Engineering Structures 177 (2018) 473–488

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Structural performance of cold-formed high strength steel tubular columns

Han Fang[a], Tak-Ming Chan[a,*], Ben Young[b]

[a] Department of Civil and Environmental Engineering, The Hong Kong Polytechnic University, Hung Hom, Hong Kong, China
[b] Department of Civil Engineering, The University of Hong Kong, Pokfulam Road, Hong Kong, China

A R T I C L E I N F O

Keywords:
Cold-formed
Column strength
Design
Finite element modelling
High strength steel
Tubular columns

A B S T R A C T

This paper presents a numerical investigation into the structural performance of cold-formed high strength steel tubular columns with square, rectangular and circular cross-sections. A finite element model was developed and validated against experimental results on the cold-formed high strength steel tubular columns. Parametric studies using the validated finite element model were carried out to determine the strengths of cold-formed tubular columns with various cross-sectional dimensions, member slenderness values, geometric imperfections and steel grades of S700, S900 and S1100. It was found that increasing the steel grade of the columns led to higher normalised column strengths which were less severely affected by geometric imperfections. The effect of material tensile strength to yield strength ratio on the column strengths was found to be insignificant. Based on the experimental results in literature and the results obtained from parametric studies, the applicability of the design rules in European, Australian and American Standards to cold-formed high strength steel tubular columns was evaluated. The reliability of the design rules was also assessed by performing reliability analysis. The design rules in these standards provide conservative predictions for the strengths of cold-formed high strength steel tubular columns. Recommendations on the column buckling curve selection are discussed. An improved column buckling curve expression considering the increment in normalised column strength with increasing steel grade is also proposed.

1. Introduction

High strength steel (HSS) with yield strength (0.2% proof stress) above 460 MPa and up to 1100 MPa has been produced and increasingly applied to civil structures used as building columns [1] and in bridge structures [2]. HSS structural members demonstrate higher strength-to-weight ratios than the structures formed by conventional-strength steel. Thus, the application of HSS structural members reduces the weight of steel consumption in construction and consequently brings cost and resources savings. Current design codes including European [3–4], American [5] and Australian standards [6,7] allow the design of structures with steel grades up to S700 with nominal 0.2% proof stress of 700 MPa, but no specifications for structural members with steel grades higher than S700 are provided in these design codes. Besides, the design rules applied to HSS structures based on European and American standards are the same as those for the structures with steel grades up to S460 and have been developed based on the data for conventional-strength steel members [8,9]. However, the behaviour of HSS members is different from that of conventional strength steel members since the material properties and the ratio of residual stress over yield strength in HSS members are different from those of conventional strength steel members [10–14]. Therefore, applying the same design rules to HSS members with the members made of conventional strength steel may lead to inaccurate structural designs [15,16].

The current study aims to contribute to developing accurate design rules for cold-formed HSS tubular columns with high local buckling resistance, high torsion resistance and ease of fabrication. Research studies have been conducted to investigate the flexural buckling behaviour of cold-formed HSS tubular columns. Somodi and Kövesdi [17] conducted experimental and numerical investigations on the flexural buckling behaviour of cold-formed HSS square tubular columns with steel grades between S420 and S960. The normalised strengths of cold-formed HSS columns were found to be higher than those of the conventional-strength steel columns since the ratios of the average residual stress over yield strength in HSS members are lower [10]. It has also been found that the design specification in European standard [3] provides conservative strength predictions for the cold-formed HSS square tubular columns. Dundu and Chabalala [18] investigated the strength of cold-formed circular tubular columns made of steel with 0.2% proof stress ranging from 484.7 to 503.7 MPa. The column strengths were found to be higher than the strengths estimated based on

* Corresponding author.
E-mail address: tak-ming.chan@polyu.edu.hk (T.-M. Chan).

https://doi.org/10.1016/j.engstruct.2018.09.082
Received 21 June 2018; Received in revised form 19 September 2018; Accepted 28 September 2018

the design specification in European code. Ma [19] experimentally investigated the structural behaviour of four cold-formed HSS tubular columns with square, rectangular and circular cross-sections made of S700 or S900 steel while Javidan et al. [20] tested two cold-formed S700 and S1100 steel columns with circular cross-section. Each of these studies focused on an individual cross-section shape or limited steel grades. The results in these studies also reveal that quite limited data were obtained for the cold-formed tubular columns with the steel grade of S1100. For each cross-section shape, no systematic studies have been carried out to investigate the behaviour of the columns with various steel grades up to S1100. The experimental and numerical data in these studies are insufficient for developing accurate design rules for the cold-formed HSS columns with various cross-section shapes and with steel grades up to S1100.

Therefore, in the current study, a numerical investigation into the structural performance of cold-formed S700, S900 and S1100 tubular columns with square hollow sections (SHS), rectangular hollow sections (RHS) and circular hollow sections (CHS) was conducted. A finite element (FE) model was developed and validated against test results. Subsequent parametric studies on the cold-formed HSS tubular columns with various cross-sectional dimensions and member slenderness values were conducted using the validated FE model. Based on the experimental results in literature and the parametric studies results, the applicability and reliability of the design rules in European, Australian and American standards, on the cold-formed HSS tubular columns was evaluated. Recommendations on column buckling curve selection for design rules in standards and a new column buckling curve expression were proposed.

2. Finite element modelling and validation

Finite element (FE) modelling using the software package ABAQUS 6.12 [21] was carried out to investigate the structural performance of cold-formed HSS columns with square, rectangular and circular hollow sections, and to obtain column strength data for the development of accurate design rules. A finite element model was first developed and validated using the test results of cold-formed HSS tubular columns [17–20]. The labels and dimensions of the columns investigated in these studies are presented in Table 1 for SHS and RHS columns and in Table 2 for CHS columns using the nomenclature defined in Fig. 1. The yield strength (f_y) and ultimate strength (f_u) of the column materials and the test results of ultimate loads ($N_{u,test}$) of the columns are also provided in the tables. In the following Sections 2.1 and 2.2, the FE model and its validation are presented.

2.1. Description of the FE model

The cold-formed HSS tubular columns were modelled using the four-noded S4R shell element with reduced integration. This type of element has been successfully used in the accurate FE modelling of steel tubular columns [22–24]. The mesh size of (B + H)/30 was adopted for the SHS and RHS columns while the mesh size for CHS columns was D/15. Measured material stress-strain curves for the columns were reported in the experimental studies [17,19–20]. For the CHS columns investigated by Dundu and Chabalala [18], material properties were presented while no measured stress-strain curves were provided. Thus, the stress–strain model proposed by Ma et al. [25] for cold-formed high strength steel CHS members was adopted and stress-strain curves were subsequently obtained by substituting the material properties measured by Dundu and Chabalala [18] in the model. These measured and calculated stress-strain curves were converted into the true stress-log plastic strain curves using Eqs. (1) and (2). The obtained true stress-log plastic strain curves were subsequently incorporated into the FE model.

$$\sigma_{true} = \sigma(1+\varepsilon) \tag{1}$$

$$\varepsilon_{ln}^{pl} = \ln(1+\varepsilon) - \frac{\sigma}{E} \tag{2}$$

In Eqs. (1) and (2), ε_{ln}^{pl} is the logarithmic plastic strain, ε is the engineering strain, E is the Young's modulus, and σ_{true} and σ are the true stress and engineering stress respectively. For cold-formed SHS and RHS columns, material strength enhancements at the corner regions were obtained due to the cold-working effect caused by the cold-forming fabrication process, as shown in Table 1. It has also been found that the strength enhancements also occur at the flat portions adjacent to the corners [26–29]. Extending the corner material properties to the flat portions adjacent to corners with the width of two times the cross-section thickness on each side of a corner region can be applied to account for the strength enhancements at these locations [26–29]. This finding was adopted in this study for modelling cold-formed SHS and RHS columns.

Initial local and global geometric imperfections exist in the column structures and can affect the buckling behaviour and strengths of the columns. Thus, both local and global geometric imperfections were included in the FE model. The lowest local and global eigenmode shapes obtained by conducting separate linear eigenvalue buckling analysis were used as the local and global geometric imperfection patterns respectively. The scaled local and global eigenmode shapes with the magnitudes of the local and global imperfections respectively were assigned to the column FE model. For the SHS columns from the study of Somodi and Kövesdi [17], local and global geometric imperfection magnitudes of 0.7*B/1000 and 1/1000 of column length respectively were provided in the study and adopted for the modelling. For the H 80*80*4-LBC, H 50*100*4-LBC and H 100*50*4-LBC columns in Table 1, the local geometric imperfection magnitudes provided as 0.0119*(B-2R)*t [19] were used while the measured global geometric imperfection magnitudes of 1/3885, 1/58268 and 1/6474 of column length were adopted [19]. For the V89*3-LBC CHS column, local and global geometric imperfection magnitudes of 0.0058D/t and 1/9711 of column length respectively [19] were used for the FE modelling. However, Dundu and Chabalala [18] only reported the geometric imperfection measured at mid-height of the columns while Javidan et al. [20] only reported the global geometric imperfection magnitudes of about 1/1000 column length for S-HST2 and S-UHST2. For the 165.1*3.0L1500, 165.1*3.0L2000, 220.0*3.5L2000 and

Table 1
Labels, dimensions and test data of the SHS and RHS columns.

Specimen	B (mm)	H (mm)	t (mm)	R (mm)	l_e (mm)	$N_{u,test}$ (kN)	Flat portion		Corner	
							f_y (MPa)	f_u (MPa)	f_y (MPa)	f_u (MPa)
CF7-R150*8_1B [17]	151.3	151.3	7.88	20	2140	2852.0	751	834	815	951
CF9-R120*6_3B [17]	120.3	120.0	6.05	14	2940	1554.0	1088	1182	1109	1271
CF9-R150*7_2B [17]	151.3	151.2	6.85	16	2940	2876.0	1114	1199	1215	1399
CF9-R150*7_1A [17]	152.4	151.6	6.90	16	1840	4237.0	1114	1199	1215	1399
H 80*80*4-LBC [19]	80.3	80.1	3.94	9.5	1655	581.9	719	840	897	983
H 50*100*4-LBC [19]	50.3	100.2	3.98	8.5	1655	642.3	719	840	897	983
H 100*50*4-LBC [19]	100.2	50.6	3.97	8.5	1655	306.9	719	840	897	983