AN EXPERIMENTAL STUDY ON THE STRENGTH OF CFT COLUMNS REINFORCED WITH CASTELLATED CRUCIFORM STEEL SECTIONS

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ABSTRACT

The steel tubular column filled with steel reinforced concrete (SRCFT), is formed by inserting a steel section into a concrete filled steel tube. In the current paper, a new shape, namely Castellated Cruciform Steel Section (CCSS), for reinforcing of CFT columns has been proposed to improve the compressive strength and hysteresis behavior of these columns under moderate and severe earthquake excitations. A comprehensive study has been conducted to investigate the strength of SRCFT columns reinforced with castellated and traditional cruciform steel sections, made with thin-walled, welded I-section. The paper describes and presents the results of the testing of four small size (155 mm*155 mm) and short column specimens. The experimental results indicate that the new steel section causes high strength and better post yield behavior of SRCFT columns, because of the increase of shear and bending strength, torsion resistance and interaction between the hollow steel section and concrete. In addition, the axial compressive capacities of those steel sections are investigated in a numerical way in the current study. The obtained results of nonlinear analyses of these columns revealed that strength and buckling behavior of castellated cruciform steel columns far outweighs and is more appropriate than that of the traditional cruciform steel columns.

1. INTRODUCTION

Composite steel concrete columns have widely been used over recent decades. Major benefits that could be attributed to composite columns include high load-bearing capacity, their inherent ductility, and toughness [1-3]. A number of reinforcement methods that have

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been proposed to improve the resistance of CFT columns against fire and increase their strength and load-bearing capacity include CFT columns reinforced with stiffeners, reinforcing CFTs with fiber-reinforced concrete, and reinforcing CFTs with binding bars. Another method to reinforce CFT columns is to create a new cross-section, which is a combination of both CFT and SRC cross sections (i.e. embedding a steel section into the CFT column). In this manner, the shortcomings of both types could be eliminating. In addition, it could bring together their corresponding advantages. Such columns are called Steel Reinforced Concrete Filled steel Tubular (SRCFT) columns. Zhuet et al. [4] conducted an experimental research study on square steel tubular columns that are filled with steel-reinforced self-consolidating high-strength concrete under the condition of axial loads. Their findings indicated that the encased steel section could function as a restraining factor against the generation of diagonal shear cracks in the core concrete. Therefore, the failure mode and post-yield behavior of short composite columns might be changed. Chen and Lin [5] analytically investigated the axial compressive capacity and the force–deformation behavior of concrete-encased steel stub columns. An analytical model for the prediction of the force–deformation response in composite stub columns with various structural steel sections and volumetric lateral reinforcement was developed. Analytical results showed that the structural steel sections could trigger a confinement effect on the concrete and enhance the axial capacity and post-peak strength. Wanget et al. [6] investigated the strength and ductility in composite columns under axial compressive loads. Their findings showed that composite columns have a very high rate of strength, ductility, and capacity to absorb energy. That is because of the interaction between the steel tube, steel section, and concrete. Chicoineet et al. [7] conducted an experimental study in order to investigate the behavior and strength of partially encased composite columns having built-up shapes and stiffened by the application of transverse links. They found that failure of all specimens happens because of the local buckling of flange plates along with concrete crushing. The findings also indicated that closer link spacing and the use of additional reinforcements could result in the improvement of post-ultimate load behavior. Chenet et al. [8] conducted an experimental research study in order to investigate the structural behavior of concrete-encased composite beam columns having T-shaped steel sections under cyclic loading. Their findings showed that cyclic behavior and modes of failures in the beam columns are affected to a large extent by the direction of bending moment. This could be attributed to the asymmetrical cross section. In addition, the concrete-encased composite beam columns could trigger in a stable hysteretic response and a large energy-absorption capacity through the provision of cross ties and decreased spacing between transverse ties.

Therefore, the composite column that is formed by the insertion of steel sections into concrete-filled steel tubes should have the advantages of both concrete-encased steel and CFT columns. The cross section of composite columns is shown in Figure 1. Hereafter, composite columns will be regarded as steel tubular columns filled with steel-reinforced concrete.
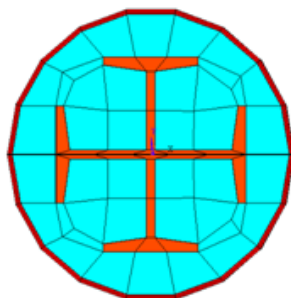
Figure 1. CFT column reinforced with traditional cruciform steel section (SRCFT)

2. RESEARCH SIGNIFICANCE

Investigation over these sections combined with concrete in the form of Steel Reinforced Concrete Filled Tubular (SRCFT) columns indicates evidences of higher applicability and more appropriate results through disappearing the local buckling effects, increasing the shear strength torsion resistance and bending strength, and decreasing the creep phenomenon in concrete (that is because of higher concrete confinement within the flange holes located in castellated columns and a larger contact surface between concrete and steel).

Based on the above study, the current study presents an experimental study on the strength of concrete-filled steel tubular columns reinforced with castellated cruciform steel section and traditional cruciform steel section. In order to achieve this objective, four column specimens were examined to investigate the centrally loaded behavior of this new type of SRCFT column under axial loading. Furthermore, the previous review demonstrates a lack of information on the buckling and compressive strength of built up castellated cruciform steel columns when buckling occurs about X or Y major axes. To the best of the researchers' knowledge, no curves for the design of castellated cruciform steel sections existed in the literature. Therefore, a section of the study has been devoted to the introduction of this feature and by the application of ANSYS software, some nonlinear analyses were conducted to make a comparison between the load bearing capacity of castellated and traditional cruciform steel columns.

3. CASTELLATED CRUCIFORM STEEL SECTION (CCSS)

Structural engineers purport to make an investigation between a common target of specially useful profiles when examining structures' bearing capacity in case that compressive loads are applied by the presence of a higher gyration radius having lesser material consumption and higher strength per weight for material. In this case, two castellated profiles are being attached to each other by the use of steel plates, being welded to columns in proportionally made distances. In this manner, the moment of inertia for sections would become similar to both X and Y major axes.

These sections, currently, are significantly being implemented in a range of geometries, which are suitable for a range of loading conditions so as to improve the seismic performance of concrete composite structures (especially when regarded as steel

reinforcement in the so called concrete filled steel tubular (SRCFT) columns). Predicting the post-buckling behavior characteristic and the related buckling capacity of 'I' shaped Castellated Cruciform Steel Columns (CCSC) during the occurrence of an earthquake is essential in order to decrease disaster loss.

Typical web openings commonly being used within exposed steel structures consist of hexagonal, octagonal, and cellular perforations. The hexagonal perforations are naturally introduced during the manufacturing of castellated steel members where the member is cut in a zigzag pattern through its web. The resulting two pieces are then reassembled together by welding as shown in Figure 2. Figure 3 illustrates a built-up castellated cross steel column. Although the main intent of the castellation process is to produce stiffer I-sections, by the way of increasing the web height and providing higher major-axis moment capacity than plain-webbed members of the same weight, it also provides access to services and optimizes the use of the costly structural steel materials. These advantages, combined with the significant development in computerized manufacturing equipments, have resulted in the widespread applications of castellated steel members in various structural applications [13].
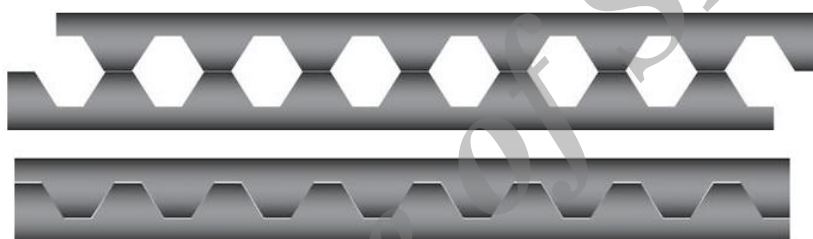

Figure 2. Manufacturing steel members with hexagonal web-castellation (Typical cut and reassembled castellated member)
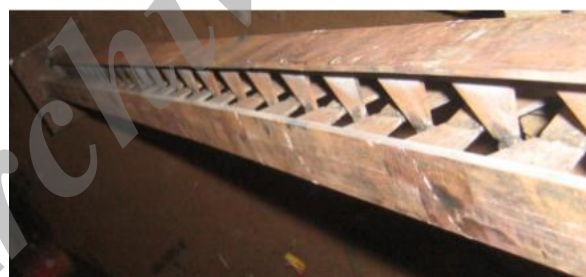

Figure 3. Built-up castellated cruciform steel column

4. EXPERIMENTAL RESULTS OF CFT COLUMNS REINFORCED WITH CASTELLATED CRUCIFORM STEEL SECTIONS

4.1 Characteristics of specimens
Four specimens were constructed and tested under axial compressive loads. From these 4 specimens, 2 were steel tubular column reinforced with castellated cruciform steel sections and the other 2 specimens were steel tubular column reinforced with traditional cruciform steel sections. The material specifications are as follows:

For steel: F_y^s=288 MPa, F_y^t=319 MPa, A_s=1070 mm^2 , A_t=1870 mm^2 , E_s=200,000 MPa and for concrete: F_c'=36.35 MPa, A_c=17793 mm^2 , E_c=29643 MPa

The concrete mix proportions are given in Table 1. The average compressive strength of the cubes at 28 days was calculated as 36.35 MPa. The details of test variables are given in Table 2.

Table 1: Mix proportions of concrete

Mix type	Cement (N/m^3)	Sand (N/m^3)	Coarse aggregate (N/m^3)	Water (N/m^3)	Water/cement ratio
NC	5000	7110	12300	2250	0.45

Table 2: Properties of specimens

Sample specifications	Shape of tube	Dimension (mm)	Type of Steel section	Tube thickness (mm)	Length (mm)	A_{s+t} (mm^2)	N_u (kN)
Ca.C800	circle	155	Castellated	3.5	800	2950	2283
Tr.C800	circle	155	Traditional	3.5	800	2950	2170
Ca.C1900	circle	155	Castellated	3.5	1800	2950	2125
Tr.C1900	circle	155	Traditional	3.5	1800	2950	1989

4.2 Specimen preparation

In the manufacturing of steel framework, first, the steel tubes and steel sections were meticulously cut to size. Then, a square steel plate was welded to the bottom end of steel section and a 15 mm steel plate was welded into the 65 mm thick solid cylinder, being placed at the top of columns as a rigid plat for applying the load to the concrete only. In addition, special attention was paid to the centering and perpendicularity of the steel plate. Later, a steel tube was applied to make a coating for the steel section (Figure 4(a), and Figure 4(b)).

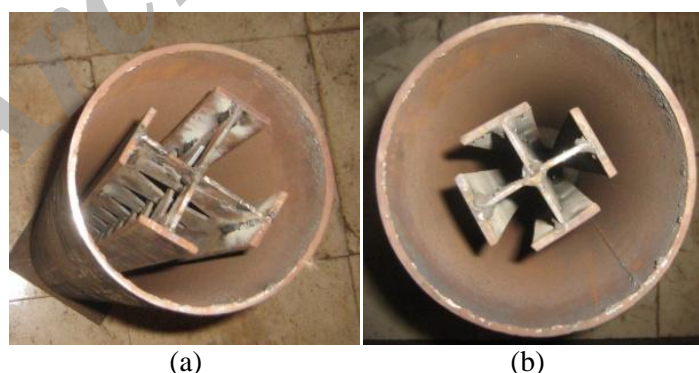

(a)   (b)

Figure 4. (a) Steel tubular column reinforced with castellated cruciform steel section
(b) Steel tubular column reinforced with traditional cruciform steel section

The steel tube was carefully adjusted to assure the steel section lies in the core of the steel

tube. Finally, the steel tube was also welded to the bottom steel plate. The specimens were cast in concrete in such a way that a very small amount of longitudinal shrinkage occurred at the top of the specimens during curing.

Therefore, prior to the testing, the top surface of each specimen was undergone the process of roughening by the use of a wire brush and a thin layer of cement was poured on the rough surface to fill the longitudinal gap. This process ensured loading on the concrete, the steel section, and the steel tube in a simultaneous way.

5. TEST SETUP AND PROCEDURES

All specimens were tested in a 5000 kN capacity universal testing instrument. Before testing began, the 15 mm-thick clamping plates were bolted into the bottom of the specimen in order to enhance the strength at the ends. Linear Variable Differential Transducers (LVDTs) were placed at the top of column to measure the axial deformation of cylinder and one more LVDT was also used to measure the lateral deflection in mid-height specimens.

The specimens were loaded under monotonically increasing axial compressive loads. By the application of a thick bolted plate to the cylinder, the load was applied onto the concrete core only. Loads were applied at a very slow rate in order that the local buckling behavior in the composite columns could be observable. When considerable deformation occurred and local wall buckling was observed, the tests were terminated.

6. RESULTS AND DISCUSSIONS

6.1 Load-deformation curves of the specimens
The ultimate strength and properties of each specimen is given in Table 2. Figures 5 and 6 indicate that the use of castellated steel sections causes the confining effect of steel sections on the concrete and the strength of composite column to increase effectively, as well.
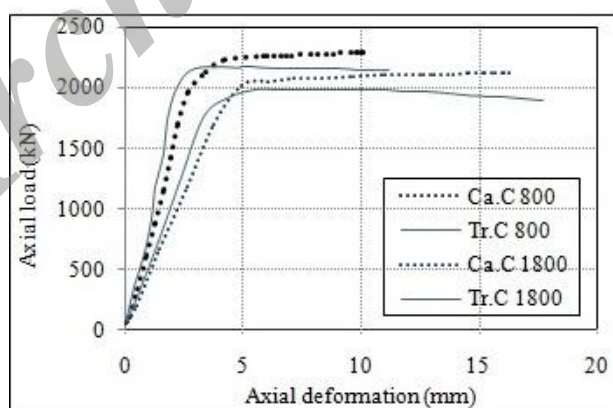

Figure 5.   Axial load–axial deformation responses of SRCFT columns

The test load versus axial deformation curves of specimens are presented in Figure 5 and the compressive behavior of specimens could be described in what follows. In general, the load–

axial deformation curve was close to linearity before the steel began to yield and beyond this load level, load–axial deformation curve diverged significantly from its initial linearity.

For the steel tubular column reinforced with traditional steel section (Tr.C), the strength gradually decreased with an increase in axial deformation after Nu was reached. The steel tubular columns reinforced with castellated steel section (Ca.C) exhibited strain-hardening characteristics long after the steel yielded.
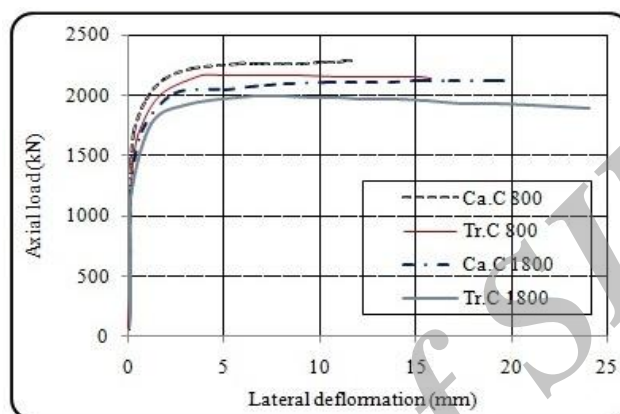

Figure 6. Axial load –lateral deflection responses of SRCFT columns

Curves of axial load N versus mid-height deflection for specimens are given in Figure 6. The deflection of specimens was not obvious before the axial load reached 0.6–0.7 Nu. Beyond this load level, the deflection increased gradually. When the load approached Nu, the deflection increased rapidly. The lateral displacement caused secondary moments, and this lead to the column failing by bending rather than by compression.

Table 3: Properties of specimens

IPE No	(A) Cm2	h_W cm	t_w cm	b_f cm	t_f cm	a-a			b-b		
						A Cm^2	I_x Cm^4	W_x Cm^3	A Cm^2	I_x Cm^4	W_x Cm^3
12	(13.2)	18	0.44	6.4	0.63	16	809	90	11	746	83
14	(16.4)	21	0.47	7.3	0.69	19.7	1374	130	13.1	1266	120
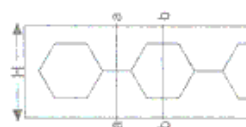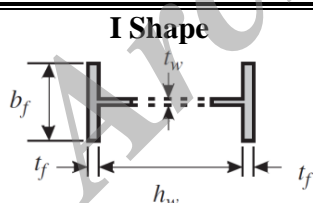
7. NUMERICAL RESULTS OF CASTELLATED AND TRADITIONAL CRUCIFORM STEEL COLUMNS

In the present study, the strength of axially-loaded castellated cruciform steel columns are modeled using the ANSYS software in order to assess the load bearing capacity of castellated cruciform steel columns and it is compared with the analytical results of traditional cruciform steel columns. Summary and specifications of results is reported about a number of castellated and traditional steel sections, shown in Table 3.

In Table 3, "IPE" is the standard steel profile section, "A" is the cross section area of IPE section, and "CPE" is castellated steel section made built-up by the application of IPE sections.

7.1 Finite element modeling

Over the past few decades, the Finite Element Method (FEM) has been developed into a key indispensable technology in the modeling and simulation of various engineering systems. As such, techniques that are related to modeling and simulation in a rapid and effective way play an increasingly important role in building advanced engineering systems. Therefore, the application of the FEM has multiplied rapidly [14]. The current study aims at investigating the geometric and material nonlinear responses of Castellated Cruciform Steel Columns (CCSC) under axial loading. The models were simulated by the application of ANSYS 10.0 In order to perform the buckling analysis according to ANSYS user's manual [9], before loading, a perturbation load was applied to specimens towards the considered buckling direction. The element shell 181 is the proper element in order to attain accurate results in this research study. The element has such characteristics as plasticity, creep, stress stiffening, large deflection, and large strain capabilities [9].

7.2 Modeling assumption

The main assumptions employed in the conducted investigation are summarized below:
1. All columns are assumed to have a bilinear nonlinear inelastic material with a modulus of elasticity equal to $E=2\times10^5$ MPa, Poisson's ratio equal to $v=0.3$, and yield stress equal to $f_y =250$ N/mm^2. The stress-strain behavior of steel columns used for nonlinear material and geometric static analysis has been illustrated in Figure 7. In addition, the effect of strain hardening of steel has been considered.
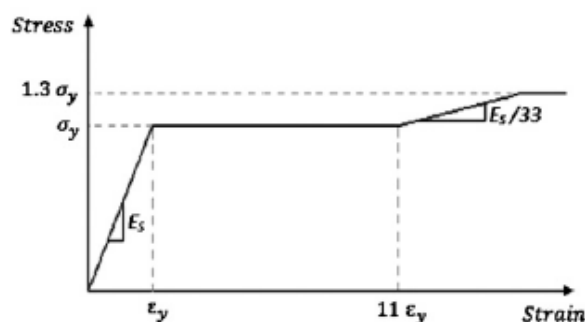
Figure 7. The stress-strain relationship for steel material [10]

2. The I-shaped column under investigation is defined by its length L, flange width b_f, flange thickness t_f, web width h_w, and web thickness t_w, as shown in Table 3.
3. Hexagonal web perforations are uniformly spaced at the distance "s" along the column axial direction (Figure 3).
4. Various end conditions are implemented to simulate different boundary conditions.
5. The column is axially loaded with concentrated loads applied at its ends.
6. To avoid any secondary stresses resulting from local deformations at the column ends (especially close to the hole), load is applied by steel plate placed at the top and bottom of columns.

Table 4. Numerical results of castellated and traditional cruciform steel column No. 12

Samples						$\dfrac{(P_b)_{Ca12-}(P_b)_{Tr12}}{(P_b)_{Tr12}}$
L (mm)	B.C	$(\frac{kL}{r})_{Tr12}$	$(\frac{kL}{r})_{Ca12}$	$(P_b)_{Tr12}$ (kN)	$(P_b)_{Ca12}$ (kN)	% Load increment percentage
	F.F	36.2	35.7	532.4	534.47	0.38
4320	F.P	50.73	49.98	510.9	517.35	1.26
	P.P	72.48	71.4	327.3	479.24	46.4
	F.Fe	144.9	142.8	96.4	204.98	112.6
	F.F	40.7	40.1	484.3	530.22	9.4
	F.P	57.1	56.2	456.6	508.71	11.4
4860	P.P	81.54	80.3	270.4	453.73	67.8
	F.Fe	163.1	160.6	76.6	166.27	117
	F.F	42.3	41.6	468.3	527.26	12.6
	F.P	59.2	58.3	439.4	503.80	14.6
5040	P.P	84.5	83.3	254.3	442.16	73.8
	F.Fe	169.1	166.6	71.4	154.70	116.6
	F.F	45.3	44.6	437.65	524.47	19.8
	F.P	63.4	62.4	403.61	497.70	23.3
5400	P.P	90.6	89.2	225.58	418.92	85.7
	F.Fe	181.2	178.5	62.4	136.76	119
	F.F	48.3	47.6	405.6	520.63	28.3
5760	F.P	67.7	66.6	369.9	488.37	32
	P.P	96.6	95.2	201.1	394.50	96.1
	F.Fe	193.3	190.4	55	120.49	119
	F.F	54.3	53.5	348.3	511.41	46.8
	F.P	76.1	74.9	308.6	465.77	50.9
6480	P.P	108.7	107.1	162.2	339.02	109
	F.Fe	217.5	214	43.5	95.92	120
	F.F	60.4	59.5	300.1	502.76	67.5
7200	F.P	84.5	83.3	258.1	440.19	70.5

P.P	120.8	119	133.2	289.57	117.3
F.Fe	242	238	35.4	77.41	118.6

7.3 Comparison of the numerical results

Modeling and nonlinear analyses of the columns have been undertaken under axial loading. Table 4 shows the obtained result using ANSYS software analyses for the castellated and traditional cruciform steel columns No 12. In Table 4, (kL/r), Tr12, Ca12, and P_b are the slenderness, castellated cruciform steel column, traditional cruciform steel column, and load bearing capacity of the specimens, respectively. L is the length of the castellated cruciform steel columns and B.C is the boundary conditions of end supports of the columns:

➢ Fixed Support – Fixed Support = F.F
➢ Fixed Support– Pinned Support = F.P
➢ Pinned Support– Pinned Support = P.P
➢ Fixed Support –Free ended = F.Fe
➢ $\frac{(P_b)_{Ca12} - (P_b)_{Tr12}}{(P_b)_{Tr12}}$ = Maximum load difference percentage of the two sets of relative samples

Considering the load carrying capacity of castellated and traditional steel columns in Table 3, it is concluded that load bearing capacity of castellated cruciform steel columns is considerably higher than the traditional cruciform steel columns. In addition, it was found that the difference between load increment percentage of castellated and traditional sections increases with the increase of the columns' slenderness. According to Table 4, castellated cruciform steel columns show a significant load difference (up to 119 percent) in comparison with traditional cruciform steel columns.

With regard to the load carrying capacity of the samples in Table 4, it is found that with the increase of support restraint of the castellated cruciform steel columns $((P_b)_{Ca12})$, strength reduction ratio of the highest length to the shortest length of the samples at the same boundary conditions decreases. For example the amount of strength reduction ratio for the Fixed support – Fixed support (F.F), Fixed support – Pinned support (F.P), Pinned support – Pinned support (P.P), and Fixed support − Free ended (F.Fe), of the castellated cruciform steel columns $((P_b)_{Ca12})$ are equal to 6% , 15% , 39% , and 61%, respectively. On the other hand, the amount of strength reduction ratio for the Fixed support – Fixed support (F.F), Fixed support – Pinned support (F.P), Pinned support – Pinned support (P.P), and Fixed support – Free ended (F.Fe), of castellated traditional steel columns $((P_b)_{Tr12})$ are equal to 77% , 97% , 145% , and 272%, respectively.

Comparing the obtained results of load bearing capacity of castellated with traditional steel columns $((P_b)_{Ca12}, (P_b)_{Tr12})$, it was found that the strength reduction ratio of castellated cruciform steel columns is considerably lower than the traditional cruciform steel columns. Therefore, it is suggested that castellated cruciform steel columns be conducted as fixed ended and lengthy column and be used at high-rise structures, industrial halls, or tower columns. That is because the strength of castellated columns (especially through increasing

the support restraint) is high and their reduction in strength does not show significant difference with the increase of length of columns.

7.4 Verification of the finite element modelling of castellated cruciform steel sections
Performance of the developed 3D finite element model is validated by evaluating the maximum load of I-shaped plain-webbed columns and comparing it to the compressive strength capacity of AISC code [10]. Therefore, the obtained results of FEM analysis have been compared according to Engesser's analytical formula, only for slender columns in the range of Elastic behavior that accounts for shear deformations in plain-webbed columns [11, 12]. The influence of the boundary conditions on the column strength is accounted for by considering various end conditions including pinned support– pinned support, fixed support– pinned support, fixed support – fixed support, and fixed support –free ended columns for which k has been estimated as 1.0, 0.7, 0.5 and 2.0, respectively. The effective buckling length factor (k) is incorporated in the load calculation which yields the following expression of the analytical buckling load P_{an} in which $n \approx A/A_w$,

$$P_{an} = \frac{P_e}{1 + (^{nP_e}/_{AG})} = \frac{EI}{(^{kL}/_{\pi})^2 + (^{nEI}/_{AG})} \qquad (1)$$

where A is the area of the cross-section and A_w is the area of the web $A_{w=} h_w.t_w$.
The finite element method takes into account the analyses of 224 different columns that are modeled using the ANSYS software to assess the maximum load P_{FE} compared to its analytical counterpart P_{an}, obtained from the equation No.1.
A sample of obtained results is presented in Table 5. Quantitative comparison between the two sets of results show an absolute maximum relative error of about 7.9 % for the case of a pinned support- pinned support column.

Table 5. Summary of the results for castellated cruciform steel columns No. 14

L (mm)	Kl/r (b-b section)	Engesser's formula	P_{an} (kN) (AISC)	P_{FE} (kN) (ANSYS)	Relative error $P_{an}-P_{FE}/P_{an}$ %
4410	31.1	-----------	654.85	671.3	-2.5
5880	41.5	------------	619.7	656.6	-5.9
4410	43.6	-------------	616.04	654.1	-6.1
5880	58.1	-------	585.35	628	-7.2
4410	62.2	---------	574.97	620.5	-7.9
5880	83	-----------	512.3	551.3	-7.6
5880	166.1	188.2	187.59	192.8	-2.7
6720	189.8	144.1	143.62	150.5	-4.7
7560	213	113.9	113.48	120.3	-6

Table 5 illustrates that theoretical behavior predicted by the nonlinear finite element static analysis followed closely the actual behavior exhibited by the analytical buckling load according to AISC code [10]. Consequently, it was found that the finite element model is

reliable enough to be used in undertaking nonlinear analyses of castellated cruciform steel columns.

7.5 The proposed equation for load capacity calculation of Castellated Cruciform Steel Columns (CCSC)

Considering the obtained numerical results of castellated cruciform steel column No. 12, 14, 16, 18, 20, 22, and 24, an equation is proposed for the calculation of castellated cruciform steel columns. In this graph, X-axis is defined as slenderness of the samples and Y-axis is defined as a parameter named $\Phi=P_b/P_y$ (both sides are without dimensions), L is the length of samples, r is the gyration radius of castellated steel columns around X or Y buckling axes (in cruciform sections the amount of r is same for both axes), P_b is load bearing capacity of the samples obtained by ANSYS finite element analysis, and P_y is nominal compressive strength which is equal to cross section area of castellated cruciform steel section multiply to yield stress of steel section. By fitting this curve using Jandel software, the following equation is suggested

$$P_b/P_y = a + b(1-(1+\exp((kl/r - 84/65)/d))^{-e}) \qquad (2)$$

In the above equation, the value of a, b, c, d, e are as follows:

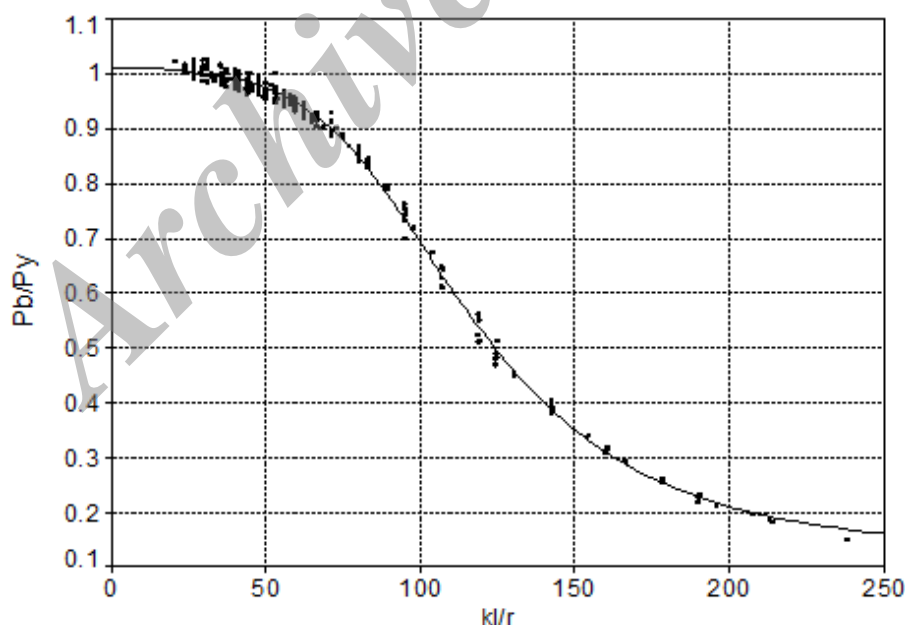a= 1.0109962   b= - 0.877074   d=17.207685   e= 0.369165

Figure 8. Design curve of castellated cruciform steel column

Equation 2 is in accordance with the load capacity of castellated cruciform steel columns and slenderness parameters kl/r (λ). This curve has correlation coefficient equal to 0.9970, shown in Figure 8. With the help of the proposed equation and under varying slenderness values, columns' load bearing capacity could be simply calculated for all kinds of castellated cruciform steel columns.

8. CONCLUSIONS

The following conclusions could be reached based on the experimental results of four centrally loaded concrete-filled steel tubular columns reinforced with castellated and common cruciform steel sections

1. Experimental results of SRCFT columns show that the strength and the rate of stiffening in post-yield range of stiffening castellated cruciform steel sections are considerably higher than the traditional cruciform steel sections, pointing to the effect of reinforcing steel shapes on the enhancement of columns' strength.
2. The measured lateral-to-axial load of the steel tube suggests that significant confinement is not present for most specimens until the axial load reached almost 70% of the ultimate strength of the columns. In addition, it was found that steel tube and steel section both yield when Nu is approached.
3. The obtained results from the analyses of steel columns show that the maximum load capacity of castellated steel columns is considerably higher than that of the traditional steel columns and the difference between the load increment percentages in castellated and traditional sections increases as the columns' slenderness goes up.
4. The strength reduction ratio of castellated cruciform steel columns is considerably lower than the traditional cruciform steel columns. Therefore, it is suggested that castellated columns be conducted as fixed ended and lengthy column and be used at high-rise structures, industrial halls, or tower columns.
5. The equation for the prediction of load capacity in castellated cruciform steel columns under axial compression was proposed. Furthermore, numerical results obtained from the ANSYS nonlinear analyses are in good agreement with the analytical results reported in this paper.

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