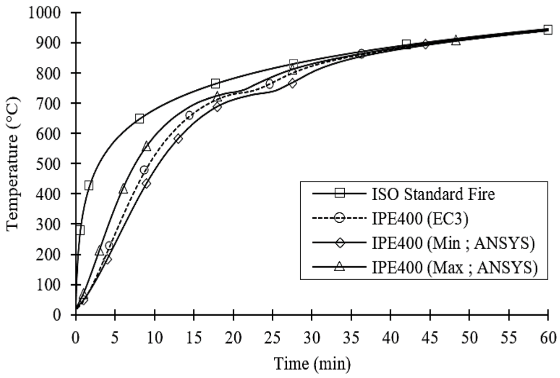


Figure 5. Temperature evolutions in the rafter (IPE400) and ISO834 fire

The temperature field is recorded for the corresponding resistance class and applied as body load to the mechanical model.

4.3 Structural analysis results

The thermal loading was set in steps on the deformed state of construction at simultaneous change of all necessary thermal and mechanical properties of the material depending on the temperature in the structure. The nonlinear material response of steel at elevated temperature is provided by EC3 [2].

The results in term of displacements (vertical and horizontal) of the portal frame, horizontal shear force developed at the column base and Von Mises stresses at the developed hinges are illustrated in the next section.

4.3.1 Effect of rafter inclination angle

In addition to the rafter inclination angle of the initial portal frame taken as 2° (3.5% slope), five other angles are studied in this analysis: 2.86°, 5.71°, 8.53°, 11.31° and 14.04° which correspond respectively to 5%, 10%, 15%, 20% and 25% slopes. Although some studies confirmed that this parameter has no effect on time resistance (or critical temperature) when rafters are used with constant profile sections, it can be relevant when failure mechanism is considered [18]. Adding to that, when haunches are used at the eaves, the whole behaviour of the portal frame may change.

Fig. 6 shows apex deflections for the analysed frame. Results show that, for different rafter inclination angles, time resistance is almost constant (≈ 16 min). However, large mid-span deflections are observed for smaller angles. For an angle of 2°, apex displacement is about 1.72 m and this value decreases to 0.45 m for an angle of 14.04°.

Fig. 7 shows the evolution of horizontal displacements at eaves. For small rafter slope (2°), the displacement increases outward until time collapse where the portal frame begins to change to the opposite way (inward). But when inclination angle increases, the horizontal displacements at eaves increase outward the portal frame with no reversible displacements (inward).

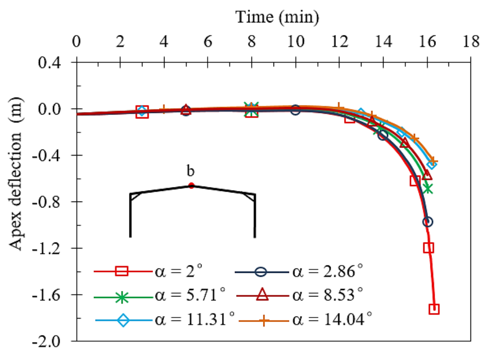


Figure 6. Apex vertical deflection (node b) for different rafter slopes

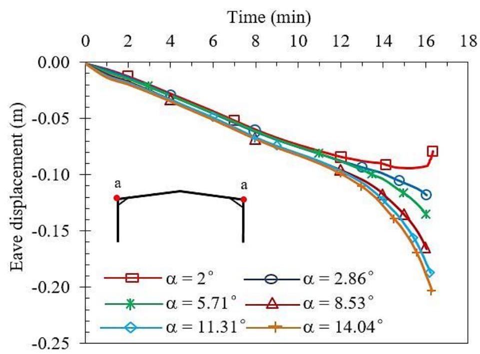


Figure 7. Eaves horizontal displacements (nodes a) for different rafter slopes

Fig. 8 shows the development of shear force at column bases (nodes d). As it can be seen from this figure, the increase in the rafter inclination angle leads to a decrease in the shear force. At heated phase, maximum values are observed within 6 and 10 minute times with 9.63% decrease which is not a significant value comparing to the decrease at ambient temperature equal to 9.2%.

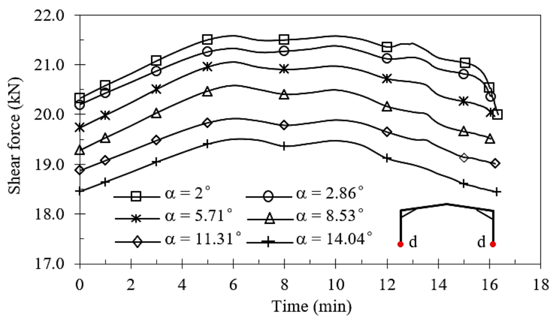


Figure 8. Shear forces at the column bases (nodes d) for different rafter slopes

Figs. 9-12 show the developed Von Mises stresses at respectively portal frame, rafter ends (eaves), haunch end and rafter mid-span (apex).

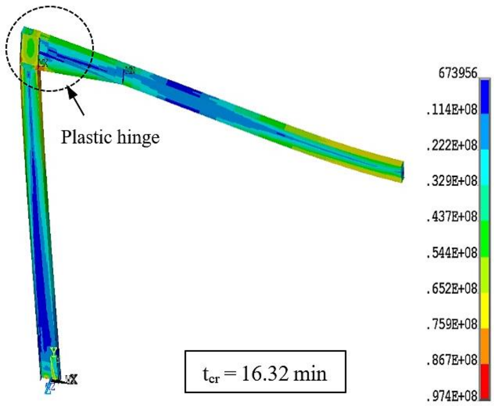


Figure 9. Von Mises stresses in the half portal frame (2.5 m haunch; 2° rafter slope)

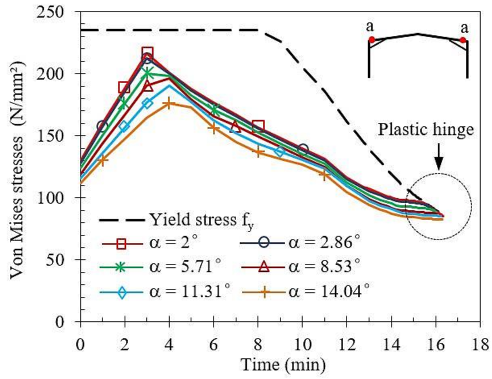


Figure 10. Von Mises stresses at eaves (nodes a) for different rafter slopes

As shown in Figs. 9-10, the plastic yielding is reached at the eaves and the maximum stresses are within the four first minutes of fire exposure. Increasing the inclination angle leads to the decrease in stresses values. This decrease is about 18.48% when inclination angle increase from 2° to 14.04°. From Figs. 11-12 and for the same increase in inclination angle, maximum decrease in stresses are about 13.8% and 23.22% at respectively haunch end and rafter mid-span.

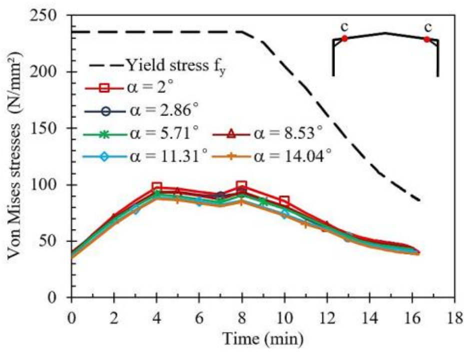


Figure 11. Von Mises stresses at haunch end (nodes c) for different rafter slopes

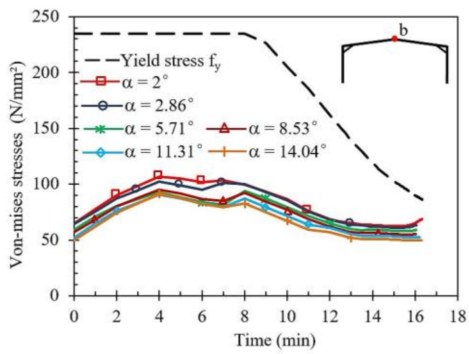


Figure 12. Von Mises stresses at apex (node b) for different rafter slopes

4.3.2 Effect of haunch length

Haunches in the rafters at the eaves are used to reduce the depth of the rafter and achieve efficient moment connection between column and rafter. However, in fire conditions and according to the current simple design method [27], the length of the haunch is limited to one-tenth of the span. To analyse the influence of haunches on the fire resistance of single portal frame, five different lengths are considered: 0 m, 0.5 m, 1 m, 2 m, 3 m and 4 m. Both load (5.04 kN/m) and profile sections of the rafter and haunch (IPE400) are taken constant.

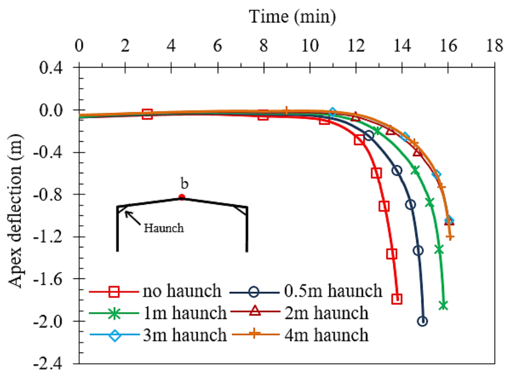


Figure 13. Apex vertical deflection (node b) for different haunch lengths

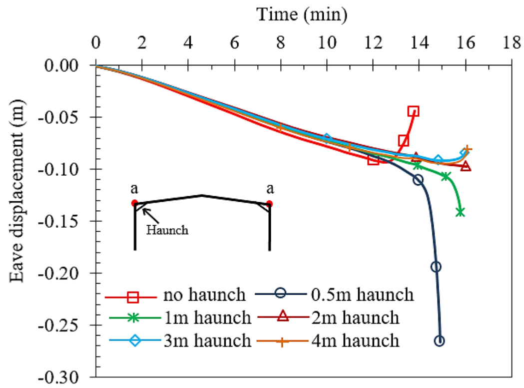


Figure 14. Eave horizontal displacement (node a) for different haunch lengths

The displacement-time curves (vertical at apex and horizontal at eaves) presented in Figs. 13-14 show that the use of haunches until one-tenth of the span (2 m) increases the time resistance of the portal frame. With the same rafter profile (IPE400), time resistance without haunches is about 13.81 minutes and with 2 m haunches (one-tenth) time resistance increases to 16 minutes (16% increase). We notice that beyond this distance (2 m), no improvement can be seen. This may be explained by the fact that weakest section can be located at the end of the haunch when shorter haunch is used (less than one-tenth) to rafter end when haunch reached one-tenth and more.

Results from Fig. 14 show that when the lengths of haunches are less than one-tenth of the rafter span, the collapse of the structure tends to happen in the outward direction.

The maximum Von Mises stresses developed in the rafter at different locations (haunch end, rafter ends and rafter mid-span) are shown in Figs. 15-18.

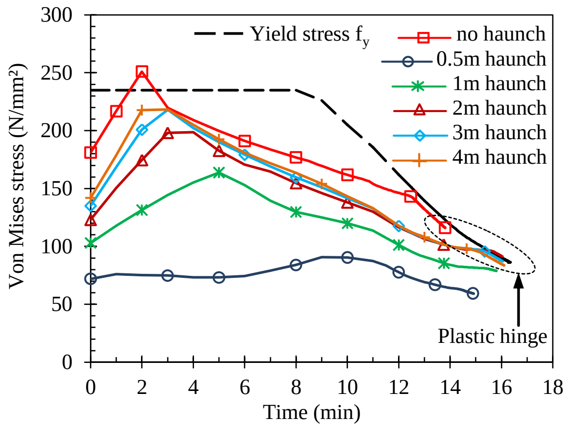


Figure 15. Von Mises stresses at rafter ends for different haunch lengths

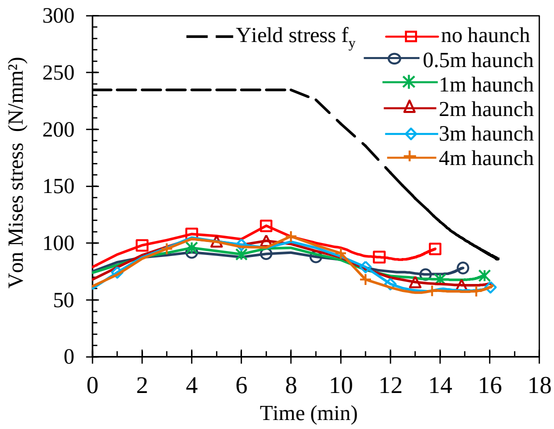


Figure 16. Von Mises stresses at mid span for different haunch lengths

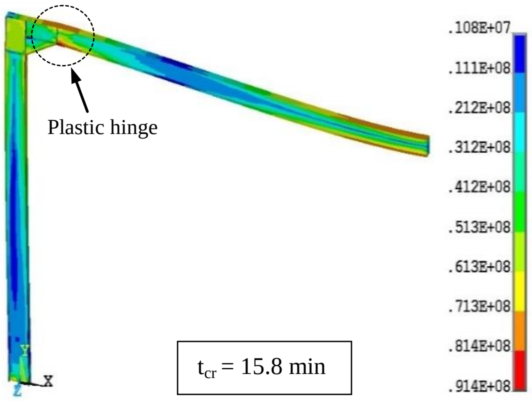


Figure 17. Von Mises stresses in the half portal frame (1m haunch; 2° rafter slope)

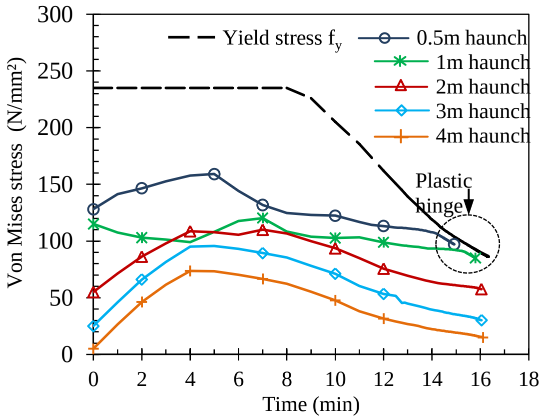


Figure 18. Von Mises stresses at haunch end for different haunch lengths

From Fig. 15, the yielding stresses are obtained at the eaves when haunch length is one-tenth (2 m) and more. For shorter haunches (less than one-tenth), the yielding stresses appear at the ends of the haunches (Figs. 17-18). This is due to the weakest section on the rafter which change from the eaves to the ends of the haunches.

From Fig. 16, Von Mises stresses at mid-span of the rafter are comparable for all the studied haunch lengths. As expected, when no haunch is used, maximum stresses are at the eaves (Fig. 15). A value of 250 N/mm² is obtained at 2 minutes heated time, which is greater than the yield stress (235 N/mm²) and this has favourably leads to the collapse of the portal frame at early time.

5. MODEL VALIDATION

The double span frame considered to validate the proposed model is shown in Fig. 19. This structure has been previously investigated by Vassart *et al*. [11] using several finite element programs (SAFIR, ABAQUS and ANSYS). The structure was represented in two dimensions but the out-of-plane displacements are allowed: see Fig. 20(a). The portal frame has been modelled using geometrically nonlinear beam-column elements. The standard ISO 834 fire model [22] has been adopted while the thermal transfer, convection and radiation, have been considered with the convective coefficient α equal to 25 W/m²K and the resultant emissivity ε chosen equal to 0.5 (no shadow effect has been taken into account). Eq. (4) has been used to evaluate the temperature curves of steel members [2]. This leads to a uniform distributed temperature in the cross section. Haunches have not been included in this model.

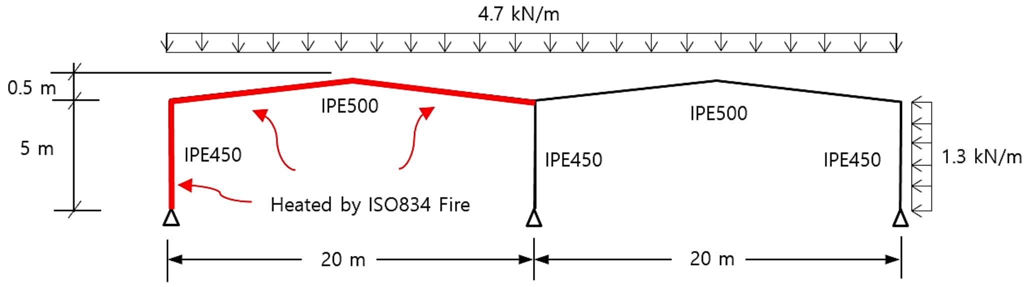
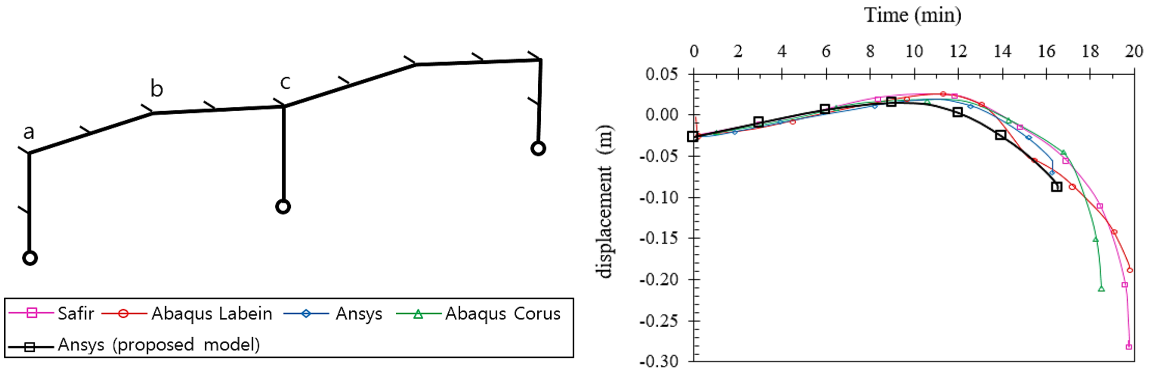


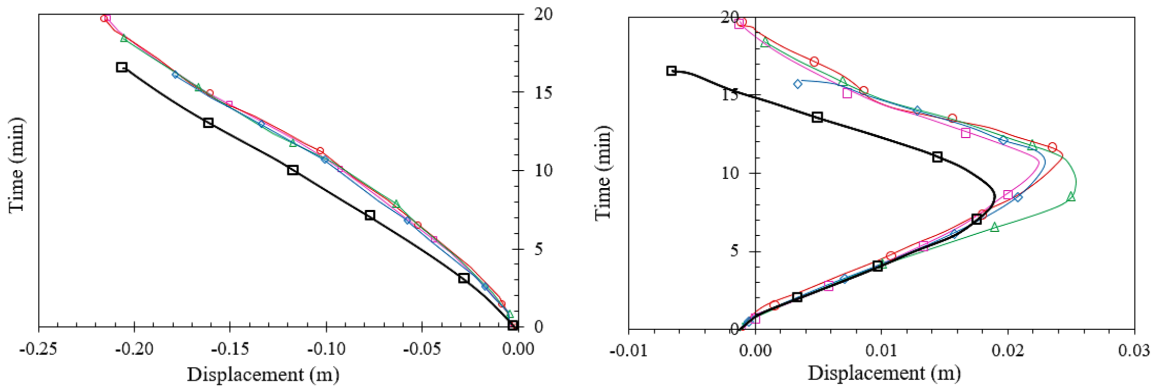
Figure 19. Details of double span after Vassart et al [11]

In this present analysis, the double portal frame was modelled with the same finite element mesh as described previously.

Figs. 20(b)-(d) show the evolution of the horizontal and the vertical displacements at different places (nodes a to c) with respect to time.



(a) Double frame model (b) Vertical displacement of node b



(c) Horizontal displacement of node a (d) Horizontal displacement of node c

Figure 20. Model validation

It seems that the different curves obtained with the proposal model are not very far from those when using Safir, Abaqus and Ansys (2D) and the difference in results could be acceptable regarding the hypothesis made for each model. Moreover, the failure time of the structure found in this study and that using Ansys (2D) are less than the other models (Safir and Abaqus). Also, the difference in results is probably attributed to the thermal model used in this study.

6. COMPARISON OF RESULTS USING THE PROPOSED MODEL AND THE SIMPLIFIED METHOD (R15)

Tables 2-3 present a comparative study between values of time resistance obtained using this study and those calculated with simplified method (R15). The later method is mainly used for industrial buildings as reported in the literature [17]. For the comparison purpose, the correction factor K_{sh} obtained from Eq. (7) is taken equal to 1 (no shadow effect).

Table 2: Time resistance using different rafter slopes (with 2.5 m haunch length): comparison between the numerical model and the simplified method (R15)

Rafter slope (deg.)	Numerical model (ANSYS)	Simplified method (R15)		
	t_{cr} (min)	$k_{y,\theta}$	θ_a (°C)	t_{cr} (min)
2	16.32	0.264	685.96	15.95 ≈ 16
2.86	16.06			
5.71	16.04			
8.33	16.03			
11.53	16.22			
14.04	16.29			

Table 3: Time resistance using different haunch lengths (with 2° rafter slope): comparison between the numerical model and the simplified method (R15)

Haunch length (m)	Numerical model (ANSYS)	Simplified method (R15)		
	t_{cr} (min)	$k_{y,\theta}$	θ_a (°C)	t_{cr} (min)
0	13.81	0.469	600.42	12.20
0.5	14.92	0.423	619.58	12.87
1	15.80	0.379	637.92	13.61
2	16.04	0.300	670.83	15.14
2.5	16.32	0.264	685.96	15.95
3	16.05	0.230	700.00	16.92
4	16.11	0.169	750.83	23.49

Results from table 2 indicate that, when inclination angles are varied, time resistance obtained from the numerical simulation are in agreement with the simplified method (R15). Therefore, using inclination angles more than 10% (5.71°) does not affect the failure time of the portal frame.

Results from table 3 indicate that when the lengths of haunches are less than one-eight (2.5 m) of the span length (20 m), time resistance using simplified method (R15) is under estimated (conservative). However, when the haunches are longer than one-eight of the rafter span, the simplified method (R15) overestimates the failure time of the portal frame according to the results study.

7. CONCLUSIONS

The paper investigates the behaviour of industrial portal frames with haunches under standard ISO834 fire. The following conclusions can be drawn:

- The time resistance of the portal frame is independent of the rafter inclination angle.
- At small angles, large deflections of the rafter are observed, which leads to inward collapse of the structure. This may probably explain the cause of limiting the rafter inclination angle at 10% (slope) for the simplified method (R15).
- The increase in the inclination angle leads to the decrease of the Von Mises stresses at rafter sections. The difference in stresses values does not exceed 24% when rafter slope varies from 3.5% to 25%.
- The use of haunches of one-tenth of the span and more increases the time resistance of the portal frame around 16%. Beyond this length, no improvement can be observed. This is due to the developed of yielding stresses at eaves. For less haunch length, the critical sections change from eaves to the ends of haunches. As a result, time resistance is reduced.
- When haunch length is less than one-tenth of the rafter span, the collapse of the structure tends to happen in the outward direction.
- Time resistance results determined by the simplified method (R15) are approximately in agreement with those of the present study when varying rafter inclination angles. However, when varying haunch length up to one-eight, time resistance results according to simplified method are under estimated (conservative). Beyond this length, the results are overestimated.

REFERENCES

1. Boudrouma A. Skikda: Incendie à répétition à la plateforme pétrochimique, Le quotidien d’Oran news paper, Algeria, 2015.
2. EC3, EN 1993-1-2: 2005. Design of steel structures. General rules, Structural fire design.
3. Naijar S, Burgess I. A nonlinear analysis for three-dimensional steel frames in fire conditions, *Engineering Structures*, **18**(1996) 77-89.
4. Landesmann A, de M. Batista E, Drummond Alves JL. Implementation of advanced analysis method for steel-framed structures under fire conditions, *Fire Safety Journal*, **40**(2005) 339-66.
5. Toric N, Harapin A, Boko I. The behaviour of structures under fire – numerical model

with experimental verification, Steel and Composite Structures, **15**(2013) 247-66.
6. O'Connor MA, Martin DM. Behavior of a multistory steel-framed building subjected to fire attack, *Journal of Constructional Research*, Nos. (1-3)**46**(1998), Page No. 169.
7. Yin YZ, Wang YC. A numerical study of large deflection behaviour of restrained steel beams at elevated temperatures, *Journal of Constructional Steel Research*, **60**(2004) 1029-47.
8. Mesquita LMR, Piloto PAG, Vaz MAP, Vila Real PMM. Experimental and numerical research on the critical temperature of laterally unrestrained steel I beams, *Journal of Constructional Steel Research*, **61**(2005) 1435-46.
9. Ahn JK, Lee CH, Park HN. Prediction of fire resistance of steel beams with considering structural and thermal parameters, *Fire Safety Journal*, **56**(2013) 65-73.
10. Henneton N, Renaud C, Zhao B. Comparing simple and advanced tools for structural fire safety engineering, *Fire And Materials*, **40**(2016) 65-79.
11. Vassart O, Brasseur M, Cajot LG, Obiala R, Spasov Y, Friffin A, Renaud C, Zhao B, Arce C, De la Quintana J. Fire safety of industrial halls and low-rise buildings: realistic fire design, active safety measures, post-local failure simulation and performance based requirements, Technical Steel Research, Final report, 2007.
12. Wong YS. The Structural Response of Industrial Portal Frame Structures in Fire, Ph.D. thesis, University of Sheffield, Sheffield, UK, 2001.
13. De Souza Junior V, Creus GJ, Franssen JM. Numerical modelling of a single storey industrial building at elevated temperature - comparison between the 2D and 3D analyses, *Mecànica Computacional*, **XXI**(2002) 1986-97.
14. Franssen JM, Kodur VKR, Mason J. A Computer Program for Analysis of Structures Submitted to the Fire, University of Liege, Liege, 2001.
15. Song YY. Analysis of Steel Portal Frames under Fire Conditions, Ph.D. thesis, University of Sheffield, Sheffield, UK, 2009.
16. Song YY, Huang Z, Burgess IW, Plank RJ. The behaviour of single-storey industrial steel frames in fire, *International Journal of Advanced Steel Construction*, **5**(2009) 289-302.
17. Renaud C, Sakji S. Ossatures en acier, Méthode de justification d'une stabilité au feu de ¼ heure (R15), CTICM report, France, 2009.
18. El-heweity M. Behavior of portal frames of steel hollow sections exposed to fire, *Alexandra Engineering Journal*, **51**(2012) 95-107.
19. Huang Z, Song Y. Analysis of industrial steel portal frames in fire, *Journal of Structural Fire Engineering*, **3**(2012) 267-84.
20. Rahman M, Lim J, Hamilton R, Komleckci T, Pritchard D, Xu Y. Effect of column base strength on steel portal frames, *Proceedings of the Institution of Civil Engineers - Structures & Buildings*, **166**(2013) pp. 197-216.
21. Gentili F. Advanced numerical analyses for the assessment of steel structures under fire, *International Journal of Lifecycle Performance Engineering*, **1**(2013) 159-84.
22. ISO 834-1: 1999. Fire resistance tests - Elements of building construction – Part 1: General requirements for fire resistance testing, International Organization for Standardization, Switzerland.
23. Kmet S, Tomko M, Demjan I, Pesekc L, Priganc S. Analysis of a damaged hall subjected

to the effects of fire, *Structural Engineering and Mechanics*, **58**(2016) 757-81.
24. ANSYS®, Academic Research, Release 16.2, Help System, Element reference, ANSYS, Inc.
25. EC1, EN 1991-1-2: 2002. Actions on structure Part 1-2: General actions. Actions on structures exposed to fire.
26. EC0, EN 1990: 2002. Eurocode. Basis Structural Design.
27. Simms WI, Newman GM. Single-storey Steel Framed Buildings in Fire Boundary Conditions, The steel Construction Institute, SCI Publication P313, Berkshire, UK, 2002.