# Failure of RHS Steel Columns and Local Buckling at Elevated Temperatures

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**Abstract**

This study investigates the behavior of RHS(Rounded Hollow Square) steel columns in fire conditions and represents the functional effect of temperature on yield strength and modulus of elasticity. A nonlinear finite element modal using **Ansys(9)** has been developed to investigate the effect of fire on the buckling of RHS and mode shape. With an increase of temperature, strength of steel and stiffness of columns decrease leading to buckling at an even much lower of external loading than at no temperature effect. An extensive parametric study on columns with RHS are discussed to study of their behavior under fire conditions. The results are used to study the different types of failure mode occur due to increasing of temperature and studying of the influence of slenderness ratio due to fire. In this study the local buckling coefficient for stiffened elements, is equal to 4.4 in 900$^{o}$C and 5.97 in 0$^{o}$C than compare with the values of some searchers is good .

**Key word**: RHS Steel Columns, Fire, Buckling, Mode shape.

**الخلاصة:**

تبحث هذه الدراسة سلوكية الأعمدة الحديدية المستطيلة المجوفة ذات المقاطع مدورة الجوانب RHS تحت تأثير درجات حرارة مختلفة تتراوح مابين (0 $^{o}$C و900 $^{o}$C). تم اعتماد نموذج العناصر المحددة الغير خطية باستخدام برنامج **ANSYS 9.0** بدراسة تأثير ارتفاع درجات الحرارة على local buckling , mode shape بالإضافة إلى تغيير قيمة yield stress , modulus of elasticity. تم التوصل في هذه الدراسة إلى انخفاض مقاومة الأعمدة الحديدية مما يؤدي الوصول إلى local buckling بوقت أسرع مما هو عليه بدون تأثير درجات الحرارة. وفي هذه الدراسة تم الحصول على إشكال مختلفة من الفشل بالإضافة إلى إن الحمل الحرج للأجزاء الصلدة والذي يعتمد على قيمة معامل الانبعاج والذي أشار إليه بعض الباحثين والذي يساوي في هذه الدراسة إلى، 4.4 في 900 $^{o}$C 5.97 في 0 $^{o}$C هذا يؤكد صحة النتائج التي تم التوصل إليها في هذه الدراسة.

**الكلمات المفتاحية:** أعمدة فولاذية RHS، حريق، التواء، شكل الوضع.

## 1.Introduction

The applications of structural hollow sections nearly cover all fields. Sometimes hollow sections are used because of the beauty of their shape, to express a lightness or in other cases their geometrical properties determine their use Hollow structural steel columns are very efficient structurally in terms resisting compression loads. Steel is vulnerable to fire, however, and steel structures potentially exposed to fire, require a particularly careful design. This especially holds true for steel columns as they are loaded in compression and are thus disposed to buckling. With an increasing of temperature, strength of steel and stiffness of columns decrease loading to buckling at an even much lower level of external loading than at room temperature. One can find numerous results of experiments on steel columns in fire. **Lie, T.,T. and Stanzak, W.,W.(1973)** presented experimental and analytical studies of the fire resistance of steel columns protected with low density insulating materials. Critical temperature for failure are derived and methods for calculation of temperature rise are described. **Burgess, I.W.,Olawale, A.,O. and Plank, R., J. (1992)** presented an analytical study on the performance of geometrically perfect steel columns under fire condition. The analysis is based on the finite strip method and includes nonlinear material characteristics as functions of temperature. **Pan, K., W. and Bennets, I., D. (1995)** Compares the numerical analysis results against the laboratory test results on steel columns under elevated temperatures, with different heating rates load eccentricities and end restraint conditions. **Talamona, et.,al. (1997)** presented numerical study on the behavior of concentrically and eccentrically loaded steel columns under fire. They proposed equations for determining the buckling coefficient of centrally and eccentrically loaded columns based on the material properties at elevated temperatures defined in Euro code 3. **Fernando and el.,at.(2007)**, proposed alternative formulas to determine the buckling length at elevated temperatures, as an improvement of the actual rule of the Euro code 3 part 1.2. The testing of a loaded structure under fire conditions can be carried out in two generic ways: **(1)** transient state testing; or **(2)** steady state testing. In transient state testing, loads are applied to the structure first. These loads are then held constant and the structure is exposed to fire attack. The test is terminated when one of the specified failure criteria is reached. In steady state testing, the temperature in the structure is raised to the pre-determined level and held constant. Loads are then applied to the structure until structural failure **[Y.C.Wang]**. Steady state testing, considered in this paper.

## 2.Behaviour of steel column under fire condition

Columns are under predominantly axial compression. The three modes of failure of a steel column are local buckling, global buckling and yielding:

### 2.1 Cross-section yield

If local buckling does not occur, complete yielding of steel in compression can only occur in short columns with a length to width ratio of not exceeding about 5 (or slenderness of about 20).

### 2.2 Global buckling behavior

**Figure 2.1** shows the typical axial deformation-temperature relationship of an axially loaded steel column with uniform heating. It may be divided into three stages: the first stage **(A-B)** is essentially due to free thermal expansion. At high steel temperatures **(B-C),** the rate of increase in the column axial deformation is reduced when the column stiffness is reduced and the mechanical shortening becomes important. Finally**(C-D),** the mechanical shortening overtakes the free thermal expansion of the column. The column axial deformation changes direction and the column starts to contract until the column cannot sustain the applied load. The column mechanical shortening is directly related to the tangent stiffness of the column at elevated temperatures. Since the tangent stiffness reduces rapidly, the final stage is short **[Wang, Y.C]**.

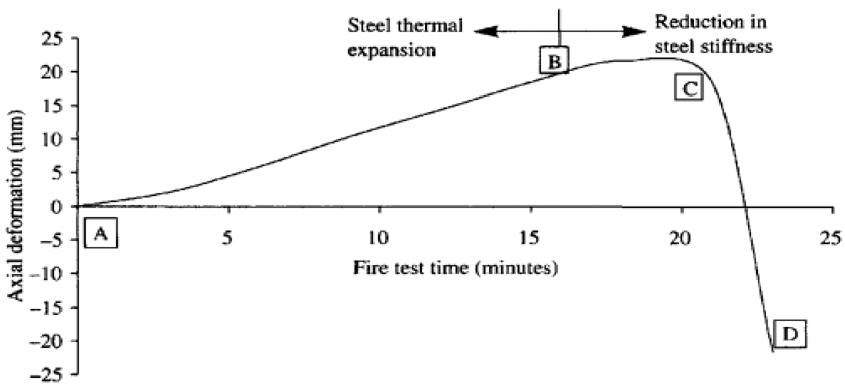



**Fig.(1)** Typical behavior of a steel column exposed to the standard fire on all sides**(Wainman and Kirby 1987**)

## 2.3 Local buckling

Buckling is a mode of failure generally resulting from structural instability due to compressive action on the structural member or element involved. **Ala-Outinen and Myllymaki (1995)** reported the results of some local buckling tests on thin-walled rectangular steel tubes at elevated temperatures. Tests were carried out under steady-state condition, i.e. the steel temperature was raised to the specified target values and the specimen was then loaded to failure.

If the member is so slender that the stress just before buckling is below the proportional limit- that is, the member is still elastic-the critical buckling load is given by:

$$P_{cr} = \frac{\pi^2 EI}{L^2} \qquad (1)$$

Due to temperature effect the equation become:

$$P_{cr} = \frac{\pi^2 E^T I}{L^2} \qquad (2)$$

Where:

$$E^T = E\left(-1.94*10^{-6}T^2 - 3.32*10^{-5}T + 1\right) \qquad (3)$$

Where:

$P_{cr}$ =critical load in (kN ), $E$ =Modulus of elasticity(N/mm$^2$ )

$E^T$ = Modulus of elasticity ( N/mm$^2$) due to temperature .

$I$ =Moment of inertia( mm$^4$) , $L$ =Effective length (mm) , $T$ =Temperature ($^o$C)

The basic requirements for compression members are covered in chapter **E** of the **AISC Specification(2005)**

The nominal compressive strength is:

$$P_n = F_{cr} A_g \qquad (4)$$

For **LRFD:**

$$P_u \le \phi_c P_n \qquad (5)$$

Where:

$P_u$ =Sum of the factored loads. $\phi_c$ =Resistance factor for compression =0.9.

$\phi_c P_n$ =Design compressive strength.

In order to present the AISC expressions for the critical stress $F_{cr}$ one first define the Eular load as:

$$P_e = \frac{\pi^2 EA}{\left(\frac{KL}{r}\right)^2} \qquad (6)$$

Due to temperature:

$$P_e = \frac{\pi^2 E^T A}{\left(\frac{KL}{r}\right)^2} \qquad (7)$$

To obtain the critical stress for elastic columns the Eular stress is reduced as follows:

**a.** $F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$ When $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ (8)

Due to temperature:

$$F_{cr}^{\ T} = \left[ 0.658^{\frac{F_y^{\ T}}{F_e^{\ T}}} \right] F_y^{\ T} \quad \text{When } \frac{KL}{r} \le 4.71\sqrt{\frac{E^T}{F_y^{\ T}}} \tag{9}$$

**b.** $$F_{cr} = 0.877 F_e \quad \text{When } \frac{KL}{r} \ge 4.71\sqrt{\frac{E}{F_y}} \tag{10}$$

Due to temperature:

$$F_{cr} = 0.877 F_e^{\ T} \quad \text{When } \frac{KL}{r} \ge 4.71\sqrt{\frac{E^T}{F_y^{\ T}}} \tag{11}$$

Where:

$$F_y^{\ T} = 1.024 F_y \left( 9.2*10^{-7} T^{\ 2} - 3.34*10^{-4} T + 1 \right) \tag{12}$$

$$F_e = P_{cr} \quad , \quad F_e^{\ T} = P_{cr}^{\ T}$$

While the local buckling stress for stiffened plate element as defined by **Salmoon[2007]** , as

$$F_{cr} = K_1 \frac{\pi^2 E}{12\left(1-\mu^2\right)\left(b/t\right)^2} \tag{13}$$

Where $K_1$ is the local buckling coefficient for stiffened elements, the value varied between (4.0-6.97) depending on the edge stiffened **Salmoon[2007]** .

## 3- The numerical modeling and parametric study

### 3-1 The Finite element modeling

The commercial finite element software **ANSYS 9.0** was adopted for the numerical simulation. Some previous researchers were used shell element to model the steel **[Xiong and Zha,2007],[Guo et .,al,2007],[Kwon et.,al,2007]**,and **[Ellobody,2007]**,while **[Mohi-Aldeen, 2008]** and**[Zinkaah,2010]** were used solid element. Thus, the type of element has been used to model the steel section; is **SOLID45** for the 3-D modeling of RHS column. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions as shown in Fig.(2). The element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities**[ANSYS, 2004].**

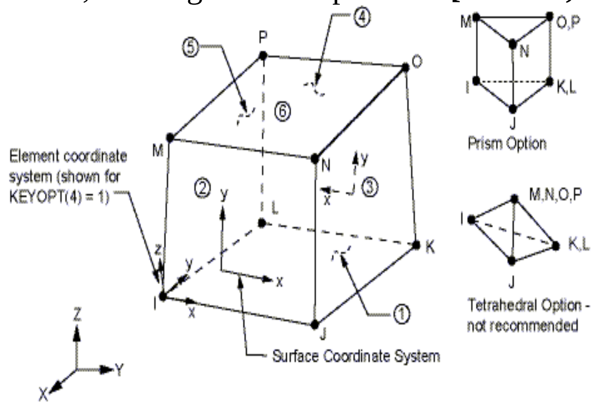



**Fig.(2)** Solid45-3D Solid**[ANSYS, 2004].**

Material properties specified in ANSYS 9.0 included a Young`s modulus of steel Es of 200000MPa and Poisson`s ratio νs of 0.3 **[Guo et al,2007].** The steel is assumed to behave as an elastic-plastic material with strain hardening in compression. The idealized stress-strain curve used in the numerical analysis is shown in Fig. 3

**[Xiong and Zha,2007], [Mohi Aldeen, 2008], and [Zinkaah, 2010].** The $F_y$ in Fig.(3) represent the ultimate stress steel.

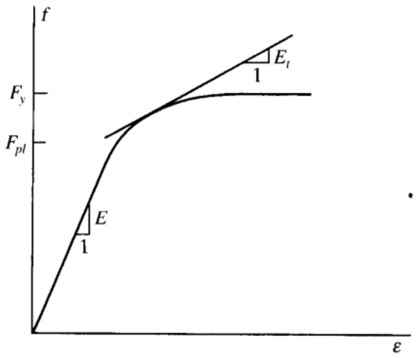



**Fig.(3)** Stress-strain curve**[Xiong and Zha,2007], [Mohi Aldeen, 2008],**

### 3-2 parametric study

A parametric study was performed to investigate the behavior of RHS steel column.The applied load levels, temperature as well as slenderness ratio and yield stress were varied and the results are calculated .

### 3.2.1 Effective length

The influence of temperature on effective length is greater for lower load. The fire resistance decrease with an increase in effective length. The decreased fire resistance for longer columns can be attributed to the increase of effective slenderness during the course of the fire which, in turn, reduces the load-carrying capacity.

The finite element modeling and the boundary conditions of the analyzing column are showed in Fig.(4). Thus, element SOLID45 is used for all other analyzed columns.

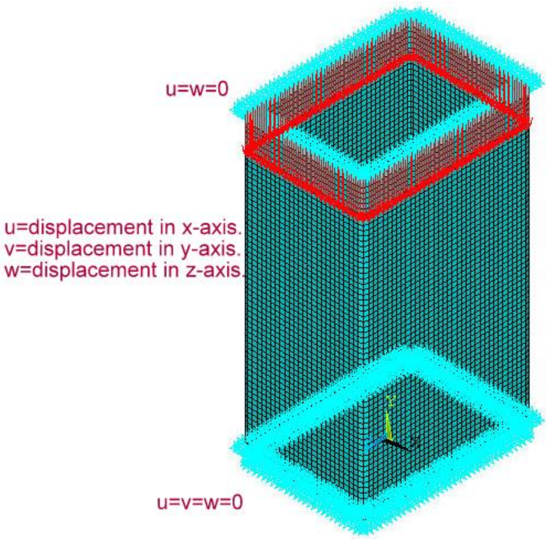



Fig.(4) Meshing and boundary conditions of column

As the specimens were short, global flexural buckling did not occur, they lost their load-bearing capacity the moment a local buckle appeared. The end temperature was the max. temperature at the level (upper, middle or lower) where buckle appeared. The axial deformation and X-displacement due to different temperatures a difference yield stress on the varying slenderness ratio. Figs. (5to 20) give the results.

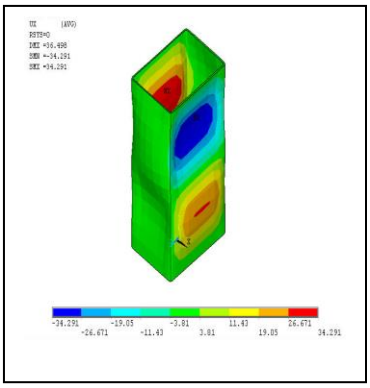


**Fig. (5)** X- Displacement(mm)T=0, Fy=250MPa, KL/r=40

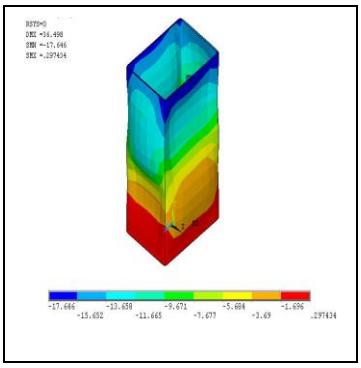


**Fig. (6)**:Axial deformation(mm) T=0, Fy= 250MPa, KL/r=40

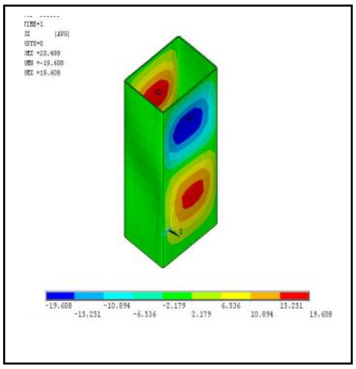


**Fig. Fig.(7)** X- Displacement(mm)T=900 $^{O}$C, Fy=250MPa, KL/r=40

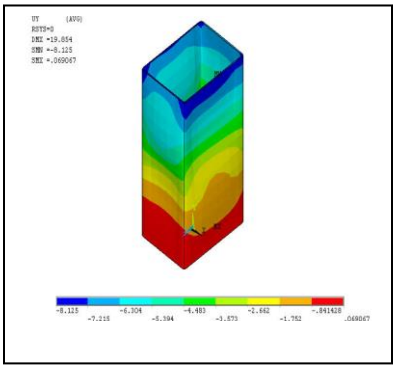


**Fig. (8)** Axial deformation(mm) T=900 $^{o}$C, Fy=250MPa, KL/r=40

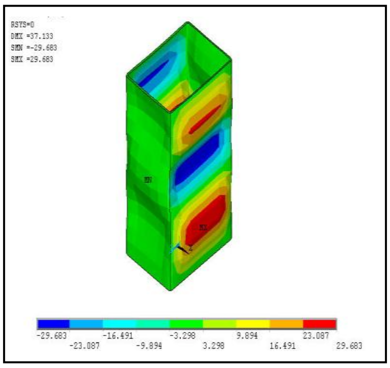


**Fig. (9)** X- Displacement(mm) T=0, Fy=414 MPa, KL/r=40

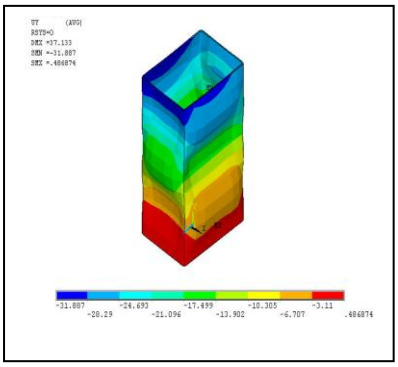


**Fig. (10)** Axial deformation(mm)T=0, Fy=414 MPa, KL/r=40

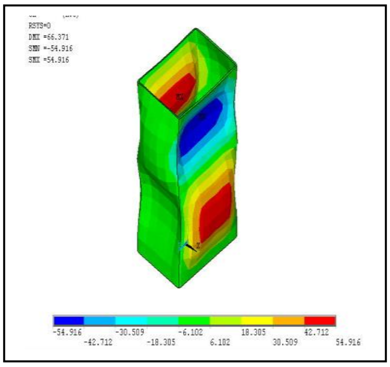


**Fig. (11)** X- Displacement(mm)T=900 $^{o}$C, Fy=414 MPa, KL/r=40

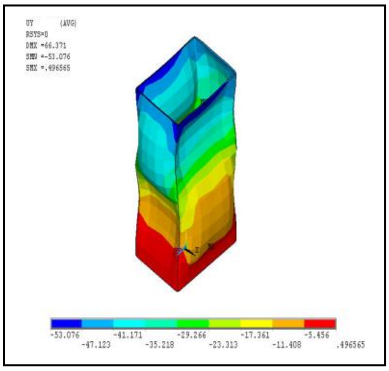


**Fig. (12)** Axial deformation(mm) T=900 $^{O}$C, Fy= 414 MPa, KL/r=40

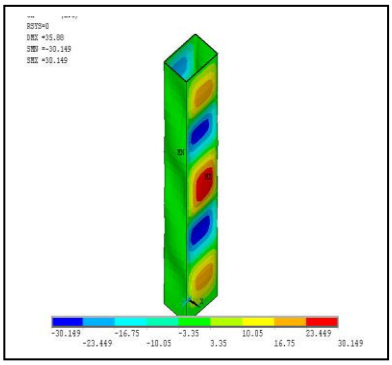


**Fig. (13)** X- Displacement(mm) T=0 $^{o}$C, Fy=250 MPa, KL/r=100

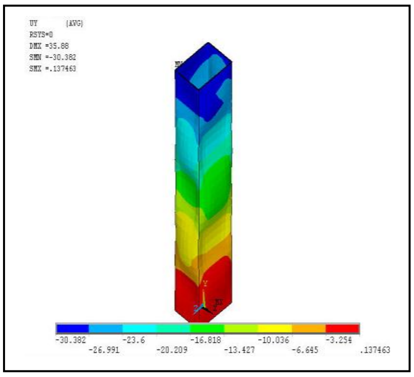


**Fig. (14)** Axial deformation(mm),T=0 $^{O}$C, Fy=250 MPa, KL/r=100

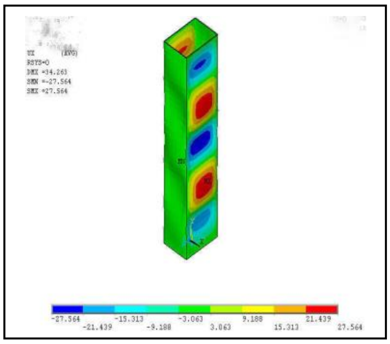


**Fig. (15)** X- Displacement(mm) T=900 $^{o}$C, Fy=250N/mm^2, KL/r=100

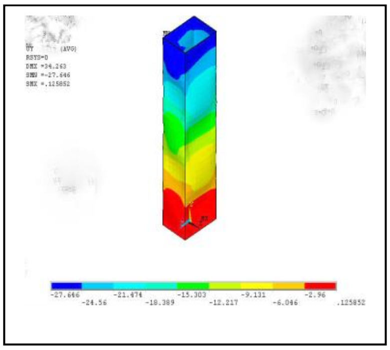


**Fig. (16)** Axial deformation(mm) T=900 $^{O}$C,Fy=250N/mm^2, KL/r=100

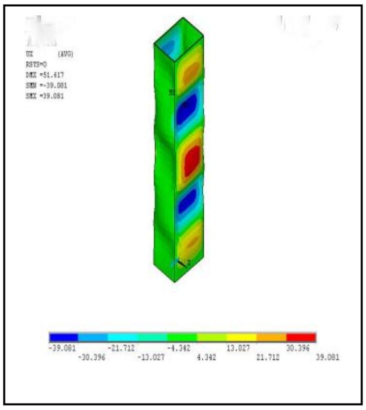


**Fig. (17)** X- Displacement(mm)T=0 $^{o}$C, Fy=414 MPa, KL/r=100

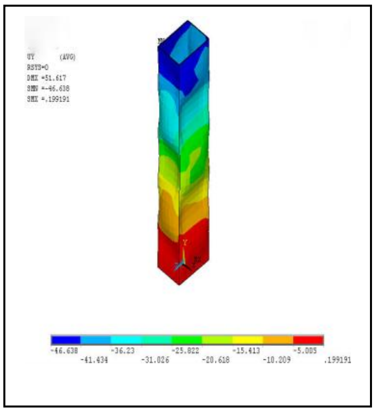


**Fig. (18)** Axial deformation(mm) T=0 $^{O}$C, Fy=414 MPa, KL/r=100

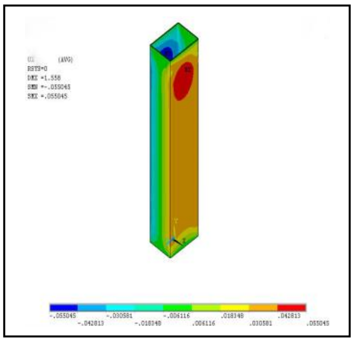


**Fig. (19)** X- Displacement(mm)T=900 $^{o}$C, Fy=414 MPa, KL/r=100

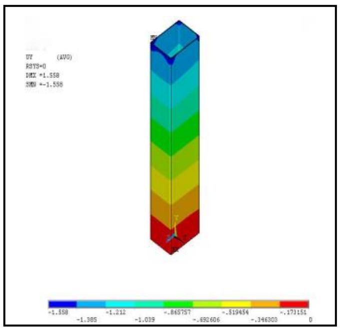
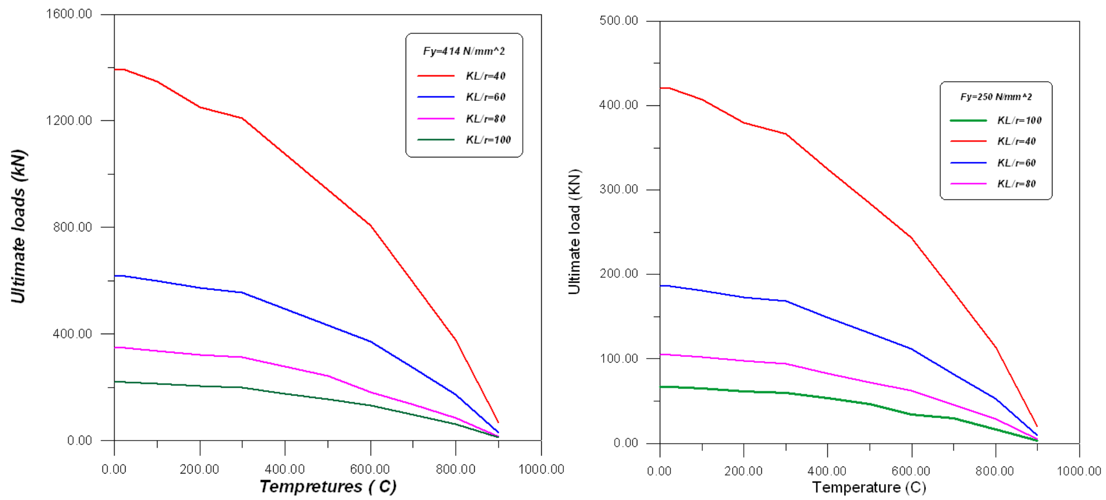


**Fig. (20)** Axial deformation(mm) T=900 $^{O}$C, Fy=414 MPa, KL/r=100

### 3.2.2 Applied load

In the event of a fire, the applied loads are much lower than the critical load specified for normal temperature conditions. Fig.(21) show the ultimate load decrease with increasing of temperature for different slenderness ratio.

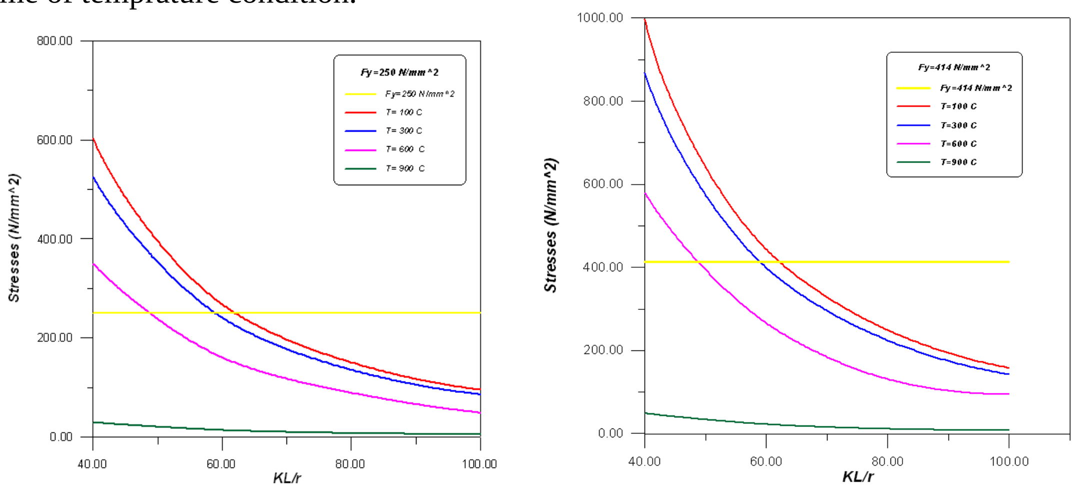



( a ) ( b)

**Fig.(21)** Ultimate load for varying temperatures and slenderness ratio
a)Fy=414 MPa b) Fy=250 MPa .

### 3.2.3 Stresses

The different failure modes are resulted from the difference stress-slenderness characteristice of steel columns.Fig.(22) shows the results of failure for different tempreatures. For T=900 $^{o}$C , the RHS steel column failed by buckling for arange of slendereness ratios. At T=100 $^{o}$C, 300 $^{o}$C and 600 $^{o}$C , the steel column failed by yielding for KL/r= 40 but the buckling failure occure for KL/r= 60,80 and 100 at the same of temprature condition.

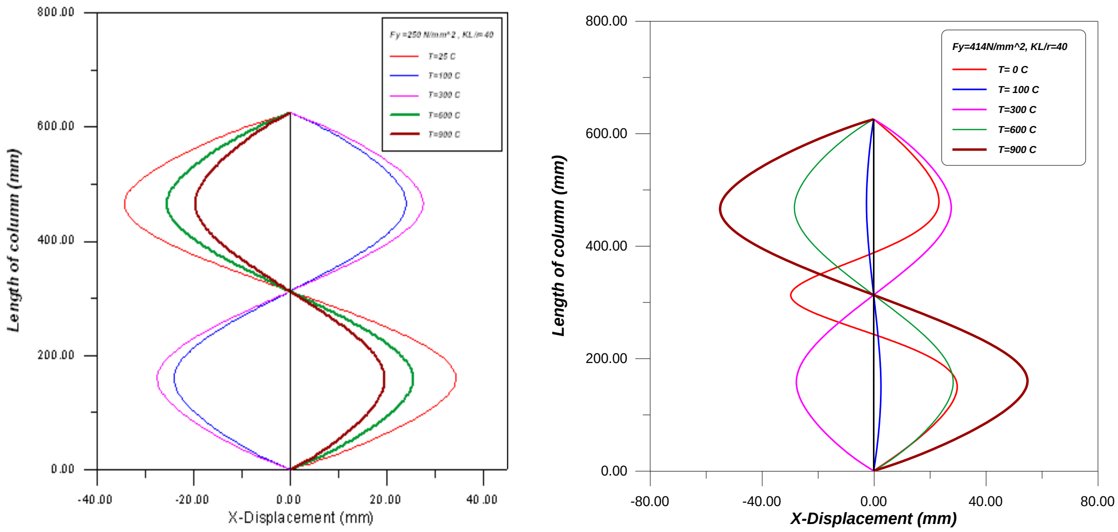



(a) (b)

**Fig.(22)** Stresses for varying slenderness and ratio temperatures
a) Fy=250N/mm^2 b) Fy=414N/mm^2 .

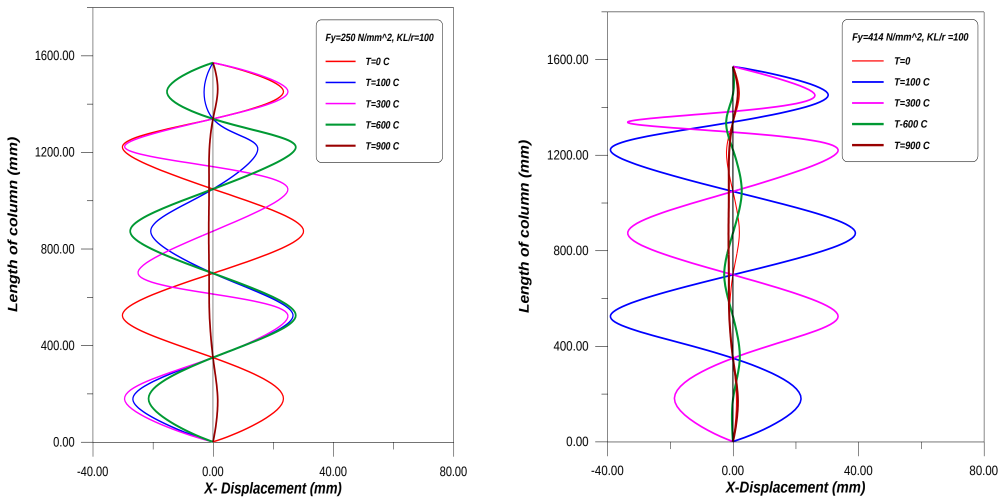
### 3.2.4 Mode Shape

In both stresses Fy=250 MPa and Fy=414 MPa the mode shape is the same for KL/r=40 but the difference occur due to increasing of temperature. Figs. (23 and 24) show the behaviour of RHS due to increasing of tepmperature for KL/r=40 and KL/r=100.




a b

**Fig.(23)** Mode shape for KL/r =40 steel column
a)Fy=250 MPa b) Fy=414 MPa




(a) (b)

**Fig.(24)** Mode shape for KL/r =100 steel column
a)Fy=250 MPa b) Fy=414 MPa m^2

### 3.2.5 local buckling Stresses

The local buckling stresses for stiffened plate element depending on the (b/t) ratio and the local buckling coeffiecient for stiffened element. The local buckling load (critical load $P_{cr}$ )for RHS in this paper occure in between the two curves repares two values of K are shown in figs(25,26,27,28 and 29). one can get the minimum local buckling load in higher temperature and the range of stresses value varied between (243 kN/mm$^2$ -4146.25 kN/mm$^2$). From the results one can get the values of the local buckling coefficient for stiffened elements ,is equal to 4.4 in 900 $^o$C and 5.97 in 0 $^o$C then compare with the values of **Salmoon[2007]** is good as shown in Fig.( 30).

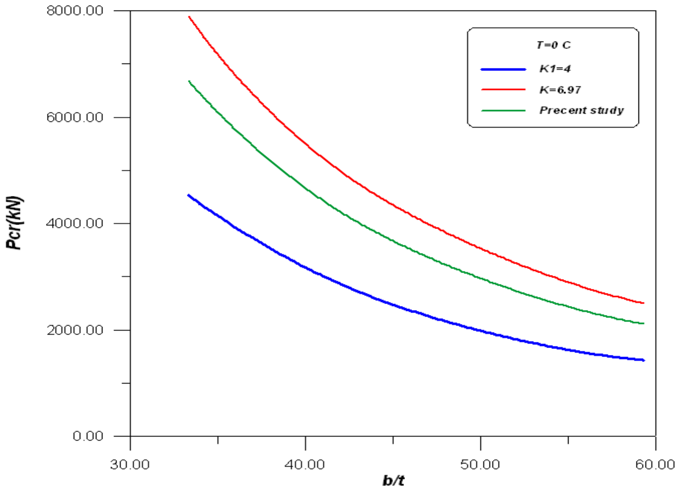



**Fig.( 25)** local buckling loads in T= 0 C,

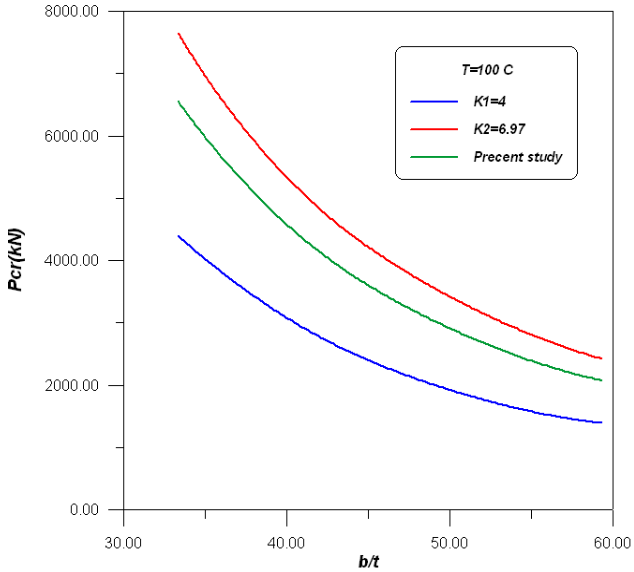



**Fig.( 26)** local buckling load in T=100 C,

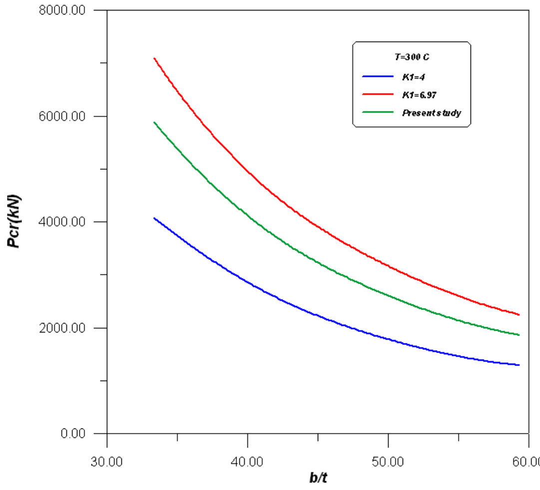



**Fig.(27)** local buckling in T=300 C,

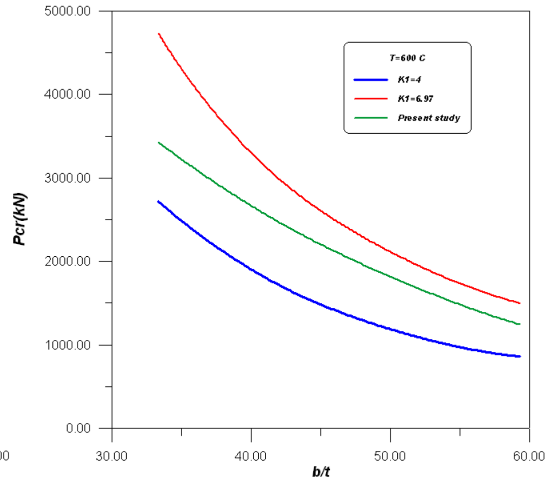



**Fig.(28)** local buckling load in T=600 C,

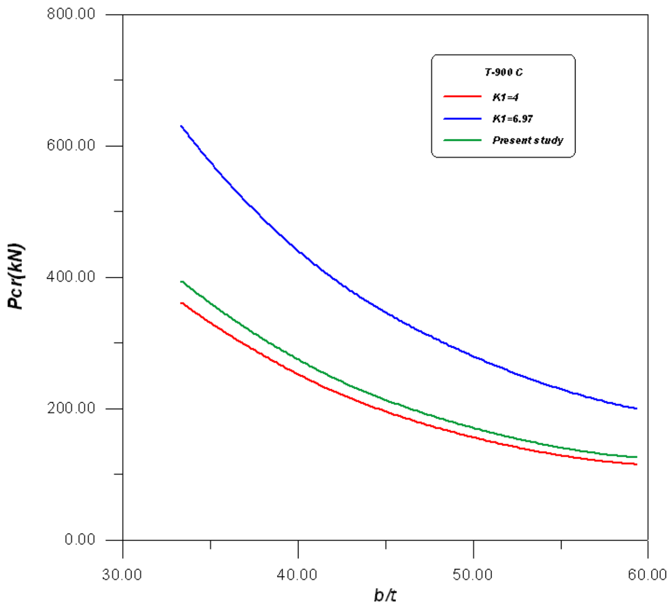



**Fig.(29)** local buckling load in T=900 C,

## 4. Conclusion

Considering that the steel at elevated temperature behaves in accordance with the equation for yield stresses and modulus of elasiticity .

Buckling appears to be the mode of frailure of columns due to fire and elevated temperature highly depend on both the slenderness of a column and the material model of steel.

From the results of analysis of RHS steel column under fire condition , the following conculusion can be obtained:

- The influence of effective length is greater for lower loads due to fire.
- The ultimate loads are decreace with increacing temperatures.
- More different types of failure occure due to increasing temperatures.
- Mode shape and X-Displacement changed through increasing temperatures.

- The minimum local buckling stresses and minimum critical load for stiffened plate element occur in higher temperatures and the value of local buckling coefficient for stiffened element varried between (4.4-5.97).

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