World Academy of Science, Engineering and Technology
International Journal of Civil and Environmental Engineering
Vol:9, No:5, 2015

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

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International Science Index, Civil and Environmental Engineering Vol:9, No:5, 2015 waset.org/Publication/10001207

***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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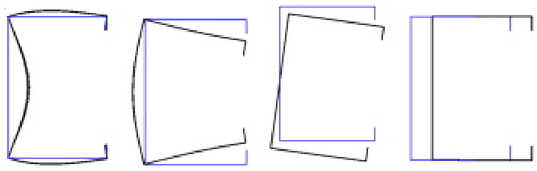


(a) (b) (c) (d)

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

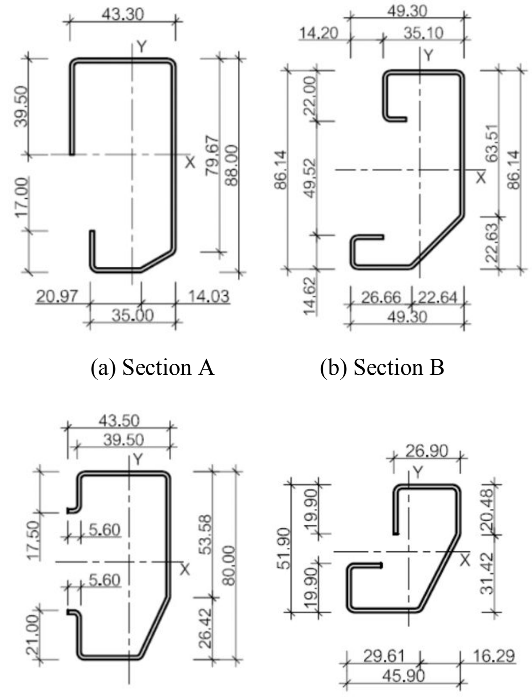
This working part contains finite element analysis. First, the 12 tested columns in part (a) were analyzed and the finite element models were validated through agreement of the two results. Then, effect of thickness was investigated varying the thickness of 1.0, 1.15, 1.2, 1.5, 1.6 and 2.0 mm.

### 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)

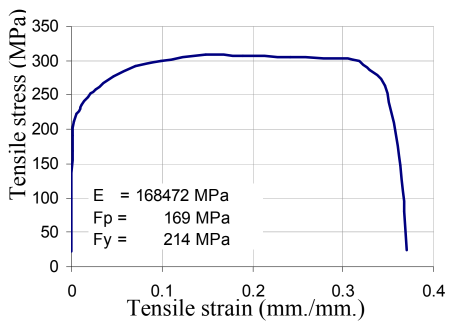



Fig. 3 Tensile stress-strain relation of the steel from coupon test

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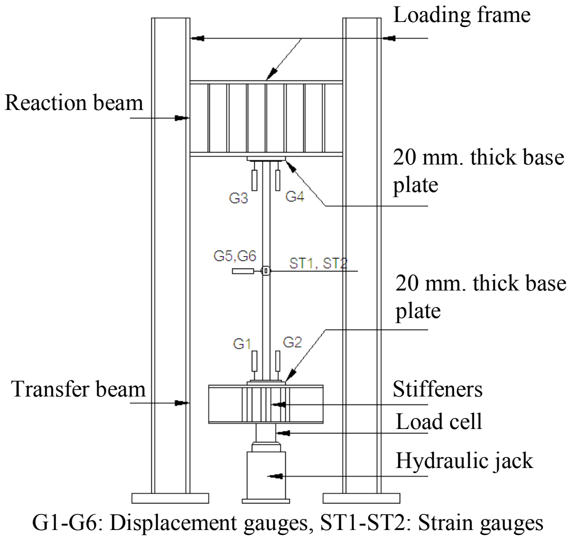


Fig. 4 Experimental set up

## 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

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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).

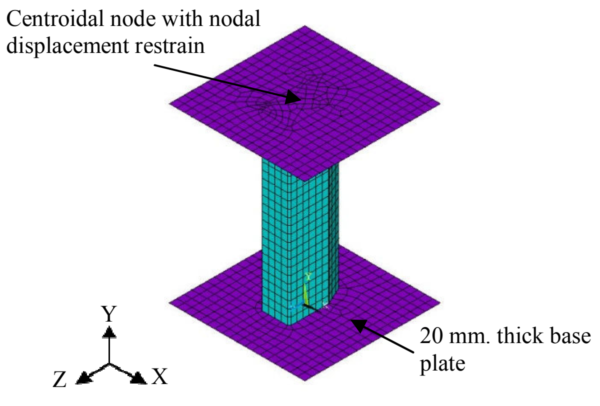


Fig. 5 Finite element model

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 $\delta_y$ 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.

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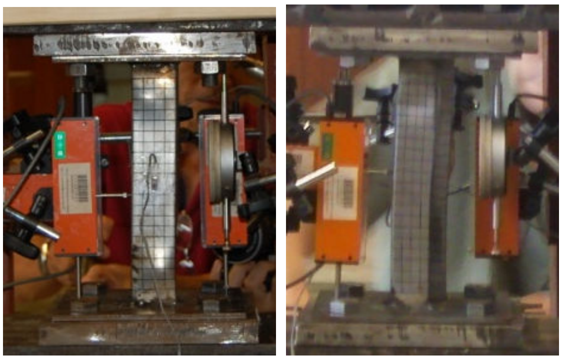
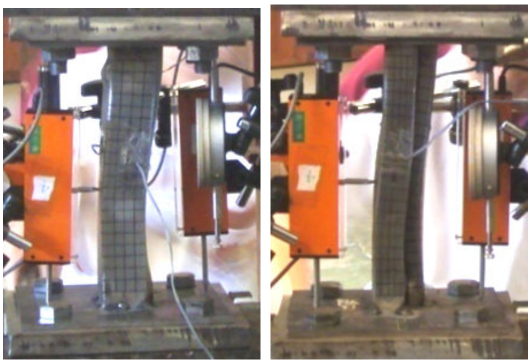


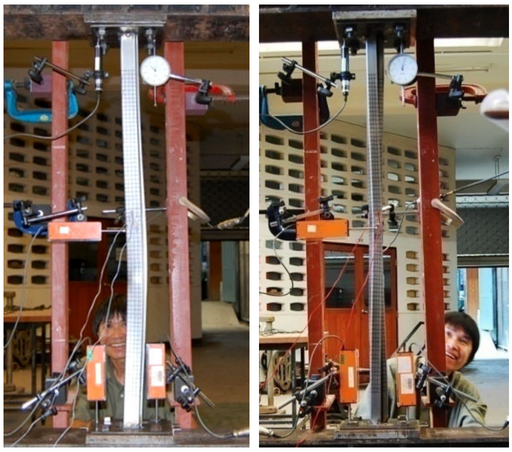
(a) A230 (b) B230



(c) C230 (d) D230

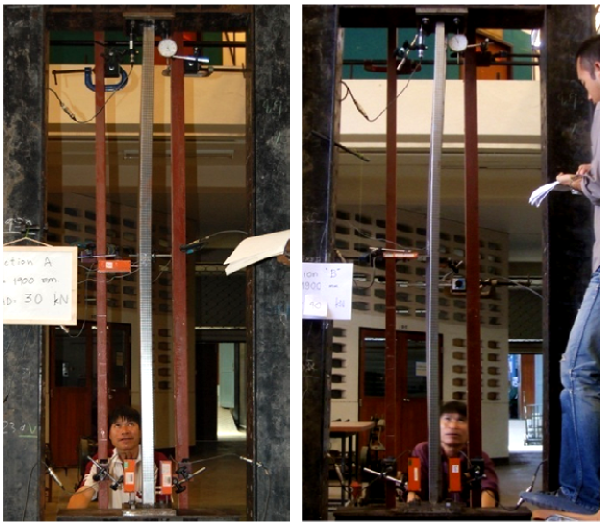
Fig. 6 Short column failure

(a) A950 (b) B950

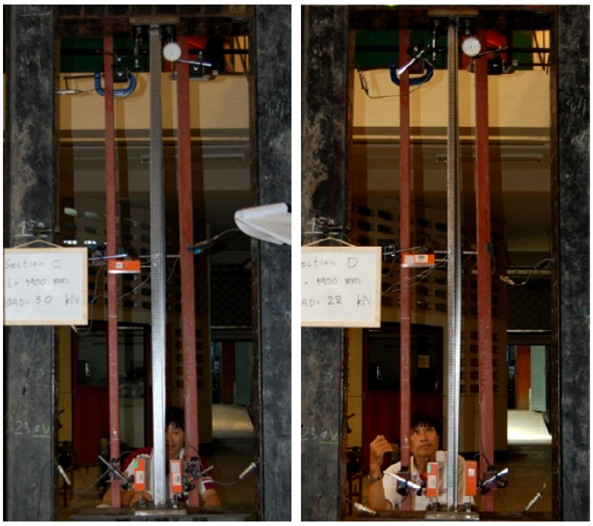


(c) C950 (d) D950

Fig. 7 Intermediate long column failure



(a) A1900 (b) B1900



(c) C1900 (d) D1900

Fig. 8 Long column failure

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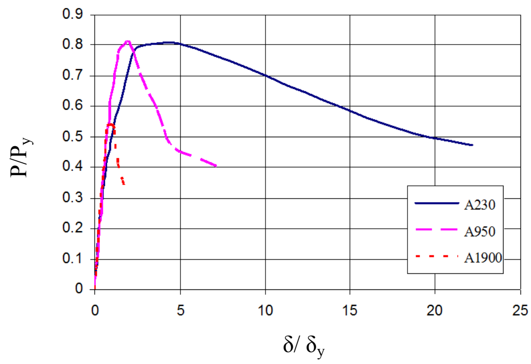


Fig. 9 $P/P_y$ and $\delta/\delta_y$ relationships of A-section columns

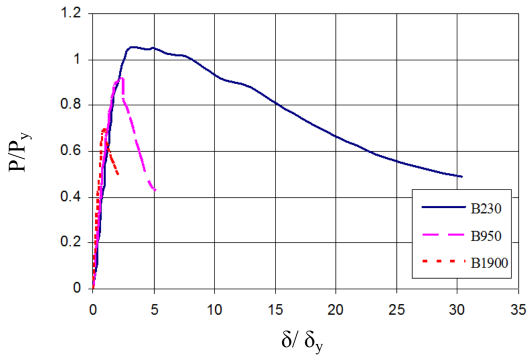


Fig. 10 $P/P_y$ and $\delta/\delta_y$ relationships of B-section columns

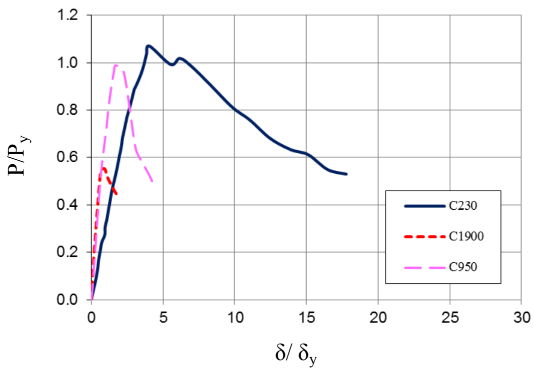


Fig. 11 $P/P_y$ and $\delta/\delta_y$ relationships of C-section columns

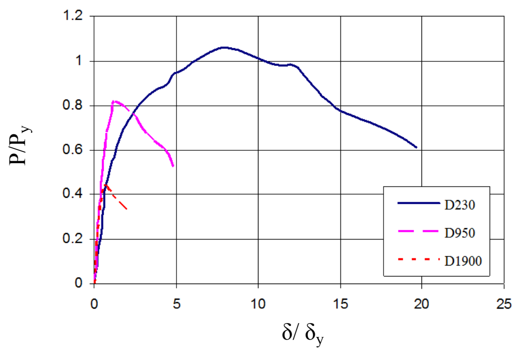


Fig. 12 $P/P_y$ and $\delta/\delta_y$ relationships of D-section columns

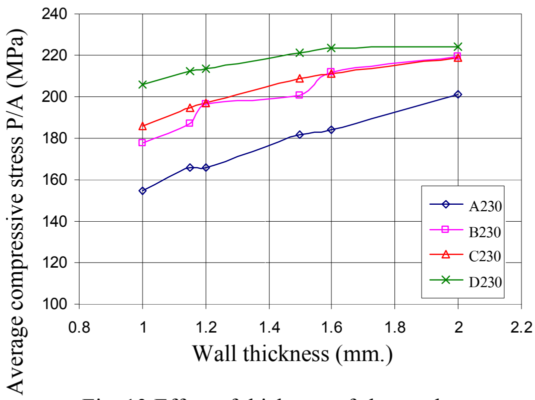


Fig. 13 Effect of thickness of short columns

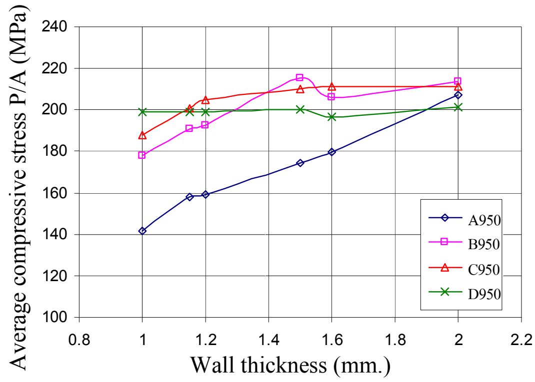


Fig. 14 Effect of thickness of intermediate long columns

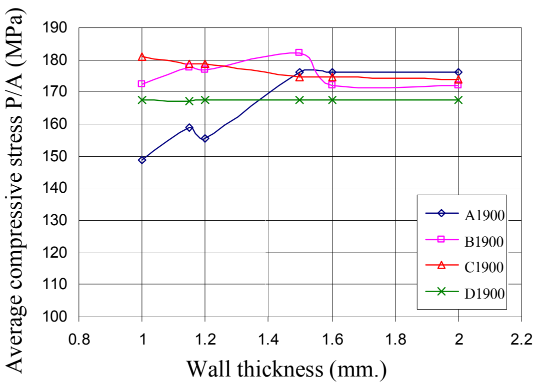


Fig. 15 Effect of thickness of long columns

### *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.

## ACKNOWLEDGMENT

The authors would like to acknowledge office of the Higher Education Commission, Thailand under the National Research University Project for the financial support. Special thanks are given to Prof. Manabu Matsushima for his guidance and ANSYS support and Mr. Nattapol Tansiri for conducting the experiments.

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