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Behaviour of Wide Flange Steel Columns under Elevated Temperature

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Abstract: ***As we know there is a lack of understanding in how structural systems perform under realistic, uncontrolled fire situations. Fire protection of steel structure is usually provided through prescriptive requirements. The development of performance-based standards and tools requires explicit consideration of the fire effects on structural components and systems. This consists a parametric study employing nonlinear finite element analysis to model the response of wide flange steel columns at elevated temperature. Different axial loads and different cross sections are included in the parametric study. The FEM modelling was used to conduct parametric studies to evaluate the effects of different heating configurations on steel column. The failure behaviour at elevated temperature depends on the column cross sectional area, axial load and temperature. Failure mode includes flexural buckling about weak and strong axis. The column sections were uniformly heated until they exhibited either inelastic or elastic buckling failure. Typical cross sectional area is addressed through limiting the element width to thickness ratio, so only member buckling occurs. As the members are heated the Young's modulus and yield strength are reduced, which results in slender elements at elevated temperature. An indication about the column strength and failure behaviour at elevated temperature is the expected outcome of this thesis work.***

Keywords: Wide flange beam, axial loading, elevated temperature, local buckling, torsional buckling

1. Introduction

Fire protection of steel structure is usually provided through prescriptive requirements. The development of performance-based standards and tools requires explicit consideration of the fire effects on structural components and systems. This study concerns hot-rolled I-section members and their failure behaviour under axially applied compression load under varying temperature. Despite the fact that I-sections are readily available commercially and can be easily obtained by lapping two channel sections or by plate sections, the failure behaviour of these members when subjected to a compressive force and elevated temperature is still not fully understood as corroborated by the numerous different design approaches for these members in various national specifications for steel structures.

A wide flange column is usually made up of component parts which may be considered as plate elements. These plate elements may buckle locally if their thickness is relatively small in comparison with the width between ribs or between component parts of the column which hold the plate elements in line. Structural sections are usually proportioned so that local buckling will not occur in the elastic range, in which case the plate elements will usually buckle "inelastically" or by "plastic buckling" at an average stress somewhere between the proportional limit and the yield point of the material. In very compact sections buckling may occur at stresses above the yield point, but the yield point usually represents the practical upper limit of strength.

The buckling behaviour of I-section columns is discussed in detail, followed by a numerical study using geometric and material nonlinear analyses to produce column strengths for a wide range of geometries of I-sections and column lengths. This study first briefly describes the buckling and post-buckling behaviour of I-section columns and singles out the two main design issues such as the similarity between local buckling and torsional buckling modes, and the shift of the effective centroid due to the effect of local buckling on slender elements. Experiments on I -section columns appear to be limited. Studies by kitipornchai and lee (1986), presents a finite-element model calibrated against the tests and consists subsequently a parametric study of the strength of concentrically loaded T-section columns covering a wide range of geometries. Then it compares the obtained numerical strengths and test strengths with strength predictions for hot rolled and fabricated steel structures, and suggests more accurate design approaches than those currently available ease of use.

The main objective of this study is to analyze the behavior of wide flange steel column under axial loading with varying temperature. Due to the problem that there is lack of understanding in how structural systems perform under realistic uncontrolled fire situations.The development of performance-based standards and tools requires explicit consideration of the fire effects on structural components and systems. This consists a parametric study employing nonlinear finite element analysis to model the response of wide flange steel columns at elevated temperature. Different axial loads and different cross sections are included in the parametric study. This study first briefly describes the buckling and post-buckling behaviour of I-section columns and singles out the two main design issues such as the similarity between local buckling and torsional buckling modes. Then it compares the obtained numerical strengths and test strengths with strength predictions for hot rolled and fabricated steel structures, and suggests more accurate design approaches than those currently available.

2. Literature Review

Study for centrally loaded columns of D. Talamona, J. M. Franssen, J. B. Schleich, and J. Kruppa, in 'Stability of steel columns in case of fire: numerical modelling', says that the shape of the buckling curve in case of fire is different from the shape observed at an ambient temperature. The results are more consistently presented when the relative slenderness is evaluated at the failure temperature. In this case, the buckling curve does not depend significantly on the temperature. The buckling coefficient increases with increasing nominal yield strength. The scatter between different sections or different buckling planes is not significant. Francisco Sena Cardoso and Kim J. R. Rasmussen, 'Behaviour and Design of Concentrically Loaded T-Section Steel Columns', motivated by the fact that the torsional and local buckling modes are similar for I - section, whereby design strength predictions may be conservative if accounting for both modes. The focus of the study is on columns compressed between pinned ends. A shell finite-element model capable of accurately predicting the failure behaviour of T-section columns when submitted to geometric and material nonlinear analyses is calibrated against available experimental data. The development of residual stresses in T-section columns produced by cutting I-sections in half is first explained, including the initial bending of each T-section that results when an I-section is cut in half, followed by a comprehensive analysis of geometric imperfections.

John L. Dawe and Gilbert Y. Grondin, 'Inelastic buckling of steel plates', had comparison with 22 carefully conducted tests of web and flange type plates buckling in the inelastic range revealed that none of thesets of material properties resulted in uniformly accurate predicted values. Based on a semi-empirical analysis, a new set of material properties was developed. These properties resulted in close agreement between predicted and experimental plate buckling capacities for both web and flange type plates. Good agreement between predicted and experimental values was also found to exist for many specimens tested independently by several other investigators. J. Y. Richard Liew, N. E. Shanmugam, and S. L. Lee, 'Behaviour of thin-walled steel box columns under biaxial loading', had tests on welded thin-walled steel box columns subjected to biaxial loading show that the ultimate capacity of the columns drops significantly with an increase in the plate slenderness ratio. The drop becomes more significant in the case of biaxially loaded columns. Extensive measurements of welding residual stresses and initial column deflections were made. They were used in computing the load-carrying capacity of the columns. Results thus obtained showed the adverse effect of these imperfections on the failure load of test columns. A comparison of the experimental and theoretical failure loads shows that the analytical method is capable of predicting the ultimate load of thin-walled box columns under biaxial loading with reasonable accuracy. Nagarjun Krishnappa, Michel Bruneau, and Gordon P. Warn, 'Weak-Axis Behaviour of Wide Flange Columns Subjected to Blast', study says that high intensity, near field detonation producing large over pressures on the web of a wide flange column can result in highly localized web deformations, fractures along the web-to-flange joint, and create a hole in the column web (depending on the proximity of the charge); this indicates that loading perpendicular to the weak axis of bending is a critical scenario to be considered for the blast resistant design of wide flange columns.

3. Materials and Methodology

The material which is mainly used in this thesis work is structural steel to analyze its behaviour when it is subjected to fire exposure situation. The structural steel section mainly preferred for the analysis is Wide Flange I-sections. Finite Element Method is used for the analysis to investigate this coupled structural and thermal behaviour.

3.1. Basic concepts of wide flange column

Learning the basic concepts such as its component parts which may be considered as plate elements or structural sections which are usually proportioned so that local buckling will not occur in the elastic range, in which case the plate elements will usually buckle "inelastically" or by "plastic buckling" at an average stress somewhere between the proportional limit and the yield point of the material. In very compact sections buckling may occur at stresses above the yield point, but the yield point usually represents the practical upper limit of strength. So wide flange I sections are selected for the analysis covering the entire range from ISWB 150 to ISWB 600. Dimensions and specifications of rolled steel beam sections are given in the table 3.1. In most of the situations, height of the column is about 3m to 6m. Thus the selected specimen having a height in this range with an interval of 0.5m.

In common circumstances, the maximum temperature of a fully developed building fire will rarely exceed 950°C. The average gas temperature in a fully developed fire is not likely to reach 800°C.Temperatures of fires that have not developed to post-flashover stage will not exceed 500°C. So that a temperature range of 27°C to 700°C selected for the analysis.

3.2. Specifications of the column specimen analyzed

- Shape –I sections
- Size ISWB150 & ISWB 175
- Height of column 3m
- Temperature range 27°C to 700°C
- Support conditions

a) Effectively held in position at both ends but not restrained against rotation
b) Effectively held in position at both ends and restrained against rotation at one end
c) Effectively held in position and restrained against rotation at both ends

3.3. Properties
3.3.1. Physical properties

Physical Properties of structural steel irrespective of its grade may be taken as:

- Density, $\rho = 7850\ kg/m^3$
- Modulus of Elasticity, $E = 2 \times 10^5\ N/mm^2$
- Poisson Ratio, $\mu = 0.3$
- Modulus of Rigidity, $G = 0.769 \times 10^5\ N/mm^2$
- Coefficient of thermal expansion, $\alpha = 12 \times 10^{-6}$ /°C

3.3.2. Mechanical properties

Commonly used Mechanical Properties can be taken as:

3.4. Ultimate Tensile Strength = 410 MPa
3.5. Yield Stress = 250 MPa ; < 20 mm = 240 MPa ; 20-40 mm = 230 MPa ; >40 mm
3.6. % Elongation at gauge length 5.65 $\sqrt{\rho_0}$ = 23
3.7. Bend Test = 3 t
t – Thickness of the section

3.3.3. Chemical Composition

Table 1: Chemical Composition of Structural Steel

Sl. No.	*Compounds*	*Percentage*
1.	Carbon	0.23
2.	Manganese	1.5
3.	Sulphur	0.045
4.	Phosphorous	0.045
5.	Silicon	0.4
6.	Carbon Equivalent	0.42

Table 2: Rolled Steel Beam Dimensions and Properties

Designation	**c/s area mm²**	**Depth (h)**	**Width of flange (b)**	**Flange thickness (t_f)**	**Web thickness (t_w)**
ISWB 150	2167	150	100	7	5.4
ISWB 175	2811	175	125	7.4	5.8

3.4. Permissible stresses and load calculation

Common hot rolled steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a, b, c, and d. Permissible stresses and loads are calculated as per IS 800: 2007

- Formulas used are
– Design Compressive Strength

$P_d = A_e \times f_{cd}$
- A_e - effective sectional area
- f_{cd} - Design compressive stress

– Design Compressive Stress,
$f_{cd} = (f_y / \Upsilon_{mo}) /(\varphi + (\varphi^2 - \lambda^2)^{0.5})$
- $\varphi = 0.5(1+\alpha(\lambda-0.2)+ \lambda^2)$
- Non –dimensional effective slenderness ratio, $\lambda = \sqrt{(f_y / f_{cc})}$

Euler buckling stress, $f_{cc} = \pi^2 E/(KL/r)^2$

Table 3: Design Compressive Strength for Column Sections

Item	**Support Condition (a) (kN)**	**Support Condition (b) (kN)**	**Support Condition (c) (kN)**
ISWB 150	161.2459	233.5905	310.3937
ISWB 175	298.9789	405.8403	490.2104

3.5. Finite Element Analysis

The finite element method (FEM) is the most popular simulation method to predict the physical behaviour of structures. Since analytical solutions are in general not available for most daily problems. The finite element method models were developed to analyze the behaviour of structural steel under elevated temperature using the ANSYS program. Element types used are SOLID 92 and SOLID 45 for structural analysis and SOLID87 and SOLID70 for thermal analysis. The Newton raphson method of analysis was used to compute the non linear response. The application of the load up to failure was done incrementally as required by the Newton raphson procedure.

4. Results

Analysis is done on two steps. First the column section is analyzed structurally, then its behaviour is compared with coupled thermal and structural analysis of column section.

4.1. Structural Analysis

Structural analysis is the determination of the effects of loads on physical structures and their components. In this analysis, the column section is assumed to be loaded with a permissible load which is calculated using permissible stress. Due to this permissible load the structure will deflects, it should be within the permissible limit i.e., approximate zero.

Columns selected for the analysis are short columns and it only undergo crushing due to compression. So the deflection due to permissible stress is zero. From the analysis results shown in table IV, it is clear that the calculated permissible stresses produce approximate zero deflection.

4.2. Coupled Thermal and Structural Analysis

Coupled analysis in which the column section is analyzed both structurally and thermally. Thus we get the behaviour of columns subjected to axial loading with fire exposure. As a structural part, the section is subjected to permissible loading with different support condition and in thermal analysis; this loaded structure is exposed to a temperature range for a period of 15 minutes.

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Table 4: Results of Structural Analysis

Support condition	***Items***	***Displacement vector sum***	***Permissible load***	**Stiffness**
Effectively held in position at both ends but not restrained against rotation	ISWB 150	0.03189	161.2459	5056.3159
	ISWB 175	0.038319	298.9789	7802.3682
Effectively held in position at both ends and restrained against rotation at one end	ISWB 150	0.031890	233.5905439	7324.8838
	ISWB 175	0.038319	405.8402951	10591.098
Effectively held in position and restrained against rotation at both ends	ISWB 150	0.031890	310.393657	9733.2599
	ISWB 175	0.038319	490.2103832	12792.88

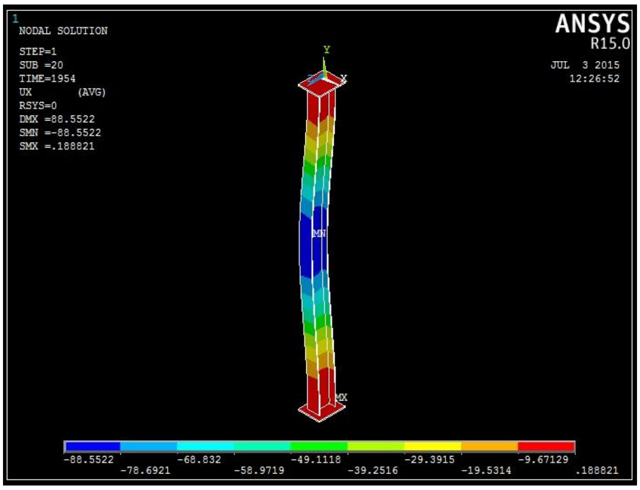

Figure 1 : Buckling pattern of ISWB 175

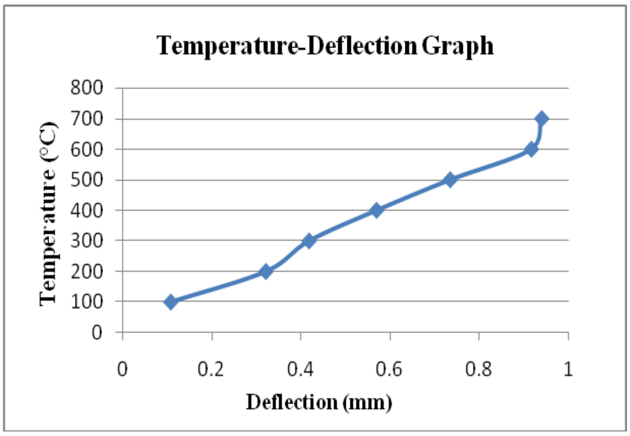

Figure 2 : Temperature-deflection graph of ISWB 150

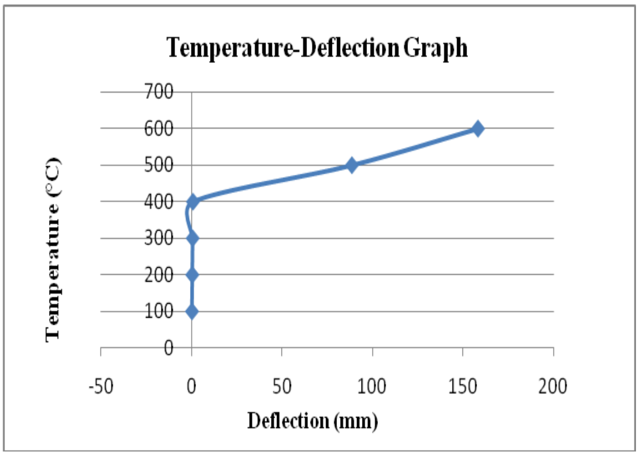

Figure 3 : Temperature-deflection graph of ISWB 175

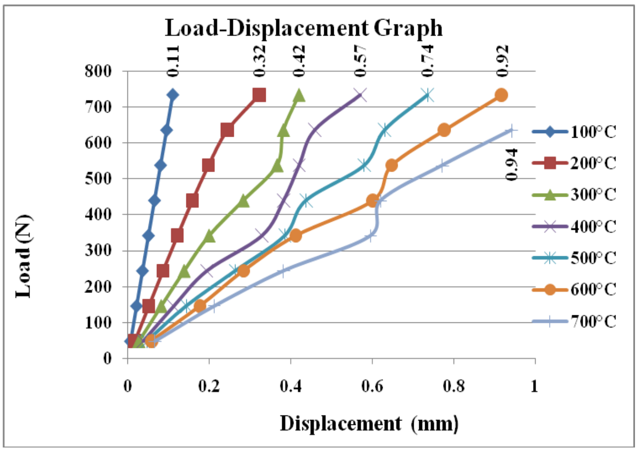

Figure 4 : Load – Displacement Graph of ISWB 150

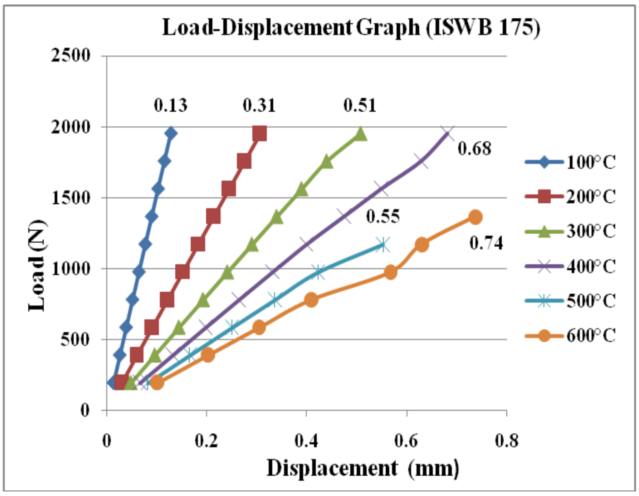

Figure 5 : Load – Displacement Graph of ISWB 175

From the coupled analysis it is clear that the column behaves as slender and buckle when the temperature increases. Figure 1 shows the buckling pattern of ISWB 175 in the temperature range 400 - 500°C. The temperature-deflection graph of ISWB 175 (Figure 3) show that it fails when the temperature increases beyond 500°C. But the temperature-deflection graph of ISWB 150 (Figure 2) show that it fails when the temperature increases just to 600°C. Figure 4 and 5 shows that the column section had a tendency to increase the deflection due to temperature increase, that it yield as the temperature increases. Here in this graph nadal loads are taken for the analysis.

As the permissible load for each section is different, but the behaviour pattern is same, it buckles at an elevated temperature range. Failure of each section occurs at different temperature range. As the cross section of the column increases, but the failure range of temperature decreases, it may due to increase in permissible load.

5. Conclusion

Failure behavior at elevated temperature is not understood under loading conditions, thus the finite element modeling gives its behavior that the short column may buckle along its weak axis due to axial loading with elevated temperature. Thus the property of short column changes as it buckles due to loading instead of crushing during elevated temperature situations. For an average loading and temperature condition most appropriate cross section to be having less cross sectional area with permissible loading condition. As the permissible loading decreases more will be the withstandable temperature limit.

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References

[1] Anil Agarwal, Lisa Choe and Amit H Varma, “Fire design of steel column: Effects of thermal gradients”, Journal of Constructional Steel Research, vol. 93, 107-118, 2014.
[2] Francisco S C and Kim J R, “Behaviour and design of concentrically loaded T-section steel columns”, Journal of Structural Engineering, 1-17, 2014
[3] John L. Dawe and Gilbert Y. Grondin, “Inelastic buckling of steel plates”, Journal of Structural engineering, 97 – 107, 1985
[4] J Y Richard Liew, N E Shanmugam, and S L Lee, “Behaviour of Thin walled steel box columns under Biaxial loading”, Journal of Structural Engineering, Vol 115, No.12, 1990
[5] Nagarjun Krishnappa, Michel Bruneau, and Gordon P, “Weak axis behaviour of wide flange columns subjected to blast”, Journal of structural engineering, 1-7, 2013
[6] D. Talamona, J. M. Franssen, J. B. Schleich, and J. Kruppa, “Stability of steel columns in case of fire: numerical modeling”, ‘Journal of Structural engineering’, Vol 123, No.6, 1997