STRENGTH AND STABILITY OF HIGH-STRENGTH STEEL TUBULAR BEAM-COLUMNS

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1. ABSTRACT

The present work examines structural strength and stability issues for steel tubular beam-columns made of high-strength steel. The tubular beam-column elements are widely used in a variety of structures such as buildings, off-shore templates, masts, towers and cranes. The research focuses on the analysis of such members for better understanding the benefits of using high strength steel in tubular structures.

Extensive numerical work has been conducted on CHS high strength steel beam-columns of two cross sections and various values of member length. First, considering initial out-of-straightness imperfection, structural member stability curves are calculated. Subsequently, an extensive beam-column analysis takes place and interaction diagrams are provided as a result. The numerical results are compared with the predictions of the current Eurocode 3 provisions.

2. INTRODUCTION

The structural behavior of tubular beam-column members under combined loading conditions has been extensively investigated through the past. The present extensive numerical parametric study is a part of a large research effort with the scope of revising the current slenderness limits for CHS member classification and examining the buckling behavior of high strength steel tubular elements under axial compression and combined loading conditions. More specifically, the structural response of high strength steel (σ_y=590MPa) imperfect tubular beam columns of various lengths and cross sections is investigated under axial compression and combined loading. Extensive finite element analysis has been conducted in order to develop buckling curves and interaction diagrams of compression and bending which are compared with the current design provisions of EN-1993. For the comparison between FEA results and European provisions, the safety factors included in the EN 1993 equations are not considered.

There exist limited information for cross sections of strength steel over 460 MPa and the current work will contribute at the enhancement of the current guidelines for stability curves, interaction diagrams and classification limits for high strength steel CHS members.

3. NUMERICAL SIMULATIONS OF BEAM-COLUMNS

General-purpose finite element program ABAQUS is employed to simulate the performance of high-strength steel tubular beam-columns of various lengths and diameter-to-thickness ratios (D/t) under combined loading conditions. The geometrical characteristics of the tubes reported in the present paper are shown in *Table 1*. The monotonic analysis considers nonlinear geometry and a J_2- flow (von-Mises) large strain plasticity model, with isotropic hardening. The tubular column is modeled with four-node reduced-integration shell elements (S4R) available in ABAQUS. The finite element mesh used has been optimized, so that the numerical model is accurate and time-effective.

Tube	Outer diameter D_o (mm)	Thickness t (mm)	D_o / t	Yield stress σ_y (MPa)	Class (EN-1993-1-1)
A	355	12	29.58	590	Class 3
B	193.7	10	19.37	590	Class 1

Table 1. Geometric-mechanical properties of the tube models

The numerical models consist of simply-supported tube elements. A stress-free initial crookedness on the longitudinal axis with amplitude equal to w_0=L/300 at the mid span of the element, is implemented in the models' geometry. In the case of axial loading, axial compressive load is applied at the element's ends and it is gradually increased so that the non linear equilibrium path of load vs. displacement is traced and the maximum buckling load is identified. In the case of monotonic bending, bending moments are applied at both ends of the members until a maximum moment is reached. For the application of either compression or bending, Riks' algorithm was used through force and moment control.

In the case of combined loading conditions, each tube is initially subjected to axial compression up to 0.25, 0.5, 0.75 of the maximum load derived from the analysis under pure axial compression for the specific model described above. Subsequently, keeping the axial load constant, bending loading is gradually applied at the end sections until the maximum bending capacity is reached, using Riks' algorithm.

4. NUMERICAL RESULTS

4.1. Stability curves

Previous numerical and experimental research [2-9] has been conducted to investigate the stability and buckling strength of beam-columns in terms of their slenderness, which is a function of the length and the material yield stress, described in *eq. (1).*

$$\lambda = \frac{KL}{\pi r} \left(\frac{F_y}{E} \right)^{0.5}$$
(1)

where K is the effective length factor, affected by the boundary conditions applied on the model, r is the radius of gyration of the tubular cross section, E is the Young's Modulus and F_y is the yield stress. For simply supported members, K=1.

The present parametric numerical analysis aims at developing stability curves, i.e. buckling strength vs member slenderness. The geometric characteristics of the tubular cross sections under consideration are presented in *Table 1* and the member length L ranges from 0.5 to 10m. The material uniaxial stress-strain curves used in the present analysis are depicted in *Fig. 1*, representing a theoretical bilinear elastic-plastic curve of a high-strength steel grade T590 with yield stress equal to σ_y=590 MPa and hardening modulus equal to E/500.
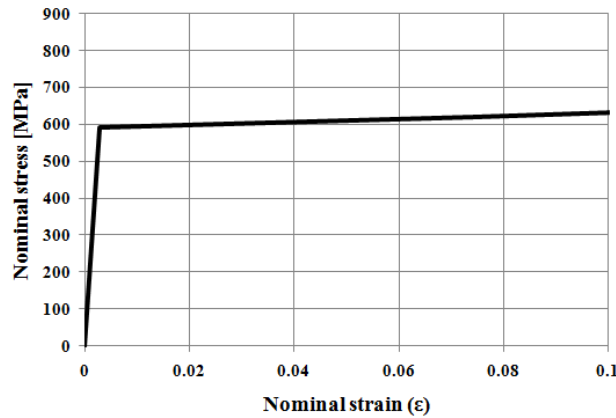

Fig. 1 Plasticity model prediction for the uniaxial material stress-strain curve

Each model is subjected to axial compression until failure, in terms of ultimate axial strength, due to local or global buckling. The deformed geometry and the buckle development of the models with cross section "A" are shown in *Fig. 2*, for a relatively short (2m long) and a 5m-long beam-column, corresponding to a member slenderness (λ) of 0.275 and 0.688, respectively.
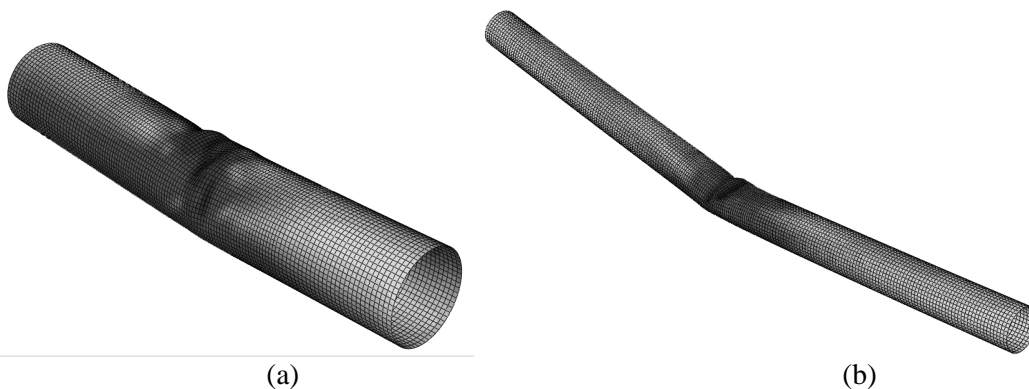

| (a) | (b) |

Fig. 2 Failure mode of the "A" type numerical model (a) 2m and (b) 5m long under axial compression

The corresponding non-linear load-displacement paths are shown in *Fig. 3*. It is observed that, in short members (λ=0.07-0.3), local buckling occurs progressively while plastic deformation is accumulated at the most critical region of the tube wall. The post buckling branch appears to be rather smooth, while the member does not deviate from the longitudinal axis until the tube wall has been excessively deformed. For intermediate and large values of member slenderness, global (Euler type) buckling occurs, and the member is suddenly deflected laterally from the initial straight position. As a result, an abrupt

transition from the pre-buckling to the post-buckling region is observed on the load-displacement curve and the post-buckling part becomes unstable.
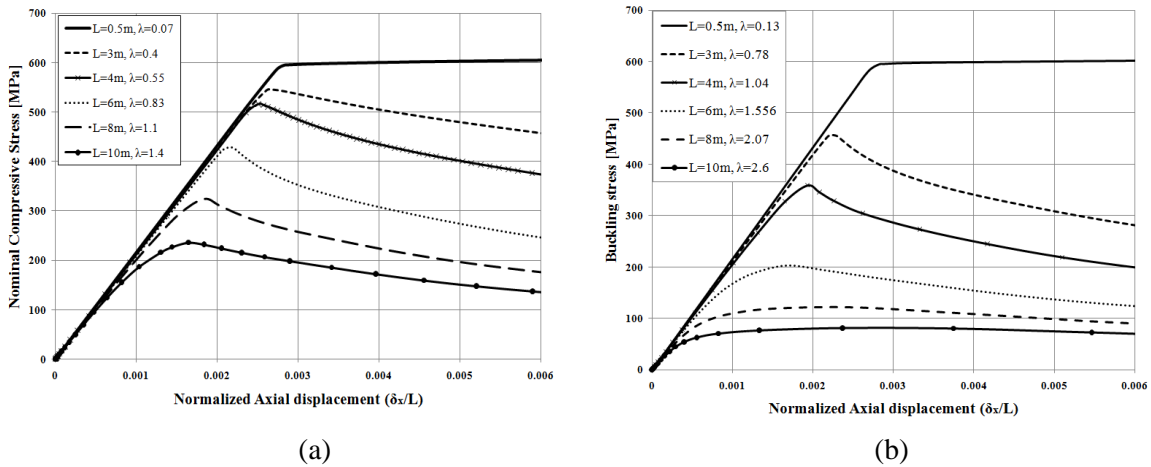

(a)   (b)

Fig. 3 Load vs Displacement curves for cross sections (a) "A" and (b) "B". Comparison for various member slenderness values

The maximum buckling load for each finite element tubular model is identified and the predicted buckling strength for various values of member slenderness is depicted in *Fig. 4*. The stability curves obtained numerically are compared with the provisions of EN-1993 [1]. It should be noted that the cross sections under consideration, are classified as class 3 and class 1 referring to "A" and "B", respectively, according to EN-1993-1-1[1] using a stress value σ_y=590 MPa.
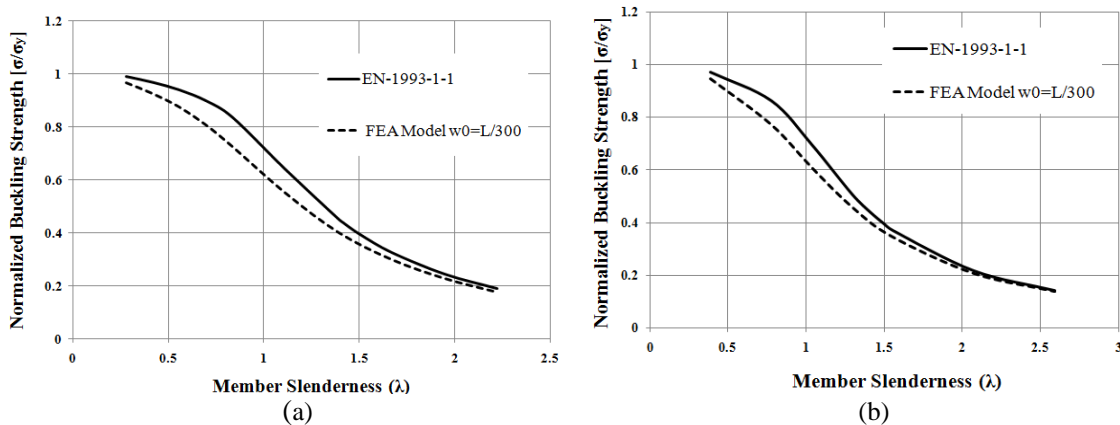

(a)   (b)

Fig. 4 Stability curves for cross sections (a) "A" and (b) "B"

Despite the fact that EN 1993-1-1[1] indicates that curve "a_0" is the most suitable for high-strength steel tubes, from *Fig. 4,* it can be observed that this overestimates the strength of the beam-columns. This indicates that a revision of buckling provisions in EN 1993-1-1 for the case of high-strength steel CHS members is necessary. It should be kept in mind that the imperfection "L/300" suggested by the European provisions for the application of "a_0" curve, and imposed in the finite element model's geometry, actually represents an "equivalent imperfection value" so that the effect of residual stresses on the tube wall are taken into account indirectly.

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4.2. Interaction diagrams

Interaction diagrams are developed for 3m, 5m and 8m-long beam-columns under combined loading conditions. The initial out-of- straightness imperfection (w_0) adopted for this analysis is equal to L/300. The geometrical and material properties are similar to the ones described previously.

The interaction diagrams, of axial compression and bending, for 3 and 8m of member length for the "A" cross section, (corresponding to member slenderness equal to 0.417 and 1.11) are shown in *Fig. 5*. The axial and bending values are normalized with the values $N_0 = \pi D_m t \sigma_y$ and $M_0 = D_m^2 t \sigma_y$, representing the fully-plastic compressive and bending strength of the cross section respectively. The results show that the bending capacity is not affected by the value of member slenderness but it depends strongly on the material yield stress and cross sectional geometry. On the other hand, buckling strength is significantly reduced by the increase of member length while global (Euler-type) buckling governs the overall member behavior.
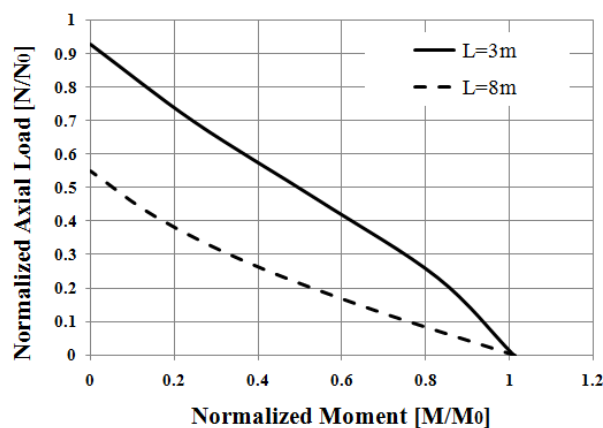

Fig. 5 Interaction Diagrams for "A" cross section of L=3m, (λ=0.417) and L=8m, (λ=1.11) in comparison

The interaction diagrams shown in *Fig 5* are compared with the curves proposed by EN-1993-1-1 using a nominal yield stress σ_y =590 MPa as shown in *Fig.6*. Two main observations can be noted from this comparison. First, EN-1993-1-1 provisions significantly penalize the bending capacity of the tubular member so that the bending strength is underestimated by approximately 20%. The reason behind this difference is that the CHS member is classified as class 3 but the finite element results indicate that it is capable of undergoing significant inelastic deformation in terms of rotational capacity without lost of strength. Therefore the EN-1993-1-1 predictions appear to be rather conservative. The second observation is that the EN-1993-1-1 applied limit for buckling strength is by 15% higher than the corresponding FE predictions especially for the 8m long tubular member, also indicated by the stability curves illustrated previously in *Fig. 4* for member slenderness equal to 1.11.
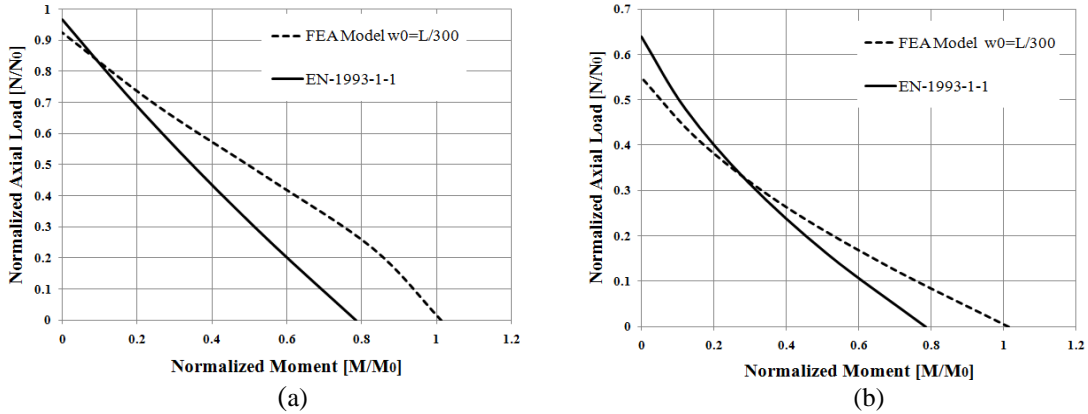
<div align="center">(a)</div>

<div align="center">(b)</div>

Fig. 6 Interaction Diagrams for "A" cross section with (a) L=3m, (λ=0.417) and (b) L=8m, (λ=1.11)
Comparison with EN-1993

The interaction diagram of a 5m-long tubular finite element model of cross section "B", (corresponding to member slenderness equal to 1.3) is compared with EN-1993-1-1 provisions as shown in *Fig. 7*. The comparison shows that the Eurocode 3 predictions for the ultimate bending moment correlate well with the finite element results. On the other hand, the buckling strength is overestimated by approximately 10%, also noted in the development of stability curves.
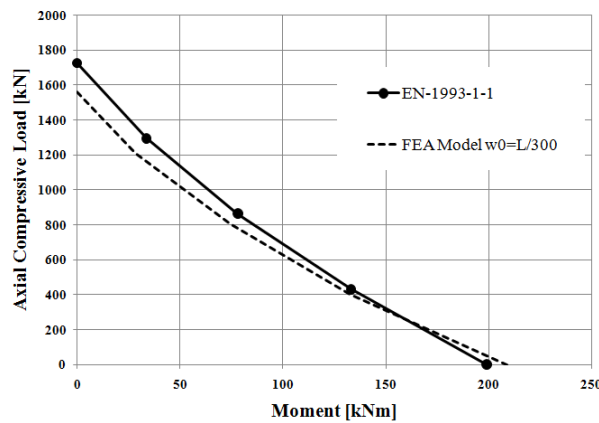

Fig. 7 Interaction Diagrams for "B" cross section with L=5m, (λ=1.3)

5. CONCLUSIONS

The above observations indicate that new provisions for high-strength steel CHS members are necessary. In particular, the use of "a_0" curve appears to be questionable; therefore, it may be necessary to introduce an enhanced stability design curve for high strength steel members. Moreover, the conservativeness of the code provisions with respect to the FEA results for the bending capacity, raises a significant concern on the accuracy of the current classification limits adopted by the EN-1993-1-1[1] for high-strength steel CHS members.

6. ACKNOWLEDGEMENT

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ΑΝΤΟΧΗ ΚΑΙ ΕΥΣΤΑΘΕΙΑ ΣΩΛΗΝΩΤΩΝ ΔΟΜΙΚΩΝ ΜΕΛΩΝ ΑΠΟ ΧΑΛΥΒΑ ΥΨΗΛΗΣ ΑΝΤΟΧΗΣ

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ΠΕΡΙΛΗΨΗ

Η παρούσα εργασία εξετάζει την δομική αντοχή και την ευστάθεια σωληνωτών μελών από χάλυβα υψηλής αντοχής που υπόκεινται σε συνδυασμένω καταπόνηση από κάμψη και αξονική δύναμη. Τα εν λόγω μέλη έχουν σημαντικές εφαρμογές σε κτηριακά έργα, θαλάσσιες πλατφόρμες πετρελαίου, πύργους και γερανούς. Η παρούσα έρευνα εστιάζει στην αντοχή τους με στόχο την καλύτερη κατανόηση της συμπεριφοράς τους και την επισήμανση πιθανών πλεονεκτημάτων ή μειονεκτημάτων στην χρήση χάλυβα υψηλής αντοχής.

Η έρευνα είναι αναλυτική/αριθμητική με την ανάπτυξη προσομοιωμάτων πεπερασμένων στοιχείων για τα σωληνωτά μέλη στο πρόγραμμα ABAQUS. Χρησιμοποιήθηκαν δύο βασικές διατομές και διάφορες τιμές του μήκους για το σωληνωτό μέλος. Αρχικώς, χρησιμοποιώντας ατέλεια της μορφής απόκλισης από την ευθυγραμμία, υπολογίστηκαν οι καμπύλες λυγισμού των δύο διατομών. Στην συνέχεια, χρησιμοποιώντας τα ίδια προσομοιώματα, υπολογίστηκε η αλληλεπίδραση μεταξύ της αξονικής θλιπτικής δύναμης και της ροπής κάμψης και τα αποτελέσματα παρουσιάστηκαν σε κατάλληλα διαγράμματα αλληλεπίδρασης. Τέλος τα αριθμητικά αποτελέσματα συγκρίθηκαν με τις αντίστοιχες διατάξεις του Ευρωκώδικα 3 (ΕΝ 1993-1-1).