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A Review on the Experimental Study of Cold-Formed Steel Columns with Holes subjected to an Axial Load

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Abstract: This paper focuses on a review of research studies of Cold- Formed Steel Columns (CFSC) with the aim to provide an overview of the current state of understanding and evaluation of existing findings. CFS members commonly has complex buckling behaviour and buckling mod with the slenderness being the main factor that affect the stability and buckling failure. The different buckling modes that occur in such columns are evaluated in the scope only for the close section type such as SHS, RHS and corrugated column with or without perforated.
From the experiment works, compression test was conducted to determine the effect of a column reaction under compression load. The tests performed with various parameters such as the slenderness, thickness, temperature and holes in the column web. Based on the observation, the perforated RHS columns are generally failed due to local buckling. The holes provided in the column also influenced the buckling behavior. In general, short column tends to experience local buckling, while slender column tends to experience global buckling.

Keywords: Cold-Formed Steel (CFS), Closed Section, Perforated, Non-Perforated

1. Introduction

Steel is an alloy that contains carbon, which can make up 2.1% of its total weight and can be used either separately or in combination with other materials [1]. The popularity of steel is based on the 'properties' of the material; good strength and ductility, easily produced, and recyclable. In this study the focus of steel is on the type of CFS. This type of steel is receiving growing attention and is increasingly being used in research and construction, particularly in industrial and European countries [2]. Malaysia is not an exception, but its widespread use is widely applied in lattice structures. In addition, the construction technology that uses CFS as a building material in buildings is becoming more and more important. As a result, the steel industry continues to focus on improving the use of SCF in construction. Indeed, the

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CFS is one of the materials increasingly used in construction because of its high strength to weight ratio, which makes it easy to produce, transport and install.

CFS column have complex buckling behaviors. If the compressive load is excessive, the column may fail due to structural instability called buckling. There are three common modes of failure in CFS, namely local buckling, distortion and global. The failure mode is not fully understood in the CFS closed-type such as Square hollow section (SHS) and rectangular hollow section (RHS).

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Column may be deformed in one of three modes, such as Flexural (Euler), Torsional and Torsional-Flexural buckling. Problems with instability and buckling failure usually occur in slender structures such as building column are undeniable. In general, the main factor that affects the failure is the slenderness ratio. According to [3] one of the major challenges in designing the CFS is the prevention of buckling. Due to the low thickness-to-width ratio, it is likely that the bar will deform at stresses less than the yield stress.

Therefore, buckling is the main design factor for all CFSs, as opposed to hot-rolled steel (HRS) properties, where steel yielding is the main design factor. The web plate commonly to be perforated and the holes are provided for many functions such as connections between columns and beams, utility service such as water supply, electricity, network and other uses.

The preparation of these holes will result in the concentration of stress, the effect on the elastic strength and subsequently the strength of the structure member. Therefore, the development of a study from the selected previous study is performed to observe the response of the perforated and non-performed column in the influence on the buckling mode.

2.0 Materials and Methods

2.1 Materials

The materials from the selected study are explained below;

Studied by Yang and Xu [4] use SHS tube grade 235 with 6 samples labelled C1, C2, C3, C4, C5, C6. The tube dimension is 200×200×2 mm as shown in Table 2.1 and Figure 2.1.

Table 2.1: Dimension specimens in research Yang and Xu [4]

Specimen	Bar	Concrete	height (mm)	Width x depth	Thickness, t (mm)	Bar dia.	Area Ratio
C1	-	-	1200	200	2	-	100%
C2	-	fill	1200	200	2	-	3.96%
C3	inside	fill	1200	200	2	12	5.07%
C4	outside	fill	1200	200	2	16	5.91%
C5	inside	fill	1200	200	2	12	5.07%
C6	outside	fill	1200	200	2	16	5.91%

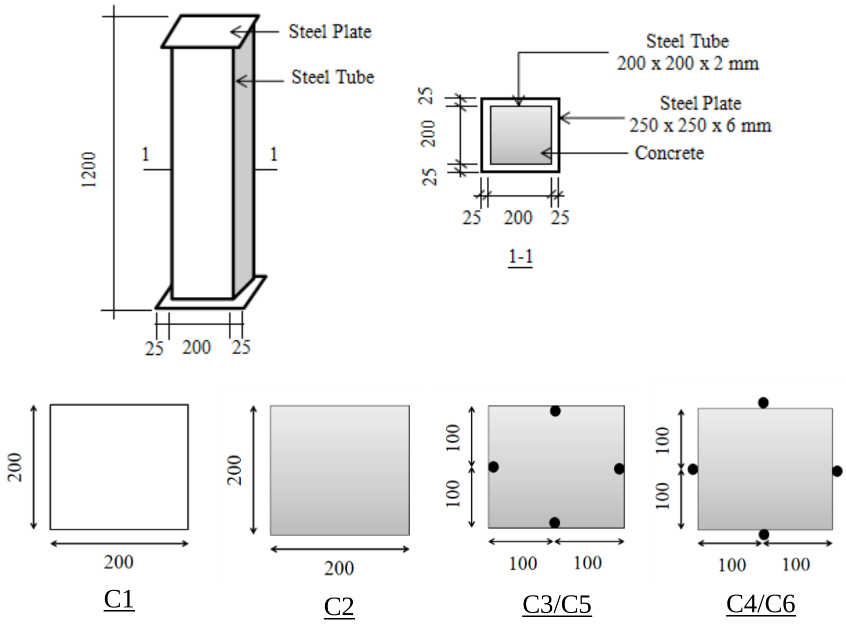

Figure 2.1: Details dimension in research by Yang and Xu [4]

Balarupan [5] used the CHS 450 grade cross-section of the CFS with dimensions of 90×90×2 and 100×100×2 for short columns and 65×65×3 and 65×65×6 for slender columns and described in Table 2.2.

Table 2.2: Dimension specimens in research Balarupan [5]

Spesimen	Width (mm)	Depth (mm)	Thickness, t (mm)	Height (mm)
90 x 90 x 2 SHS	90	90	2	270
100 x 100 x 2 SHS	100	100	2	300
65 x 65 x 3 SHS	65	65	3	1800
65 x 65 x 6 SHS	65	65	6	1800

Varghese and Krishnakumar [6] used Corrugated SHS column with CR2 grade and tested 3 samples: Type 1 (two corrugated and flat edge surfaces), Type 2 (two corrugated and flat surfaces where both of two flat surfaces have 3 holes with 82 mm size and distance of 175 mm) and type 3 (same as type 2 but only perforated on one side only) with a height of 700 mm and a thickness of 1.6 mm. Details of this study are provided in Table 2.3 and Figure 2.2.

Table 2.3: Dimension specimens in research Varghese dan Krishnakumar [6]

Material	α	a	h	t	c	l	b	d	E	Fy	Holes dia. (mm)
CFS Grade CR2	45	20	15	1.6	21.21	70	210	15	204	295	82

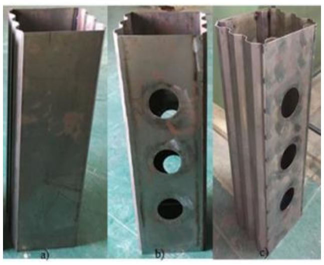

Figure 2.2: a) Column Type 1, b) Column Type 2, c) Column Type 3

Finally, Al-Shareef [7] used an innovative Section C column converted into RHS and prepared 24 specimens labelled C1 to C24 (Table 2.4). The variable involves being; $\alpha = 0.2h$, $\alpha = 0.4h$, $\alpha = 0.6h$ and $\alpha = 0.8h$ are used. The variable serves as a parameter for determining the width of the rectangular shape of the hole, as shown in Figure 2.3. All specimen dimensions are 50×80 mm and with different thicknesses of 6, 4, and 1.25 mm and height of 250 mm and 500 mm.

Table 2.4: Dimension specimens research AL-Shareef [7]

Cross section	Length, L (mm)	B x D (mm)	Thickness, t (mm)	α
RHS	250	50x80	6	0.2
	500		4	0.4
			1.25	0.6
				0.8

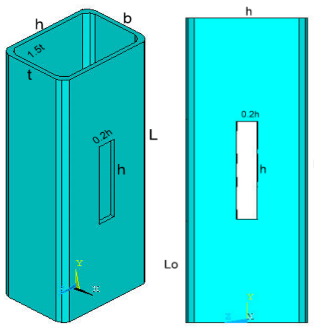

Figure 2.3: Geometry specimen column in research AL-Shareef [7]

2.2 Methods

The method used is a compression test on the CFS column with closed cross section. Compression test used to determine the behaviour of materials when it subjected to compression load. The test was conducted by placing the sample between two plates, then applying a compression load under the sample.

As part of research conducted by Yang and Xu (2018), they performed a compression test of the thin-walled steel tube column filled with concrete and steel bars on the outside and inside of the tube. Compression test carried out using a hydraulic pressure testing machine with a capacity of 5000 kN.

Balarupan [5] performed axial compression tests at uniform temperatures ranging up to 700 (20, 200, 400, 500, 600 and 700). Fixed support was applied at the end of the columns and an electric furnace compression machine with the compressive load applied using a hydraulic pump with capacity 600kN. Figure 2.4 shows the spesimens tested in electric furnace.

a) Stub Column **b) Slender Column**

Figure 2.4: Compression test for research Balarupan [5] using electric furnace

Varghese and Krishnakumar [6] used UTM machine by applying 1000kN load capacity to conduct a compression test on the corrugated SHS column.

Meanwhile, Al-Shareef [7] used hydraulic compression test machine on C type column that converted to RHS and to ensure the specimen members act as one member, the slenderness of the individual components of the specimen should not be greater than 3/4 slenderness of RHS according to specifications in AISC. The formula is as in Equation (1).

$$\frac{Ka}{ri} \leq \frac{3}{4}\left(\frac{KL}{r}\right) RHS \tag{1}$$

Where:

a = Distance between connectors (mm)

ri = Minimum gyration radius for connected components (mm)

Holes with different sizes were located at the midpoint of the steel web.

3. Results and Discussion

Table 3.1 and Figure 3.1 shows the experimental result from a study done by Yang and Xu [4]. From the figure it can be seen that C6 has the highest ultimate load value (1353 kN) which is expected from the presence of concrete support and steel bars outside the tube. This further strengthen the structure of the column because the element can withstand and strengthen the steel wall when it is loaded. Therefore, steel bars are very well used as reinforcement in the construction for example, in RC column because it can strengtern the column structure. C4 and C6 have the highest bearing capacity due to the presence of steel bars placed outside the tube and this can act as a support to the column. While C1 has a low bearing capacity because the column does not contain concrete and steel bars, where there are no supports that support the steel.

Table 3.1: Result Axial bearing capacity in research Yang and Xu [4]

Specimen	Area (mm^2)	Displacement (mm)	Ultimate Load, Nue (kN)	Nominal Bearing Capacity, No (kN)	Nue/No
C1	-	4.2	145.5	384.9	0.377
C2	-	5.4	802.0	1014.9	0.790
C3	452.4	10.1	1154.5	1119.6	1.031
C4	804.2	7.4	1200.0	1200.9	0.999
C5	452.4	7.0	1100.0	1119.6	0.983
C6	804.2	4.6	1353.0	1200.9	1.127

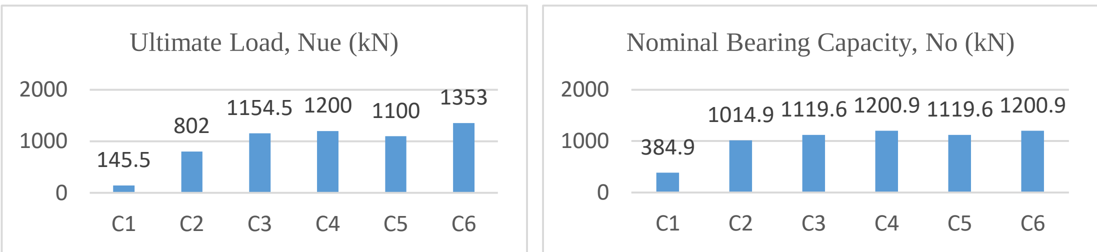

Figure 3.1: Graph shows the Ultimate Load and Nominal Bearing Capacity

Results from study by Balarupan [5] is shown in Figure 3.2 and Table 3.2. The figure shows the failure mod of columns under the compression load. CI shows the concave and convex local buckling on the wall of the column, meanwhile C2 that filled with concrete shows in the middle of the column shear failure was observed indicate by three buckle waves. This happens because of the concrete failure inside the tube. For specimens C3 and C4 which were filled with concrete and a bar inside the tube, experienced shear failure and the local buckling which occurs at the top of the column. For C5 and C6 filled with concrete and bar outside tube also experienced shear failure and local buckling occurs at the bottom part of the column.

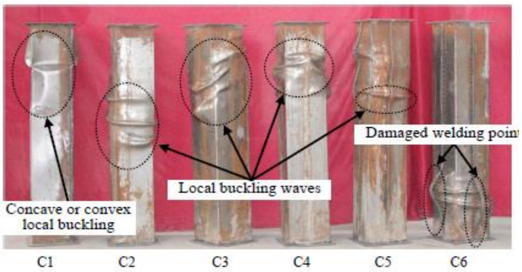

Figure 3.2: Failure Mod for column specimen research Yang and Xu [4].

Table 3.2: Result testing in research Balarupan [5]

Temperature (°C)	Ultimate Failure Load (kN)			
	90 x 90 x 2	100 x 100 x 2	65 x 65 x 3	65 x 65 x 6
20	212.6	202.7	277.6	-
200	207.0	196.7	271.9	-
400	178.3	179.6	229.2	-
500	119.4	130.0	151.3	298.8
600	89.9	84.6	93.2	179.4
700	46.6	35.3	37.1	72.0
Failure Mod	Local Buckling & yield		Global Buckling - Flexural Buckling	

Based on Figure 3.3 and Table 3.3 from a study by Varghese and Krishnakumar [6], the steel column undergoes expansion as soon as the column heating with high temperature. Such expansion causes an axial compression load on the column. The enhancement of local buckling deformation can be observed against the ultimate load. Short column specimens are clearly buckling locally before they reach the ultimate load.

Figure 3.3: Failure Mod for stub column specimen from research Balarupan [5].

Table 3.3: Result testing in research Varghese and Krishnakumar [6]

Specimen	Peak Load (kN)	Maximum Displacement (mm)	Cost/Load
Conventional Column	103.52	6.6	5.25
Type 1	205	2.15	3.14
Type 2	196	4.8	3.79
Type 3	284.98	2.3	2.82

As for the slender column, all specimens experienced flexural buckling failure. Based on the observations, the middle part of the column experiences maximum lateral elasticity as predicted. The buckling behavior of the SHS column is not limited to a particular direction, not even parallel to the column surface, but the direction of failure on the column occurs randomly and at different angles. This can be concluded that the direction of column failure also affects the imperfections of the global buckling as the random direction may be due to load alignment.

Figure 3.4: Failure Mod for slender column specimen from research Balarupan [5].

From Table 3.3 conventional columns show higher maximum displacement (6.6 mm) while column type 1 shows the lowest maximum displacement (2.15 mm). Figure 3.5 shows a picture of the results obtained from the experiment and compared with the results from finite element analysis. From Figure 3.6 the graph of the relationship between load against displacement that occur in the column can be observed from both experiment and the finite element analysis. The graph shows that all the column reaches the peak load phase before they experience a failure phase.

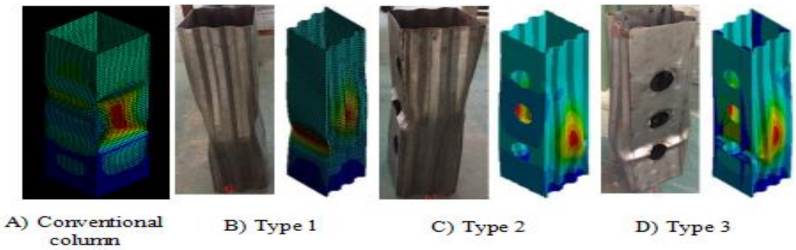

Figure 3.5: Failure Mod for column specimen from research Varghese and Krishnakumar [6]

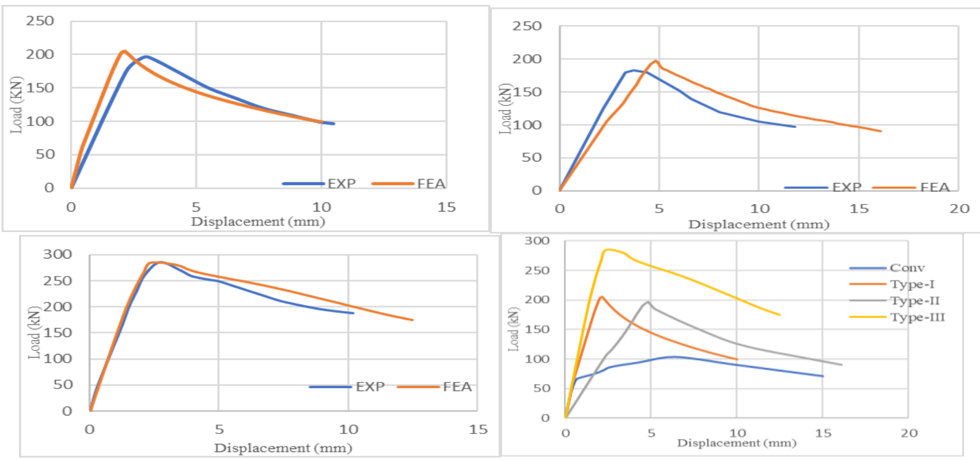

Figure 3.6: Relationship between load against displacement

Based on the result from Table 3.4, in Figure 3.7 for the thickness of t = 6mm, it can be seen that the steel is deformed from its original shape which is C1 (16%), C2 (13%), C3 (12%), C4 (10%), C5 (14%), C6 (13%), C7 (12%) and C8 (10%). For column with L = 250 mm, C1 has the higher deformation (16%) and the lowest is from C4 (10%), while for column with L = 500 mm C5 has higher deformation (14%) and the lowest is C8 (10%). It can be concluded that the hole using variable α = 0.2h has higher compare with α = 0.8h. This shows that the holes size also affect the mod of failures.

Table 3.4: Result testing in research AL-Shareef [7]

Specimen	L (mm)	t (mm)	α	P_{Exp} (kN)	Deformation (mm)
C1	250	6	0.2	456.6	2.35
C2	250	6	0.4	397	2.04
C3	250	6	0.6	349	1.79
C4	250	6	0.8	286.4	1.47
C5	500	6	0.2	444	2.15
C6	500	6	0.4	395.2	2.02
C7	500	6	0.6	372.4	1.87
C8	500	6	0.8	287	1.50
C9	250	4	0.2	290.7	1.58
C10	250	4	0.4	257	1.32
C11	250	4	0.6	212	1.15
C12	250	4	0.8	177.8	0.92
C13	500	4	0.2	259	1.35
C14	500	4	0.4	240.87	1.27
C15	500	4	0.6	211.97	1.12
C16	500	4	0.8	184	0.98
C17	250	1.25	0.2	24.14	0.57
C18	250	1.25	0.4	19.7	0.48
C19	250	1.25	0.6	13.8	0.42
C20	250	1.25	0.8	6.92	0.35
C21	500	1.25	0.2	24.89	0.67
C22	500	1.25	0.4	19.32	0.46
C23	500	1.25	0.6	15.65	0.43
C24	500	1.25	0.8	7.23	0.40

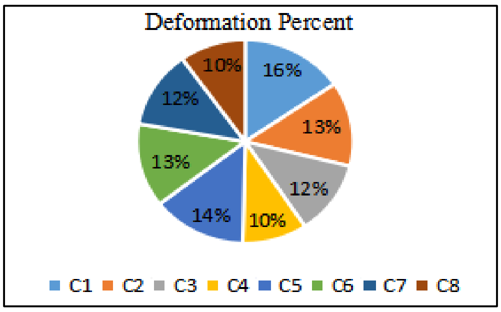

Figure 3.7: Pie chart shows the deformation percent (t = 6mm)

Figure 3.8 shows the differences in maximum load value when columns tested using the same hole size but with different thicknesses. C1 and C5 (t = 6mm), C9 and C13 (t = 4 mm), C17 and C21 (t = 1.25 mm). In figure shows C1 is the highest maximum load at 4556.6 kN and followed by C9 (290.7 kN) and C17 (24.14 kN) for L = 250 mm. While for L = 500 mm, C5 has the highest load of 444 kN and followed by C13 of 259 kN and C21 of 24.89 kN. C1 and C5 have the highest maximum load values due to the material properties because C1 and C5 have the highest thickness of 6 mm. This can be concluded that the higher the thickness of the column the higher the maximum load obtained. This because the thickness can increase the stability of the material properties and it's able to withstand high loads when subjected to compressive loads.

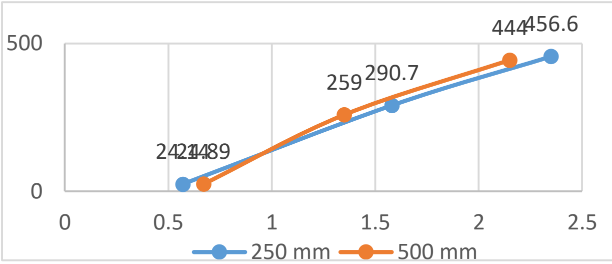

Figure 3.8: The graph shows the maximum load of P_{Exp} kN (α = 0.2h)

Figure 3.9 shows the shape changes that occur in the column during the test. Based on the observations in the study the results also explain that the column with a thickness of 1.25 mm failed due to local buckling. This is because the column fails before the yield stress was achieved. In addition, the best behavior can be done by using a hole size of 0.4h mm.

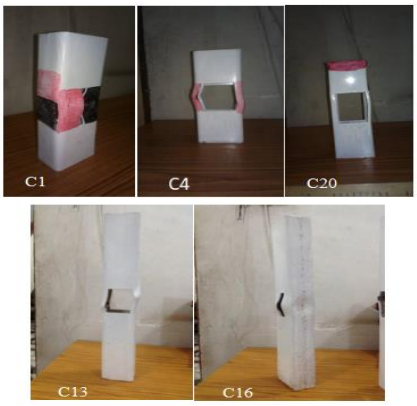

Figure 3.9: The picture shows a local buckling on a perforated RHS cross section

4. Conclusion

In conclusion, through this study there are several effects influencing the buckling modes. The hole size affects the behavior of the columns where the area around the hole experience local buckling, then followed by a global buckling due to increasing load applied to the column. The slenderness of the column affects the buckling mode where short column tend to experience local buckling, while slender column tend to experience global buckling. In addition, column filled with concrete and steel bars affect the ultimate load and the local buckling load.

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