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Research Article

INVESTIGATING DURABILITY TIME INTERVAL OF THREE-DIMENSIONAL STEEL FRAME WITH ANGLE CONNECTIONS UNDER ELEVATED TEMPRATURE

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ABSTRACT

During recent years, much attention has been paid to the analysis of fire effect on steel structures because of fire importance and its effect on the stability of steel structures. Considering steel sensitivity to heat and high costs of steel frames retrofitting with the help of thermal covers, accurate behavior analysis of metal frames in elevated temperatures is required in order to reduce undesirable effects of temperature increase. To do so and taking into account the high costs of lab tests and their limitations in investigation of numerous parameters during any test, finite element method is used as a powerful and valuable tool in modeling of steel frames under thermal conditions. In this study, the fire effect on steel beams is studied considering the softening of connection and decrease of materials strength. Then, structure durability in fire will be analyzed in the ANSYS software. The analyzed frame is a single span three dimensional one and different conditions including connection type, longitudinal expansion effect, thermal loading and the kind of fire will be analyzed and compared. The obtained computer results will be compared with other researches results.

Keywords: steel structures; fire loading; temperature; connection.

INTRODUCTION

In this paper, the behavior of a member inside a compartment in a steel building is analyzed under fire loading. Due to the complexity of design and structures analysis under fire conditions, the compartment fire test is used in most of researches [1-4]. Since the compartment test is an analytical method and design rules are obtained with its help, recent studies on steel structures behavior under fire loading demonstrate that the effect of node behavior on the structure's overall response is important and significant. Lack of experimental results from steel nodes behavior under fire conditions, and use of numerical models which are dependent on experimental relations and are based on experiments in the room temperature with low limited temperatures, lead to simple characteristics of the current codes. Considering the European code 3 section 1.2 reference 2 and part 2 of section 1.8 reference 3, mass concentration inside the node region leads to a delay in temperature increase at this region in comparison to the connection members and thus it is suggested that the nodes must be neglected in fire conditions. However, unlike the recent experiments results, this study shows that it is needed to assess the behavior of steel nodes in elevated temperatures since they exhibit significant reduction of strength and hardness in elevated temperatures which will have obvious effect on the structure's overall response.

Among researches, development of member method can be mentioned as a method to predict steel nodes response under fire loading which has been done by Simoes et al. In this study, the member method is suggested to predict steel nodes behavior under fire loading and an analytical process is proposed to assess steel nodes behavior under fire loading [4]. Bradford et al. presented numerical study of a steel beam subjected to fire in a steel compartment which analyzed members' response in elevated fire temperatures under members' slenderness ratio and the restraints provided by cooler adjacent members and the applied thermal regime were investigated [5].

Sokol et al. presented a numerical test study of a structure in fire. This study is focused on fire test of an eight-floor building, large building test facility (LBTF) of Europe, in Cardington. The aim of fire test on the whole frame structure of Cardington is to gather information about the performance of usual beam to column and beam to beam connections in firing conditions. By frame simulating, the real structure response can be determined and proper firing and mechanical loads can be applied to the structure [6].

According to above descriptions and considering that the structure behavior is so complicated in elevated temperatures and also the cost and complexity of experimental analyses, the structure behavior is studied separately during recent years including the analysis of beam and column behavior; and the effects of this element's behavior under this special loading is applied to the whole structure and its effect is investigated on the whole structure.

Table 1. Decrease of steel hardness and strength according to regulation EC3 [3]

Steel temperature, θ_S [°C]	Reduction factor for yield stress f_y, and Young's modulus E_S, at steel temperature θ_S	
	$k_{y,\theta} = f_{y,\theta} / f_y$	$k_{E,\theta} = E_{S,\theta} / E_S$
20	1.0	1.0
100	1.0	1.0
200	1.0	0.9
300	1.0	0.8
400	1.0	0.7
500	0.78	0.6
600	0.47	0.31
700	0.23	0.13
800	0.11	0.09
900	0.06	0.0675
1000	0.04	0.0450
1100	0.02	0.0225
1200	0.0	0.0

STEEL MECHANICAL CHARACTERISTICS IN ELEVATED TEMPERATURES

Steel behavior simulation and decrease of its characteristics under elevated temperatures are so important in behavior modeling of steel structures under thermal conditions. In this study, this simulation is done based on the relations of European regulations [3].

Table 2. Variations of steel specific heat with temperature increase [3]

Steel temperature, θ_S [°C]	Specific heat [J/(kg·K)]
20	440
100	448
200	530
300	565
400	606
500	667
600	760
700	1009

Table 3. Variations of steel thermal conductivity with temperature increase [3]

Steel temperature, θ_S [°C]	Conductivity [W/(m·K)]
20	53
100	51
200	47
300	44
400	41
500	37
600	34
700	31

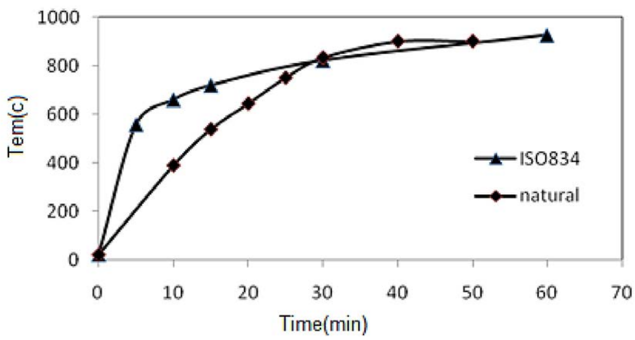


Fig. 1. Time-temperature diagram in standard and natural fires [3]

YAHYAI AND SAEDI EXPERIMENTS

Connection geometry

Two kinds of experiments are performed on angle bolted connections in order to show behavior of steel connections in normal temperatures and under fire conditions:

- 1st group of connections: specimens without web angle (SOW),
- 2nd group of connections: specimens with web angle (SWW).

Connections of the first group (SOW) are consisted of two angles; one is connected to the topflange of the beam and the other is connected to the beam's bottom flange and then the total system is bolted to the column flange. Every angle is connected to the beam flangeby*6*bolts of *M16* and is connected to the column flangeby*2*bolts of *M16*. Details of this group of connections are demonstrated in Figure 3.

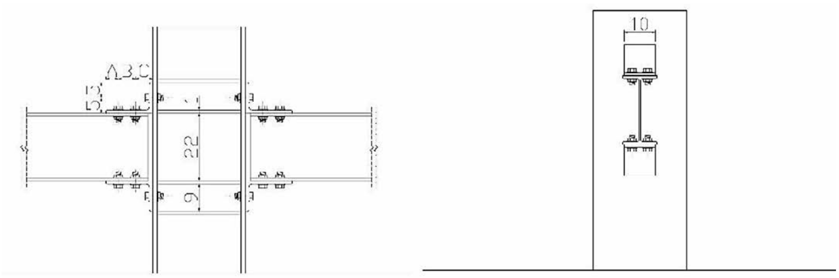
Connections of the second group (SWW) have two additional angles comparedto two angles of the SOW group. They are connected to the beam web from one side and are bolted to the column flange from the other side. Web angles are connected to the beam webby two *M16*bolts or to the column flangeby two *M16*bolts. The details are illustrated in Figure 2. Considering the failure of the tested specimens in the reference, and the finite element modeling limitations in case of modeling the boltsthreads and investigating their effect on behavior of the whole structure, four samples of connections which are analyzed in the reference are chosen. Two of them are from connections of group one and the two others are related to the connections of group two. Characteristics of the specimens are presented in Table 4 for every specimen [7, 8].

Since moment-rotation diagrams are so important and usable in connections design, moment-rotation diagrams of these four types of connections are shown in different temperatures in Figure 3 with the help of finite element model. For all of the experiments, reduction of connection characteristics is predictable with the increase of temperature.

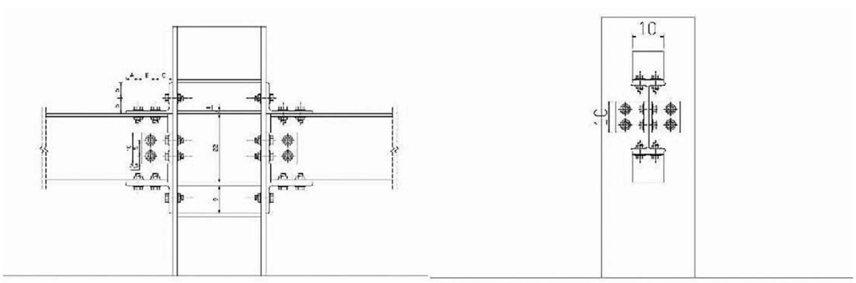
Figure 3 shows that with increase of temperature, the strength of connection moment decreases and in general, it can be said that for these kinds of angle connections which are made with normal building steel and bolts, the connection strength can be neglected for temperatures above 800 °C.

Table 4. Characteristics of the chosen specimens from Yahyai and Saedi experiments

Specimen number	Group number	Angle size [mm]	Grade of bolt
3	1	100×100×10	8/8
5	2	150×100×15	8/8
9	1	150×100×15	8/8
13	2	100×100×10	8/8



Connections of the 1st group



Connections of the 2ndgroup

Fig. 2. Details of Yahyai and Saediexperiments

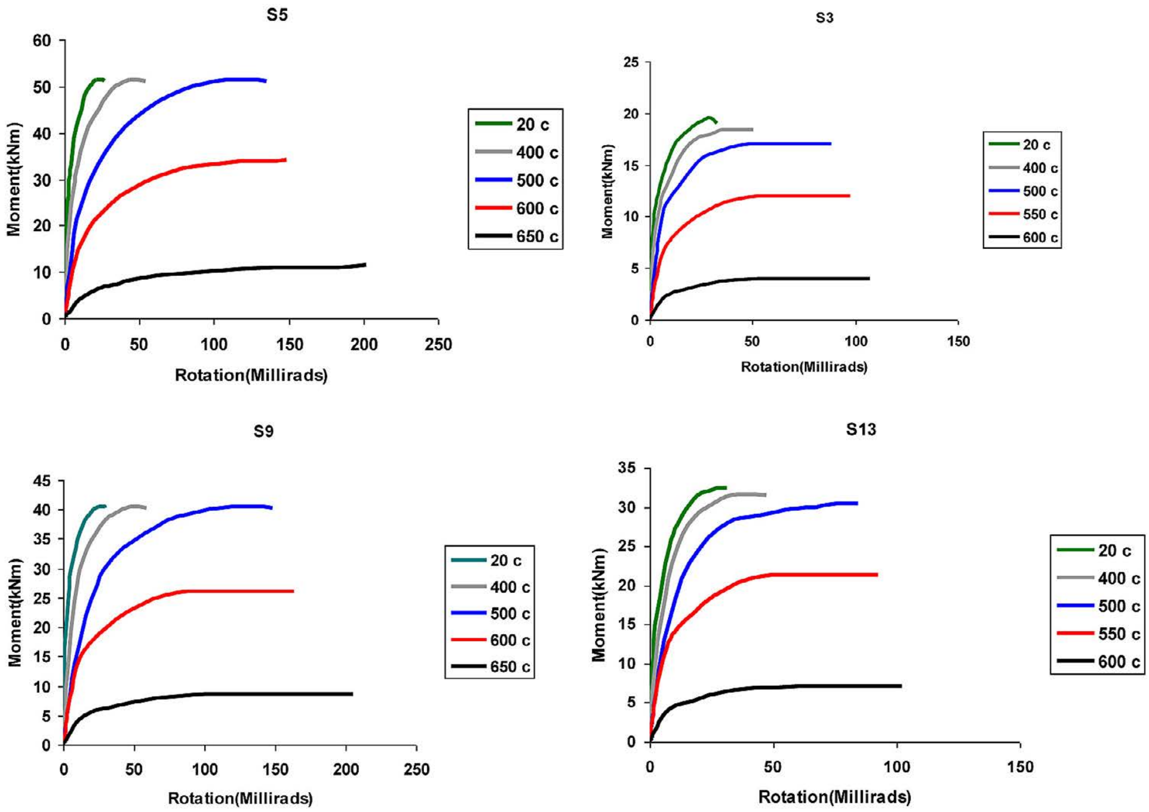


Fig. 3. diagram of moment-rotation-temperature [7]

Software validation

In this section, validation of ANSYS software with beam model of sub-assembly, which is considered by Bradford with almost the same assumptions of this study, is done by considering different conditions for the model support [5].

In Bradford's study, numerical analysis is done on the beam with *460UB82.1* section (Australian) which is located in the depth of cross section in the section geometric center with linear thermal gradient of ζ and is under specific thermal regimes with increase of T_c (mean of temperatures T_1 and T_2). The member is under uniform load of q=5 N/mm which is equal to tolerable operation load of the member. The end restraints are modeled with elastic springs and their stiffness is obtained as:

$$r_{\mathrm{L}} = r_{\mathrm{R}} = \mathrm{r} = \beta\frac{\mathrm{E_{20}I}}{\mathrm{2L}} \tag{1}$$

$$k_{\mathrm{L}} = k_{\mathrm{R}} = \mathrm{k} = \beta\frac{\mathrm{E_{20}A}}{\mathrm{2L}} \tag{2}$$

where: E_{20} – is the steel elasticity modulus in room temperature and is considered equal to 200×10^3 N/mm^{2}E$_s$,
I and *A* – are inertial moment and cross section area,
L – is the member half-lengthand,
β – is the relative stiffness of the member by which the spring stiffness can be changed easily (Figure 4).

The relative stiffness changes from B=1 (member with a great support) to B=0.01 (highly restraint member) when ζ=0.25 °C/min, the uniform load is q= 5 N/mm and the member length is 2L=6000 mm.

High levels of end restraint, unlike increase of the memberaxial force, decrease the mid-spandeflection. As it is seen in Table 5, with the increase of end restraints, deflection of the mid-span de-

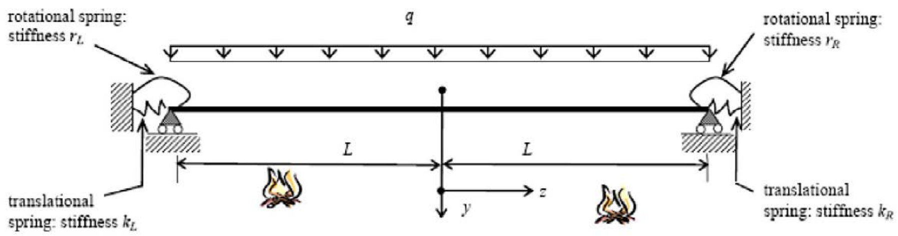


Fig. 4. Bending device system modeled by Bradford

Table 5. Comparison of present results with Bradford results

β	Bradford study, y_{max} [mm]	Present study, y_{max} [mm]
0.1	9.5	10
0.2	9	9.6
0.5	7.5	8.2
1	6	6.6

creases significantly. As it can be observed Bradford results are in good consistent with the results obtained from the ANSYS software.

PROBLEM STRUCTURE

Theory of study

Due to complexity of design and the structures analysis under firing conditions, the compartment fire test is used in most of researches because the firing test is an analytical method and prescriptive design rules are obtained with its help and numerical analysis results of the steel beam sub-assembly of the frame are presented in this study for elevated temperatures. A comprehensive beam study is presented based on generic nonlinear modeling; the effect of mechanical characteristics decrease, connection softening and also different loading conditions are analyzed in this paper in order to obtain the axial force, the mid-spandeflection, the member's critical temperature and finally the time-temperature diagram which includes the time-temperature diagram of the standard fire ISO834 and

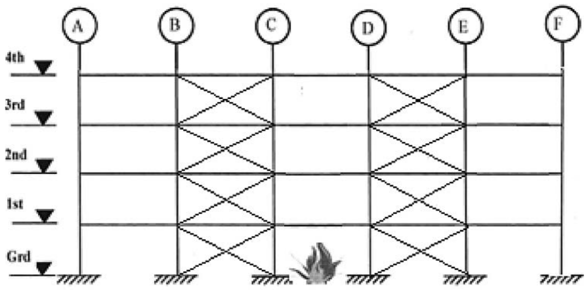


Fig. 5. lateral frame modeled under fire

the experimental diagram which is obtained from natural fire of Cardington are used to calculate the structure's durability time.

To do this study, a four-floor building with a concentric bracing frame system is modeled in the ETABS software (for seismic analysis, regulation *2800* and for design, regulation *AISC* are used). Then the lateral frame which is shown in Figure 5 is separated from this building and the fire is applied to the center beam in the ground floor and the behavior of this member (beam) in a compartment inside the steel building under fire loading is investigated.

As it is seen from Figures 6a and 6b, the steel beam with length of L is in a sub-assembly of the frame and the end lateral deflections of the beam are considered $V(Z=0, L) = 0$ and the two ends are restrained with non-elastic rotational springs (r_L, r_R) and elastic transitional springs (K_L, K_R). This member is under uniform load of q. Nonlinear behavior of the rotational spring obtained from m-Θ diagram from Yahyai and Saedi experiments for

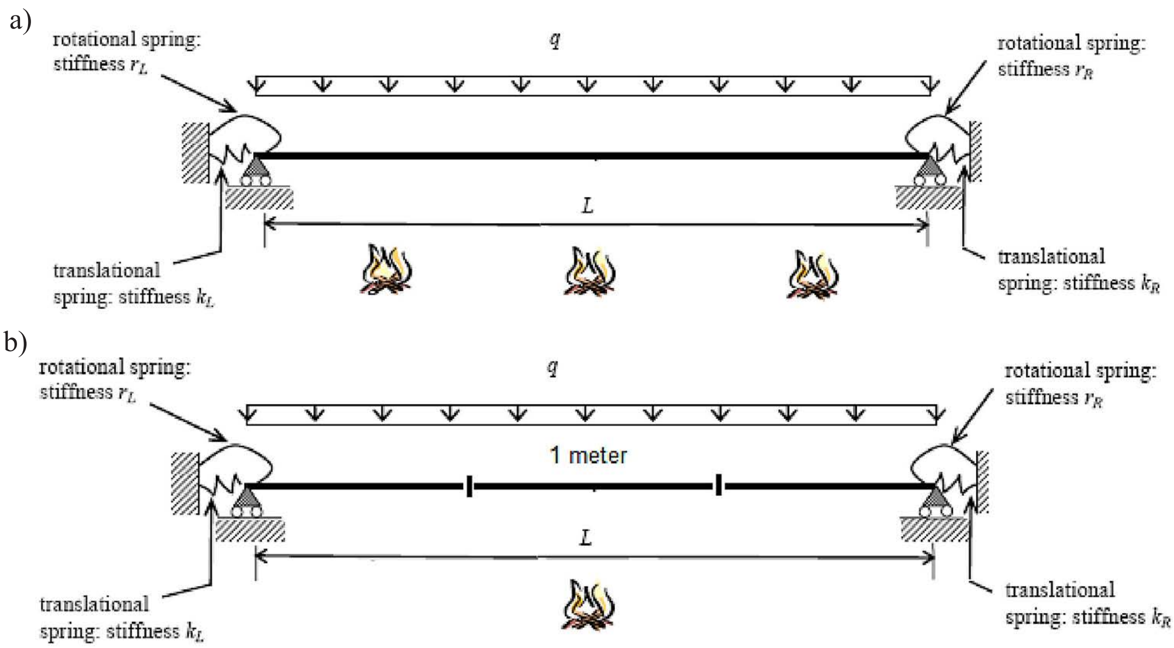


Fig. 6. Bending device system modeled in the case of: a) uniform thermal loading, b) partial thermal loading

four kinds of bolt connections (5, 13, 3, 9). Also, for linear elastic behavior of the linear spring, the lateral stiffness of the frame is considered.

The aim of this study is to compare the connection type with the slenderness ratio and their effect on the structure behavior under thermal loading. The ANSYS software is used for modeling. Two cases of loading are considered in this paper: as can be seen in Figure 6a for the first one, the temperature is applied uniformly to the whole beam and connection and all the members are under equal temperature increase; for the second one (Figure 6b), the beam's temperature increase is applied to the length of *1* meter at the center of the beam and temperature of other sections is applied, considering heat distribution of Figure 7 and Table 6.

Finally, durability time of either cases of thermal loading is obtained considering two kinds of fires including standard fire (ISO) and natural fire (Cardington).

Effect of connection type on the structure behavior

Colder beams make significant restraint for the hot steel beam during the compartment fire; also, semi-rigid connections are considered as rotational restraint of the steel beam. Warmer members which have high restraints against thermal expansion get under compressive forces. In this paper, uniform load of q=17 Kg/cm is considered which is equal to the tolerable operation load of the member and the member length is L=4000 mm.

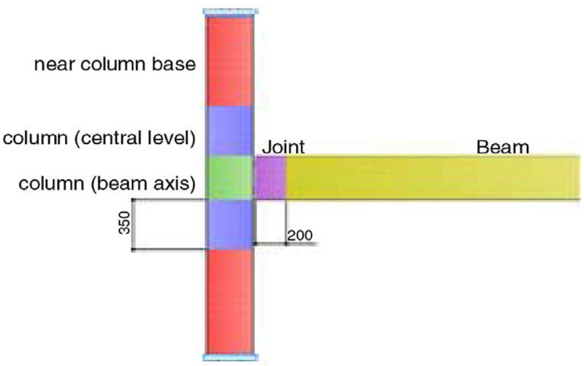


Fig. 7. Classification of different sections for natural fire thermal load applying [4]

Effect of connection type on mid-span deflectionduring uniform loading

Figure 8 shows mid-span deflection in the case that the beam is under uniform thermal loading throughout its length and connection. In this diagram, the connection type changes and other beam specifications remain constant. As it is seen, with increase of connection stiffness, mid-span deflection decreases. However, considering that the modeled connections stiffness is close to each other, the effect of connection type on the structure behavior is not so clear (especially the axial force). According to Figure 8, slope of deflection increase becomes sharp from 400 °C upward and its reason can be found in steel mechanical characteristics after this temperature (elasticity modulus reduces severely).

Effect of connection type on mid-span deflectionduring partial loading

Figure 9 shows mid-span deflection in the case that the heat is applied to the length of *1* meter of the mid-span and the temperatures of other sections are obtained from Table 6. In this case, since temperature of a part of the beam is lower than the applied temperature to the beam center and also the connection temperature is lower than the applied temperature, the system demonstrates stiffer behavior than the first case and the mid-span deflection in constant temperature in the sec-

Table 6. Temperature of beam's different sections during thermal loading similar to natural fire

Time [min]	Temperature [°C]					
	beam bottom flange (joint)	beam web (joint)	beam top flange (joint)	beam bottom flange (beam)	beam web (beam)	beam top flange (beam)
0	20.0	20.0	20.0	20.0	20.0	20.0
10	245.4	209.0	125.3	387.8	291.5	197.2
15	373.5	304.7	214.8	537.1	466.2	333.0
20	528.1	448.0	373.5	642.8	608.8	495.0
25	661.7	607.1	537.7	749.4	717.6	594.3
30	741.4	699.5	615.9	832.1	806.7	694.0
40	848.5	792.1	748.0	900.0	878.1	813.3
50	855.4	790.0	757.8	900.0	868.9	838.5

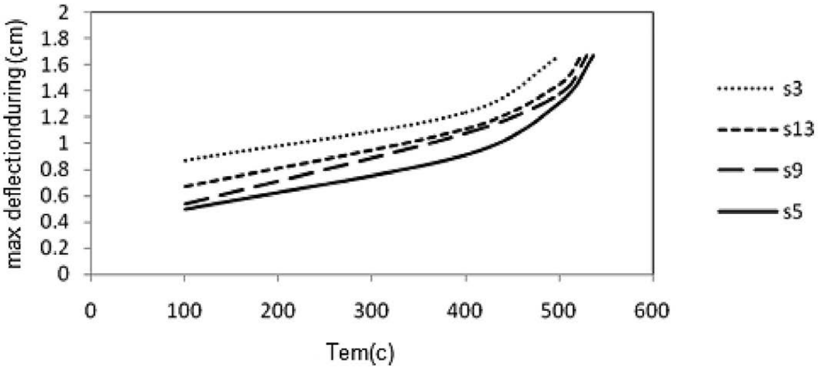


Fig. 8. Mid-span deflectionduring uniform loading

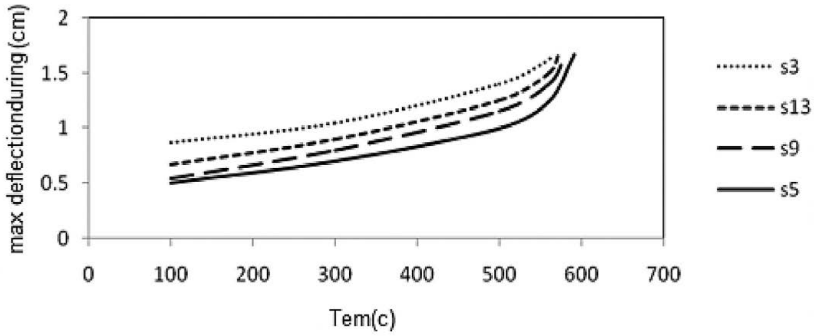


Fig. 9. Mid-span deflectionduring partial loading

ond case will be lower than the first one. This can be clearly seen by comparing Figures 8 and 9.

Comparison of structure behavior in different thermal loadings

In Figures 10 and 11, the effect of two types of thermal loadings including mid-span deflection and axial force are observed. Based on Figures 7, 8 and 9, when the applied temperature to the beam exceeds 500 °C, significant increase happens in the mid-span deflection due to high decrease of beam's elasticity coefficient at this temperature and with change of loading type from the first case to the second one, mid-span deflection increases (e.g. mid-span deflection at temperature of 500 °C is 1.1 cm for the second case and is *1.3 m* for the first case).

According to Figure 11, at first, the beam axial force increases with temperature increase and this increase continues to about 500 °C; from this temperature on, the axial force reduces until the structure fails but the deflection enhances with temperature increase. Consideration of these effects can be beneficial in designs. According to Figure 11, the axial force in the second loading is lower than the first one. The reason of this reduction is that in the same applied temperature for both beams, temperature of a part of the beam is lower than the temperature applied to the beam for the second case of loading and this decreases axial force of the second loading case, in comparison to the first one.

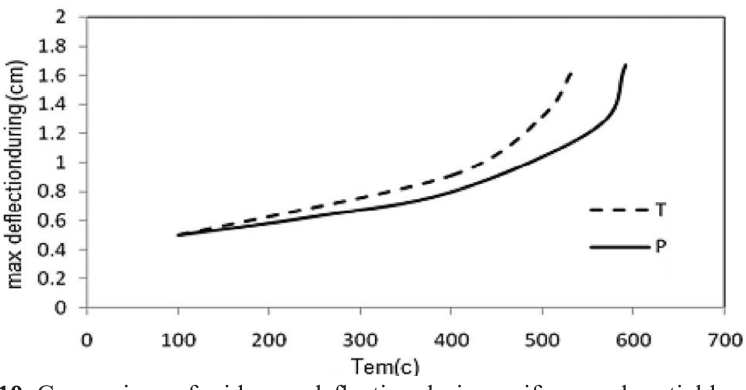


Fig. 10. Comparison of mid-span deflection during uniform and partial loadings

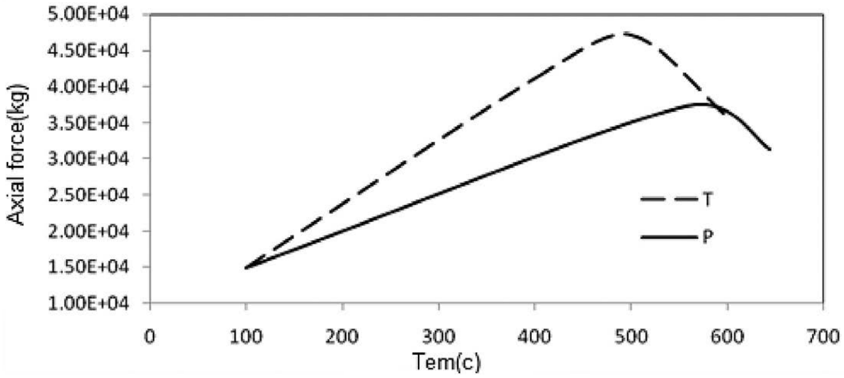


Fig. 11. Comparison of axial force during uniform and partial loadings

Comparison of critical temperature and structure durability time in different loadings

It is expected that the beam's critical temperature (temperature at which the system loses its performance) for the second loading is higher than the first one with decrease of deflection and axial force and with change of loading type and the beam fails at higher temperatures. It can be clearly seen in Figure 12 for the second case of loading that the connection is at lower temperature than the applied temperature and still has more stiffness than the first case and this makes the structure in the second case of loading more stable than the first one and certainly increases the critical temperature of the structure. In both cases, the structure critical temperature is between 500 °C and 600 °C and the structure loses its performance in this range.

As it is seen in Figures 13 and 14, the structure's durability time increases with the increase of connection stiffness. This shows direct effect of connection behavior on the structure behavior. Also, with change of loading type, the beam's durability time increases. Important point from Figure 13 is that the difference of critical temperature in most connections is almost a constant value; however, in *S5* connection, the difference measure increases, which is due to slope change of time-standard fire temperature from 580 °C on. Relative increase of durability time can be seen by bearing of this temperature by the structure and transition from this temperature.

Comparison of critical temperature and structure durability time for standard and natural fires

For comparison of time-standard fire temperature diagram with time-natural fire temperature, natural fire is used in modeling in addition

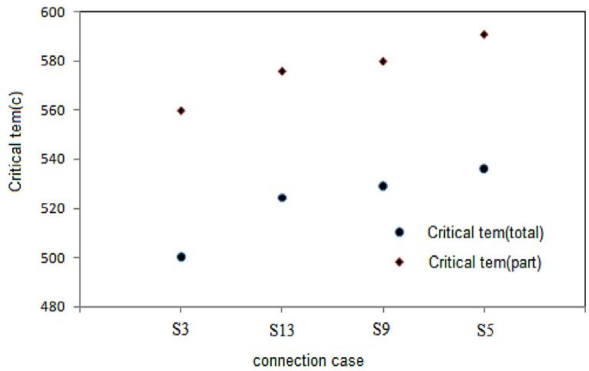


Fig. 12. Comparison of critical temperature during uniform and partial loadings

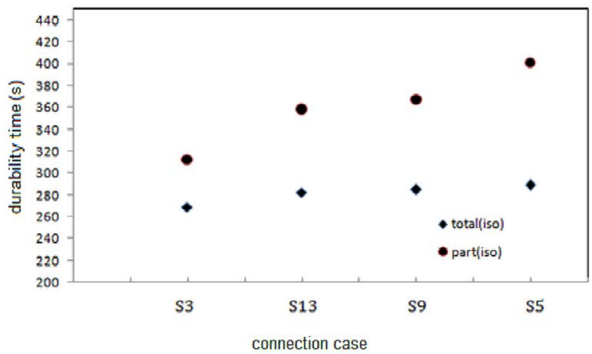


Fig. 13. Comparison of beam's durability time during uniform and partial loadings in standard fire

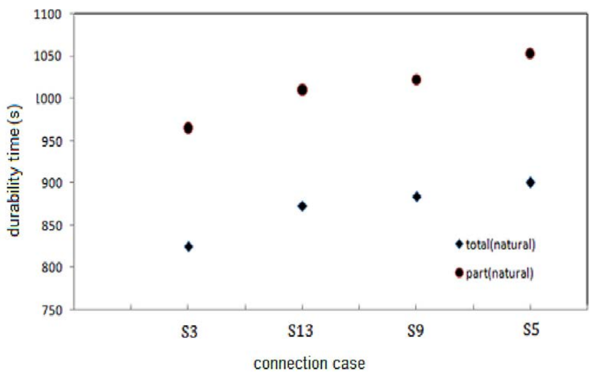


Fig. 14. Comparison of beam's durability time during uniform and partial loadings in natural fire

to the standard fire. As it is seen in Figures 15 and 16, the structure durability time in natural fire is 2-3 times the durability time of the standard fire. The reason of this great difference can be found in standard and natural diagrams of Figure 12. As mentioned before, the structure critical temperature is between 500 °C and 600 °C and considering, it can be observed that temperature increase for the standard loading has severe slope up to 580 °C and this, causes the structure durability time in the standard fire to be lower than the natural fire. Diagram of temperature increase in natural case has milder slope than the standard case and this, causes the structure durability time in natural fire to be higher than the standard fire.

Slenderness ratio effect on mid-span deflection during uniform loading

In this case, the beam's uniform load is *q*=17 Kg/cm and the connection type is *S5*; the measures of 3, 3.5, 4 and 4.5 are considered for members length and also the beam section is regarded as IPE22. In this study, the slenderness ratio is defined as the ratio of member length to the section's radius of gyration about the strong axis and the values of slenderness ratios are 32.9, 38.4, 43.9 and 49.4. As it is seen in Figure 17, the slenderness ratio has significant effect on the beam deflection and the mid-span deflection increases with increase of slenderness ratio. At temperature of 500 °C, the members' deflection with slenderness ratios of 38.42, 43.9 and 49.4 relative to the shortest member with slenderness ratio of 32.93 have increase of 33%, 67% and 167%.

Slenderness ratio effect on axial force during uniform loading

Since the beam forces are due to its length increase and also the measure of this increase has direct relationship with beam length, the formed axial force and thermal deflection in a long beam is more than a short member. Although in the first processes of the fire, thermal expansion affects the created axial load in the member, but with temperature increase, especially higher than 500 °C, due to severe decrease of steel mechanical specifications (stiffness and strength), the member axial force reduces and the mid-span deflection exhibits the maximum change (Figure 18).

Slenderness ratio effect on mid-span deflection and axial force during partial loading

In this kind of loading, the created axial force and thermal deflection are more in a long beam than a short one. In the first processes of fire,

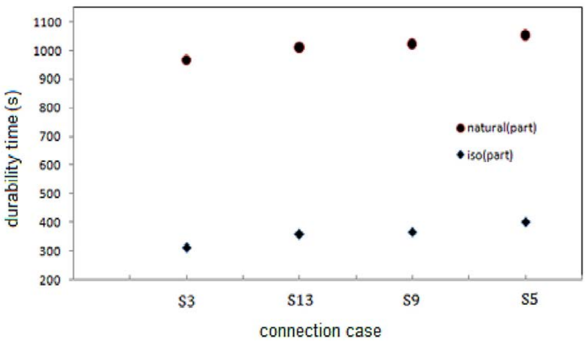


Fig. 15. Comparison of beam durability time in natural and standard fires during partial loading

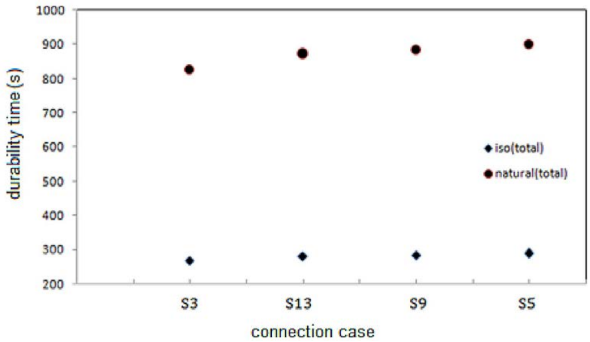


Fig. 16. Comparison of beam durability time in natural and standard fires during uniform loading

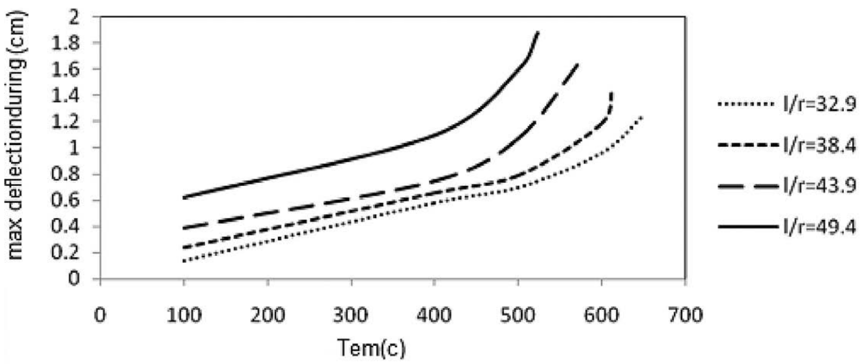


Fig. 17. Mid-span deflection during uniform loading

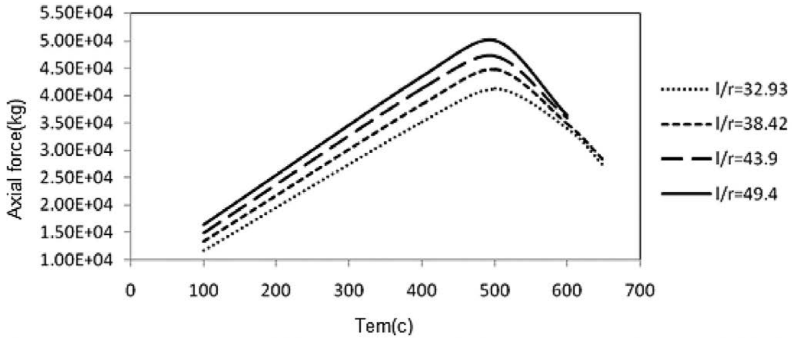


Fig. 18. Axial force during uniform loading

thermal expansion affects the created axial force but with temperature increase, especially higher than 580 °C, the axial force of the member decreases and the mid-span deflection illustrates a maximum change. As it is seen in Figure 19, the mid-span deflection in members with slenderness ratios of 38.42, 43.9 and 49.4 relative to the mid-span deflection of the shortest member with slenderness ratio of 32.93 are higher than 33%, 67% and 167%.

Comparison of critical temperature during two different loadings

Critical temperature of the second loading case is higher than the first one and this can be seen in Figure 21. The beam fails at higher temperatures and loses its performance. In both cases, the structure critical temperature decreases with the increase of slenderness ratio and the critical temperature is between 500 °C and 700 °C and the structure loses its performance in this range and it can be concluded that the slenderness ratio has the most effect at the critical temperature.

Comparison of structure durability time (standard and natural fires) during two different loadings

With increase of the beam slenderness ratio, the critical temperature decreases and hence the structure durability time reduces. Also, change of loading type from the first case to the second one increases beam durability time. This can be seen clearly in Figures 22 and 23.

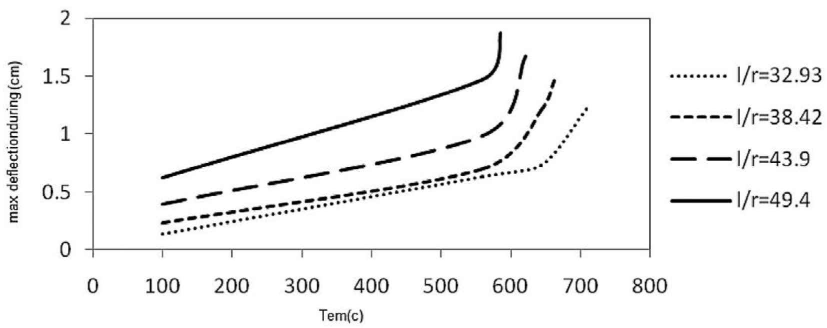


Fig. 19. Mid-span deflection during partial loading

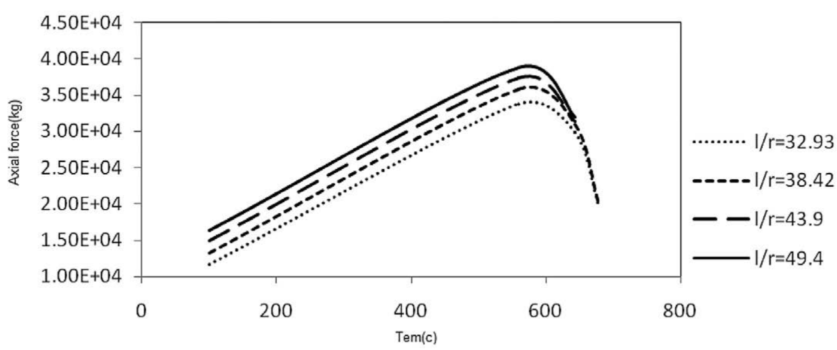


Fig. 20. Axial load during partial loading

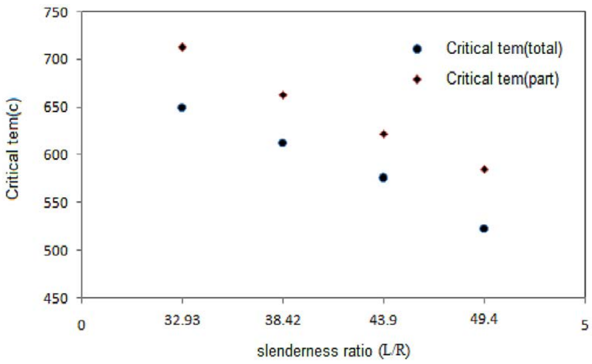


Fig. 21. Comparison of critical temperature during uniform and partial loadings

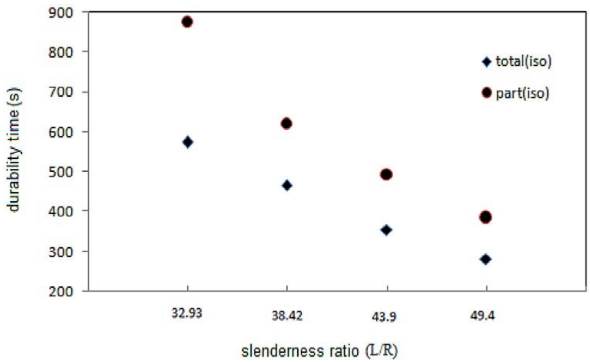


Fig. 22. Comparison of beam durability time during uniform and partial loadings in standard fire

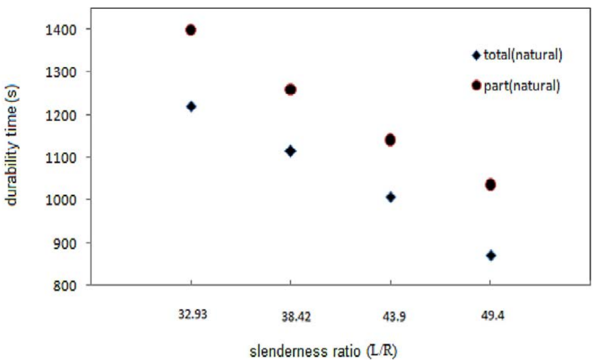


Fig. 23. Comparison of beam durability time during uniform and partial loadings in natural fire

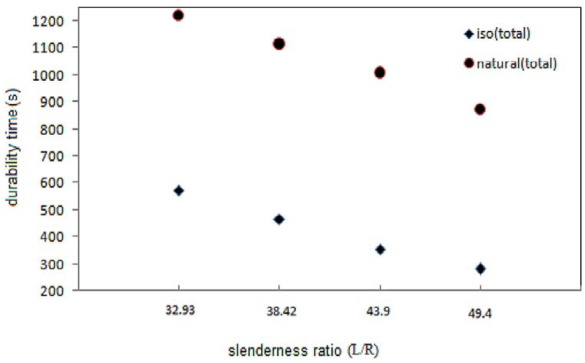


Fig. 24. Comparison of beam durability time in natural and standard fires during uniform loading

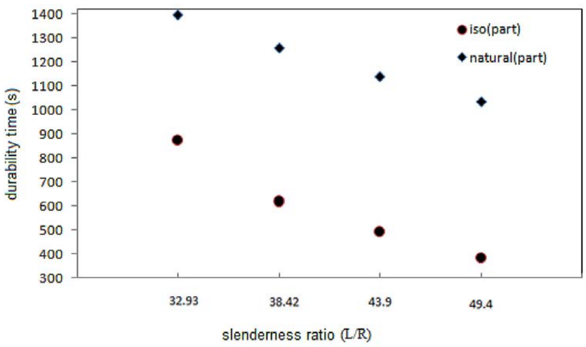


Fig. 25. Comparison of beam durability time in natural and standard fires during partial loading

Comparison of structure durability time in standard and natural fires

To compare fire standards with natural fire, in addition to the standard fire, natural fire is used in beam modeling. As it is seen in Figure (21), structure durability time in natural fire has much difference with standard fire. The reason of this difference can be found in diagrams of standard and natural fires. As mentioned before, the structure critical temperature is between 500 °C and 700 °C and from Figures 24 and 25, it can be observed that temperature increase in standard loading case has sharp slope until 580 °C and this, causes the structure durability time in standard fire to be lower than the natural fire; however, temperature increase in natural case has milder slope than the standard one. If the structure temperature reaches 650 °C, the structure durability time exhibits significant increase.

CONCLUSIONS

1. Slenderness ratio of the member in elevated temperatures, the created restraints by adjacent colder elements and the applied thermal regimes affect significantly the behavior of steel beam under heat.
   - With increase of the beam slenderness ratio, maximum deflection and axial force of the beam increase. A steel beam which has support restraints against thermal expansion and rotation and is under similar thermal loading, the created axial force and thermal deflection of a long beam are more than a short one.
   - With increase of the beam slenderness ratio, critical temperature (temperature

at which the temperature loses its performance) and beam durability time (time at which the structure reaches the critical one) decrease.

2. Type of connection at elevated temperatures affects steel beam behavior under thermal loading.
   - With stiffness increase, the maximum mid-span deflection decreases.
   - With stiffness increase of the beam, critical temperature and the beam durability time increase.
3. Structure durability time in natural fire is more than the standard fire (*2-3* times). This shows that diagram of time-temperature of standard fire is so over estimate.
4. In the same temperature, the mid-span deflection and the axial force during partial loading are lower than the uniform loading while in most of the researches, the uniform loading is used.
5. Critical temperature and durability time of the structure during partial loading are higher than the uniform loading.
6. The beam axial force has direct relationship with temperature and the elasticity coefficient and according to Table 3, it can be seen that from temperature of 500 °C upward, severe decrease happens in the steel elasticity modulus and this decreases axial load of the beam from this temperature.
7. Although it can be said that in most models, the beam failure occurs before connection failure, but considering the beam-connection interaction, the connection effect on the beam behavior cannot be neglected.
8. As it can be seen, in the case of using cover in the connection, the beam durability time increases significantly which can be a simple solution for increasing the structure strength against fire.
9. Analysis of two factors are important with temperature increase:
   - Force increase by the longitudinal expansion due to temperature increase,
   - Decrease of steel strength and stiffness due to temperature increase,

   The structure critical temperature can be found considering these two factors.
10. Under real fire conditions, the temperature distribution may be non-uniform and this makes deflections and re-distribution of the forces to be more complicated. Although local heat may be so high in the member, but the member has yet the possibility of resisting due to continuity with other members. Special attention must be paid to instability out of the frames plane. Thermal growth rate can be effective on the steel structures strength against fire. Generally, quicker heating and higher temperature cause faster failure.
11. Considering that researches are done on real buildings and in these kinds of structures, the slenderness ratio is negligible, the buckling can be disregarded.
12. When the building next to the heated part is cold, all the boundaries are assumed against rotation and deflection and this assumption provides high level of resistance against length increase due to heat and conservative answers are obtained for supports forces of the beams and columns.
13. Although in the first processes of fire, thermal expansion affects the created force in the member, but it affects the member deflection too. Consideration of these effects is beneficial in design so that the ideal design can be achieved by considering these effects.

REFERENCES

1. Wong M.B. Elastic and plastic methods for numerical modelling of steel structures subject to fire. Journal of Constructional Steel Research, 57, 2001, 1–14.
2. Lawson R.M. Behaviour of steel-beam-to-column connections in fire. The Structural Engineer, 68(14), 1990, 263–271.
3. European Committee for Standardization – CEN. ENV - 1993-1-2-1995, Eurocode 3: Design of Steel Structures, Part 1.2: General Rules – Structural fire design, Brussels, 1995.
4. Wald F., Simões da Silva L., Moore D., Lennon T., Chladná M., Santiago A., Beneš M. and Borges L. Experimental behaviour of steel structure under natural fire. Fire Safety Journal;, 41(7), 2006, 509–522.
5. Wong M.B. Szafranski M.. Elastic method for design of 3-D steelstructures subject to fire. Journal of Constructional Steel Research, 60, 2004, 1095–1108.
6. Jones SW, Kirby PA, Nethercot DA. The analysis of frames with semi-rigid connections—a state-of-the-art report. Department of Civil and Structural Engineering, University of Sheffield, UK; 1981.
7. Lawson RM. Behaviour of steel beam-to-column connections in fire. Struct Eng., 68(14), 1990, 263–271.

8. Usmani A.S., Rotter J.M., Lamont S., Sanad A.M., Gillie M. Fundamental principles of structural behaviour under thermal effects. Fire Saf. J. 36, 2001, 721–744.
9. Wang Y.C., Lennon T., Moore D.B. The behaviour of steel frames subject to fire. J. Construct. Steel Res., 35, 1995, 291–322.
10. Lien K.H., Chiou Y.J., Wang R.Z., Hsiao P.A. Nonlinear behavior of steel structures considering the cooling phase of a fire. 2009.
11. Saedi Daryan A., Bahrampoor H. The study of behavior of Khorjini Connections in Fire. Accepted in Fire Safety Journal.
12. Saedi Daryan A., Yahyai M. Behavior of welded top-seat angle connections exposed to fire. Fire Safety Journal 44, 2009, 603–611.
13. Saedi Daryan A., Yahyai M. Modeling of bolted angle connections in fire. Fire Safety Journal 44 2009, 976–988.
14. Saedi Daryan A., Yahyai M. Behavior of bolted top-seat angle connections in fire. Journal of Constructional Steel Research 65, 2009, 531–541.
15. British Steel, The performance of beam/column/beam connections in the BS476: art 8 fire test. Reports T/RS/1380/33/82D and T/RS/1380/34/82D, 1982.
16. Leston-Jones L.C., Burgess I.W., Lennon T., Plank R.J. Elevated temperature moment-rotation tests on steelwork connections. Proc. Instn Civil Engrs, Structures & Buildings, 122, 1997, 410–419.
17. Al-Jabri K.S., Lennon T., Burgess I.W. and Plank R.J. Behaviour of steel and composite beam-column connections in fire. Journal of Constructional Steel Research, 46, paper no. 180, 1998, 1–3.
18. Lou G.B. and Li, G.Q. Nonlinear finite element modelling of behaviour of extended end-plate bolted moment connections in fire. Fourth International Workshop – Structures in Fire, Aveiro, Portugal, 2006, 327–349.
19. Armer G.S.T. and Moore, D.B. Full-scale testing on complete multistory structures. The Structural Engineer, 72(2), 1994, 30–31.
20. Rubert A., Schaumann P. Structural steel and plane frame assemblies under fire action. Fire Safety Journal, 10, 1986, 173–184.
21. Cooke G.M.E. and Latham, D.J. The inherent fire resistance of a loaded steel framework. Steel Construction Today, 1, 1987, 49–58.
22. Liu T.C.H., Fahad, M.K. and Davies, J.M. Experimental investigation of behavior of axial restrained steel beams in fire. Journal of Constructional Steel Research, 58, 2002, 1211–1230.
23. Qian Z.H., Tan K.H. and Burgess I.W. Behaviour and mechanical model of steel beam-to-column joints at elevated temperatures. Proceedings of Fifth International Conference on Advances in Steel Structures. Singapore, 2007, 783–790.
24. Quintiere J.G., Marzo M. and Becken R. A suggested cause of the fireinduced collapse of the World Trade Towers. Fire Safety Journal, 37, 2002, 707–716.
25. Bravery P.N.R. Cardington Large Building Test Facility. Construction details for the first building. Building Research Establishment, Internal paper, Watford 1993, pp.158.
26. Moore D.B. and Lennon T. Fire engineering design of steel structures. Progress in Structural Engineering and Materials, 1(1), 1997, 4–9.
27. ISO 834. Fire resistance tests – elements of building construction. 2002.
28. Wei-Yong Wang, Guo-Qiang Li, Yu-li Dongc. A practical approach for fire resistance design of extended end-plate joints. Journal of Constructional Steel Research, 64, 2008, 1456–1462.
29. Wei-Yong Wang, Guo-Qiang Li. Behavior of steel columns in a fire with partial damage to fire protection. Journal of Constructional Steel Research, 65, 2009, 1392–1400.
30. Bradford M.A., Luu K.T. and Heidarpour A. Numerical studies of a steel beam in a frame subassembly at elevated Temperatures. Construction and Maintenance of Structures - Hanoi, Vietnam, December 2007.
31. Bradford M.A., Luu K.T., Heidarpour A. Generic nonlinear modelling of a steel beam in a frame subassembly at elevated temperatures. The International Colloquium on Stability and Ductility of Steel Structures (Edited by: D. Camo-tim, N. Silvestre, P.B. Dinis) IST Press, Lisbon 2006, 293–298.
32. Sokol E., Wald F., Pultar M., Bene M. Numerical simulation of cardington fire test on structural.
33. ANSYS User's manual, version 12.
34. Kirby B.R. The behavior of high-strength grade 8.8 bolts in fire. J. Constr. Steel Res. 33, 1995, 3–38.
35. Saedi Daryan A. A Study on Behavior of Connections in Fire. MS thesis. K.N. Toosi University, Tehran, Iran 2006.