RELIABILITY ANALYSIS OF CONCRETE FILLED STEEL TUBE BEAMS OF QUADRILATERAL CROSS SECTIONS

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ABSTRACT

Concrete filled steel tube beams is a structural composition that brings the advantages of the combination of the surrounding steel with the core concrete, this structural member had been investigated under different conditions but still, there is a significant difference among the well-known design codes provisions and limitations. Assessing the limit states of the codes can be conducted based on structural reliability analysis. Bending capacity of the beams can be characterized by the reliability index in terms that can be readily understood by structural engineers with only a basic knowledge of probability theory. In this study, First-order Reliability Method (FORM) was used in assessing the selected tested specimens. More than 250 concrete filled steel tube beams of square and rectangular cross-section had been collected from recently introduced literature (from 2000 up to 2018); moreover, the selected specimens were compared with the provisions of many codes. The judgment was based on the convergences between the calculated beams load capacity in term of bending strength according to mentioned codes and the recorded load capacity that obtained from the experimental work. Different parameters were evaluated from the selected beams, while the geometry of the beams were the main parameters, the tube thickness is ranging between 1 to 12 mm, the width of the beams were ranging between 20 to 450 mm, this lead to a different relative (b/t and L/h) up to 105 and 50 respectively. The material properties are also altered in order to evaluate its effect on the beam strength, where the concrete compressive strength was ranging between 17 to 154 MPa, and the steel yield strength was less than 560 MPa. The results show that both codes (Eurocode 2004 (EC4-2004) and American Institute of Steel Construction 2010 (AISC360-16)) have percentages of error within the unsafe zone, and EC4-2004 is less conservative than the AISC360-16, moreover, The reliability in percentages were 55.7 and 54% for the predicted moment resistance based on the EC4-2004 and AISC360-10, respectively, while the C.O.Vs the specimens predicted according to EC4-2004 was 0.37 compared to 0.31 according to AISC.

Keywords: CFST beams, Bending strength, Design codes, FORM, Reliability

INTRODUCTION

Steel-concrete composite beams have been widely used in different structures, because of its considerable advantages such as high load holding capacity, ductility, rigidity, and stability due to homogeneous materials using in construction. Concrete Filled Steel Tube (CFST) is one of the most used types of composite member, which has the advantages of ductility and high strength of the steel plate surrounding the high loading capacity of concrete [1], The strength of concrete solves the rigidity of CFST tube, where, there are statistics suggesting that the increase in concrete strength may increase the strength of the composite beams [2]. The use of these types of CFST results in significant reduction of the beam size when compared to regular reinforced concrete beams needed to carry the same load. Therefore, considerable economic savings can be obtained. Also, the beam size reduction is Advantageous where floor space is at a premium, such as in car parkings and office blocks. In addition, closely spaced composite columns connected to the spun-Outside a skyscraper for lateral load resistance by a flat concept [3]. There are five types of steel-concrete a) Encased Steel b) CFST c) combination of CES d) hollow CFST section and, e) double skin section.

Any structures shall be designed and constructed with an appropriate degree of reliability that holds the expected load, serviceable, and durable. The level of the reliability is based on the consequences of damages of the structure, EN 1990-2002 adopted three classes of reliability (RC) based on the reliability index (β) with a minimum value ranging between 3.3 for RC1 and 4.3 for RC3 all for 50 year reference period. FORM is a common method used to assess the structural analysis, which is in turn related to the reliability of the safety factor adopted by many standards. In the reliability analysis, the adopted limitations produce stable results if the estimated parameters modified in various situations. Certain quality inspections are determined by acquiring an accurate variety range at the scale that must be possible by determining the relationship between the scores obtained from the different organizations of the scale. Thus, if the relationship in the reliability survey is high, the scale gives predictable results and thus is safe [2].

In this research, 158 experimental results of CFST rectangular beams were evaluated. The effect of the cross-sectional area, materials strength and the relative size effects on the beams moment strength were introduced. Moreover, based on the comparison between the calculated moment capacity according to the EC4-2004 and AISC-360-16 provisions, and the actual test result, many points were concluded.

Elected Experimental Data

The data are collected from 14 sources found in the literature, which are included 139 CFST rectangular beams were elected with different strength of concrete (ranging between 17 and 154 MPa), steel yield strength (ranging between 86 and 560 MPa), and relative size as listed in Table1-3 show the reliability percentage for all samples and reliability percentage for each article.

The elected data that imported from the literature review were tested under laboratory condition by References [4-15],

Table 1 specimens relative size range

Relative size	B/H	B/t	H/t	L/B	L/H
Max.	2.00	105.26	105.26	50.00	50.00
Min	0.26	12.50	12.50	5.00	5.00

Table 2 The reliability percentage for all specimen

	M_{test}	σ_{test}	M_{code}	σ_{code}	β	pf	R%
EC4	86.01	194.0067	54.91	93.39192	-0.14441	0.4443	55.57
AISC	86.01	194.0067	61.64	122.7774	-0.10611	0.4602	53.98

Table 3 The reliability percentage for each article

Source	NO.of sample	β_{EC4}	Pf	R%	β_{AISC}	Pf	R%
M.F. Hassanein 2017; [4]	6	0.99	0.83	16.90	-1.063	0.144	85.54
M.R. Bambach 2008;[5]	3	-0.14	0.44	55.57	-0.216	0.420	58.00
Yulin Zhan et at all 2016;[7]	8	-3.12	0.00	99.91	-2.013	0.023	97.72
Ran Feng a at at all 2017;[8]	20	-0.07	0.47	52.79	-0.348	0.366	63.31
lin han 2004; [9]	6	2.20	0.97	0.03	2.351	0.980	2.00
A. SILVAa,et at all 2016;[10]	2	-2.26	0.01	98.90	0.462	0.677	32.28
Wei-Hua Wang 2014;[11]	4	-0.84	0.20	79.95	-0.884	0.189	81.06
F.W. Lua 2007;[12]	3	-3.87	0.00	99.99	-3.433	0.001	99.97
Cheng Fang 2018;[13]	10	0.30	0.62	38.30	0.468	0.677	32.30
L.-H. Han et al 2006;[14]	18	-0.24	0.41	59.48	-0.588	0.281	71.90
L.-H. Han et al 2004;[9]	2	-1.13	0.13	87.10	-3.236	0.001	99.93
Arivalagan 2008;[15]	15	-2.26	0.01	98.81	-1.354	0.088	91.15
Lin han 2004 ;[16]	10	-0.56	0.29	71.23	-0.294	0.385	61.41
Zhijian Yang 2017;[17]	8	-0.36	0.36	64.06	-1.11243	0.1357	86.43
A. SILVAa 2016;[10]	8	-0.35	0.37	63.40	-0.35369	0.3632	63.68
Wei-Hua Wang 2014;[11]	2	-3.71	0.00	99.99	-3.85022	0.0001	99.99
M.F. Hassanein 2017;[4]	2	1.02	0.84	16.00	-1.06328	0.144	85.6

Methodology

The reliability index (β) used in this research is calculated based on the following equation:

$$\beta = \frac{M_r - M_s}{\sqrt{\sigma_r^2 + \sigma_s^2}} \qquad \text{Eq. 1}$$

where, M is the mean of bending moment strength, and σ is the standard deviation, the subscript r and s refer to the moment obtained from the adopted codes calculation and actual moment resistance that recorded from the experimental work, respectively.

The reliability percentage (R%) can be calculated based on the corresponding probability of failure P_f that can be introduced in a cumulative distribution function of the standard normal distribution function in term of the β ($Pf = \varphi[-\beta]$)) as follows;

$$R\% = (1 - P_f) \times 100 \qquad \text{Eq. 2}$$

Where Φ = Cumulative distribution function of the standard ordinary variant taken from z-table. The reliability percentage.

Two codes methods are used to predict the strength of the CFST rectangular beam. The first is the plastic stress distribution method that adopted by AISC360-16 and the second is based on the EC4-2004 provisions. In order to consider all the constraints cases found in the literature, and the relative size (B/t and H/t) that govern the slenderness limit, three cases can be found in rectangular beams that is compact with limit λ_P, non-compact with limit λ_r, and slender flanges same cases are available for web but without slender web, these cases determine the behavior of the steel in both tension and compression based on the compactness and non-compactness limits λ_P and λ_r, respectively, where the CFST beam is governed by the steel yielding and the local buckling. The nominal flexural strength M_n can be calculated as plastic moment M_p as follows.

$$M_n = M_p = F_y b t_f\left(a_p - \frac{t_f}{2}\right) + F_y b_i t_f\left(H - a_p - \frac{t_f}{2}\right) + F_y a_p 2t_w\left(\frac{a_p}{2}\right) + F_y\left(H - a_p\right)2t_w\left(\frac{H - a_p}{2}\right) + 0.85f'_c\left(a_p - t_f\right)b\left(\frac{a_p - t_f}{2}\right) \text{ for } \lambda \le \lambda_p \qquad \text{Eq. 3}$$

$$M_n = M_y = F_y b t_f\left(a_y - \frac{t_f}{2}\right) + F_y b t_f\left(H - a_y - \frac{t_f}{2}\right) + 0.5\,F_y a_y 2t_w\left(\frac{2a_y}{3}\right) + F_y\left(H - a_y\right)2t_w\left(\frac{d}{2}\right) + 0.35f'_c\left(a_p - t_f\right)b\left(\frac{2\left(a_p - t_f\right)}{3}\right) \text{ for } \lambda = \lambda_r \qquad Eq. 4$$

$$M_n = M_p - \frac{\left(M_p - M_y\right)\left(\lambda - \lambda_p\right)}{\lambda_r - \lambda_p} \text{ for } \lambda_p < \lambda \le \lambda_r \qquad Eq. 5$$

where, a is the neutral axis, H, b, t are the tube depth, width, and thickness, the subscript w and f refer to the web and the flange.[18]

As mentioned previously, the second analysis method is that what adopted by EN1994-1-1-2004, the code suggested a design procedure based on the slenderness and three different buckling curves that introduced in EC3-1-1, each curve is dependent on the reinforcement ratio, in order to design the hollow section filled with plain concrete exposed to the plastic moment resistance (M_p) with zero axial loads for the case of simply supported. M_p can be calculated as follows;

$$M_p = f_y(W_a - W_{an}) + 0.5f_c(W_c - W_{cn}) + f_s(W_s - W_{sn}) \qquad \text{Eq. 6}$$

where, f_y, f_s, are the yield strength of the steel and reinforcement, respectively. f_c is the concrete compressive strength, W_a , W_c , and W_s is the plastic section moduli for steel section, concrete, and the reinforcement, and W_{an} , W_{cn} , and W_{sn} the plastic section moduli of the section distance 2h from the neutral axis.[19]

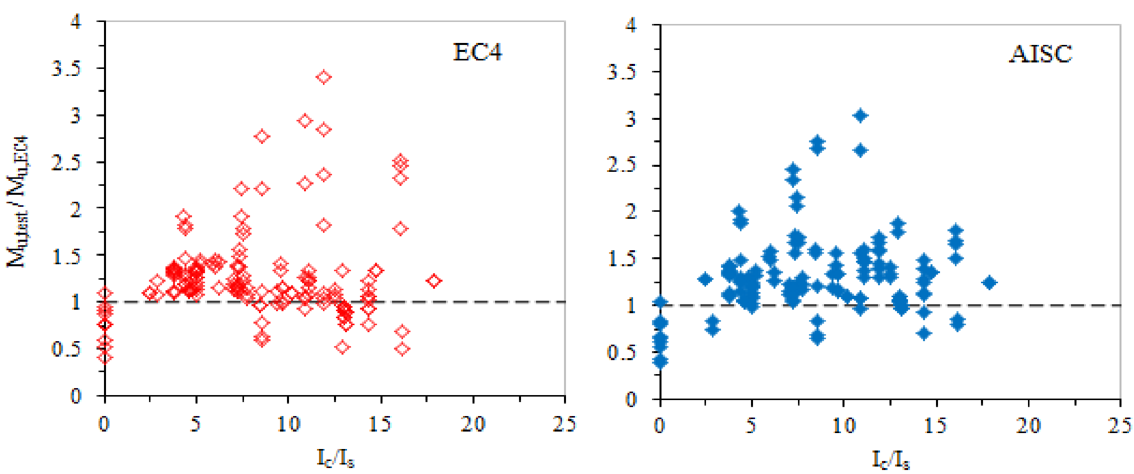
Evaluation of the Codes' Equations

As a first step of the evaluation, the of the effectiveness of the codes prediction of the ultimate moment resistance is by evaluating the effectiveness of the main parameters such as the relative size of the composite materials (concrete and steel). Figure 1 shows the effect of variation of the moment of inertia of the concrete core with respect to the moment of inertia of the surrounding steel on the ultimate moment. The comparisons between the two codes (EC4-2004 and AISC360-16) shows that the EC4-2004 is less conservative than the AISC360-16, where the 23% of the elected specimens was overestimated by the code, while 14% of the specimens were overestimated based on the AISC360-16. Although the mean of the normalized moments ($M_{n,test}$ / $M_{n,EC4}$) is 25% more than the unit and the $M_{n,test}$ / $M_{n,AISC}$ is 31% more than the united, but the coefficient of variance (C.O.V) of $M_{n,test}$ / $M_{n,EC4}$ was 0.37 compared to the less C.O.V of the proportion $M_{n,test}$ / $M_{n,AISC}$ (0.31). This is may be due to the sensitivity of the C.O.V to the small change in the mean.

Figure 2 shows the effect of the differences in the materials' strength on the normalized moments. In spite of the confinement of the concrete is increasing the compressive strength, but the low differences between the steel and concrete strength show a significant influence on the prediction of the moment resistance for both codes, AISC360-16 shows significant scattering for low f_y/f'_c compared to EC4-2004.

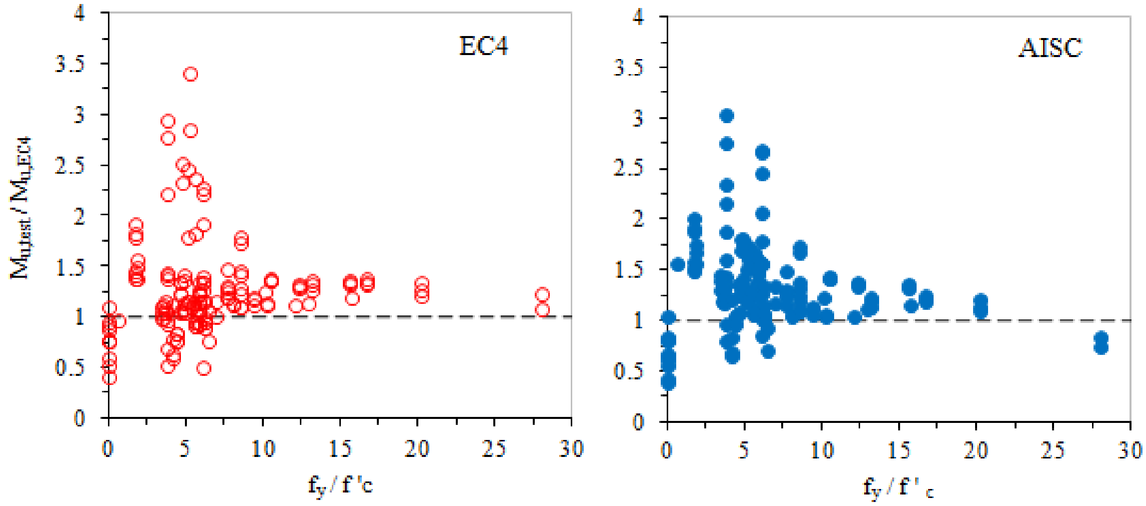
Variation of the relative size does not have significant effect of the prediction of the moment resistance according to both codes, which is agreed with Zhichao, 2014 [6], except for L/H, where the increase in L/H are decreases the error between the predicted moments, this is clearly can be seen in AISC360-10. Figure 3 illustrated the effect of the relative size (B/H, B/t, L/B, and L/H) on the normalized moment resistance.

Comparison between the assessment based on the C.O.V and the FORM shows that the linear approximation errors may be magnified by the FORM because it is underestimated the confidence percentages in term of probability of failure, due to considering the contribution of the zone between the design and experimental [20], where the reliability in percentage was 55.7 and 54% for the predicted moment resistance based on the EC4-2004 and AISC360-16, respectively, i.e. less than 3% variation. While the differences in C.O.V was 19% for differences between the two codes as mentioned above.



(a) (b)

Figure 1 effect of the composite size on the normalized ultimate moment according to a) EC4-2004 b) AISC360-16



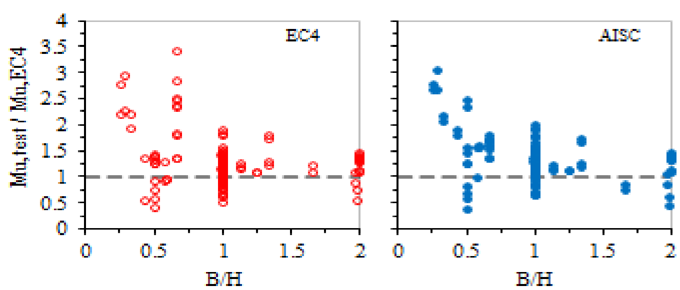
(a) (b)

Figure 2 effect of the materials strength percentages on the normalized ultimate moment according to a) EC4-2004 b) AISC360-16

1. Conclusion

Data collected from the literature are used in assessing the reliability of the moment resistance of the CFST rectangular beam according to EC4-2004 and AISC360-16, moreover, the effect of the cross-sectional area (in term of the moment of inertia), the materials' strength, and the relative size were compared. From the aforementioned calculation and comparison following points can be concluded; 52

- FORM has underestimated the differences between the predicted methods between the two codes, may be the second order reliability method is more convenient.
- The reliability in percentages were 55.7 and 54% for the predicted moment resistance based on the EC4-2004 and AISC360-16, respectively, while the C.O.Vs the specimens predicted according to EC4-2004 was 0.37 compared to 0.31 according to AISC360-16.
- Comparisons between the two codes provision showed that both codes have percentages of error within the unsafe zone, and EC4-2004 is less conservative than the AISC360-16.
- Increasing the strength of the steel compared to the concrete, are decreases the differences between the predicted in designed specimens for both codes.



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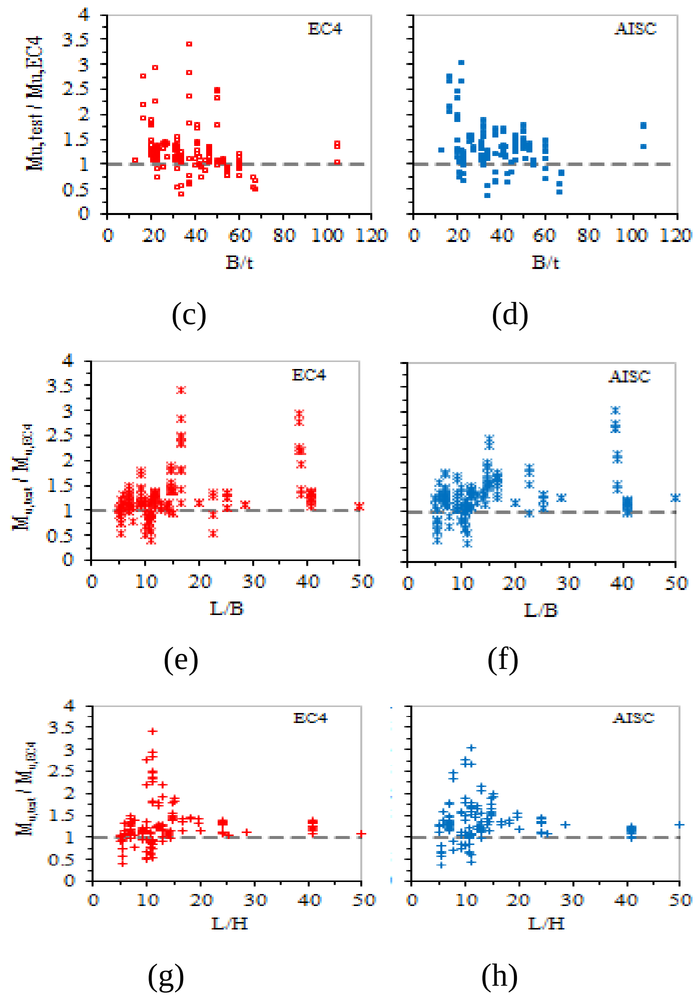

Figure 3 effect of the relative size on predicted moment resistance according to EC4-2004 and AISC360-16

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