Behaviour of Columns Made From High Strength Steel

Mohamed Said, Rishicca Kamalarajah, Katherine A. Cashell, Mahmoud Chizari

Abstract— **The buckling behaviour of compression members is very important in structural engineering especially those that are made from high strength steel (HSS). This paper presents an investigation into the behaviour of four columns which were designed according to EN 1993-1-1 (2005) so that the buckling capacity of both HSS and mild steel column was equal. A finite element model was created which included imperfections due to residual stresses as well as local and global imperfections. It was found that the buckling capacity of the columns made from 460 MPa steel was lower than that for similar capacity mild steel columns with a yield strength of 275 MPa. Accordingly, a buckling curve for HSS is proposed.**

Index Terms— **buckling behaviour, finite element, columns, imperfection**

I. Introduction

THERE has been a surge in the usage of structural members made from high strength steel (HSS) due to their high strength to weight ratio, structural safety and lower environmental impact. Most design specifications such as EN 1993-1-1 (2005), ANSI/AISC 360-10 (2010) and the Chinese code GB50017-2003(2006) are limited to steel with a yield strength of between 235 and 460 MPa. The overall compression behaviour of a column is determined by many factors including the yield strength of the steel, non-dimensional slenderness, boundary conditions and imperfections associated with the column (such as residual stresses and geometric imperfections) (Ban et al., 2013).

Recently, there has been an increase in the number of research studies into the overall buckling behaviour of HSS columns. Ban et al., (2012) carried out an experimental investigation to study the overall buckling behaviour of 460 MPa high strength steel compression members. A total of 12 columns, which included both box and welded I-sections, were tested. Initial imperfections such as residual stresses and initial bending were measured. Experimental results showed that all specimens failed by overall global buckling.

Furthermore, these researchers also did a detailed numerical study using finite element analysis (FEA) software. The results of this study revealed a poor correlation between the numerical data and the buckling response predicted by EN 1993-1-1 (2005). Therefore, a new buckling curve was proposed for HSS with an imperfection of 0.254 which is between curve A and B in Eurocode 3.

Residual stresses play a major role in the behaviour of compression members, especially those that fail due to flexural buckling. This is a result of their premature yielding and loss of stiffness as previously mentioned. Although the account of residual stress taken in EN 1993-1-1 (2005) for normal strength steel (NSS) columns has shown good correlation, this is not the case for HSS columns, for the following reasons: (1) the material properties and the manufacturing process for HSS and NSS are not the same leading to different metallurgical arrangements; (2), the maximum tensile residual stresses close to the welded region is taken as its yield strength for normal steel members whereas for HSS members, the maximum tensile residual stresses close to the welded region can be lower than the yield strength (A.W. Huber et al., 1954).

The most important failure modes in the design of steel columns are local buckling failure, global buckling failure and the interaction between these two modes. If a very thin column is designed then it will likely fail because of premature local buckling. However, if a column is designed with a smaller width and/or greater thickness, it will probably fail via global buckling (Becque, 2014).

Van Der Neut (1973) carried out a comprehensive research into the interaction between local and global buckling. The focus of his research was to see what extent this imperfection can have on the ultimate buckling capacity. By looking at different box section model with pin ended condition, the following results were obtained: (1) if the Euler buckling load is significantly greater than the local buckling load then the column will fail by pure elastic buckling; (2) if the local buckling load is close to the Euler buckling load then the flanges will buckle first (local buckling) however this does not result in a decrease in its load bearing capacity meaning they can continue to carry load post flange buckling.

II. Methods

A. FEA Model

This section begins by looking at the flexural buckling behaviour of high strength steel with a yield strength of 460 MPa and comparing this with mild steel with a yield strength of 275 MPa. Using EN 1993-1-1 (2005) clause 6.3.1.1, a mild steel column and a high strength steel column were designed so that the theoretical buckling capacity and length for both

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steel grade would be equal while the cross-sectional area will differ.

Once the column was designed using EN 1993-1-1(2005), ABAQUS was used to carry out a linear and non-linear analysis in order to obtain a buckling capacity which will then be compared to EN 1993-1-1(2005). The material properties of both steel grades were implemented in ABAQUS based on the tensile coupon test carried out by Ban et al., (2012). Young's modulus for steel is taken as 2.1×10^5 MPa. For both models, the local imperfections were considered in the FEA analysis by updating the geometry of the column with an amplitude of 0.16% based on the first order eigenvalue buckling mode (Ban et al., 2012). The distribution of residual stresses is also quantified by the model L1-460 (Ban et al., 2012). The Global imperfection was proposed in this paper and the amplitude of the imperfection is taken as $\frac{\mathbf{Tf}}{\mathbf{100}}$ where T_f is the thickness of the flange. Poisson's ratio is taken as 0.3. Boundary condition of both the columns will be pinned-pinned. A Linear analysis was carried out to quantify Euler's buckling load. The buckling capacity was obtained by RIKS method to quantify the overall buckling strength.

TABLE I
GEOMETRIC PROPERTIES

Symbol	Quantity
A	Area
I	Second moment of area
NCR	Euler Buckling load
$\bar{\lambda}$	Non-dimensional slender
fy	Yield strength
Φ	Intermediate factor
χ	Reduction factor
α	Imperfection factor
Φ_{EC3}	Buckling resistance obtained from EUROCODE 3
φ_{FEA}	Buckling resistance obtained from ABAQUS

B. *Linear analysis*

A linear analysis was first carried out using ABAQUS. The Linear analysis in ABAQUS solves Eigenvalue problems defined by geometric matrices and elastic stiffness of the column. A solution will be obtained by subspace iteration method. As a result, mode shape and their corresponding EigenValue will be obtained, and since both columns are under pinned-pinned conditions, Euler's buckling load of the column should equate to the corresponding EigenValues. Euler's buckling load occurred in the first mode shape of both columns. The displacement of the modes was saved by modifying the keywords and entering the code 'NODE FILE U'. This will be used as a reference for the local imperfection and global imperfection in the non-linear analysis.

The geometry details of the column are given in Table II.

TABLE II
THE RELEVANT GEOMETRIC DETAIL OF THE 2 COLUMN

Model	M1-275	H1-460	M2-275	H2-460
B, mm	290	205	308.9	291
H, mm	300	280	305.3	298
T_f, mm	14	11	15.4	12
T_w, mm	8.5	6	9.9	8
A, mm^2	11200	7900	12300	9850
I_x, mm^4	6.31×10^7	4.02×10^7	7.36×10^7	4.92×10^7
L_o, mm	6000	6000	7000	7000
N_{cr}, N	3.63×10^6	2.31×10^6	3.11×10^6	2.08×10^6
Φ_{EC3}, N	1.81×10^6	1.81×10^6	1.735×10^6	1.735×10^6

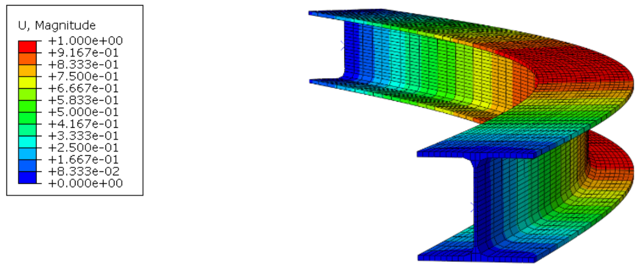

Fig 1. Model M1-275 undergoing linear analysis. Eigenvalue of M1-275 is 3.610×10^6 N. Euler Buckling Load for M1-275 is 3.63×10^6

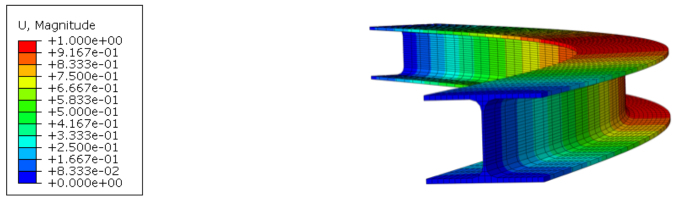

Fig 2. Model H1-460 undergoing linear analysis. Eigenvalue of H1-460 is 2.309×10^6 N. Euler Buckling Load for H1-460 is 2.31×10^6

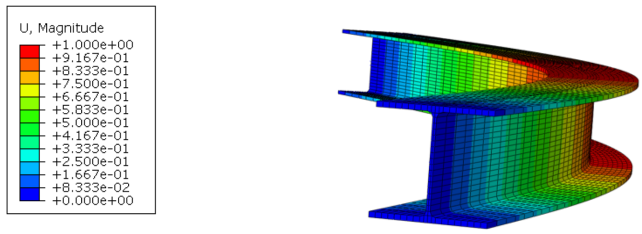

Fig 3. Model M2-275 undergoing linear analysis. Eigenvalue of M2-275 is 3.075×10^6 N. Euler Buckling Load for M1-275 is 3.11×10^6

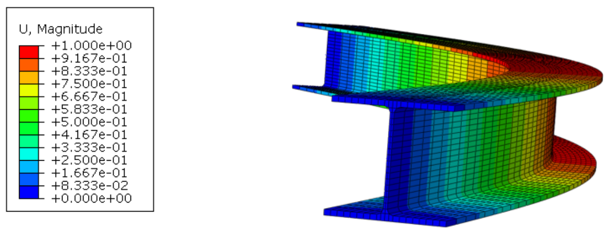

Fig 4. Model H2-460 undergoing linear analysis. Eigenvalue of H2-460 is 2.070×10^6 N. Euler Buckling Load for H1-460 is 2.08×10^6

C. *Non-Linear analysis*

Once a linear analysis has completed, a nonlinear analysis was performed to determine the buckling resistance φ_{FEA}. In this work, RIKS method was used along with predefined increment and tolerance parameter. A total of 100 increments was set. Imperfection such as global, local, and residual stresses was also implemented. Table 3 shows the residual distribution stresses used in both models. Table 4 illustrates the amplitude of both global and local imperfection.

TABLE III
DISTRIBUTION OF RESIDUAL STRESSES OF USED FOR ALL SPECIMEN IN FEA ANALYSIS
(BAN ET AL., 2012)

σ_{frt} MPa	σ_{frte}, MPa	σ_{frc}, MPa	σ_{wrt}, MPa	σ_{wrc}, MPa
345	35	-254.1	345	-302.4

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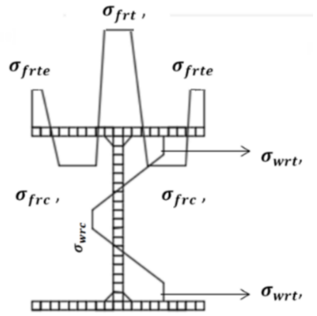

Fig 5 Distribution of residual stresses of used for all specimens in FEA analysis (Ban et al., 2012)

TABLE IV

AMPLITUDE OF BOTH GLOBAL AND LOCAL IMPERFECTION USED IN FEA

Specimen	Global imperfection %	Local imperfection
M1-275	0.16	0.14
H1-460	0.16	0.11
M2-275	0.16	0.154
H2-460	0.16	0.12

The non-dimensional buckling capacity φ_{FEA} was obtained from ABAQUS using the peak loading graph in figure 5 and 6 for both mild steel and high strength steel. φ_{FEA} will also be compared with those in design curves in EUROCODE 3, and this can be seen from figure 7 and 8

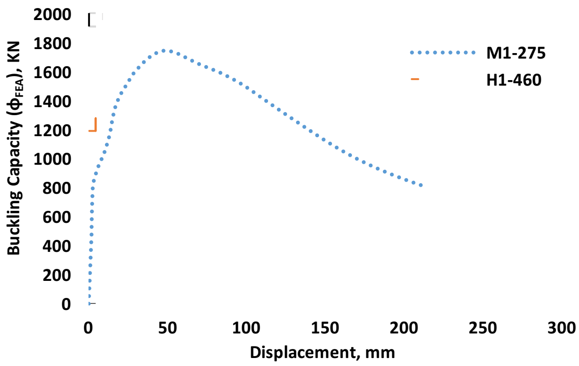

Fig 6. Comparisons of the buckling capacity (φ_{FEA}) of model M1-275 and H1-460

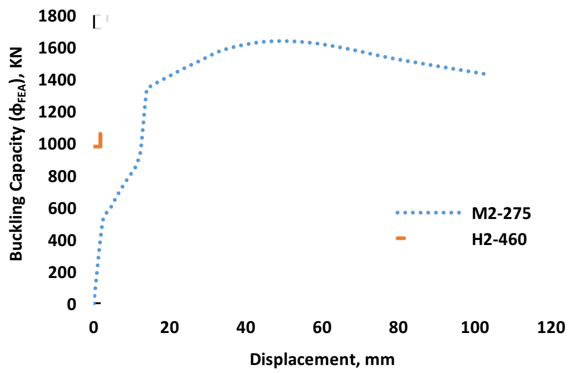

Fig 7. Comparisons of the buckling capacity (φ_{FEA}) of model M2-275 and H2-460

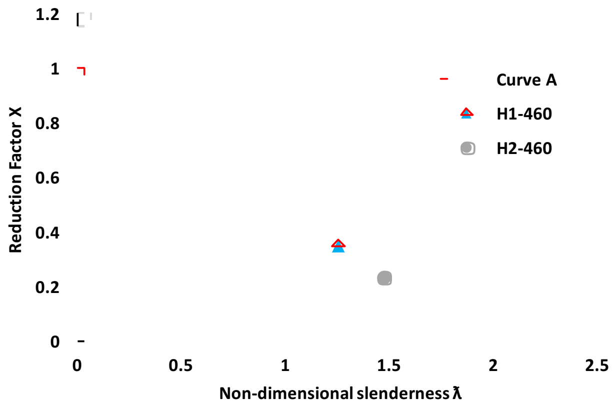

Fig 8. Comparisons of the FEA results with design curve A of EUROCODE 3.

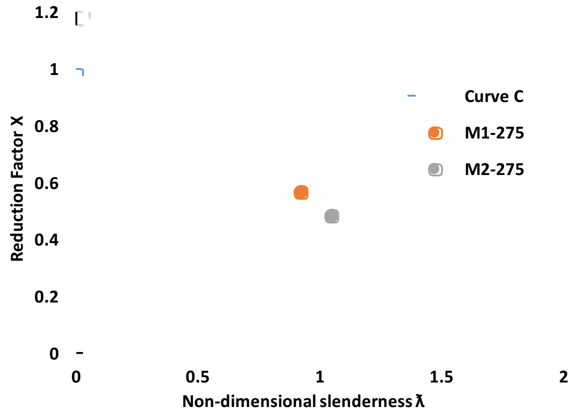

Fig 9. Comparisons of the FEA results with design curve C of EUROCODE 3

It was found that the FEA result was very close to the buckling resistance in EUROCODE 3 for mild steel. Model M1-275 φ_{FEA} was 1.735×10^6 N whereas φ_{EC3} was calculated as 1.81×10^6 N showing a percentage error of 4.14%. Model M2-275 achieved a φ_{FEA} of 1.64×10^6 N whereas as φ_{EC3} is calculated as 1.735×10^6 N showing a percentage error of 5.47%.

However, with high strength Steel, there is no correlation between the buckling capacities calculated in Eurocode 3 with the buckling capacity obtained from ABAQUS. Theoretical buckling capacity of H1-460 is 1.81×10^6N and the buckling capacity obtained from FEA analysis were 1.27×10^6 N showing a percentage error of 30%. H2-460 theoretical buckling capacity was 1.735×10^6 and the buckling capacity obtained from ABAQUS was 1.06×10^6 showing a percentage error of 39%

D. Proposed column design curves

In order to propose a design buckling curve, 11 pin-ended column with the same cross sectional area but different length under axial compression around the minor axis was calculated using ABAQUS. Table 5 summarizes the dimension of the columns. For each column, the initial geometric imperfection and global imperfection and residual stresses are taken as exactly as those in Table 3 and 4. Linear and Non-linear analysis was carried out to determine the buckling capacity φ_{FEA}.

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TABLE V
DIMENSION OF COLUMN SECTIONS FOR PARAMETRIC ANALYSIS

Model	H mm	B mm	T_f mm	T_w mm	L_0 mm	λ	φ_{FEA}, N
H1	195	280	11	7	2000	0.420	3.4×10^6
H2	195	280	11	7	2400	0.509	3.2×10^6
H3	195	280	11	7	3000	0.632	3.05×10^6
H4	195	280	11	7	3400	0.716	2.9×10^6
H5	195	280	11	7	4000	0.841 3	2.3×10^6
H6	195	280	11	7	4400	0.927	2.0×10^6
H7	195	280	11	7	5000	1.053 9	1.8×10^6
H8	195	280	11	7	5400	1.13	1.6×10^6
H9	195	280	11	7	6000	1.264	1.4×10^6
H10	195	280	11	7	7000	1.475	1.2×10^6
H11	195	280	11	7	8000	1.686	1.0×10^6

III. RESULTS AND DISCUSSION

Figure 10 shows the FEA results of the 11 specimens and their comparisons with all five buckling curves. The proposed column is derived from the Perry equation that is adopted in EN 1993-1-1 (2005) clause 6.3.1.1

$$\chi = \frac{1}{\Phi+\sqrt{\Phi^2-\lambda^2}} = \frac{1}{0.5[1+\alpha(\lambda-0.2)+\lambda^2]+\sqrt{\{0.5[1+\alpha(\lambda-0.2)+\lambda^2]\}^2-\lambda^2}}, \&\leq 1.0$$

Where χ is the reduction factor and λ is the non-dimensional slenderness and α is an imperfection factor. Figure 10 shows that H1, H2, H3 and H4 lie just above curve B. H5 lies on point C, and H6-H11 all lie above curve D. As a result, the purpose curve will be curve D since all the specimen failed above curve D.

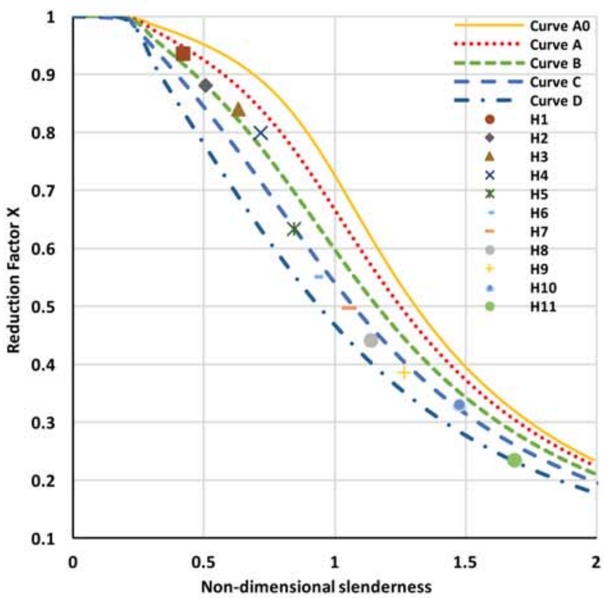

Fig 10. FEA results of the 11 specimens and their comparisons with all 5 buckling curves

IV. CONCLUSION

As a conclusion, it can be said that the FEA result of model H1-460 and H2-460 were not in agreement with their corresponding curves in EUROCODE 3. Whereas mild steel, both models were in good agreement with EUROCODE 3. As a result, 11 HSS specimens were designed in ABAQUS to compare their buckling capacity with all five buckling curves in EUROCODE 3. The result has shown that all buckling capacity of 11 models lies above curve D, hence curve D will be an adequate curve to use for 460 MPa HSS.

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