STABILITY OF STEEL PORTAL FRAMES IN INDUSTRIAL BUILDINGS UNDER NATURAL FIRE CONDITIONS

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Abstract

Fire poses high level of accidental risks for the integrity of industrial buildings. National Fire Protection Association reported over 30.000 industrial building fires in the U.S. between 2011 and 2015, which resulted in 16 civilian death and approximately $1.2 billion direct property damage. Therefore, it is crucial to understand structural behavior of these buildings under elevated temperature conditions for proper design against fire, and to be able to perform safe and effective fire-fighting activities. This study investigates the snap-through and frame sway buckling stability issues of steel portal frames in industrial buildings under a local fire event. A parametric study is performed under a specific local fire condition to investigate effects of fire, loading scenario as well as column height-to-span length ratio of steel portal frames. Three modes of frame buckling are observed: sway buckling mode, braced buckling mode and snap-through buckling mode. Results suggest that fire temperature significantly affects the critical frame buckling load at which instability of the portal frame occurs. At peak fire temperatures near 900 °C, instability capacity of the frame reduces near the corresponding column yield capacity. Further, fire affects the braced frame buckling mode more significantly than the sway frame buckling mode. With 1:1 column height-to-span length ratio, snap-through buckling is not observed. For 1:2 column height-to-span length ratio, snap-through buckling is observed as the buckling mode.

Keywords: structural fire; fire engineering; stability; steel structures.

1. Introduction

Industrial buildings are mainly constructed with steel portal frames. These buildings are single storey buildings with vast storage areas that cover large clear span distances which can only be achieved with steel. Although steel has indispensable advantages, it has low fire resistance as a construction material. Most of the industrial buildings are designed for ambient temperatures and considered as safe for stability problems. Thus, in a fire scenario where the steel temperature goes above 100 °C, steel starts to lose its stiffness and the structure might become unstable at a critical temperature.

Wong's 2003 parametric studies (cited in Song 2008) found that 2D models can accurately represent the behaviour of the steel portal frame. Steel portal frame under a localized fire is investigated; because the main load bearing steel members that are located around the localized fire region mostly determine the structural performance of the entire industrial building (Yolaçan, 2014). It is essential to comprehend and successfully determine the failing buckling mode that occurs under fire in order to prevent it with correct fire-fighting methods. The industrial building constructed with steel portal frames is given in Figure 1. The purpose of this study is to capture the change in buckling modes and buckling resistance (i.e. the critical temperature) of the portal frame at elevated temperatures under a localized fire. The heated portal frame is expected to fail in column buckling, snap-through buckling or frame sway buckling, not necessarily in that order. The order of the buckling modes is subject to change since the column is non-uniformly heated under a local fire scenario. This paper further studies the effect of height-to-span aspect ratio of the portal frame to the shift in buckling modes.

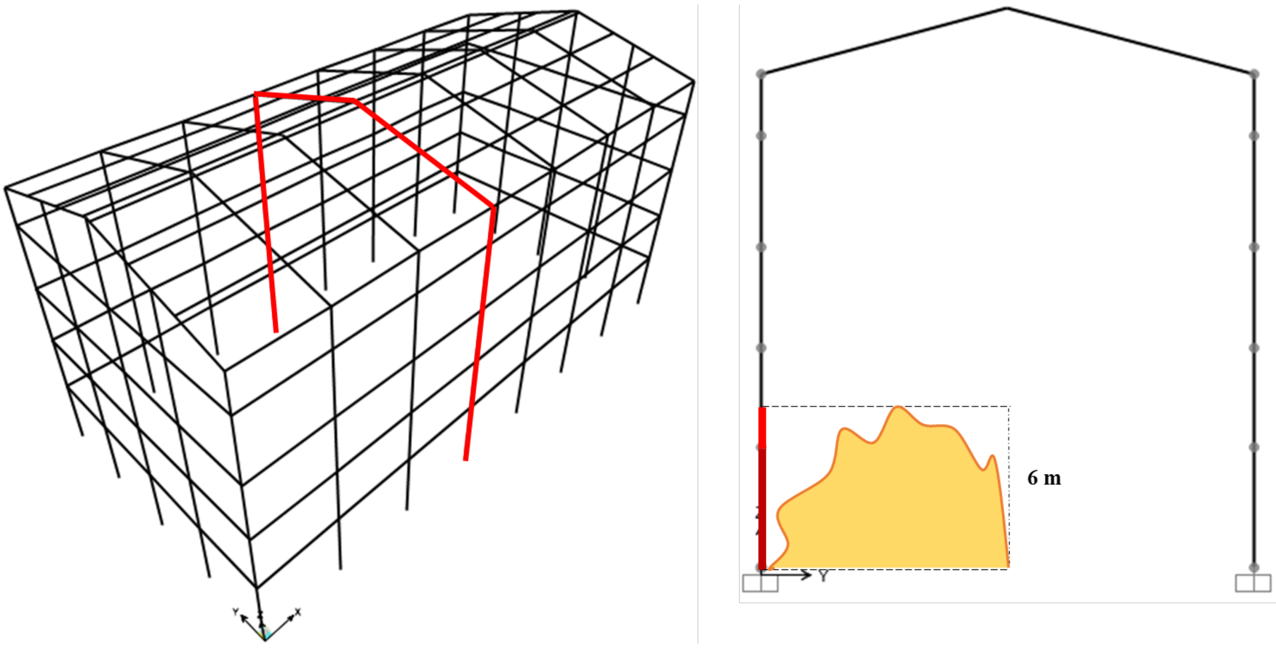


Figure 1. Illustration of the steel portal framed industrial building (Yolaçan, 2014)

2. Stability Modes of Portal Frames

In this study the load on the portal frame is held constant while the stiffness of the structural members are reduced as the temperature increases. Hence, the critical load gets closer to the applied structural load during fire. Illustration of the buckling types explained in 2.1, 2.2 and 2.3 sections are shown in Figure 2.

2.1 Column Buckling

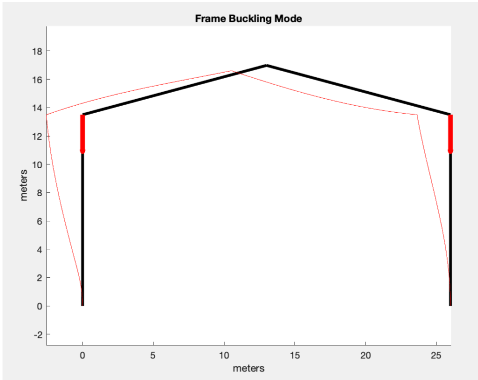
Column buckling happens when the applied load reaches the critical load and at that load column becomes unstable. Addition of any lateral force to the column will cause the column to buckle. Column buckling is less dangerous compared to the other buckling modes; because of the post-buckling strength of column and redistribution of these loads from buckled column to undamaged frame members.

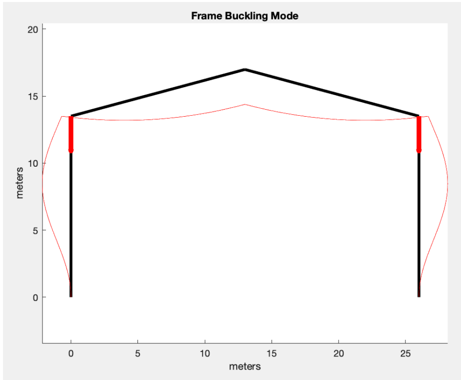
2.2 Frame Sway Buckling

Frame buckling mainly occurs during the construction of the building; because the building has no braces installed yet. The portal frame is assumed to have no braces installed. In sidesway buckling, deflection of the frame starts immediately with the load application and as the difference between applied load and the critical load, deflection of the frame is significantly amplified and eventually the structure collapses.

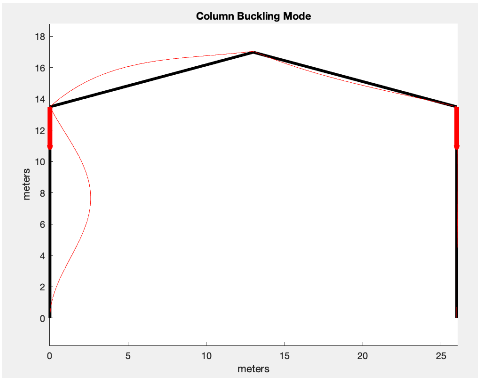
2.3 Snap Through Buckling

Snap-through buckling is a phenomenon that happens suddenly, and the failure might be catastrophic. It can be observed in structures under a compressive load on a two-bar (double pitched roof, truss system, etc.) where the two rigid bars are at an angle to each other. The bars start to deflect slowly until the snap-through point and after that point bars snap-through to buckled shape instantaneously.

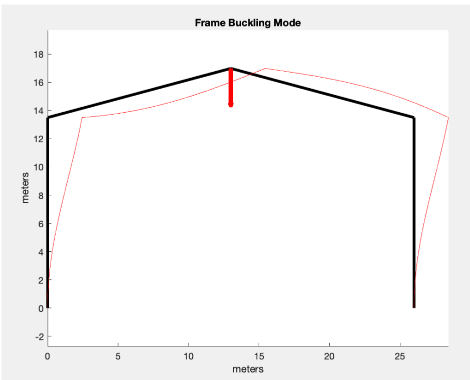


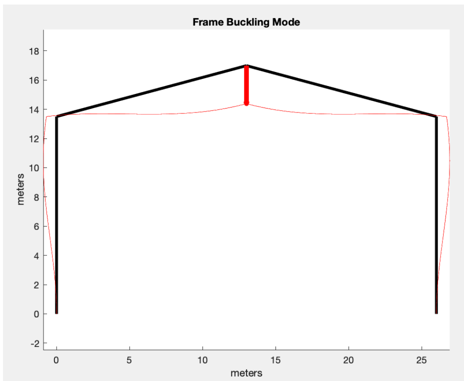


(a) (b)



(c)





(d) (e)

Figure 2. Illustration of buckling types: (a) Loading on 2 columns: Sway frame buckling, (b) Loading on 2 columns: Braced frame buckling, (c) Loading on 2 columns: Heated column buckling, (d) Loading on roof top: Sway frame buckling, (e) Loading on roof top: Snap-through buckling.

3. Case Study: Industrial Building Portal Frame

The building is 42.5 m in length, 26 m in width and 17m height. It has a floor area of 1105 m^2 and the total surface area of the building is 4235.5 m^2. Figure 3 shows the distances between each portal frame, the heated frame and the inclination angle of the roof. Heavy HEA500 steel sections are used in the portal frame for the structural members. Steel portal frames that are braced with tie-beams and purlins in out-of-plane direction are utilized as main load carrying structural system for the building.

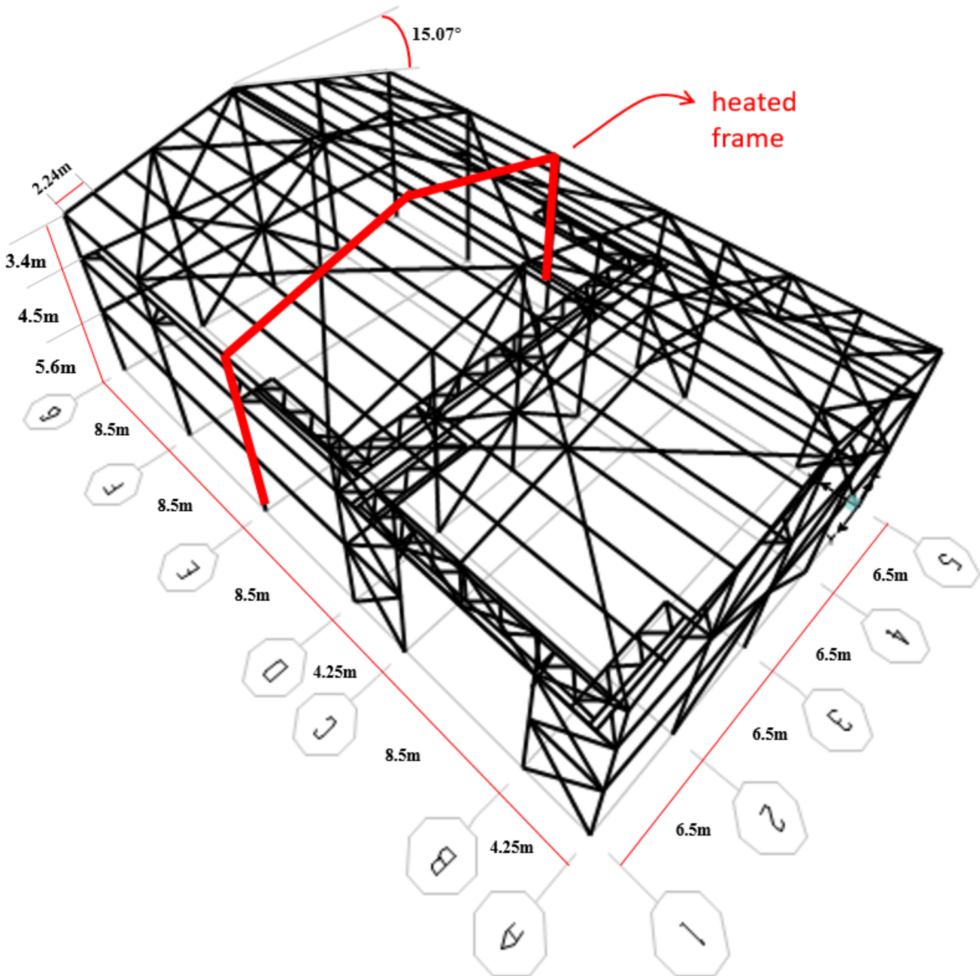


Figure 3. Dimensions and heated frame of case study portal frame (Yolaçan, 2014)

3.1 Local Fire

Local fire time-temperature history is estimated using the heat release rate simulated in Ozone software (Cadiron et al., 2001) and using Equations 1-3. Fire simulation is performed for 2 column height-to-span length ratios: 1:2 (Scenario A) and 1:1 (Scenario B). For each fire simulation, the compartment depth and the maximum fire area are changed according to the frame dimensions. General parameters used for fire simulations are given in Table 1 and Table 2; compartment details and lining materials used in ceiling, walls and floors are given in Table 3. Total fire load for the industrial building is determined with a field survey carried by the operators of the building. Fast growth fire is assumed due to large amount of highly flammable lubricant, diesel and solvent contained in the building. The maximum fire area is estimated to enclose half of the frame span and half of the distance between neighboring frames (i.e. tributary area of the heated frame). The heat release rate from the fire simulation is used as an input to calculate the time-temperature history of the local fire using Equations 1-3.

Table 1. Fire simulation parameters of the building (Scenario A)

Fire Load Density (MJ/m^2)	136.35
Fire Growth Rate	150
Total Fire Load (MJ)	15066
Maximum Fire Area (m^2)	110.5
Compartment Depth(m)	26

Table 2. Fire simulation parameters of the building (Scenario B)

Fire Load Density (MJ/m^2)	136.35
Fire Growth Rate	150
Total Fire Load (MJ)	7821
Maximum Fire Area (m^2)	57.36
Compartment Depth(m)	13.5

Table 3. Temperature properties of lining materials (Compartment boundaries)

LINING MATERIALS	Thickness (m)	Unit Mass (kg/ m^3)	Conductivity (W/mK)	Specific Heat (J/kgK)	Emissivity H.S.	Emissivity C.S.
WALLS						
S.S. Metal Panel (PUR Insulation)	15	30	0.026	1470	0.8	0.8
FLOOR						
N.W. Concrete	20	2300	1.6	1000	0.8	0.8
CEILING						
S.S. Metal Panel (PUR Insulation)	15	30	0.026	1470	0.8	0.8
N.W. =Normal Weight, H.S.= Hot-Surface, C.S.= Cold-Surface, S.S = Self-supporting, PUR = Rigid Polyurethane						

Fire simulation results for Scenario A are shown in Figure 4. Flame length temperatures 1-3 m, 4-6 m, fire area and the hot zone temperatures for the entire duration of the fire simulation are given in Figure 4a-4d, respectively. Fire simulation results for Scenario B are shown in Figure 5a-5d. Flashover does not occur in any of the fire scenarios due to hot zone temperatures remaining smaller than 500 ˚C. Maximum flame length reached for fire Scenario A and Scenario B are 6 m and 5.44 m, respectively. The flames do not impact the ceiling. The flame lengths are calculated using Heskestad method Equations 1-3. Here, D is the flame diameter in meters, Q is the rate of heat release in Watt. For the calculation of the temperatures (T_z) which is given in Equation 2; z, flame height for the desired flame temperature and z_0, virtual origin of the fire source is needed; z_0 is calculated with Equation 3.

$$L_f = -1.02\,D + 0.0148\,Q^{0.4} \tag{1}$$

$$T_z = 20 + 0.25(0.8\,Q)^{2/3} + (z - z_0\,)^{-\frac{5}{3}} \leq\; 900^oC \tag{2}$$

$$z_0 = 0.00524\,Q^{0.4} - 1.02\,D \tag{3}$$

Flame length temperatures are calculated up to 6m and given in Table 4 and Table 5 for Scenario A and Scenario B, respectively. The difference of fire temperatures between Scenario A and Scenario B is more pronounced at and above 3 meters. Beyond 6m, the fire temperature drops significantly. Overall, flame temperature values are higher for Scenario A compared to Scenario B.

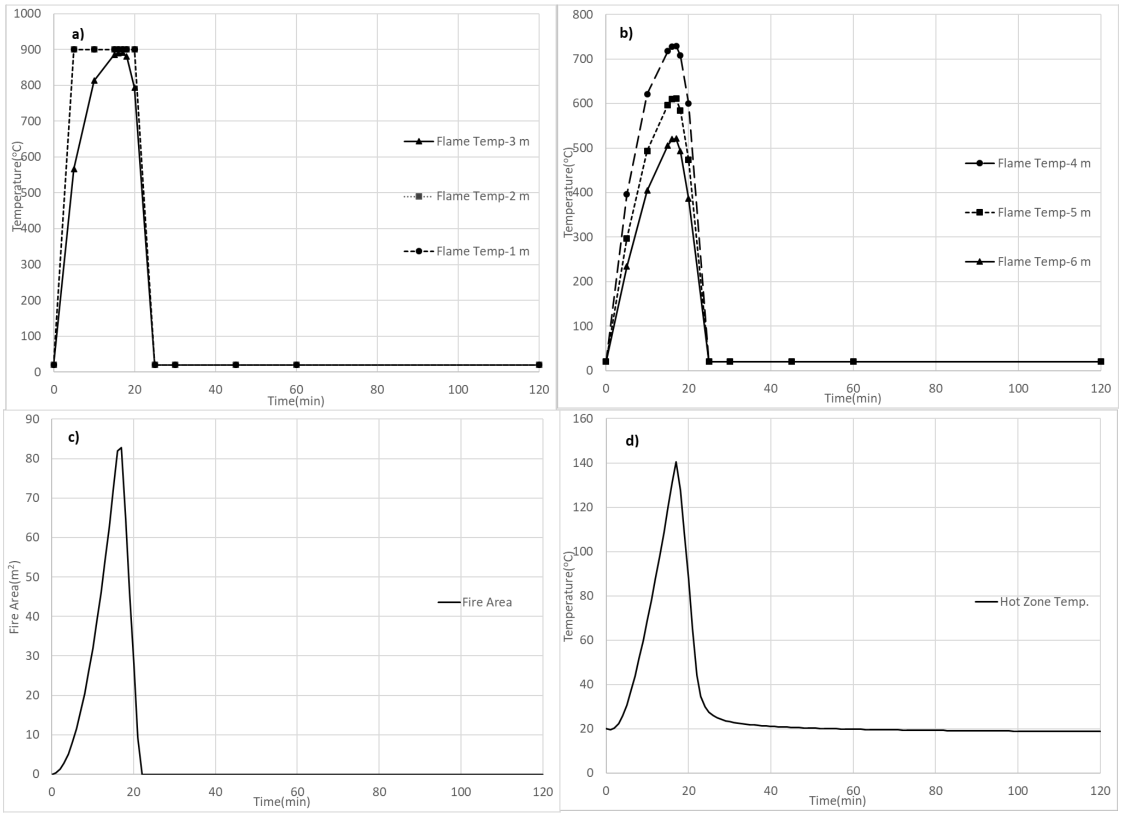


Figure 4. Scenario A, (a) Flame length temperature (1-3 m) vs time, (b) Flame length temperature (4-6 m) vs time, (c) Fire area (82.8m^2 at 17 min). (d) Hot zone temperature (Peak:140 $^\circ$C at 17 min).

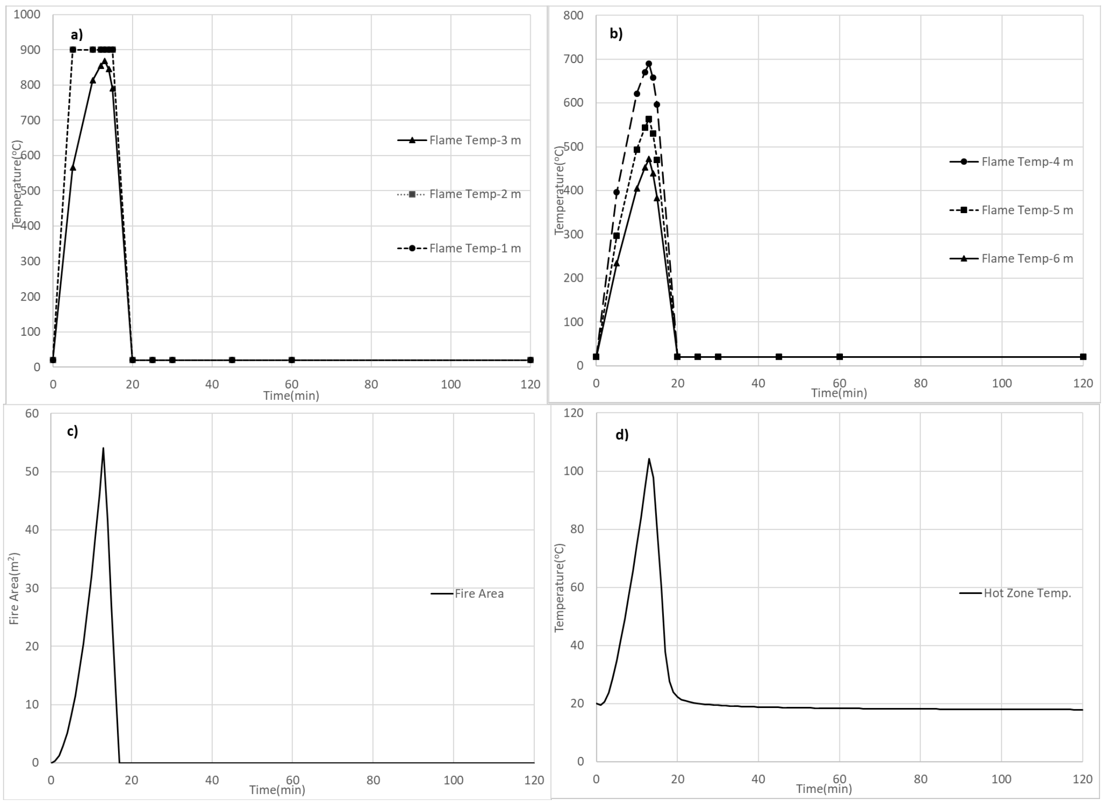


Figure 5. Scenario B, (a) Flame length temperature (1-3 m) vs time, (b) Flame length temperature (4-6 m) vs time, (c) Fire area (54.1m^2 at 13 min), (d) Hot zone temperature (Peak:104 $^\circ$C at 13 min).

Table 4. Scenario A: Flame length temperatures

Scenario A - Fire 1:2 Ratio								
Time (min)	L_f (m)	z_0	Temp (°C) at 1m	Temp (°C) at 2m	Temp (°C) at 3m	Temp (°C) at 4m	Temp (°C) at 5m	Temp (°C) at 6m
0	0.00	0.00	20	20	20	20	20	20
5	3.22	-0.96	900	900	567	396	297	234
10	4.76	-2.52	900	900	813	621	494	405
15	5.82	-4.25	900	900	885	718	597	506
16	6.00	-4.61	900	900	890	728	610	520
17	6.01	-4.64	900	900	891	729	611	521
18	5.67	-3.96	900	900	880	708	584	493
20	4.58	-2.29	900	900	793	600	473	386
45	0.00	0.00	20	20	20	20	20	20
60	0.00	0.00	20	20	20	20	20	20

Table 5. Scenario B: Flame length temperatures

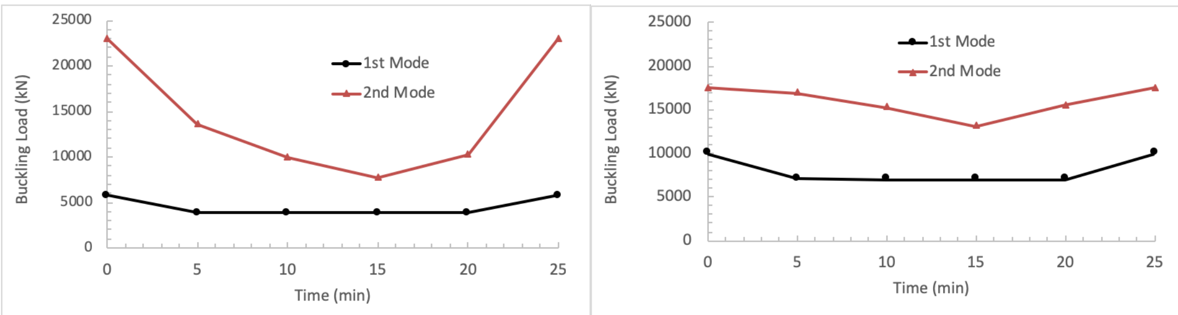
Scenario B - Fire 1:1 Ratio								
Time (min)	L_f (m)	z_0	Temp (°C) at 1m	Temp (°C) at 2m	Temp (°C) at 3m	Temp (°C) at 4m	Temp (°C) at 5m	Temp (°C) at 6m
0	0.00	0.00	20	20	20	20	20	20
5	3.22	-0.96	900	900	567	396	297	234
10	4.76	-2.52	900	900	813	621	494	405
12	5.23	-3.20	900	900	855	670	544	452
13	5.44	-3.54	900	900	868	689	564	472
14	5.10	-3.00	900	900	845	658	530	439
15	4.55	-2.25	900	900	790	596	470	383
20	0.00	0.00	20	20	20	20	20	20
45	0.00	0.00	20	20	20	20	20	20
60	0.00	0.00	20	20	20	20	20	20

3.2. Buckling Analysis of the Portal Frame

In this section, the buckling analysis of the portal frame illustrated in Figure 1 is conducted. The columns in the frame are HEA500 sections with 13.5m in height and the beams have a span of 26m. The column length-to-span length ratio is for this frame is 0.5. The frame has no fire protection. The columns are assumed to be fixed to the ground. The beams on the roof are sloped at 15 degrees where the ultimate height of the roof reaches to 17m. The buckling load and buckling modes for the frame are found by conducting an eigenvalue analysis in Matlab using structural frame element stiffness and geometric frame element stiffness matrices, which are shown below. Here, $\overline{K_e}$ is the frame element stiffness matrix, EI is the flexural rigidity, EA is the cross sectional axial rigidity, $\overline{K_g}$ is the geometric stiffness matrix, N is the axial load and L is the length of the frame element, respectively:

$$\bar{\boldsymbol{K}}_{\mathrm{e}} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \qquad \bar{\boldsymbol{K}}_{\mathrm{g}} = \frac{N}{L} \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{6}{5} & \frac{L}{10} & 0 & -\frac{6}{5} & \frac{L}{10} \\ 0 & \frac{L}{10} & \frac{2L^2}{15} & 0 & -\frac{L}{10} & -\frac{L^2}{30} \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{6}{5} & -\frac{L}{10} & 0 & \frac{6}{5} & -\frac{L}{10} \\ 0 & \frac{L}{10} & -\frac{L^2}{30} & 0 & -\frac{L}{10} & -\frac{2L^2}{15} \end{bmatrix}$$

The frame is loaded with two different loading types as depicted in Figure 2. Local fire conditions are applied to the column on the left hand side at 1m increments up to 6m as discussed in the previous section. The elasticity modulus E is reduced with respect to local fire temperatures according to Eurocode 3 (CEN, 2005). Since the steel sections are left unprotected, the HEA500 steel section temperature is assumed to be approximately equal to the local fire temperatures at column heights 1m through 6m. For reference, it is important to note that HEA500 column nominal yield capacity is 6812kN for Grade 50 steel. Although the column is tall, due to its small slenderness, its nominal buckling strength is estimated as 19220kN with fixed-pinned boundary conditions. However, as the next section will explain, the frame buckling capacity yields to much smaller values compared to column buckling only. Hence, this section focuses on the frame buckling capacities in fire conditions only. As a parametric study, the column height-to-span length ratio is varied from 1:2 (Scenario A) to 1:1 ratio (Scenario B) in order to see the effect of column length-span length ratio on the frame buckling strength reduction with fire.



(a) (b)

Figure 7. (a) Scenario A, Loading Type 1: Frame buckling capacity versus time, (b) Scenario A, Loading Type 2: Frame buckling capacity versus time.

For Scenario A (column-to-span ratio 1:2), frame buckling loads and their corresponding first two buckling modes at the peak temperature of local fire are shown in Table 6 below. The frame is loaded on two columns with equal loads (i.e. loading type 1). The 1st mode is *frame sway buckling* and the 2nd mode is *unsway (braced) frame buckling*. As Figure 7a suggests, at ambient temperature (20 °C), the frame buckling load of the 1st mode is 15% smaller than the column yield capacity. As the local fire temperatures at the heated column increase, the frame buckling load reduces to 3829kN (i.e. 33% decrease compared to ambient temperatures). A more significant reduction (up to 66%) is seen in the 2nd mode. This suggests that the 2nd mode (braced frame buckling) is more susceptible to column heating.

Table 6. Buckling load and buckling modes for Loading Type 1

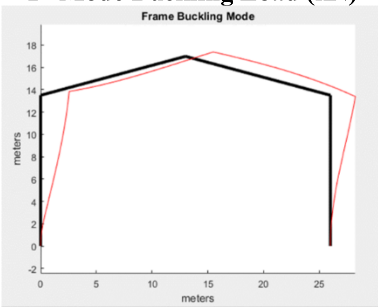
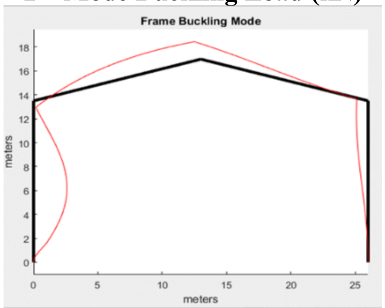
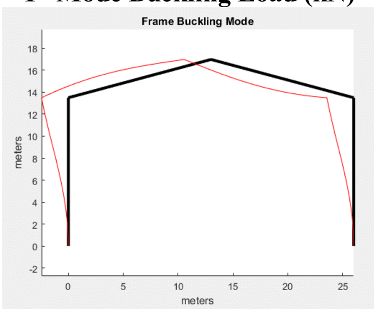
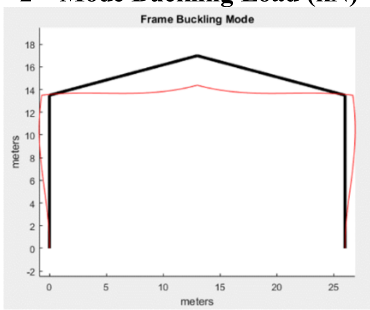
Time (min)	1st Mode Buckling Load (kN) 	2nd Mode Buckling Load (kN) 
0	5760	22993
5	3881	13584
10	3852	9929
15	3829	7780
20	3854	10266
25	5760	22993

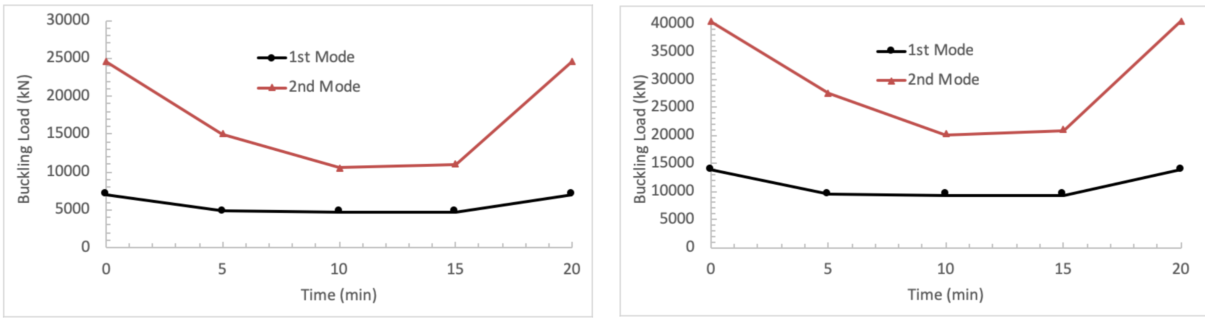
Table 7. Buckling load and buckling modes for Loading Type 2

Time (min)	1st Mode Buckling Load (kN) 	2nd Mode Buckling Load (kN) 
0	9996	17515
5	7050	16873
10	6997	15282
15	6959	13117
20	7002	15514
25	9996	17515

When the portal frame is loaded from the roof only (i.e. loading type 2), the 1st mode is *frame sway buckling* but the 2nd mode changes to *snap-through buckling* as seen in Table 7. As Figure 7b shows, at ambient temperature (20 °C), the frame buckling load of the 1st mode is 45% larger than the column yield capacity. As the local fire temperatures at the heated column increase, the frame buckling load reduces to almost column yield capacity at 6959kN. For loading type 2, both 1st and 2nd modes reduce similarly with fire, but the reduction is not as significant compared to the results from loading type 1.

For Scenario B (column-to-span ratio 1:1), frame buckling loads and their corresponding first two buckling modes at the peak temperature of local fire are shown in Table 8. The frame is loaded on two columns with equal loads (i.e. loading type 1). The 1st mode is *sway frame buckling* and the 2nd mode is *unsway (braced) frame buckling*. As Figure 8a suggests, at ambient temperature (20 °C), the frame buckling load of the 1st mode is only 3% larger than the column yield capacity. As the local fire temperatures at the heated column increase, the frame buckling load reduces to 4740kN (i.e. 33% decrease compared to ambient temperatures). A more significant reduction (up to 57%) is seen in the 2nd mode. This suggests that the 2nd mode (braced frame buckling) is more susceptible to column heating.

When the portal frame is loaded from the roof only (i.e. loading type 2), the 1st mode is *sway frame buckling* but the 2nd is *braced frame buckling* as seen in Table 9. As Figure 8b shows, at ambient temperature (20 °C), the frame buckling load of the 1st mode is 103% larger than the column yield capacity. As the local fire temperatures at the heated column increase, the frame buckling load reduces to 9344kN. For loading type 2, 1st mode does not reduce significantly, but 2nd mode reduces at a larger rate with fire as seen in Figure 8b. Again, this suggests that the 2nd mode (braced frame buckling) is more susceptible to column heating for both loading type 1 and loading type 2.



(a) (b)

Figure 8. (a) Scenario B, Loading Type 1: Frame buckling capacity versus time, (b) Scenario B, Loading Type 2: Frame buckling capacity versus time.

Table 8. Buckling load and buckling modes for Loading Type 1

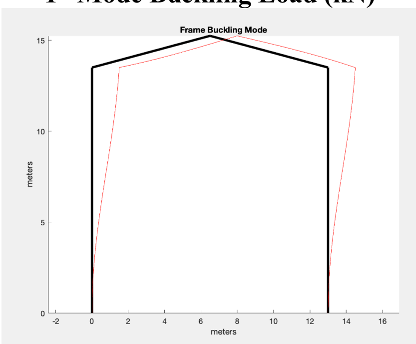
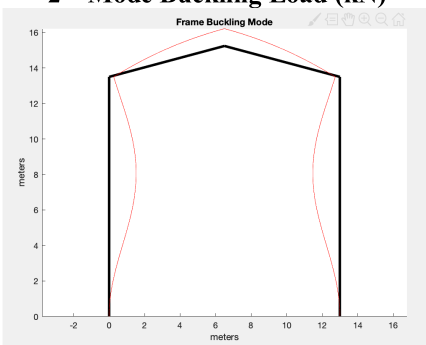
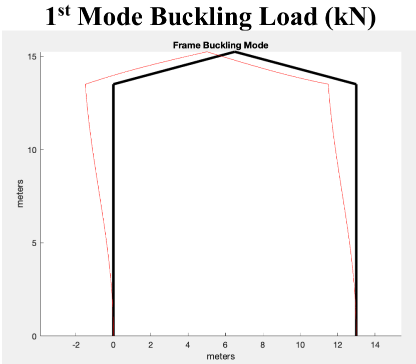
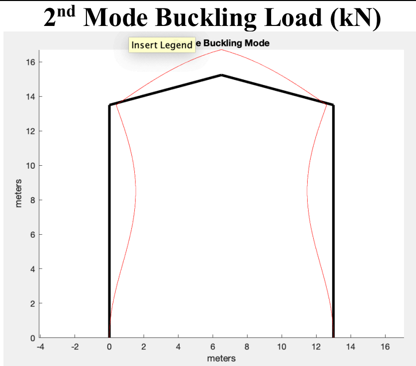
Time (min)	1st Mode Buckling Load (kN) 	2nd Mode Buckling Load (kN) 
0	7037	24541
5	4779	14994
10	4740	10566
15	4746	11027
20	7037	24541

Table 9. Buckling load and buckling modes for Loading Type 2

Time (min)	1st Mode Buckling Load (kN) 	2nd Mode Buckling Load (kN) 
0	13854	40311
5	9422	27478
10	9344	20122
15	9356	20947
20	13854	40311

4. Conclusion

Results of this study show that higher span length will yield to a local fire with larger fire area, fast growth and higher peak temperature compared to localized fires in building with a small span length. Buckling analysis of the portal frame under local fire (i.e. with the heated column) shows that the buckling capacity reduces for both loading type 1 (two column loading) and loading type 2 (roof top loading). The 1st mode is generally the sway frame buckling mode, which does not change with different column height-to-span length ratio. As fire reaches to peak temperatures, there is a significant reduction in the buckling capacity of the 2nd mode, which is generally braced frame buckling for loading type 1 and snap-through buckling for loading type 2. The 2nd mode is snap-to-through buckling if the span is larger relative to the column height. The reduction in the capacity of the 2nd buckling mode suggests that the braced frame buckling is more susceptible to local fire conditions than the sway frame buckling. In this study, a shift in the buckling modes is not observed; but results show that stability design must be rechecked for fire conditions. For different beam and column sections, it is possible that the capacity of the braced frame buckling (or snap-through buckling) reduces below the capacity of the sway frame buckling, which means that the 1st mode could become braced frame buckling rather than sway frame buckling (i.e. a shift in the buckling modes).

Future research could include fire induced forces and large deformations for stability design. Another parametric study could be conducted with different beam and column sections. Moreover, the integrity of lateral bracings on the tall columns against fire could be investigated.

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