Structural behavior of I-section steel columns under transverse thermal gradient

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In this paper, a general finite element model is presented to analyze the structural behavior of unprotected steel columns under fire conditions. The results obtained are compared with the available experimental and theoretical results. The effect of thermal gradient resulted from fire over the column cross-section is evaluated. Three cases of temperature distributions are suggested to study this effect. The first case assumes uniform fire temperature over the whole cross-section. While, in the second case the two flanges of the cross-section are exposed to fire and the web is exposed to room temperature. In the third case, one flange only is exposed to fire while the web and the other flange are exposed to room temperature. For these cases, three boundary conditions are examined assuming that the column is fixed at its lower end and varies at its upper end from free to fixed passing through hinged end. Also, the effect of changing the column height on the behavior of columns under fire is studied. As a result of the numerical analysis performed, interaction curves between the column-bearing load and the column-resisted temperature are introduced. From the research work done, it can be concluded that changing both the temperature distributions over the cross-section and the boundary conditions of the column have a great influence on the behavior of steel columns.

فى هذا البحث، تم تقديم نموذج عام باستخدام طريقة العناصر المحددة لتحليل السلوك الانشائى للأعمدة الحديدية غير المحماه تحت تاثير التدرج الحرارى العرضى الناتج من الحريق. تم مقارنة النتائج التى تم الحصول عليها باستخدام هذا النموذج بالنتائج المعملية والنظرية المتاحة. تم تقييم تأثير التدرج الحرارى الناتج من الحريق على مقطع العمود، ولدراسة هذا التأثير تم اختيار ثلاث حالات للتوزيع الحرارى. الحالة الأولى هى افتراض أن توزيع الحرارة الناتجة من الحريق منتظمة على كامل مقطع العمود. وفى الحالة الثانية تم فرض أن شفتى مقطع العمود هما المعرضان فقط للحريق. أما الحالة الثالثة فهى تفترض أن شفة واحدة فقط من مقطع العمود هى التى تتعرض للحريق وباقى القطاع يكون فى درجة حرارة الغرفة. ولهذه الحالات، تم اختيار ثلاث شروط لحافة العمود وذلك بافتراض أن العمود مثبت تماما عند طرفه السفلى أما الطرف العلوى فهو متغير من طرف حر الى طرف مثبت تماما مرورا بطرف مفصلى. كذلك تم دراسة تأثير تغير طول العمود على السلوك أثناء الحريق. من خلال النتائج التى تم الحصول عليها تم تقديم منحنيات التأثير التبادلى بين حمل العمود والحرارة التى يستطيع العمود أن يتحملها. ومن نتائج هذا البحث نستطيع أن نستنتج أن تغير كلا" من التدرج الحرارى العرضى وشروط الحافة للأعمدة يؤثران تأثيرا كبيرا على سلوك الأعمدة الحديدية.

Keywords: Structural behavior, Unprotected steel columns, Fire conditions, Standard fire curve, Boundary conditions

1. Introduction

The provision of structural fire-resistance requirements in building design and building codes is intended primarily to ensure building integrity for a certain period of time under fire conditions and to permit evacuation of occupants and access for firefighters.

To meet fire-resistance requirements specified in building codes, the fire resistance of individual building elements has to be determined. Often, this is done by subjecting specimens to costly and time-consuming fire tests. Recent developments, however, have made it possible in many cases to calculate fire resistance using mathematical models. Developed to simulate the behavior of building elements exposed to fire, such as beams, columns, floors and walls, these models make it possible to predict the fire resistance of most building elements for a wide range of practical conditions.

The analysis of the behavior of load-bearing members in buildings during a fire can be complicated. Various factors that influence the behavior of the members should be considered. These factors include: variation of member temperature with time, variation of temperature over cross section and along the member, temperature effects on material properties, material and geometric non-linearity, combined actions, initial Imperfections and finally, external restraints.

The structural behavior at elevated temperature of steel members that are mainly submitted to bending actions has been thoroughly investigated. It is generally accepted that a good description of this behavior can be obtained by classical analytical or numerical methods. This is because, in addition to the temperature distribution, the main factor that influences this behavior is the non-linear stress-strain relationship at the elevated temperature [1].

The behavior of steel members subjected to axial compressive forces is by far much more difficult to be analyzed because it is very sensitive to second-order effects caused by structural imperfections such as initial out of straightness and residual stresses. Various authors have investigated the behavior of steel columns at elevated temperatures with the aid of numerical or analytical tools, some of them specifically dedicated to this problem [2-13].

Due to the fact that experimental tests of loaded columns at elevated temperatures are rather expensive, the numerical or analytical tools are usually validated against a limited number of experimental test results. Some authors have applied their calculation tools on some cross sections, in order to define an analytical formula to calculate the ultimate load of a heated column. There is a number of different solutions proposed by different authors [6,14,15] or appeared in various recommendations and codes such as ERFSSS [16], EC3-10 [17], EC3-1.2 [1] and GFSDB [18]. Most of the existing recommendations use the concept of buckling coefficients defined as the ratio between the ultimate load and the plastic load of a centrally loaded column.

A survey of the available literature concerning experimental tests on columns and beam-columns allowed the formulation of database of compression tests conducted at elevated temperature on steel elements. The general considerations taken into account in these tests are:

1. Tests were not considered if the actual yield strength of the specimen had not been measured [19, 20].
2. Nominal values were taken for geometrical properties when they had not been measured.
3. The influence of the yield strength has been found to be overwhelming when compared to the influence of the geometrical properties [21].

These tests are nevertheless valuable for the validation of numerical tools, provided that the measured temperature distribution is introduced [9]. Finally, a total of 81 test results on columns with small eccentricities were obtained. One of these tests was carried out in Borehainwood [22], 16 results from Gent [23], 3 from stuttgart [24], 25 results from Braunschweig [25], 14 from Rennes [15], 3 more from Rennes for columns under longitudinal temperature gradient [26] and finally, 19 results from Berlin [27, 28]. With the 14 new tests performed in Bilbao, the database thus comprises 95 test results on columns loaded with a small eccentricity. In addition to the above mentioned tests, there are 21 recent fire tests performed in Spain in the LABEIN laboratory, and more eight tests at the fire station of CTICM, France [29].

Several finite element models are available to conduct structural response analyses. Frannssen et al. [29] and Sullivan et al. [30] provide extensive reviews and comparisons of many of the existing finite element models for structural fire protection applications. As a result of the review, they provide the following comments:

- The predictive capability of the structural models is less than that for the thermal finite element models due to inadequate material models, uncertain material property values, and sensitivity of the structural response to elevated temperatures.
- The stress history of an assembly is ignored.
- Creep is compensated for by defining other mechanical properties as effective properties.
- Because the models are based on the Navier-Bernoulli assumption of small displacements,

large displacements are not accurately modeled.

- The maximum load or failure temperature predicted by any two models differs by less than 6% for any of the tests.

2. Objective of the study

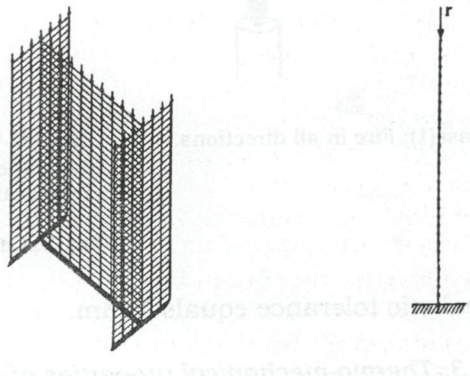
This paper presents a finite element model to evaluate the behavior of unprotected steel I-section columns under transverse thermal gradient. The model is applicable to members of any cross-sectional shape and constructed of any ductile material as long as the elevated temperature stress-strain relationships of the material are known. All the aforementioned factors influencing the behavior of a member at elevated temperature can be taken into account.

In this model, two new approaches are implemented. The two approaches are:
1. The column is modeled, using the finite element method, as a three-dimensional model comprised of thick shell elements. In the previous analyses, the column is modeled as a two- or three-dimensional beam element. This approach is important in predicting the local buckling failure mode which can be occurred in the column cross-section. Fig. 1 shows the finite element model presented and the finite element model used in the previous analyses.
2. In the previous analyses, the distribution of the temperature over the cross-section of the column was assumed to be uniform. This assumption means that the column is exposed to fire in all directions. This is not a realistic assumption because the fire may be affected the column in one or two directions only. So, in this paper three temperature distributions resulting from fire are examined to evaluate their effect on the behavior of the column. Fig. 2 describes the suggested temperature distributions over the cross section of a column.

3. The model

3.1. Finite element mesh

The powerful COSMOS/M finite element analysis program [31] is used for the non-linear 3-D analysis and for linear buckling analysis. The plates of the I-section are modeled using quadrilateral shell elements based on the Mindlin plate theory, in which the transverse shear deformations are considered. Selective reduced integration scheme is adopted to prevent the transverse shear and membrane locking. The bearing load is applied as a distributed load over the perimeter of the cross-section.

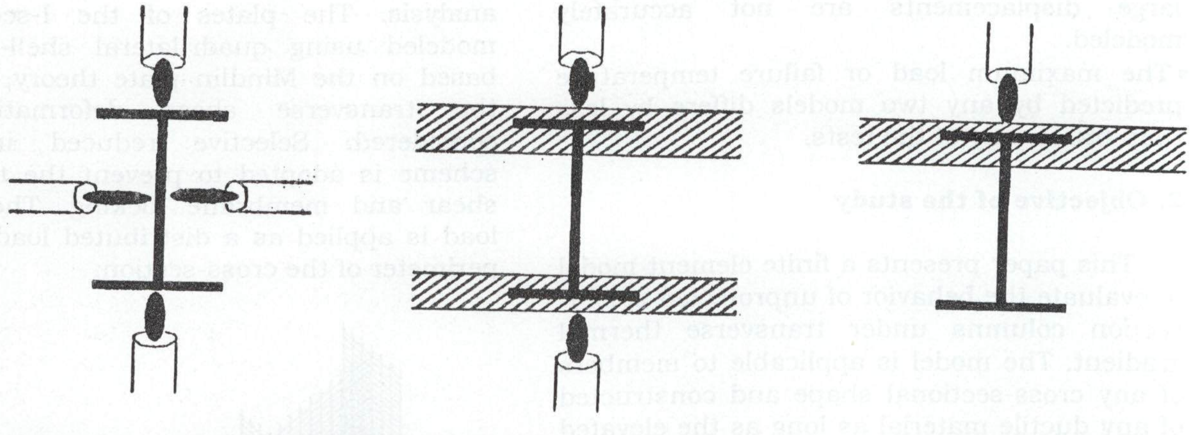


(a) (b)

Fig. 1. Proposed (a) and previous analyses (b) finite element models.

3.2. Material and geometric non-linearity

In order to model the behavior of the column under study, large deformation, but small strain, is considered on the bases of Total Lagrangian formulation, which uses Green strain tensor and the second-Piola Kirchoff stress tensor. The plastic flow theory is applied considering the Mises's yield function as a plastic potential. The material is assumed to follow the isotropic hardening law. This requires a non-linear analysis with a successful numerical procedure that includes:
- A force control technique capable of controlling the progress of computations along the equilibrium path of the system.
- A modified Newton-Raphson iterative technique to solve the simultaneous non-linear equations governing the equilibrium state along the path. In this technique, the tangential stiffness matrix is formed and decomposed at the beginning of each load step, and used throughout the iterations within this step.
- A displacement convergence criterion to terminate the iterative process within a



Case(1): Fire in all directions. Case(2): The 2 flanges are encased in any material while the web is at room Temperature remain at ambient temperature. Case(3): The upper flange is encased in any material while the web and the lower flange

Fig. 2. Case the studied transverse fire conditions

realistic tolerance equals 1 mm.

3.3. Thermo-mechanical properties of steel

Any increase in temperature leads to changes in the mechanical properties of steels. In this case, a decrease in the ultimate strength and of the yield value is occurred. So, to implement the changes in the steel properties (i.e. yield stress, ultimate stress, modulus of elasticity and coefficient of thermal expansion), formulas given by CTICM [32] are used in simulating the thermo-mechanical properties of steel. A description of the formulas used are presented hereafter and shown in fig. 3.

3.3.1. Yield stress (F_y)

The following equations represent the recommended variation of the yield value for fire resistance calculations according to CTICM [32].

- For $0 \le T \le 600^\circ$ C:

$$\frac{f_{yT}}{f_y} = 1 + \frac{T}{900 \log_e \frac{T}{1750}}. \quad (1)$$

- For 600° C $\le T \le 1000^\circ$ C:

$$\frac{f_{yT}}{f_y} = \frac{340 - 0.34T}{T - 240}. \quad (2)$$

3.3.2. Modulus of elasticity (E)

The variation of the Young's modulus as recommended by the CTICM can be expressed by the equation:

$$\frac{E_{aT}}{E_a} = 1 + \frac{T}{2000 \log_e \frac{T}{1100}}. \quad (3)$$

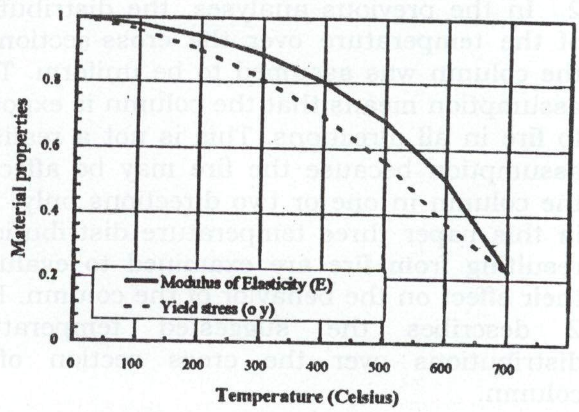


Fig. 3. Change of material properties of steel with temperature according to CTICM [32].

3.3.3. Coefficient of thermal expansion (α)

A final aspect to consider when using steel in construction is its significant coefficient of linear expansion under thermal loads. At temperature of up to 1000° C, it is given as:

$$\alpha = (0.004T + 12)\times 10^{-6} \quad , \tag{4}$$

where, α is the coefficient of thermal expansion, in degrees Celsius^{-1}, and T is the temperature in degrees Celsius.

3.4. Initial imperfections

In the presented model, the out-of-straightness is considered to be 1/1000 of the column length. This assumption is typical to most of the above mentioned research work.

For residual stresses, some authors suppose that residual stresses vary with the temperature as the Young's modulus, or as the yield strength, while others do not consider residual stresses. In fact, the problem of residual stress at elevated temperature has been recently developed according to Frannssen [29] by means of initial and constant strain. It should be pointed out herein that the value of the residual stress itself is not any more the well known values 0.3 f_y or 0.5 f_y. From the results of stub-column tests, Aasen [33] estimated the residual stress at the tips of the flanges. It was found to be 20% of the yield stress.

So, in this model the values estimated by Aasen [33] is used as the values of the residual stresses at the tips of the flanges. Fig. 4 shows the variation of the residual stresses over the column cross-section.

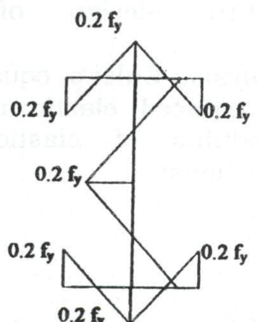


Fig. 4. Distribution of the residual stresses over the cross-section.

3.5. Temperature distribution

In the case of fire problems, two types of temperature distribution must be satisfied as input in the problem. The first distribution is the along temperature distribution which deals with the temperature distribution along the column height. The second distribution is the transverse temperature distribution that represents the temperature distribution over the column cross-section.

In the presented model, the along temperature distribution is assumed to be a uniform distribution along the column height. While for the transverse temperature distribution, three cases of temperature distributions are examined. The three cases are:

- Case (1): uniform temperature over the column cross-section (this is the popular approach in the available reviewed research work).
- Case (2): the two flanges of the I-section are exposed to fire while the web of the section is at room temperature.
- Case (3): the temperature is assumed to be on one flange only of the cross-section and the other flange and the web are exposed to room temperature.

Fig. 5 shows the assumed transverse distribution and the expected final distribution for the three cases.

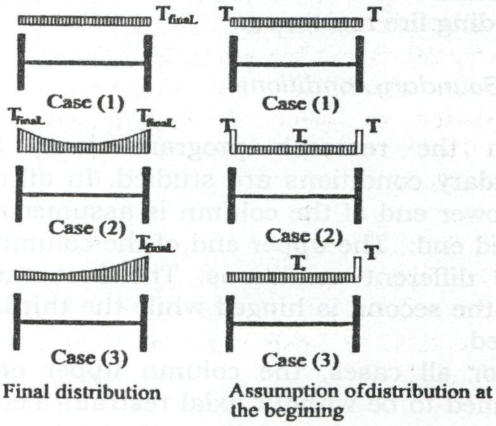


Fig. 5. Temperature distribution of fire over the cross-section for the three cases.

3.5.1. Standard fire curve

In the presented model, the fire process is generally simulated by means of a standardized curve relating the rise in temperature to the duration of the fire. The curve established by the ISO takes the form of the time-temperature graph shown on fig. 6, which can be expressed as:

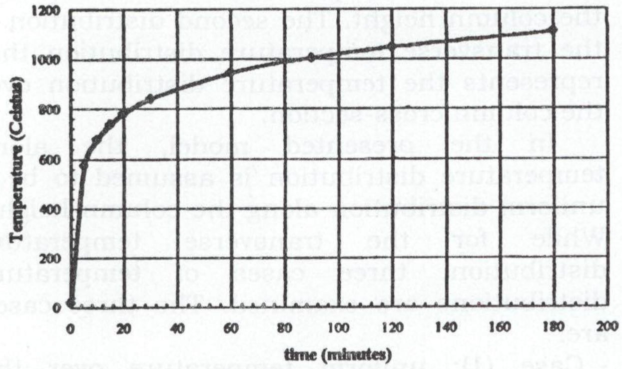


Fig. 6. Time-temperature relationship of standard fire.

$$T = T_0 + 345\log(1+8t)\ , \tag{5}$$

where, T is expressed in $^{\circ}$C, t in minutes and T_o is the ambient temperature.

The standard fire curve represents an initial, quickly increasing temperature that rises up to 1000° C after 1 hour, then continually rises. This case is absolutely a standard test that does not represent the measured output from real fires. It is the standard by which the performance of all materials and components are judged regarding fire resistance.

3.6. Boundary conditions

In the research program done, three boundary conditions are studied. In all cases the lower end of the column is assumed to be a fixed end. The upper end of the column has three different conditions. The first case is free, the second is hinged while the third case is fixed.

For all cases, the column upper end is assumed to be without axial restraint because if the column is axially restrained against displacement, the expansion due to heat will be translated into thermal stresses that will increase the overall stress level in the column and cause earlier collapse.

4. Verification of the presented model

In order to verify the proposed model, a comparison between the results of the presented model and the results obtained analytically and experimentally in the available literature is done. Two analytical models and one experiment are used in the comparison. A short description of the two analytical models and the experiment are presented hereafter.

4.1. Analytical models

4.1.1. AISC standard

This model depends mainly on a buckling or stability analysis involves the application of Euler's buckling equation, with the modulus of elasticity evaluated at the temperature of the column. In this analysis, all columns are assumed to be concentrically loaded and have a uniform temperature along the length of the column and over the column cross-section.

The related expressions are included in the AISC standard [34]:

$$f = \frac{f_{yT}}{\lambda_c^2}\ . \tag{6}$$

The column slenderness ratio, λ_c, is defined as $\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{f_{y,T}}{E}}$.

The modulus of elasticity is evaluated at the column temperature resulting from the fire exposure. Where, r is the minimum radius of gyration and K is the effective factor on ambient temperature design of steel structures.

For plastic analysis, Euler's equation is applied, with the reduced elastic modulus replacing the modulus of elasticity for rectangular cross-sections:

$$E_r = \frac{4EE_T}{\left(\sqrt{E}+\sqrt{E_T}\right)^2}\ . \tag{7}$$

While Euler's equation assumes idealistic conditions of load and uniformity of properties and does not account for residual stresses, an empirically derived equation can be applied that implicitly accounts for these effects [35]. This buckling analysis of fire-exposed steel columns, presented as eq. (8), was developed from a regression analysis of ambient temperature buckling behavior:

$$N_{cr,T} = \frac{f_{y,T}}{f_{yo}} \left\{ \frac{1 + 0.489(\bar{\lambda} - 0.2) + \bar{\lambda}^2}{2\bar{\lambda}^2} - \frac{1}{2\bar{\lambda}^2} \sqrt{\left[1 + 0.489(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]^2 - 4\bar{\lambda}^2} \right\}. \quad (8)$$

Eq. (8) tends to overestimate the fire resistance of slender columns, i.e., where plastic buckling may occur.

4.1.2. Franssen et al. formula

From the observations made during the numerical analysis, Franssen et al proposed an analytical formula that formulated for failure temperatures in the range 400-800°C range. It can be safely applied for temperatures higher than 800°C or lower than 400°C. The formula is given as follows:

$$P_u(\theta, H) = \chi(\theta) K_{fy}(\theta) f_y \Omega, \quad (9)$$

with

$$\chi(\theta) = \frac{1}{\varphi(\theta) + \sqrt{\varphi^2(\theta) - \bar{\lambda}^2(\theta)}}, \quad (10)$$

$$\varphi(\theta) = \frac{1}{2}\left[1 + \alpha\bar{\lambda}(\theta) + \bar{\lambda}^2(\theta)\right], \quad (11)$$

where Ω = cross-sectional area of the profile and $K_{fy}(\theta)$ is the ratio between $f_y(\theta)$ and f_y.

The relative slenderness, $\bar{\lambda}$, is the ratio between the slenderness of the column and the Eulerian slenderness. It is calculated as follows:

$$\bar{\lambda} = \frac{\lambda}{\lambda_E} = \frac{\lambda}{\pi\sqrt{E/f_y}}. \quad (12)$$

The relative slenderness is evaluated by eq. (12) at room temperature, but the results concern columns buckling at elevated temperatures. It is also possible, as in the proposal of EC3-1.2 [1], to evaluate the slenderness at the failure temperature:

$$\bar{\lambda}(\theta) = \frac{\lambda}{\lambda_E(\theta)} = \frac{\lambda}{\pi\sqrt{\frac{E(\theta)}{f_y(\theta)}}} = K\lambda(\theta)\bar{\lambda}, \quad (13)$$

where

$$K\lambda(\theta) = \sqrt{\frac{K_{fy}(\theta)}{KE(\theta)}}, \quad (14)$$

$$K_{fy}(\theta) = \frac{f_y(\theta)}{f_y}, \quad (15)$$

$$KE(\theta) = \frac{E(\theta)}{E}, \text{ and} \quad (16)$$

$$\alpha = \beta\varepsilon, \quad (17)$$

where, β=severity factor, to be chosen in order to ensure the appropriate safety level.

$$\varepsilon = \sqrt{\frac{235}{f_y}}, \quad (18)$$

where, f_y = yield strength at room temperature (N/mm^2).

The value of β is calibrated with experimental test results [29]. After the calibration, the value 1.20 was taken because it safely covers the numerical results. This value is very conservative because it has been derived from simulations made with characteristic values for the imperfections.

5. Experimental results

Aasen [33] conducted a series of steel columns at the Norwegian Institute of Technology. All the columns were made from the European rolled I-section IPE 160 (fig. 7). One of these columns was chosen for the comparison with the presented model. Table 1 shows the dimensions and the support conditions. The heating conditions were chosen to give a nominal steel temperature rise rate of 20°C/min.

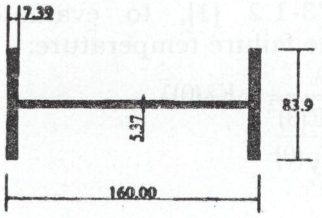


Fig. 7. Cross-sectional dimensions of column (IPE 160).

5.1. *Comparison between the results of the presented model and the results obtained analytically and experimentally*

For the examined column used in the comparison, fig. 8 shows the lateral deflection at the mid-height and the axial expansion of the columns obtained from the proposed model and those obtained from experiments and calculated from the analytical models.

From the figure, the results of the proposed model show a good agreement with the experimental results while the results of the analytical models are far conservative.

6. Parametric study

In this study, three sets of parametric study are carried out. The first is to study the effect of the transverse fire conditions on the structural behavior of the columns. In this case six values of the column axial force are taken as a ratio from the buckling load of the column. These ratios, P/P_{cr}, are 0, 0.128, 0.32, 0.64, 0.85 and 1.0. The second parametric study is to investigate the effect of the boundary conditions of the column on the failure temperature of the columns. In this case three boundary conditions are considered. These boundary conditions are fixed-free, fixed-hinged and fixed-fixed. In all cases the lower end of the column is set to be totally fixed and the upper end is changed without axial restraint. The third set of parametric study is to evaluate the effect of column height on the column behavior. The column heights are taken 3.11, 2.21 and 1.75 meter.

The column was made from the European rolled I-section IPE 160. The material properties of this section at ambient temperature are yield stress, f_y=240 MN/m², Young's modulus, E=210 GN/m², Poisson's ratio, ν=0.30, shear modulus, G=8.4 GN/m², and tangent modulus, E_T=5.94 GN/m² (i.e. E_T=E/35).

Table 1
Dimensions and support conditions of the column tested by Aasen [33]

Column No.	Length (mm)	Slenderness ratio	P (kN)	Initial imperfection (mm)	Support conditions Lower end	Support conditions Upper end
2	3110	169	91.2	4.21	Pin	Pin

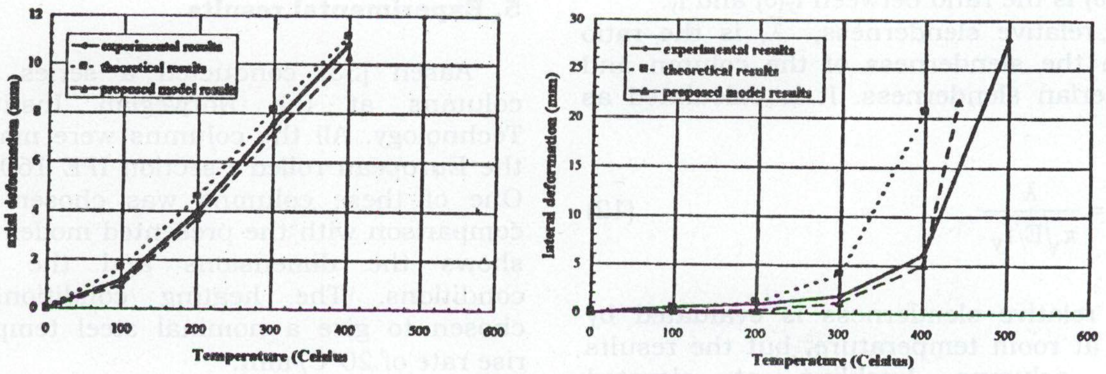


Fig. 8. Comparison between the results obtained experimentally and theoretically and the results of the proposed model.

6.1. Numerical results

The results of the research program are listed in tables 2 and 3. Table 2 shows the failure temperature for the three fire conditions with different boundary conditions for a column height equals 3.11 meter. While in table 3 the failure temperature is illustrated for columns having height equal 3.11, 2.21 and 1.75 meter for fixed-free boundary condition and for fire conditions having two flanges exposed to fire.

Figs. 9 to 11 show the effect of transverse fire conditions, boundary conditions and column height on the behavior of the columns (i.e. the lateral deformation and the axial deformation). Also, interaction curves between the failure temperature and the axial load acting on the column are presented in figs. 11 and 12. In the following subsections the numerical results obtained are discussed.

6.1.1. Effect of fire conditions

From table 2 and figs. 9 to 11, the following conclusions are observed:

1. The failure temperature increases by decreasing the bearing load of the column.
2. Generally, the transverse distribution of fire over the cross-section has a great influence on determining the failure temperature of the column.
3. For a fixed-free column, the transverse fire conditions have a great influence for columns having load level, $P/P_{cr} \leq 0.5$. In this case, the failure temperature decreases approximately by 15% when the fire condition changes from case (1) to case (2) and by 30% for fire case (3).
4. For columns having the upper end fixed or hinged, the influence of fire conditions on the failure temperature is observed to be at any load level. For example, the failure temperature decreases in case of fixed-fixed column by 57% and 68% by changing the fire condition from (1) to (2) and (3) respectively at load level, $P/P_{cr}=0.5$.

Table 2
Numerical results for column height equals 3.11 meter

Fire conditions	Boundary conditions		Failure temperature in Celsius						Buck load (kN)
			Load level (P/Pcr)						
	Lower end	Upper end	0	0.128	0.32	0.64	0.85	1.0	
Uniform over the cross-section	Fixed	Free	711	712	662.85	524.53	332	108.07	47.0
	Fixed	Hinged	505.7	543.89	474.29	519.48	321.9	184.61	368.4
	Fixed	Fixed	678.42	678.43	654.4	476.17	261.15	20.00	526.8
The 2 flanges are exposed to temperature	Fixed	Free	454.65	455.55	461.17	462.18	331.64	208.9	
	Fixed	Hinged	466.02	481.74	276.06	160.78	176.06	108.07	
	Fixed	Fixed	452.92	472.05	238.03	145.1	53.43	20.00	
1 flange only is exposed to temperature	Fixed	Free	568.61	567.37	560.87	551.85	411.69	108.07	
	Fixed	Hinged	340.41	308.86	245.62	164.88	123.85	108.07	
	Fixed	Fixed	343.92	307.54	232.32	122.16	80.75	20.00	

Table 3
Numerical results for column height equals 2.21 and 1.75 meter

Column height	Fire	Boundary conditions		Load level (P/Pcr)						Buck load (kN)
L (m)	Conditions	Lower end	Upper end	0	0.128	0.32	0.64	0.85	1.0	
2.21	The 2 flanges are exposed to Temperature	Fixed	Free	444.51	453.20	451.90	366.31	305.62	108.07	85.8
1.75		Fixed	Free	444.50	450.00	411.42	317.60	273.78	108.07	132.00

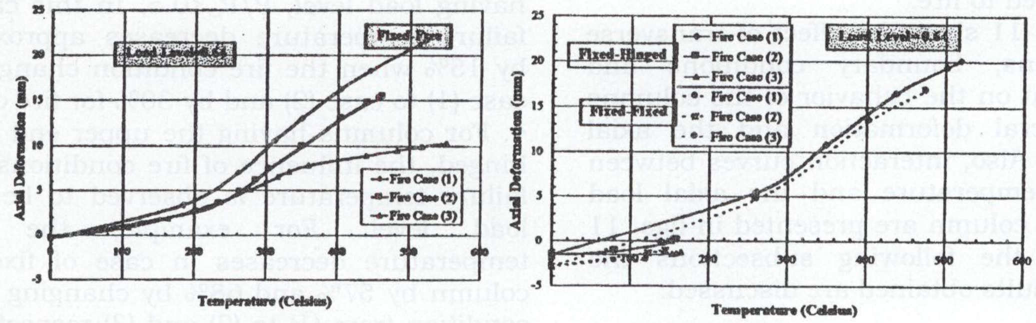


Fig. 9. Effect of boundary conditions on the axial deformation at columns of height, L=3.11 n for different fire cases at load level = 0.64 .

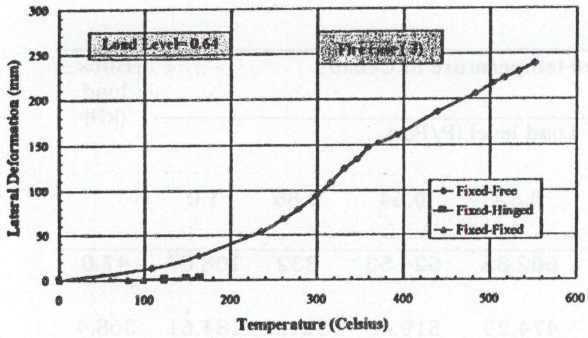


Fig. 10. Effect of boundary conditions on the lateral deformation of columns of height, L=3.11 m for fire case (3) at load level = 0.64.

6.1.2. Effect of boundary conditions

From table 2 and figs. 9 to 11, some important conclusions may be found.

1. For fire case (1), no significant changes have occurred for the column due to changing the boundary conditions. This is because the fire is distributed uniformly over the cross-section and the axial deformation is not prevented. So failure occurs when the axial deformations reached approximately the same limit for all different boundary conditions.
2. For fire case (2), significant changes have occurred by changing the upper end condition of the column. This may be because the two flanges are exposed only to fire and the web is in room temperature. So, in case of hinged or fixed upper end the column is restraint to move and/or rotate. Thus failure occurs rapid than that of free upper end (fig. 8). From fig. 11, it can be noticed that the failure temperature decreases by 65% by changing the upper end condition from free to hinged end, while it decreases by 79% when it is fixed.
3. For fire case (3), it can be noticed that also a great effect is happened on the behavior of the column by changing the upper end condition as fire case (2). The reason for that effect is the lateral deformations. For the free upper end column, lateral deformations occurred at the top of the column due to the un-symmetrical fire condition. So, by replacing the upper end from free to be hinged or fixed end the lateral deformations are prevented at the top of the column which make the column to fail at an earlier stage than the free end (fig. 8).

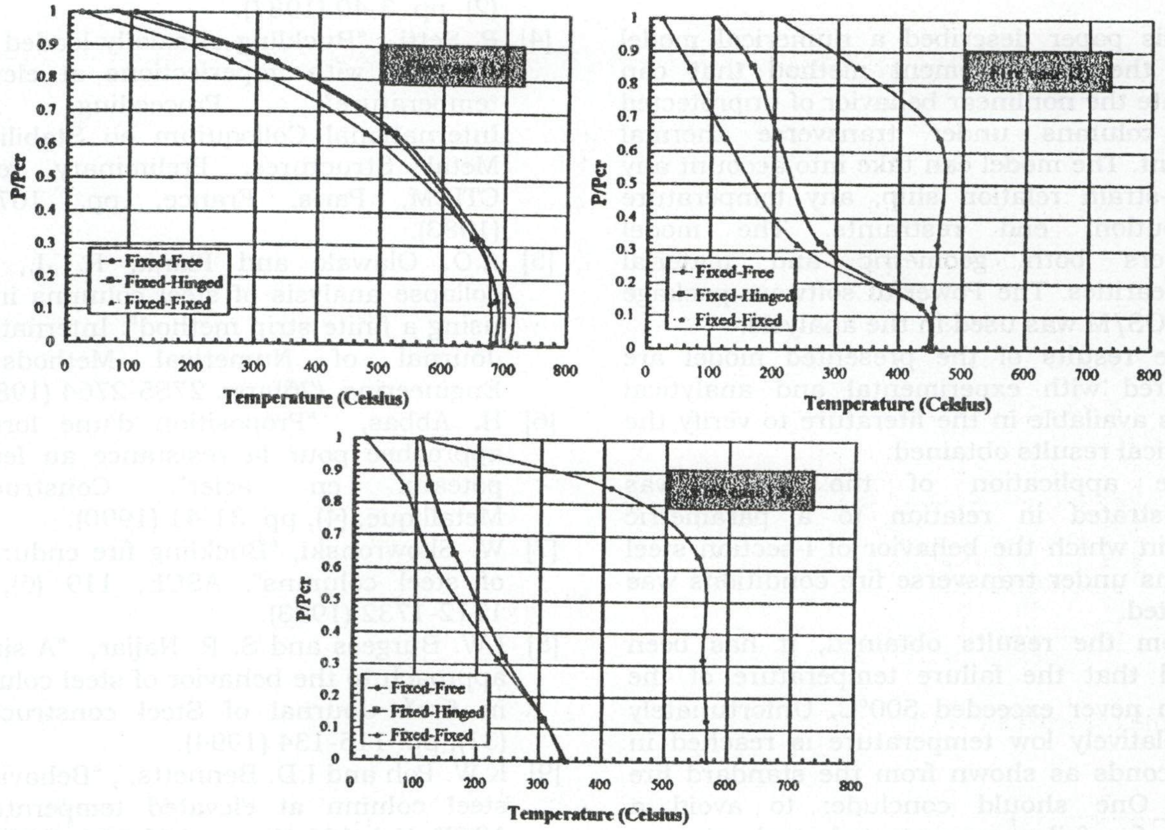


Fig. 11. Interaction curves between the failure temperature and the axial load acting on the column for different transverse fire conditions with different boundary conditions for column height, L = 3.11 m.

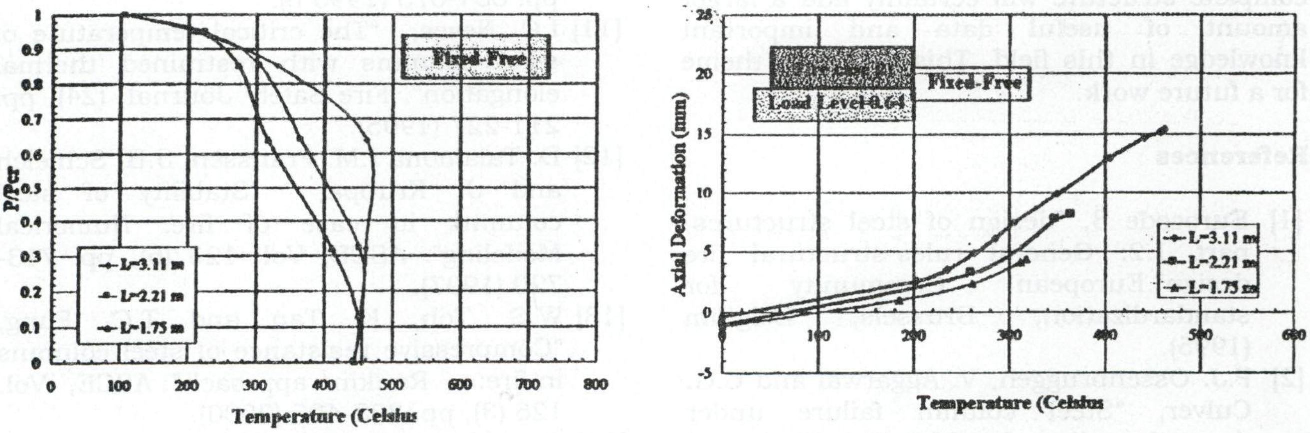


Fig. 12. Effect of column height on the behavior and failure temperature of columns.

6.1.3. *Effect of column height*

From table 3 and fig. 12, we can found that the column height has an influenced effect on the column failure temperature for columns having a load level greater than 0.15 and lower than 0.90. The great influence may be reached at load level equals 0.60 at this level increasing the height by 78% the failure temperature increases by 40%.

7. Conclusions

This paper described a numerical model using the finite element method that can evaluate the nonlinear behavior of unprotected steel columns under transverse thermal gradient. The model can take into account any stress-strain relation ship, any temperature distribution, end restraints. The model considers both geometric and material nonlinearities. The Powerful software package COSMOS/M was used in the analysis.

The results of the presented model are compared with experimental and analytical models available in the literature to verify the numerical results obtained.

The application of the model was demonstrated in relation to a parametric study in which the behavior of I-section steel columns under transverse fire conditions was evaluated.

From the results obtained, it has been noticed that the failure temperature of the column never exceeded 500°C. Unfortunately this relatively low temperature is reached in few seconds as shown from the standard fire curve. One should conclude: to avoid a sudden fire failure unprotected steel columns should never be used.

Also, freely expandable column is a rare case. The study of columns as a part of a complete structure will certainly add a larger amount of useful data and important knowledge in this field. This will be a theme for a future work.

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