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EFFECTIVENESS OF EHST COLUMNS BY STRENGTHENING OF IN-FILLED CONCRETE

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Summary: *The issues of semi-circle flange and flat-web distortional buckling effects due to the shape and differing flexural rigidities by two primary axes of the cold-formed Elliptical Hollow Steel Tubular (EHST) section columns become a key issue, which particularly need a strengthening form to postpone such failure criteria. A 3D Finite Element (FE) model of EHST column with discrete rigid ended plates was created, and one rigid plate was loaded at centric axis with applying fixed support at the next end. Good agreement in terms of axial load and shortening responses was achieved in the responses of FE model with considering experimental analysis found in the open literature. The validated FE model was then analysed with in-filled material of concrete with various strengths in terms of the increases on the strength and stiffness of the column. The variables in concrete strengths examined in the FE model investigation were significant, and this was found as a probablistic method of strengthening to avoid such inward local buckling failure criteria. In addition, the ultimate axial load in the strengthened cold-formed EHST columns is described with developed stress resultants due to the external axial centric load.*

Keywords: *Elliptical hollow Steel tubular columns, Finite element analysis, Ultimate axial centric load, Concrete infill, Parametric studies, Failure criteria*

1. INTRODUCTION

In terms of the effects of semi-circle flange and flat-web distortional buckling arisen on cold-formed Elliptical Hollow Steel Tubular (EHST) section columns due to its slenderness and flexural rigidities by differing major and minor axes properties, such section encountered local and global instability issues, which have not been thoroughly investigated in previous research studies. Non-linear behaviour of the columns was insisted on the prediction of the interaction between local and global failure criteria in this research study by considering the material non-linear behaviours such as steel and concrete. Finite element analysis of CFT columns subjected to an axial compressive force and bending moment in combination was carried out by Teh Hu et al (1) and the

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confining effect was analysed in the sections of square and circle. Their research exposed that the concrete core is a good confining method when the axial force is large. Prabhu et al (2) did experimental studies in his research studies of behaviour of concrete filled steel tubular (CFST) short columns externally reinforced using CFRP strips composite and they found from their square sections that the stiffness and strength of CFRP confined columns increases depending on the number of layers increased.
It was revealed that the section of elliptical steel tubular has recently been adapted to the research area and limited research studies are available in the published literature. Zhu and Young (3) investigated the design of cold-formed steel oval hollow section columns and they demonstrated the failure criteria of the columns by their experiments by comparing the design strengths in the established code of practices. Cold-formed steel oval hollow sections under axial compression was presented by Zhu and Young (4) in terms of reliability analysis of local stability and design strength of the sections with curved and straight elements. There can be possibility to analyse the strengthening method in the development of research studies in terms of delaying local buckling by strengthening, it is investigated with concrete in-filled material components and presented by 3D non-linear FE analysis in this paper.

2. EXPERIMENTAL REVIEWS

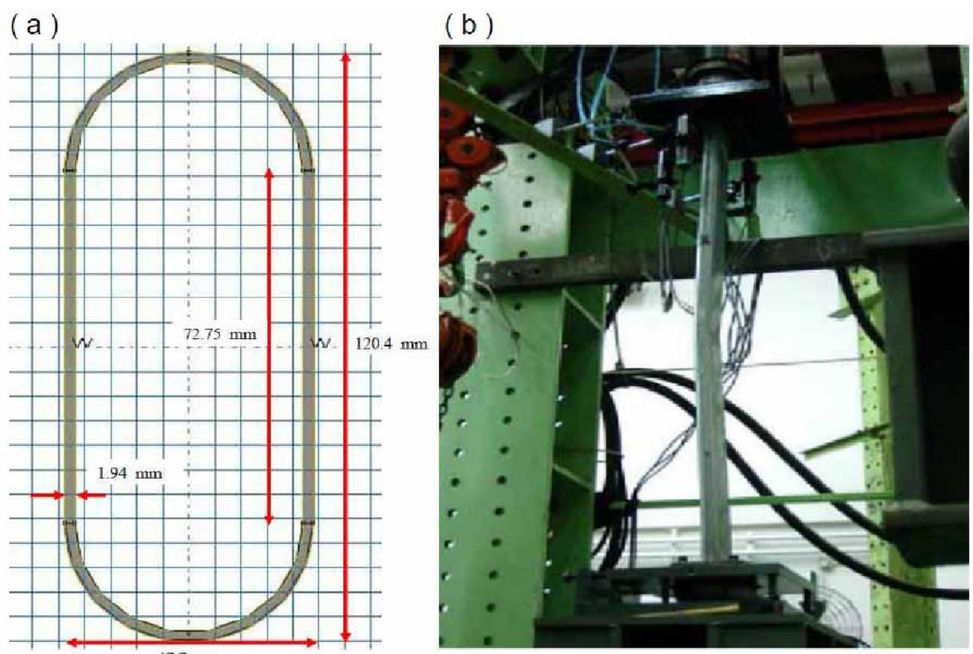

Figure 1 (a) Dimensions of the cross section of the EHST column (b) Fixed ended column test (Source: Zhu, J. and Young, B.)

A series of tests were done in the studies of ultimate strength of cold-formed-steel elliptical hollow sections subjected to axial compression by Zhu and Young (3, 4). The section named as A360, consisted a length of 361.2 mm and a cross section area of 556.9 square millimetres was taken in this study. A set of 25 mm end plates were welded at both ends and the fixed ended criteria was made at both ends by bolts with bearings from restraining as of twisting, rotating and wrapping by both axes. Figure 1(a) shows the dimensions of the section of hollow section. The load was applied at the upper end of the column by hydraulic actuated as displacement control with a speed of 0.1 mm/min. The load application and support of experimental analysis done by Zhu and Young (3, 4) by using servo controlled hydraulic testing machine are shown in the Figure 1(b). The readings in the applied load and shortening of the column were recorded throughout the analysis.

3. FINITE ELEMENT MODEL GENERATION AND VALIDATION

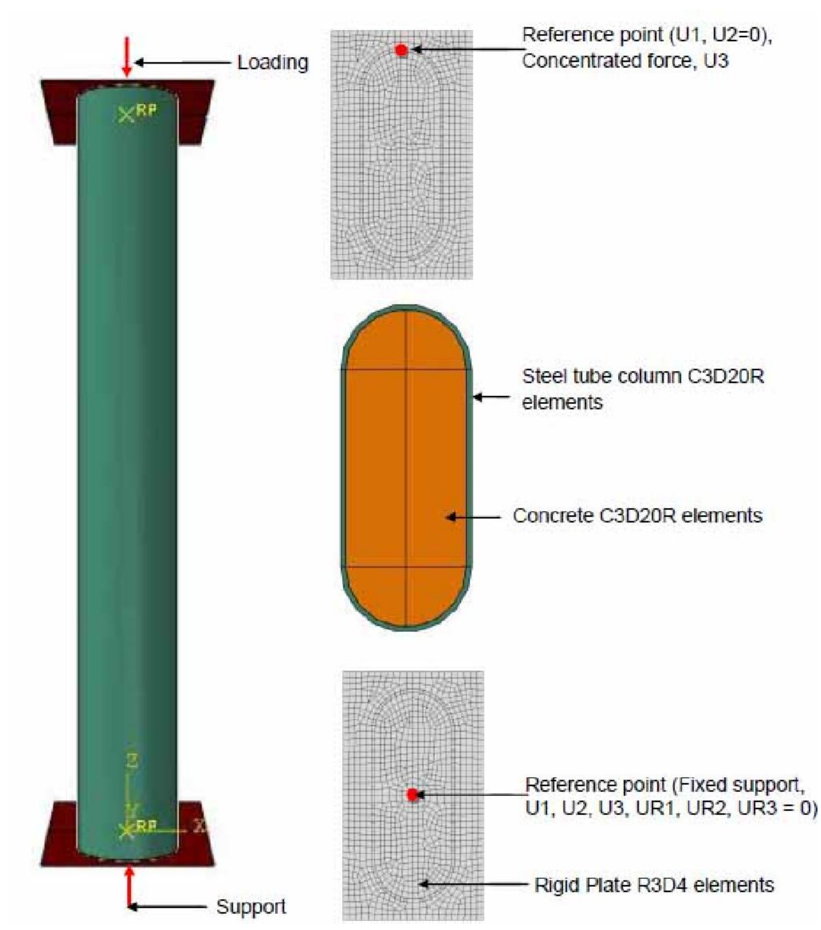

Figure 2. Elements and boundary conditions used in the FE model of the column

A section of cold-formed steel tube elliptical column connected with similar geometries of end plated steel section, which has been experimentally carried out by Zhu and Young

(3, 4) was developed by three dimensional finite element method with a non-linear material component of steel section and rigid end plates. The connections between end plate and steel tube column, and deformation of end plates have not been important in this study and thus, with considering the welding connection, two rigid end plates were fixed by tie constraint at both ends. A detailed description of the developed model for validation and for parametric studies are shown in the Figure 2. In the rigid plate, which is connected in steel tube column at one end, a reference point was selected at the centre of the rigid plate with fixed support conditions as shown in the Figure 2. The concentrated load was applied at the reference point, which is the centre of the rigid plate of the next end, and the point was allowed to move in the vertical direction while the next two directions have been restricted. Initially, ABAQUS implicit solver was used to validate the model and the model was included with various sizes of mesh as the convergence studies and the types of element. Three dimensional solid element C3D20 available in ABAQUS was used to steel tube column and it could be observed in the analysis with various types of element that such element, which contains 20 nodes with three degrees of freedom in each node, is suitable to the steel tube column in the prediction of local buckling effects. In addition, R3D4 element was applied to the rigid plates connected at both ends. The non-linear material behaviour of structural steel was specified in the FE model according to the proposed equation derived by Gattesco (5) with adopting the material test values of yield and ultimate stress states done by Zhu and Young (3, 4). Figure 3 shows the material behaviour of the steel indicated in three states such as elastic state, perfect plastic state and non-linear state, and the stress-strain responses in the non-linear behaviour of steel developed by Equation (1). The flat and curved sections were determined separately according to those ultimate and yielding states. The beginning stage of the linear material behaviour, which is elastic state, was considered with the measured values of young modules in both curved and flat portions promptly from the test done by Zhu and Young (3, 4), and the Poision's ratio was considered as 0.3. The measures test values, which are young modules, static proof stress, ultimate tensile strength and elongation after fracture have been taken from coupon tests with both different geometric sections of curved and flat column portions in the development of those non-linear states individually.

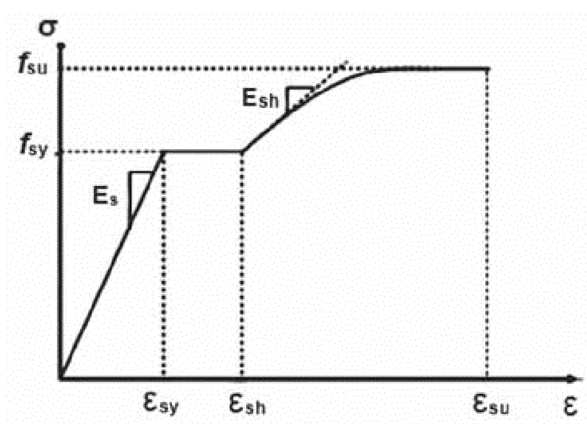

Figure 3. Developed stress strain curve of steel.

$$\sigma_y = f_{sy} + (f_{su} - f_{sy})\left[1 - e^{\left[\frac{(\varepsilon_{sh} - \varepsilon_s)}{k}\right]}\right] \quad (1)$$

where, σ_y and ε_s are stress and strain values, respectively; f_{sy} and f_{su} are yield and ultimate stresses, respectively; and ε_{sh} is the strain at the beginning of hardening stage.

The developed FE model was verified against the experimental analysis undertaken by Zhu and Young (3, 4). The applied load at ultimate limit state was predicted with a value of 185.9 kN and the FE model was shown the ultimate limit state of the applied load as 186.1 kN, which was very close to each other. In addition, ABAQUS explicit solver also was used to predict the reliable behaviour in the concerns of predicting post buckling effects with considering the shortening of the columns at constant deformation speeds similar to the experimental analysis of Zhu and Young (3, 4). In this, a quasi static solution has been determined by the comparison of kinetic and internal energies and kinetic energy was maintained within five percentage of internal energy throughout the analysis. The applied load and corresponding support reaction forces were compared throughout the analysis to confirm the static solution in the explicit solver. The applied load was predicted as 199kN in the FE model, which was a coefficient of deviation of 7%. Meanwhile, it could be observed very close applied load at ultimate limit state when the similar steel plates were used instead of rigid plates. The major concerns of the analysis were in terms of flexural and local buckling effects of EHST columns and thus, it was analysed with rigid plated end supports in an acceptable deviation of the ultimate limit state as 7% with comparative experimental analysis. The material fracture behaviour of the column was observed throughout the analysis and the first yielding point predicted in the FE model is shown in the Figure 4. In terms of the behaviour of the columns and predicted ultimate limit state in the FE model, the FE model was reliable to do the further parametric studies.

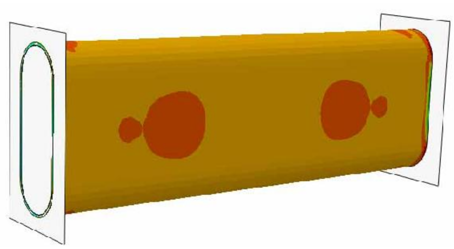

Figure 4. Initial yielding point of column

4. RESULTS AND DISCUSSIONS

The validated FE model was then extended in the prediction of in-filled effcets in terms of posphoning the local buckling with increases in ultimate strength capacity. While maintaining the similar hollow section, rigid plated end support and loading conditions, the concrete in-filled material with variable strengths was applied, and the applied axial load and shortening responses were predicted. The concrete material behaviour in uniaxial compressive state in the non-linear behaviour was developed as indicated in the Equation (2) proposed by Desay et al. (6). Figure 5(a) shows the compressive behaviour of concrete material, which starts from the origin until 40% of ultimate compressive state as elastic state and it further starts with non-linear state from that state of 40% of ultimate compressive state. The uniaxial stress-strain behaviour of concrete in tension was determined with Eurocode (7) as shown in the Figure 5(b). The two states, which are elastic state and non-linear state were determined as indicated in Equations (3) and (4), respectively. The elastic state has ultil the cracking strain from origin and it started from cracking strain as non-linear behaviour with exponential function as it gives most appropriate results after cracking in fracture energy concept.

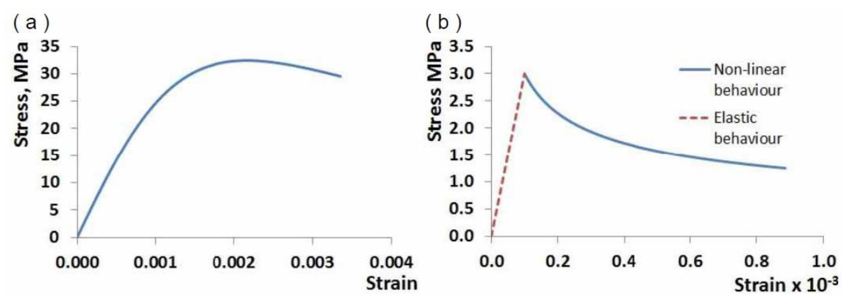

Figure 5. Developed concrete material model (a) Compressive behaviour (b) Tensile behaviour

$$\sigma_c = = \frac{E_c \varepsilon_c}{1 + \left[\frac{\varepsilon_c}{\varepsilon_{c1}}\right]^2} \tag{2}$$

$$\sigma_t = E_c\, \varepsilon_t \text{ if } \varepsilon_t \leq \varepsilon_{cr} \tag{3}$$

$$\sigma_t = f_{cm} \left[\frac{\varepsilon_{cr}}{\varepsilon_t}\right]^{0.4} \text{ if } \varepsilon_t > \varepsilon_{cr} \tag{4}$$

Where in the compressive behaviour, E_c is the longitudinal modulus of elasticity, σ_c is compressive stress, ε_c is strain and ε_{c1} is strain at the peak point. In the tensile behaviour, σ_t is the tensile stress, ε_{cr} is the strain at concrete cracking and ε_t is the tensile strain. The ultimate strengths of concrete compressive strength were 25, 30, 35, 40, 45 and 50 Mpa and the respective tensile behaviours were 3.2, 3.4, 3.6 and 3.8 Mpa. The effects axial load and shortening responses of EHST column with in-filled concrete core are shown in the Figure 6. The ultimate strength of cold-formed EHST column mostly was shown an increses by 175 %. It was observed all over the cases that the inward local buckling of the EHST column was not happening and the column in all cases has buckled outward locally. The analyses with concrete in-filled were shown in all cases that ultimate strengths were stable after reached its ultimate strength and failure of the EHST column was gradually occurred while the column without in-filled concrete material was a sudden failure. This stability issue could be rectified in the EHST column by in-filled material in the slender EHST column with large depth-to-thickness ratio. The developed stress and buckling effects of the EHST column with an in-filled concrete material of 25 MPa strength, concrete cracking behaviour in maximum principal strain and concrete crushing behaviour in minimum principal strain are shown in the Figures 7 (a), 7(b) and 7(c), respectively. Similarly, the Figures 8(a), 8(b) and 8(c) show the stresses developed in the EHST column with concrete in-filled material strength of 50 MPa. It could be observed that there is not much difference in the failure criteria of the concrete material and the ultimate strength of EHST column was possible to an increse due to the ductile behaviour of the concrete material.

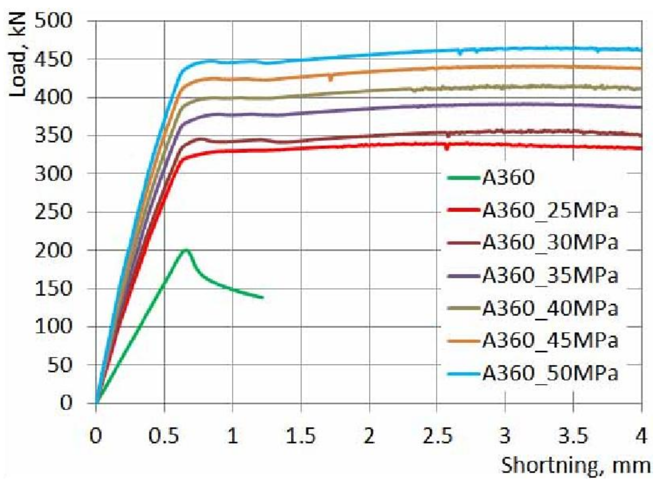

Figure 6. Comparison of applied load and shortening responses with various concrete strengths.

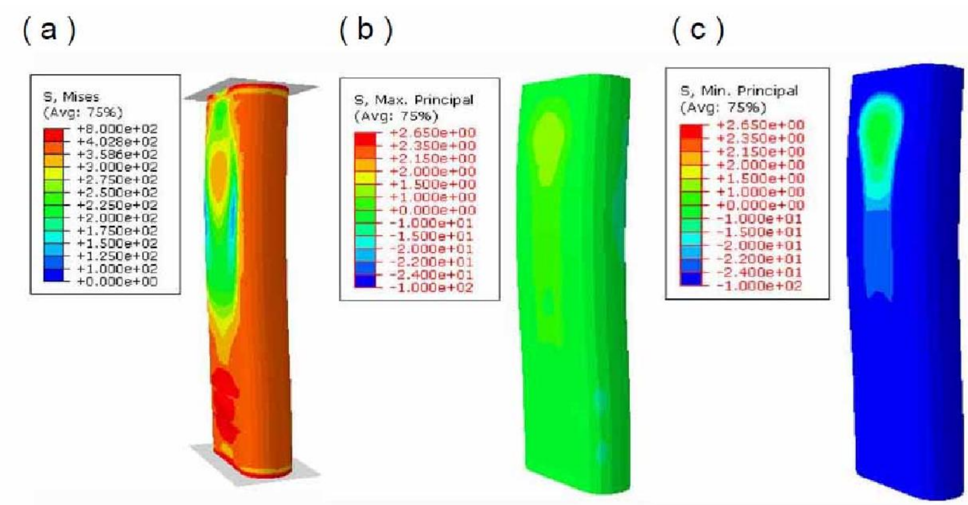

Figure 7. EHST column at concrete in-filled strength of 25 MPa (a) Mises stress contour of steel column (b) Maximum principal stress of concrete core (c) Minimum principal stress of concrete core

5. CONCLUSIONS AND SUGGESTION FOR FUTURE RESEARCH

In the construction, the initial stress and deformation may occur due to the preloads on the steel tube section and it may reduce the strength and stiffness of EHST column. In addition, the EHST column is a more adequate section in the corner area due to its section properties and it may induce early local buckling due to the combined axial loads and bending moment by axes owing to its shapes. In this regards, the effects of cold-formed EHST column with including concrete in-filled material with variable strengths were analysed and it could be observed that the ultimate strength of EHST column increases by concrete core with avoiding early inward local buckling. The stiffness of EHST column also significantly increase due to the combined capacity of the steel and concrete material components. This research study can be extended with analysing the section behaviour with the effects of column slenderness ratio and column depth-to-thickness ratio.

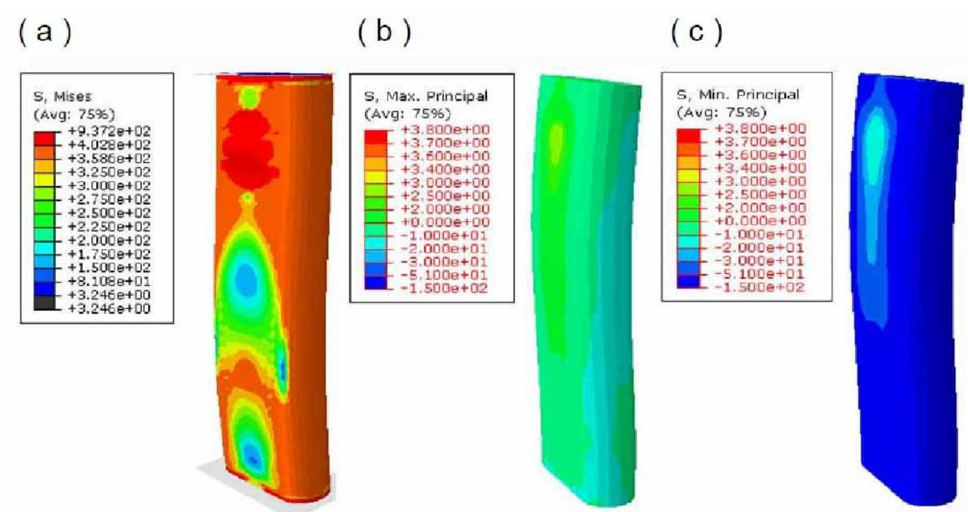

Figure 8. EHST column at concrete in-filled strength of 50 MPa (a) Mises stress contour of steel column (b) Maximum principal stress of concrete core (c) Minimum principal stress of concrete core

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ЕФИКАСНОСТ ОЈАЧАЊА EHST СТУБОВА ИСПУНОМ ОД БЕТОНА

Резиме: *Проблематика избочавања полукружних фланши и равних ребара као последица геометрије и различите савојне крутости око две главне осе савијања хладно обликованих челичних стубова са елиптичним шупљим попречним пресеком (EHST стубови) је постала нарочито значајна и захтева метод ојачања у циљу одлагања појаве лома. 3Д модел EHST стубова сачињен помоћу методе коначних елемената са крутим плочама на крајевима је оптерећен центричним оптерећењем на једном крају и непокретним ослонцем на другом крају. Утврђено је добро слагање у смислу аксијалног оптерећења и деформације скраћења добијених помоћу МКЕ и експерименталних резултата из доступне литературе. Потврђени МКЕ модел је након тога анализиран са испуном од бетона различитих чврстоћа у смислу повећања носивости и крутости стуба. Варијације носивости анализираних стубава помоћу МКЕ су биле значајне, те се овај метод показао као добар за спречавање локалног избочавања према унутрашњости пресека. Такође, гранична центрична носивост ојачаних хладно обликованих EHST стубова је описана помоћу резултанти напона услед спољашњег аксијалног, центричног, оптерећења.*

Кључне речи: *Челични стубови са шупљим елиптичним пресеком, метод коначних елемената, гранична носивост на центрични притисак, испуна од бетона, параметарска анализа, критеријум лома*