Buckling Performance of Irregular Section Cold-Formed Steel Columns under Axially Concentric Loading

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Abstract—This paper presents experimental investigation and finite element analysis on buckling behavior of irregular section cold-formed steel columns under axially concentric loading. For the experimental study, four different sections of columns were tested to investigate effect of stiffening and width-to-thickness ratio on buckling behavior. For each of the section, three lengths of 230, 950 and 1900 mm. were studied representing short, intermediate long and long columns, respectively. Then, nonlinear finite element analyses of the tested columns were performed. The comparisons in terms of load-deformation response and buckling mode show good agreement and hence the FEM models were validated. Parametric study of stiffening element and thickness of 1.0, 1.15, 1.2, 1.5, 1.6 and 2.0 mm. was analyzed. The test results showed that stiffening effect pays a large contribution to prevent distortional mode. The increase in wall thickness enhanced buckling stress beyond the yielding strength in short and intermediate columns, but not for the long columns.

Keywords—Buckling behavior, Irregular section, Cold-formed steel, Concentric loading.

I. INTRODUCTION

COLD-formed steel member is made from press braking thin steel sheet under ambient condition to optimize structural performance. The optimization can be done through increasing individual sectional element capacity and in turn overall member response. The parameters related to width-to-thickness ratio of the individual element and the presence of stiffening elements are the main factor governing local and distortional buckling phenomena. However, overall buckling can be critical when the former buckling modes are prevented. These buckling behaviors made cold-formed steel structure very complicate.

There have been a lot of past researches studying on cold-formed steel columns under concentric loading. Young and Chen [1] carried column tests of cold-formed steel non-symmetric angle sections. The test results have shown conservative estimation of the AISI design equation based on effective width concept. Innovation cold-formed steel columns were studied by [2] through experimental works and FEA. From the results, ultimate capacity obtained from the finite element analysis gave 6 percent higher in average compared with the tests, while the calculated capacities based on AS/NZS 4600 [3] shown 12 percent higher. From the study, finite element analysis can be used as a tool to capture nonlinear buckling behavior of thin-wall member.

The application of finite element analysis on buckling analysis has also been confirmed by [4]. The study performed buckling analysis of high strength stainless steel of hollow long columns. The results indicated the improvement of buckling behavior with the provided stiffening elements. Compared with the calculated capacity based on AISI specification [5], Australian/New Zealand Standard [6] and European Code [7] shown that all the codes provided lower and higher values for sections with and without stiffening elements respectively. Freitas, Freites and Souza [8] also adopted the finite element analysis to investigate buckling behavior of steel rack columns and compared with tested results. The structures were modeled using shell element and stress-strain relation obtained from tensile test was used in material input. The study also confirmed the applicability of the finite element program for predicting buckling modes.

From the literature reviews, there have been a few researches investigating buckling capacity of irregular section cold-formed steel columns. Due to the deviation of the shear center, the buckling behavior of such the section always shows buckling mode coupled with torsional response. In this paper, buckling behavior of irregular section cold-formed steel columns under concentric loading was studied. The columns are generally used for steel rack or cabinet for electronic equipment in which the unfair buckling phenomena may leads to damage of the installed equipment [9].

II. SUSCEPTIBLE TO BUCKLING OF IRREGULAR SECTION COLD-FORMED STEEL COLUMNS

Thin wall columns can be buckled in either one of three different modes or combination. Local buckling can be a dominant mode if column contain very flexible constrained element i.e. high width-to-thickness ratio. If movement of an end flexible element is not prevented by edge stiffening, the column load capacity may be controlled by presence of distortional buckle mode. However, in case that the two buckle modes are avoided, the column with long length can be terminated by overall buckling mode. Fig. 1 shows the cross-sectional movement of the buckled C-column.

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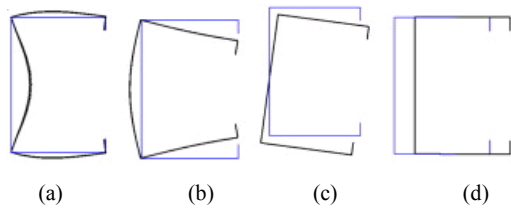
Fig. 1 Buckling modes: (a) local buckling, (b) Distortional buckling
(c) Overall torsional buckling (d) Overall flexural buckling

For irregular section cold-formed steel columns, the buckling behavior is quite complicated. The normal stress distribution under concentric loading is no longer uniform since the shear center is not the same location of the sectional centroid which means the deviation of the shear center from the centroid. The more the deviation distance, the higher the effect of torsion on buckling capacity of the column.

III. METHOD STATEMENT

A. References

The study is composed of two parts:

1. Experimental Work

Cold-formed column tests with four irregular sections, as shown in Fig. 2, were performed in this part. The columns were made from press braking of 1.5 mm thick steel sheet. Three column lengths of 230, 950 and 1900 mm. were selected representing short, intermediate long and long columns. Totally, the test comprises 12 column specimens. Table I shows geometrical related properties of the columns and the terms x_o and y_o exist which means the deviation of the shear center from the centroid and Fig. 3 shows stress-strain relation from coupon test.

2. Finite Element Analysis

This working part contains finite element analysis. First, the 12 tested columns in part (a) were analyzed and the finite element models were validated through agreement of the two results. Then, effect of thickness was investigated varying the thickness of 1.0, 1.15, 1.2, 1.5, 1.6 and 2.0 mm.

B. Column Tests

The test set up is shown in Fig. 4. The upper end of column was fixed between welded plated and loading frame and concentric load was incrementally applied at the bottom end using hydraulic jack. Hence, the experimental set up implies fix-hinge column. Deformation measurements included column axial deformation, lateral movement at mid-height. Strain gauges were also attached at the level.

C. Finite Element Analysis

Eight-node shell element was used in this study. Each node contains 6 degree-of-freedom i.e. x, y, z-translations and x, y, z-rotations. Fig. 5 shows finite element model of short column. Two end plates with thickness of 20 mm. were modeled continuously imitating welding joint of the test columns. At upper end, the centriodal point of section was

completely fixed. For the lower end, rigid link was introduced for the transferring steel beam. The continuity between the link element and bottom plate was maintained and boundary condition at the loading point was hinged. Displacement was incrementally controlled at lower node of the rigid link element.
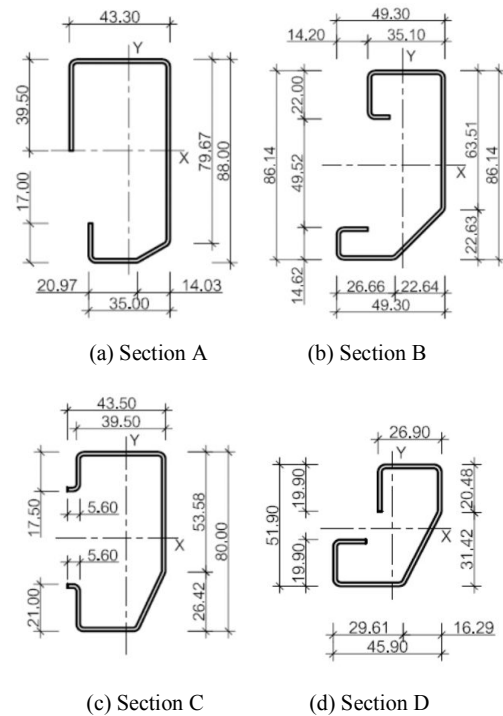

(a) Section A (b) Section B

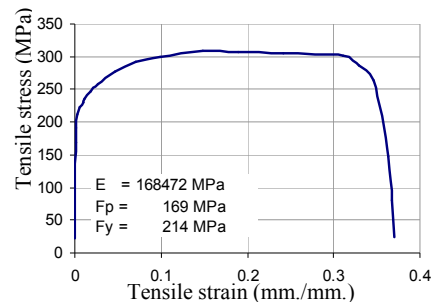
(c) Section C (d) Section D

Fig. 2 Column sections (Unit: mm)

Fig. 3 Tensile stress-strain relation of the steel from coupon test

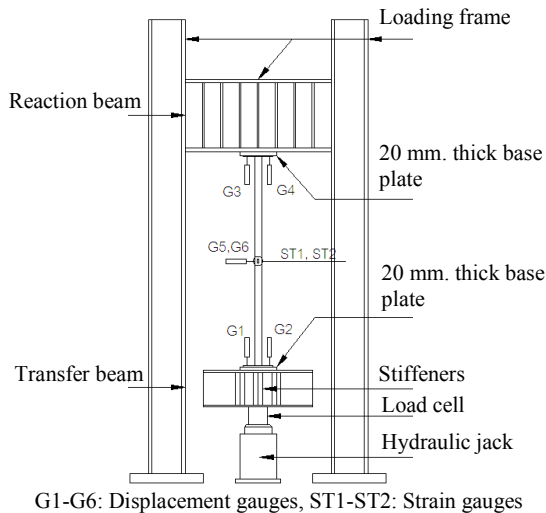G1-G6: Displacement gauges, ST1-ST2: Strain gauges

Fig. 4 Experimental set up

IV. RESULTS

A. Buckling Capacity

Table II shows buckling capacity and corresponding failure mode obtained from column tests and finite element analyses. The two results are well agreed in which the test ones are slightly higher for the short and intermediate long columns. On the contrary, the finite element analyses give higher column capacity in the long columns. The average ratio of P_{FEM}/P_{test} is about 1.001.

B. Buckling Behaviors

1. Short Columns

With the increase of axial loading magnitude, the short columns experienced first local buckling phenomena and followed by distortional mode, as seen in Fig. 6. This is due to the fact that, the length of columns are short enough to prevent overall buckling mode and shear center deviates from the section centroid, as seen in Table I. With the presence of distortional buckling mode, the loading capacity dropped immediately.

TABLE I
PROPERTIES OF COLUMN SECTIONS

Section	A	I_x	I_y	x_o	y_o	$(KL/r)_x$ with different "L" *			$(KL/r)_y$ with different "L" *		
	mm^2	mm^4	mm^4	mm	mm	230 mm	950 mm	1900 mm	230 mm	950 mm	1900 mm
A	307.35	296458	91694	38.87	29.08	5.18	21.41	42.82	9.32	38.50	77.00
B	304.98	313606	83735	43.23	1.26	5.02	20.74	41.48	9.72	40.13	80.27
C	277.85	267314	69803	39.59	6.06	5.19	21.44	42.88	10.16	41.96	83.91
D	230.59	83752	51044	31.91	14.37	8.45	34.89	69.79	10.82	44.70	89.39

TABLE II
ULTIMATE CAPACITIES AND FAILURE MODES

Specimen*	Test results		Finite element analysis		P_{FEM}/P_{test}
	P_{test}	Failure mode**	P_{FEM}	Failure mode**	
A230	53,900	L/D	52,996	L/D	0.983
A950	54,020	D/FT	52,008	L/F	0.963
A1900	36,500	FT	38,662	FT	1.059
B230	68,810	L/D	60,972	L/D	0.886
B950	60,020	FT	55,045	F	0.917
B1900	45,560	FT	50,952	FT	1.118
C230	63,620	L/D	58,916	L/D	0.926
C950	58,150	D/F	48,228	D/F	0.829
C1900	32,370	FT	42,847	FT	1.324
D230	52,360	L/D	49,005	D	0.936
D950	40,230	FT	37,742	F	0.938
D1900	21,980	FT	24,927	F	1.134
				average	1.001

* Specimen nomenclature start with section type (A B C or D) and followed by column length. For example, A950 means column section A with 950 mm long.
** Failure modes: L = local buckling, D = Distortional buckling, F = Flexural buckling, FT = Flexural-torsional buckling

TABLE III
COMPARISONS OF AXIALLY LOAD CAPACITY BETWEEN FINITE ELEMENT ANALYSIS AND AISI CALCULATED VALUES

		Section A			Section B			Section C			Section D		
	Specimen	P_{FEM}	P_n	P_{FEM}/P_n	Specimen	P_{FEM}	P_n	P_{FEM}/P_n	Specimen	P_{FEM}	P_n	P_{FEM}/P_n	Specimen	P_{FEM}	P_n	P_{FEM}/P_n
Short column (L = 230 mm.)	A230t1.0	32,053	33,172	0.966	B230t1.0	36,112	40,042	0.902	C230t1.0	34,180	37,345	0.915	D230t1.0	31,604	32,054	0.986
	A230t1.15	39,478	40,203	0.982	B230t1.15	43,761	47,164	0.928	C230t1.15	41,198	43,912	0.938	D230t1.15	37,505	37,186	1.009
	A230t1.2	41,279	42,580	0.969	B230t1.2	47,872	49,575	0.966	C230t1.2	43,424	46,124	0.941	D230t1.2	39,392	38,905	1.013
	A230t1.5	56,437	57,374	0.984	B230t1.5	61,145	64,313	0.951	C230t1.5	57,526	58,854	0.977	D230t1.5	50,936	49,107	1.037
	A230t1.6	61,084	62,534	0.977	B230t1.6	68,867	69,287	0.994	C230t1.6	62,032	62,777	0.988	D230t1.6	55,011	52,381	1.050
	A230t2.0	83,421	83,969	0.993	B230t2.0	89,108	86,817	1.026	C230t2.0	80,358	78,472	1.024	D230t2.0	68,910	65,477	1.052
		Avg.	0.979			Avg.	0.961			Avg.	0.964			Avg.	1.024
Intermediate long column (L = 950 mm.)	A950t1.0	29,346	31,169	0.942	B950t1.0	36,180	37,338	0.969	C950t1.0	34,473	34,761	0.992	D950t1.0	30,592	28,842	1.061
	A950t1.15	37,680	37,745	0.998	B950t1.15	44,602	43,985	1.014	C950t1.15	42,477	40,870	1.039	D950t1.15	35,183	33,468	1.051
	A950t1.2	39,559	39,952	0.990	B950t1.2	46,975	46,235	1.016	C950t1.2	45,133	42,927	1.051	D950t1.2	36,716	35,017	1.049
	A950t1.5	54,202	53,842	1.007	B950t1.5	65,736	59,964	1.096	C950t1.5	57,901	54,472	1.063	D950t1.5	46,135	44,084	1.047
	A950t1.6	59,539	58,679	1.015	B950t1.6	67,095	64,371	1.042	C950t1.2	45,133	42,927	1.051	D950t1.6	48,407	47,037	1.029
	A950t2.0	85,827	78,738	1.090	B950t2.0	86,897	80,525	1.079	C950t1.5	57,901	54,472	1.063	D950t2.0	61,829	58,873	1.050
		Avg.	1.007			Avg.	1.036			Avg.	1.043			Avg.	1.048
Long column (L = 1900 mm.)	A1900t1.0	30,765	25,616	1.201	B1900t1.0	35,081	30,014	1.169	C1900t1.0	33,272	27,793	1.197	D1900t1.0	25,741	20,813	1.237
	A1900t1.15	37,800	30,902	1.223	B1900t1.15	41,513	35,411	1.172	C1900t1.15	37,740	32,707	1.154	D1900t1.15	29,502	24,218	1.218
	A1900t1.2	38,624	32,723	1.180	B1900t1.2	43,183	37,236	1.160	C1900t1.2	39,380	34,271	1.149	D1900t1.2	30,889	25,312	1.220
	A1900t1.5	54,710	44,160	1.239	B1900t1.5	55,523	47,874	1.160	C1900t1.5	48,111	43,084	1.117	D1900t1.5	38,640	31,982	1.208
	A1900t1.6	58,358	48,131	1.212	B1900t1.6	55,970	51,019	1.097	C1900t1.6	51,318	46,055	1.114	D1900t1.6	41,216	34,248	1.203
	A1900t2.0	72,930	63,181	1.154	B1900t2.0	69,940	64,483	1.085	C1900t2.0	63,885	58,115	1.099	D1900t2.0	51,467	43,538	1.182
		Avg.	1.202			Avg.	1.140			Avg.	1.138			Avg.	1.211

Nomenclature starts with section type and followed by column length (mm) and thickness (mm).

Fig. 5 Finite element model
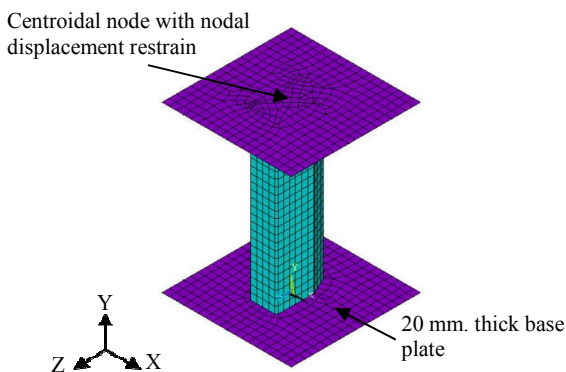
Centroidal node with nodal displacement restrain

20 mm. thick base plate

2. Intermediate Long Columns

The test results of these columns show flexural torsional buckling mode. However, specimen with section composing a flexible element without edge stiffener (A950) incorporated distortional mode. The failure started with distortional buckling of the end elements and then followed by flexural-torsional buckling, as can be seen in Fig. 7.

3. Long Columns

The failure of the tested long columns is shown in Fig. 8. Ultimate capacity of the column attained mainly by the presence of overall buckling. Sectional distortional could be observed especially for A-section column.

Figs. 9-12 show normalized compressive load-axial deformation responses of the tested columns. P_y is yielding loading based on uniform yielding stress on full cross-sectional area and δ_y is yielding shortening based on material yielding strain. Short columns excepting A-section column were failed after yielding. The longer columns indicated less ductile behavior after peak load.

C. Effect of Thickness

With the increase in wall thickness, the cross-sectional area and second moment of area are increased (Table I). However, the deviation between the area centroid and the shear center are not changed. This makes sections become more compacted. Figs. 13 and 14 show the effect of thickness of short and intermediate long columns on compressive loading capacity. Ultimate compressive stress in y-axis defined by the ultimate compressive load divided by cross-sectional area of the columns was increased with the higher thickness of the walls. The effect was obviously seen at the thinner ranges, less than 1.6 mm. For the long columns, as seen in Fig. 15, with thickness less than 1.6 mm, the ultimate compressive stress increased with the increase of thickness for A1900 column. However, the enhancement could not be seen for other long columns. This is due to the columns were compacted section and overall column buckling governed the failure mode of the columns.
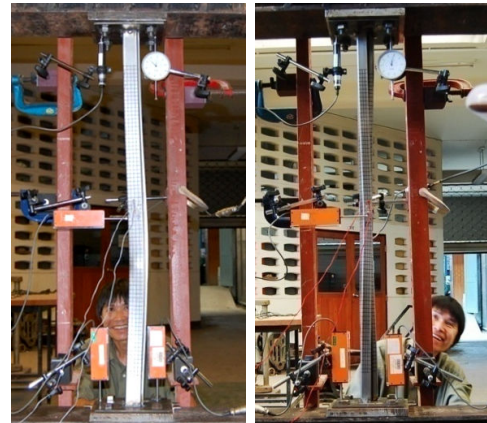
(a) A230 (b) B230
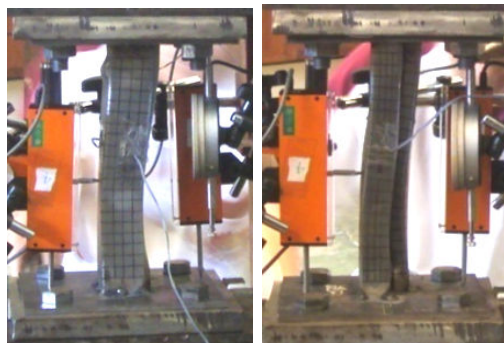
(c) C230 (d) D230

Fig. 6 Short column failure

(a) A950 (b) B950

(c) C950 (d) D950

Fig. 7 Intermediate long column failure

(a) A1900 (b) B1900
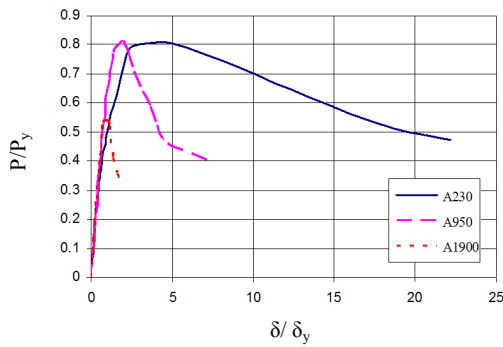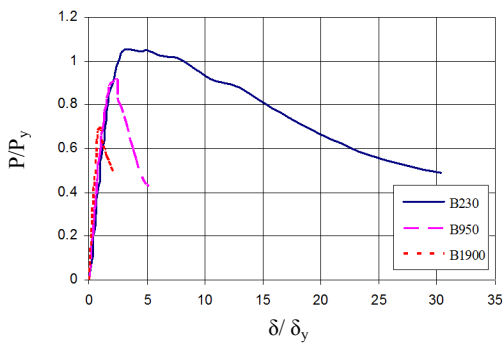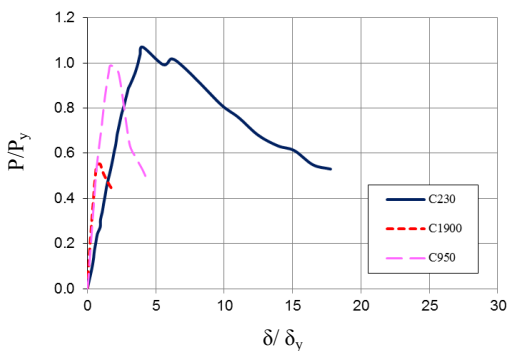

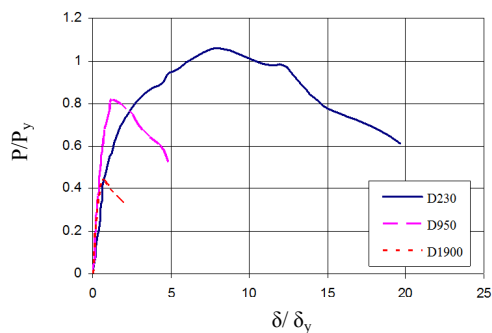
(c) C1900 (d) D1900

Fig. 8 Long column failure

Fig. 9 P/P_y and δ/δ_y relationships of A-section columns

Fig. 10 P/P_y and δ/δ_y relationships of B-section columns

Fig. 11 P/P_v and δ/δ_v relationships of C-section columns

Fig. 12 P/P_y and δ/δ_y relationships of D-section columns
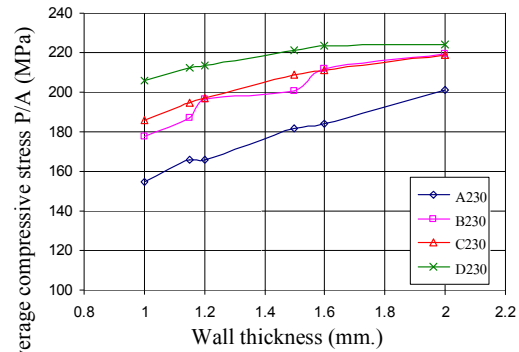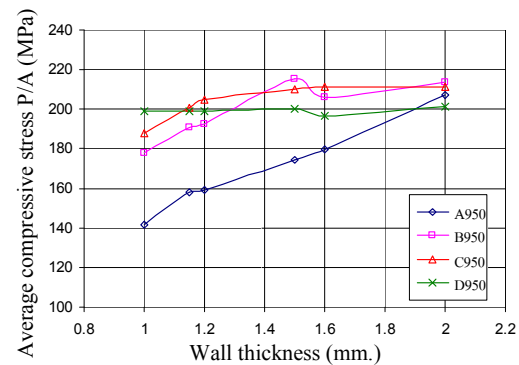

Fig. 13 Effect of thickness of short columns

Fig. 14 Effect of thickness of intermediate long columns
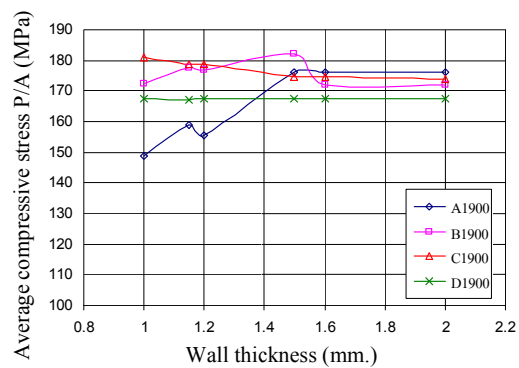

Fig. 15 Effect of thickness of long columns

D. Calculated Axial Capacity

The results of ultimate axial loads from finite element analysis with varying wall thickness are compared with the calculated values based on AISI standard [5], as shown in Table III. The differences between the two values, P_{FEM}/P_n are 0.979, 0.961, 0.964 and 1.024, respectively for the columns having sections A, B, C and D.

V. Conclusion

This research conducts 12 tests for buckling behavior of irregular section cold-formed steel columns under concentric loading and 72 nonlinear finite element analyses of the buckling behavior with varying wall thickness of 1.0, 1.15,

1.2, 1.5, 1.6 and 2.0 mm. Buckling mode depends very much on sectional geometry, stiffeners, deviation of the centroid and shear center and column length. Short columns with the cancroid and shear centers are coinciding, local buckling is dominated with ultimate compressive stress is higher than yielding strength. However, distortional becomes appearance when the stiffeners are not provided. For intermediate long column, combination between local buckling and distortional buckling can be seen. Thicker wall can enhance the ultimate capacity of the short and intermediate long columns. For long column, overall buckling governs the failure mode and thicker wall pays a little role in the capacity enhancement.

REFERENCES

[1] Young, B. and Chen, J. (2008). "Column Tests of Cold-formed Steel non Symmetric Lipped Angle Section", Journal of Constructional Steel Research, 64: 808-815.
[2] Narayanan, S. and Mahendran, M. (2003). "Ultimate capacity of innovation cold-formed steel columns", Journal of constructional steel research, 59: 480-508.
[3] Standards Australia. (1996). Australian/ New Zealand Standard AS/NZS4600. Cold formed steel structures. Sydney.
[4] Ellobody, E. (2007). "Buckling analysis of high strength stainless steel stiffened and unstiffened slender hollow section columns", Journal of Constructional Steel Research, 63: 145-155.
[5] American Iron and Steel Institute. (2001). North American Specification for the Design of Cold-Formed Steel Structure Member. Washington, DC : American Iron and Steel Institute
[6] AS/NZS. Cold-formed stainless steel structures. Australian/New Zealand Standard, AS/NZS 4673:2001. Sydney (Australia): Standards Australia; 2001.
[7] EC3. Eurocode 3: Design of steel structures — Part 1.4: General rules — Supplementary rules for stainless steels. European Committee for Standardization, ENV 1993-1-4. Brussels: CEN; 1996.
[8] Freitas, A.M.S., Freites, M.S.R and Souza, F. T. (2005). "Analysis of steel storage rack columns", Journal of Constructional Steel Research, 61: 1135-1146.
[9] Nawikkawanit, O. and Hansapinyo, C. (2010) "Dynamic Behavior of Electronic Cabinet" Third International Conference on Geoinformation Technology for Natural Disaster Management & Rehabilitation, GIT4NDM 2010, October 19-20, 2010, Chiang Mai, Thailand.

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