INVESTIGATION ON FLEXURAL BEHAVIOR OF COLD-FORMED STEEL C BACK-TO-BACK BEAMS

BY

KIMCHENG KANG

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE (ENGINEERING AND TECHNOLOGY)
SIRINDHORN INTERNATIONAL INSTITUTE OF TECHNOLOGY
THAMMASAT UNIVERSITY
ACADEMIC YEAR 2016
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A Thesis Presented

By

KIMCHENG KANG

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INVESTIGATION ON FLEXURAL BEHAVIOR OF COLD-FORMED STEEL C BACK-TO-BACK BEAMS

by

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Investigation on flexural behavior of cold-formed steel (CFS) C back-to-back beams has been conducted in this study. In the present experiment, the C back-to-back beam specimens were tested under four-point loading. Bolts were used for connecting two C section beams. For the twelve specimens, there are three different cross-sectional dimensions. C10012 (depth = 100 mm, thickness = 1.2 mm), C10015 (depth = 100 mm, thickness = 1.5 mm), and C15015 (depth = 150 mm, thickness = 1.5 mm). For each section, the connection spacings are varied as L/2, L/3, L/4, L/6, where L is overall length of the beam equal to 4 meter. From the experimental results, the influences of the thickness and spacing of the C back-to-back beams were observed, and all specimens failed by lateral torsional buckling (LTB) and distortional buckling (DB). Nonlinear finite element analysis of the beam specimens was performed and compared with experimental results. It was found that the failure modes from finite element analysis comparatively agree with the modes observed in the experiments. For the ultimate load comparison, the difference between experimental and numerical results is less than 21 percent for the section C10012, 27 percent for C10015, and 36
percent for C15015. Hence, improvement of modeling of C back-to-back beams, e.g. bolt connection and support condition, is recommended for further study.

**Keywords:** C back-to-back beam, Cold-formed steel, Flexural Behavior, Finite Element Method.
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Chapter 1

Introduction

1.1 Background

Cold-formed steel is widely applied in the warehouses, factories, automobiles, equipments, utility poles, and storage racks. Many types of cold-formed steel sections were shown in Figure 1.1.

In the 21st century, cold-formed steel is increasingly researched since it has lots of advantages such as light weight, high strength. The thickness ranges from 0.4 mm to 6 mm.

In north America, after America Iron Steel Institute (AISI) specification in 1946 was published, cold-formed steel was more popular in the building construction.

Figure 1.1 Various sections of cold-formed steel
There are two kinds of structural steel members which are hot-rolled steel and cold-formed steel. Hot-rolled steel shapes are produced at high temperature. On the other hand, cold-formed steel is produced from steel coil and formed into the shape through the press-braking method (Figure 1.2) or normally by cold-roll forming machine (Figure 1.3) at room temperature.

Figure 1.2 Press-braking method

Figure 1.3 Cold-roll forming machine
Advantages of cold-formed steel structural members:
- Cold-formed steel elements are lighter than hot-rolled steel shapes and they used for short span.
- Cold-roll forming machine could produce the complicated sections easily and economically
- Cold-formed steel can be used as formwork for composite construction.

If compared with other materials such as: timber and concrete, the benefit of cold-formed steel is:
- High strength
- Lightness
- Fast installation
- Recycling materials
- Attractive appearance
- Low maintenance

Roles of cold-formed steel:
- Individual structure framing members (beams, columns)
- Panels and decks (slab; formwork)

Figure 1.4 showed the application of cold-formed steel for the real structure.
1.2 Section properties of C section

Normally, C was used as purlin, truss, and column in some structure such as: warehouses, villas, factories, and power plants (Figure 1.5). When the section has not enough resistance, built-up C section is needed, and used as columns, beams, and rafters. Moreover, built-up C back-to-back section beam can be easily connected by using bolt or screw at the web of the section without additional plates such as stiffening plates or gusset plates.

![Figure 1.5 C sections in Lysaght catalog](image)

From NS Bluescope Lysaght Limited, there are 16 choices of the C section which range from a depth of 102 mm to 350 mm and a width of 51 mm to 125 mm. The thickness of steel used for the beams ranges from 1.0 mm to 3.0 mm. The cold-formed steel (CFS) products are available custom-cut in any transportable length. Table 1.1 showed the sectional dimensions for the range of commercially available C sections.
Table 1.1 Catalogue of C section

<table>
<thead>
<tr>
<th>Catalogue Number</th>
<th>t (mm)</th>
<th>D (mm)</th>
<th>B (mm)</th>
<th>l (mm)</th>
<th>Mass per unit length (kg/m)</th>
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<td>C10010</td>
<td>1.0</td>
<td>102</td>
<td>51</td>
<td>12.5</td>
<td>1.78</td>
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<td>C10012</td>
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<td>102</td>
<td>51</td>
<td>12.5</td>
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<td>51</td>
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<td>51</td>
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<td>152</td>
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<td>1.5</td>
<td>152</td>
<td>64</td>
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<tr>
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<td>152</td>
<td>64</td>
<td>16.5</td>
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<tr>
<td>C15024</td>
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<td>152</td>
<td>64</td>
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<tr>
<td>C35030</td>
<td>3.0</td>
<td>350</td>
<td>125</td>
<td>30.0</td>
<td>15.23</td>
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1.3 Statement of problem

In most of the past researches, cold-formed steel built-up box sections as cold-formed C face-to-face (Figure 1.6(a)), nested section (Figure 1.6(c)), were studied. In the construction of warehouses and factories, cold-formed steel C back-to-back sections (Figure 1.6(b)) were applied. However, there were a few researches on this type of built-up section. Since the torsional resistance is much smaller than the box section (C face-to-face), more detailed investigation on flexural behavior of the C back-to-back section is needed.
In North American Specification for the Design of Cold-Formed Steel Structural Members, the formula to calculate the strength of cold-formed steel C back-to-back beam is not clearly specified. In this AISI, there is only the guideline to calculate the maximum spacing for applying the cold-formed steel C back-to-back beam.

For the connection type of cold-formed steel, the weld connection should not be used, since heating may destroy the coating material of cold-formed steel. As shown in Figure 1.7, bolt connection was normally used to connect two C-sections to form built-up I-shape section.
1.4 Purpose of the study

The purpose of this study is to investigate the flexural behavior of cold-formed steel C back-to-back beam by using bolt connection on the web plates. The influence of the connection spacing of the bolts, and thickness of the section on the ultimate load and failure mode of the C back-to-back beams are examined. Moreover, the numerical simulation using ABAQUS program is performed, and the comparison between experimental and numerical results is made.

The tests were conducted on built-up open sections with different bolts spacings. The specimens were loaded with four points loading to produce uniform bending in the middle span with the length of beam = 4 m. The sections are C10012, C10015, and C15015. The connection spacings are L/2 = 2,000 mm, L/3 = 1,333 mm, L/4 = 1,000 mm, and L/6 = 667 mm.
Chapter 2

Literature review

2.1 General

The literature review in this chapter provides the background and the current design practice, experimental and numerical studies relevant to investigate on flexural behavior of cold-formed steel C back-to-back beam. There are two sections: a brief review of design specification of CFS structural members and previous researches on experimental and numerical studies of built-up C section.

2.2 Method to calculate the design strength of cold-formed steel beam.

Normally, cold-formed steel such as Z section and C section are applied as purlin or beam. The built-up sections made of back-to-back C-sections, nested C-sections forming a box girder or face-to-face C-sections are introduced when single C sections are not enough for ultimate loads or allowable deflection. The strength of the beam might be limited by lateral torsional buckling, distortional buckling or local buckling of the beam depending on the geometry of the section, the type of actions and lateral support given (Figure 2.1). Because of very small thickness, the cold-formed flexural members are also failed by distortional buckling in contrast to hot-rolled sections. The failure mode of buckling of cold-formed steel C section from the experiment is clearly illustrated in Figure 2.2.
Figure 2.1 Buckling failure mode of CFS C section

Figure 2.2 Buckling of CFS C section
The current North American Specification for the Design of Cold-Formed Steel Structural Members (AISI, Cold-formed steel, American Iron and Steel Institute, 2012) provides two methods for calculating the design strength of cold-formed steel beam: effective width method and direct strength method.

2.1.1 Effective width method

AISI, North American Specification for Design of Cold-Formed Steel Structural Members, 2012 edition, provides the formula to calculate the section strength of flexural single members. The nominal section strength for the initiation of yielding is calculated by using the following equation:

\[ M_n = M_y = S_e F_y \]  

(2.1)

Fy= design yield stress, and Se= effective elastic section modulus. Effective elastic section modulus Se is calculated based on the effective width of individual elements of the section under design yield stress.

In the design of cold-formed steel flexural members, the moment-resisting capacity of the member could be limited by lateral buckling of the beam, particularly when the open section is fabricated from thin material and laterally supported at relatively large intervals. Cold-formed steel beams generally fail due to material yielding, local buckling, distortional buckling, and lateral torsional buckling.

2.1.2 Direct strength method

Based on the AISI, North American Specification for Design of Cold-Formed Steel Structural Members, 2012 edition, nominal flexural strength of cold-formed steel member \( M_n \) is equal to min \( (M_{ne}, M_{ni}, M_{nd}) \).

- \( M_{ne} \): lateral torsional buckling strength
  - For \( M_{cre} < 0.56M_y \) : \( M_{ne} = M_{cre} \)  
    (2.2)
  - For \( 2.78M_y \geq M_{cre} \geq 0.56M_y \) : \( M_{ne}= \frac{10}{9} M_y \left(1-\frac{10M_y}{36M_{cre}}\right) \)  
    (2.3)
  - For \( M_{cre} > 2.78M_y \) : \( M_{ne} = M_y \)  
    (2.4)

\( M_{cre} = S_{fcre} \): critical elastic lateral torsional buckling moment
Where, $S_f$: gross section modulus to the extreme compression fiber

$f_{cre}$: elastic critical lateral torsional buckling stress. $f_{cre} = F_e$ of main specification section C3.1.2.2.

$M_y = S_fF_y$: Member yield moment

- $M_{nl}$: Local buckling strength
  - For $\lambda_l = \sqrt{M_{ne}/M_{cre}} \leq 0.776$: $M_{nl} = M_{ne}$ (2.5)
  - For $\lambda_l = \sqrt{M_{ne}/M_{cre}} > 0.776$: $M_{nl} = \left(1 - 0.15 \left(\frac{M_{cre}}{M_{ne}}\right)^{0.4}\right) \left(\frac{M_{cre}}{M_{ne}}\right)^{0.4} M_{ne}$ (2.6)

$M_{ne}$: Lateral torsional buckling strength

$M_{crf} = S_ff_{crf}$: Critical elastic local buckling moment

Where, $f_{crf} = k \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{w}\right)^2$ : local buckling stress at the extreme compression fiber, determined in accordance with Section 1.1.2 of Appendix 1: commentary on Direct Strength Method

- $M_{nd}$: Distortional buckling strength
  - For $\lambda_d = \sqrt{M_y/M_{crd}} \leq 0.673$: $M_{nd} = M_y$ (2.7)
  - For $\lambda_d = \sqrt{M_y/M_{crd}} > 0.673$: $M_{nd} = \left(1 - 0.22 \left(\frac{M_{crd}}{M_y}\right)^{0.5}\right) \left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y$ (2.8)

$M_y = S_fF_y$: Member yield moment

$M_{nd} = S_ff_{crd}$: Critical elastic distortional buckling moment

Where, $f_{crd}$: elastic distortional buckling stress, determined in accordance with Section 1.1.2 of Appendix 1: commentary on Direct Strength Method

For more details, it is available on the AISI, North American Specification for Design of Cold-Formed Steel Structural Members, 2012 edition.
2.2 Past experimental and numerical studies

Luis Laim et al. (2013) researched about the investigation on flexural behavior of cold-formed steel beam with various sections such as: C section, R section, I section and 2R section based on experiment tests and numerical simulations (Figure 2.3, Figure 2.4). Twelve specimens were tested to obtain the maximum load, maximum deflection, and failure mode. Each section was tested the same three times to better results. R section was got from C section and U section connected in the flanges while 2R section consisted of two R sections connected to the web. For the section geometry, the height of C section and U section was 250 mm, and 255 mm, respectively. The thickness of C and U section was 2.5 mm. Self-drilling screw was used to connect the sections. The test set-up schema was illustrated in Figure 2.5. Lateral supports were also needed to prevent torsion. Figure 2.5 showed load-displacement curves of C, I, R and 2R beams. It was observed that during unloading stage, some curves had discontinuities due to failure of the screws. The test result proven the good agreement between the numerical and experimental. The failure buckling modes shown in the numerical analysis are also consistent with that of the tests. Furthermore, so as to evaluate the effect of height, thickness, and length of the beam to the moment capacity of the beam, fifty two finite element models were undertaken.

Figure 2.3 Various built-up sections (Luis Laim et al. (2013))
Cheng Yu et al. (2003) studied about local buckling experiment. The test was shown to fully restrain with distortional buckling and gave the beam to fail with local
buckling for C section and Z section. The experimental result showed the AISI, NAS and S136 design methods gave the suitable strength predictions. The direct strength method gave the best predicted strength for slender and unslender beam among three methods.

L. Laim et al. (2015) have studied about the flexural behavior of beams made of cold-formed steel sigma-shaped sections at ambient and fire condition. Two kinds of section such as single sigma and built-up I section by two sigma sections (Figure 2.6) were tested by four-point bending test. Three repeated tests of one section were considered in their research. The span length of their specimens was 3 meters. Screws with self-drilling was used as the connection between two of the sigma sections.

![Figure 2.6 Cross sections of sigma and 2-sigma (Luis Laim et al. (2015)).](image)

Restrain system of roller and pinned support of their study was shown in Figure 2.7. Moreover, lateral deflection was restrained at the position of the support.

![Figure 2.7 Beam supports (L. Laim et al. (2015))](image)
Their results show that the load capacity of the beam is affected by the shape of the section and the ultimate load of built-up I of two sigma sections was 2.9 times higher than the single sigma section. It means that symmetry section of 2 sigma was better than the asymmetry section of one sigma section. Local, distortional, and lateral torsional bucking were found for built-up I section with two sigma as shown in Figure 2.8.

![Figure 2.8 Failure mode of 2 sigma beam (L. Laim et al. (2015)).](image)

Cheng Yu et al. (2016) investigated on distortional buckling experiment. When the compression flange of the beam is not laterally braced over a distance, 1.626 m in constant moment shows that distortional buckling is most failed. An average loss, 17% when laterally unrestrained C and Z section compared to the same beam which the section was restrained and failed in local buckling.

Liping Wang and Ben Young (2015) investigated on flexural behavior of cold-formed steel C back-to-back and built-up box beams with holes at mid span. All forty-three beams with different ten section dimensions and holes were tested to obtain the maximum force and failure modes. The test set-up with four-point bending was shown in Figure 2.9. C back-to-back section composed two C sections connected in the web (Figure 2.10(a)) while built-up box section was gotten from two U sections connected by screws at the top and bottom flanges (Figure 2.10(b)). Bolts were installed when the beam was connected to the loading bearing plates and support bearing plates. The screw diameter was 4.8 mm. The proof stress at 0.2% used in the test were 450, 500, and 550 MPa. The different thickness with 0.42, 1.2, and 1.9 mm
was observed. The overall length of the beam was 1600 mm. The ranges of web depth were from 88 mm to 138 mm. The different ratio between hole diameter and web depth 0.25, 0.5, and 0.7. In the result, the reduction of maximum moment with a maximum value of 6% when \( d_h/h_w \) was increased from 0.25 to 0.5. The reduction of
maximum moment with a maximum value of 16% when $d_w/h_w$ was increased from 0.25 to 0.7. It was observed from the test that some beams were failed by distortional buckling (D), local buckling (L), and flexural buckling (F). Some beams were failed by local buckling (L), and flexural buckling (F) while some beams were failed by local buckling (L), and distortional buckling (D). Direct strength method (DSM) in North American Specification (NAS) was compared with the experimental test. In the result, DSM could be used to calculate the strength of C back-to-back and built-up box beams with holes.

In Australian and New Zealand code, the properties of cold-formed steel are determined as following step. First, the specimen will be taken longitudinally from a main flat part of the section (not include the corner) and the specimen will be taken from the flat part with the smallest yield stress increase from cold-roll forming machine. Then, yield stress ($f_y$) and tensile strength ($F_u$) were designed by using AS 1391.

C and Z sections (Hancock et al.) are usually applied for the beams. C section is singly symmetric and Z section is point symmetric. In the case of full lateral constraint, C section, and Z section will have flexure in a plane parallel with the web and bending theory can be used to calculate the normal stress and shear stress in the section. In case of C and Z sections have the same dimensions so that the shear stress distributions are similar the same.

The bolt connection behavior in cold-formed (CFS) steel structure (Wei-Wen Yu et al.) is unlike from that in hot-rolled steel heavy structure. There are four failure modes of bolted connection: longitudinal plate shearing, bearing of sheet in front of the bolt, tearing of the plate in the net section, and shear of the bolt. For the limitation of the thickness, AISI specification shall be applied when the thickness is less than 4.8 mm. In contrast, when the thickness is not less than 4.8 mm, AISC specification shall be used.

Haiming W. et al. (2009) investigated on cold-formed steel section beams with inclined, upright and complicated lips in experiment and finite element method. It was observed that some beams were failed by distortional buckling. Some beams were failed by local buckling, and some beams were failed by interaction between local buckling and distortional buckling. The section lip has much influence on the strength.
of the beam. The flexural strength of upright lip beams is more than that of inclined lip beam in both fourpoint loading and non four point loading tests. It is a good agreement between experimental and numerical results.

Self-piercing rivet connections, pop rivet connections, and screw connection of similar thickness of two layers of steel 1.0, 1.2, 1.6, and 2.0 mm thick have been tested by Sinha et al. (1999). Their conclusions is that the corresponding load displacement curve are in the large of variation between those different types of connections. Press joins gave a nonlinear behavior in smaller range of load. Self-piercing shows high initial stiffness behavior than the other type of connection. Whereas, self-tapping screws shows low initial stiffness behavior. Self-piercing rivets show a high peak load and high ductility.

Jenitha G. et al. (2016) analysis of cold-formed steel C section beams and built-up beams. Two cold-formed C section beams, two cold-formed steel C back-to-back beams and one hot-rolled steel I beam were tested to obtain the maximum load and deflection at maximum load. The beams were also modelled in ANSYS 14 program in order to compare with experimental results. The maximum load of cold-formed steel C back-to-back beam was higher compared to the normal beam. Cold-formed steel beams are better than hot-rolled steel beam such as: thermal insulation, easy moulding, saving and light weight nature.
Chapter 3
Experimental Study

3.1 Type of Connection

In this study, bolts were used for connections cold-formed steel C back-to-back section on the web plates. It is noted that when cold-formed steel C back-to-back beam was used, the thickness of the web of the built-up C back-to-back section became double of the single C section. Bolt joints were suitable and effective for applying to the cold-formed steel section with the condition that total thickness should be enough for installation. For the installation, the wrench (Figure 3.1(a)) and the impact wrench (Figure 3.1(b)) are adopted. For the bolts M12 which are practically used to connect both C section beams, the tensile strength was 800 MPa and the diameter was 12 mm as shown in Figure 3.2. The M12 bolts are adopted to make all beam specimens in the present test.

![Wrench and impact wrench](image_url)

a. Wrench  b. Impact wrench

Figure 3.1 Wrench and impact wrench
3.2 Material property

From NS BlueScope Lysaght Limited, Cee sections are roll-formed from GALVASPAN steel complying with AS1397-1993. The cold-formed steel in this test was cold reduced to the required thickness and coated with a zinc alloy.

The yield strength and ultimate strength vary with the thickness of the beam section. For the section with thickness 1.2 mm, yield strength (F_y) was 518 MPa and ultimate strength (F_u) was 599 MPa and for the thickness 1.5 mm, yield strength (F_y) was 523 MPa and ultimate strength (F_u) was 610 MPa.

3.3 Beam specimens

12 specimens were fabricated to form C back-to-back sections with the overall beam length of 4 m. The specimens were subjected to bending on the major x-axis in order to observe the behavior of cold-formed steel C back-to-back beam. For the span length, it was 3.8 m. The distance between R1 at corner top flange to the centre of the bolt is the same as the distance from the bolt to the bolt and from the bolt to R2 at corner bottom flange, as shown in Figure 3.3. The properties of C section were shown in Table 3.1. The cold-formed steel C back-to-back sections used in the present
experiment were IC10012, IC10015 and IC15015 with the spacing $L/2 = 2,000$ mm, $L/3 = 1,333$ mm, $L/4 = 1,000$ mm and $L/6 = 667$ mm, where $L$ was the overall length of the beam (Table 3.2). For the specimen IC10012L/2, it refers to cold-formed steel C back-to-back with the depth was 100 mm, the thickness 1.2 mm, and connector spacing $L/2$.

Table 3.1 Properties of C section

<table>
<thead>
<tr>
<th>Catalogue Number</th>
<th>t (mm)</th>
<th>D (mm)</th>
<th>B (mm)</th>
<th>l (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C10012</td>
<td>1.2</td>
<td>102</td>
<td>51</td>
<td>12.5</td>
</tr>
<tr>
<td>C10015</td>
<td>1.5</td>
<td>102</td>
<td>51</td>
<td>13.5</td>
</tr>
<tr>
<td>C15015</td>
<td>1.5</td>
<td>152</td>
<td>64</td>
<td>15.5</td>
</tr>
</tbody>
</table>

![Figure 3.3. C back-to-back section C15015.](image)

Table 3.2 Beam specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Section</th>
<th>Connection spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>IC10012L/2</td>
<td>C10012</td>
<td>L/2</td>
</tr>
<tr>
<td>IC10012L/3</td>
<td>C10012</td>
<td>L/3</td>
</tr>
<tr>
<td>IC10012L/4</td>
<td>C10012</td>
<td>L/4</td>
</tr>
<tr>
<td>IC10012L/6</td>
<td>C10012</td>
<td>L/6</td>
</tr>
<tr>
<td>IC10015L/2</td>
<td>C10015</td>
<td>L/2</td>
</tr>
<tr>
<td>IC10015L/3</td>
<td>C10015</td>
<td>L/3</td>
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<tr>
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<td>C10015</td>
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<tr>
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<td>L/6</td>
</tr>
<tr>
<td>IC15015L/2</td>
<td>C15015</td>
<td>L/2</td>
</tr>
<tr>
<td>IC15015L/3</td>
<td>C15015</td>
<td>L/3</td>
</tr>
<tr>
<td>IC15015L/4</td>
<td>C15015</td>
<td>L/4</td>
</tr>
<tr>
<td>IC15015L/6</td>
<td>C15015</td>
<td>L/6</td>
</tr>
</tbody>
</table>
3.4 Test set-up

The present tests of all beam specimens were conducted at Civil Engineering Laboratory, Chulachomklao Royal Military Academy (CRMA). For the test set-up, both C sections with the overall 4 m length, were connected together at both webs by two bolts with the spacing equal to L/2, L/3, L/4 and L/6. For the connection near support (Figure 3.4), one spacing of bolts was very small in order to make the support more stiffened. The experimental installation of the back-to-back built-up C beams was illustrated in Figure 3.4. The beam was loaded at two bearing plates 1.2 m from each support of the beam to create a pure bending moment in the middle without shear force. The four-point bending test set-up was shown in Figure 3.4. The loading was applied by a hydraulic jack (no. 1 in Figure 3.4) which was connected to


Figure 3.4 Real set-up of cold-formed steel C back-to-back beam
hydraulic pump (Figure 3.7) and was hung from steel frame (no. 10 in Figure 3.4). To control the applied load during the test, a load cell of 50 kN capacity was attached beneath the hydraulic jack and connected directly to the data logger (no. 2 in Figure 3.4). In order to transfer the loading from hydraulic jack to the tested beam, I steel beam (no. 3 in Figure 3.4) was used and applied at two points on the test beam. Moreover, the loading bearing plates (no. 4 in Figure 3.4) was also put under the I steel beam in order to distribute the concentrated loading along the test beam. A spherical plain bearing was used to make a roller support condition to prevent the vertical displacement of the beams (no. 8 in Figure 3.4) while other side of support was also fixed by clamp (no. 11 in Figure 3.4) to prevent the horizontal displacement of the beams, namely pinned support (no. 9 in Figure 3.4 and in Figure 3.5). The vertical displacement was measured using linear variable displacement transducer LVDTs of 10 cm, maximum displacement capacity (no. 7 in Figure 3.4 and Figure 3.6) and the lateral displacement was measured using LVDT of 5 cm, maximum displacement capacity (no. 12 in Figure 3.4). Longitudinal strain gauges were attached on the top and bottom flange (no. 6 in Figure 3.4). All data measurement was recorded by using a data logger (Figure 3.8).
Figure 3.6 Detail of measurement LVDTs

Figure 3.7 Hydraulic pump

Figure 3.8 Data logger
In the first trial of this experiment, the support condition with no lateral restraint were installed at both ends of the beam specimens in Figure 3.9. It was observed that the beam failed at very small level of load due to lateral-torsional buckling in Figure 3.9, due to the low torsional resistance of such an open section (I-shape). Therefore, in the following test series, the lateral supports (Figure 3.10) are provided at both ends of the beam specimen to prevent the torsional deformation at these end supports. It is noted that the conditions of no torsional deformation are commonly assumed at the simply-supported ends.

![Figure 3.9 Failure mode with torsion](image1)

![Figure 3.10 Lateral support of the beam](image2)
3.5 Test procedure

Four-point bending tests were conducted to obtain the ultimate load capacity of the CFS C back-to-back beams. Steps of test procedure were described as follows: The two CFS C sections were assembled by bolt connection to form C back-to-back built-up beams. During the bending test, vertical deflections on the left and right bottom flanges and lateral deflection on the web were measured by LVDT (Figure 3.11(a)). Four strain gauges for measurement of longitudinal strain were attached at the mid-span section on the top flange and bottom flange (Figure 3.11(b)). Then, the load was applied gradually under loading control by mean of hydraulic pump until the specimen failed where the lateral displacement or rotation or deflection was too large and load was immediately dropped. For each load step, the displacements of the beams and strain gauges in the beams were recorded.

![Diagram with LVDT and strain gauge at mid-span](image)

Figure 3.11. Location of LVDT and strain gauge at mid-span

3.6 Test result

There were 12 tested specimens with different cross-sections and connection spacings. Table 3.3 showed summary of the experiment results: maximum load,
vertical deflection at the maximum load, and failure modes. From the test results, all specimens were failed by lateral torsional buckling (LTB) for section C-1 and distortional buckling (DB) for section C-2.

Table 3.3 Summary of the experimental results

<table>
<thead>
<tr>
<th>No</th>
<th>Specimen name</th>
<th>Max. Load (kN)</th>
<th>Vertical Deflection at max. load (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>IC10012L/2</td>
<td>8.89</td>
<td>47.78</td>
<td>LTB</td>
</tr>
<tr>
<td>2</td>
<td>IC10012L/3</td>
<td>8</td>
<td>40.7</td>
<td>LTB</td>
</tr>
<tr>
<td>3</td>
<td>IC10012L/4</td>
<td>8.1</td>
<td>41.01</td>
<td>LTB</td>
</tr>
<tr>
<td>4</td>
<td>IC10012L/6</td>
<td>6.51</td>
<td>33.48</td>
<td>LTB</td>
</tr>
<tr>
<td>5</td>
<td>IC10015L/2</td>
<td>9.16</td>
<td>38.26</td>
<td>LTB</td>
</tr>
<tr>
<td>6</td>
<td>IC10015L/3</td>
<td>11.23</td>
<td>50.59</td>
<td>LTB</td>
</tr>
<tr>
<td>7</td>
<td>IC10015L/4</td>
<td>9.6</td>
<td>40.26</td>
<td>LTB</td>
</tr>
<tr>
<td>8</td>
<td>IC10015L/6</td>
<td>12.89</td>
<td>51.65</td>
<td>LTB</td>
</tr>
<tr>
<td>9</td>
<td>IC15015L/2</td>
<td>16.29</td>
<td>26.62</td>
<td>LTB</td>
</tr>
<tr>
<td>10</td>
<td>IC15015L/3</td>
<td>18.25</td>
<td>31.21</td>
<td>LTB</td>
</tr>
<tr>
<td>11</td>
<td>IC15015L/4</td>
<td>18.12</td>
<td>31.96</td>
<td>LTB</td>
</tr>
<tr>
<td>12</td>
<td>IC15015L/6</td>
<td>16.76</td>
<td>28.58</td>
<td>LTB</td>
</tr>
</tbody>
</table>

From Table 3.3, the maximum load of IC10015L/2 and IC10015L/6 was 9.16 kN and 12.89 kN respectively. The increase of strength 40.72% was observed when the connection spacing decreased from L/2 to L/6. The maximum load of IC10015L/3, IC10015L/4 and IC10015L/6 was 1.23, 1.05 and 1.41 times higher than the one of IC10015L/2, respectively. Similarly, the maximum load of IC15015L/2, 16.29 kN was less than that of IC15015L/6, 16.79 kN around 2.89%. The maximum load of IC15015L/3, IC15015L/4 and IC15015L/6 was 1.12, 1.11 and 1.03 times higher than the one of IC15015L/2, respectively. However, for the section IC10012, the maximum load of connector spacing L/2, 8.89 kN is larger than that of the connector spacing L/6, 6.51 kN. It is also observed that for the section IC15015, the maximum load in case of L/3 or L/4 is more than that of L/6. The reason why the maximum load in case of small connection spacing is smaller than that in case of large connection spacing might be due to the eccentricity of applied load on the beam during the test, the material, and geometric imperfection, and the rate of applying load.
(a). Load-Vertical Deflection curve of IC10012

(b). Load-Vertical Deflection curve of IC10015
Figure 3.12 Load-Vertical Deflection curves on types of sections and connector spacings

Figure 3.12 shows load-vertical deflection curves for all 12 specimens. The values of vertical deflection plotted in the curves are the largest values among two LVDTs measured on the left and right bottom flange. The tendency of almost the same slope of the curves for each section confirmed that before reaching maximum load, the behavior of the beam with different spacings is linear, and then becomes nonlinear at the level of maximum load. This observation can be seen in the results of load-strain curves for each section in Figure 3.14, Figure 3.15, and Figure 3.16.
Figure 3.13 Load-Lateral Deflection curves on various types of sections

The failure mode of all twelve beams is the lateral-torsional buckling on one C section because the torsional stiffness of C back-to-back section was quite low. This buckling mode indicates the increase of lateral deflection after reaching the maximum load, or in the softening part of load-lateral deflection curves in Figure 3.13.
Figure 3.14 Load-Strain curves of IC10012 with different spacing

Figure 3.15 Load-Strain curves of IC10015 with different spacing
As shown in Figure 3.14 - Figure 3.16, the load-strain curves of all beam specimens are plotted. It can be seen that due to different mode of buckling on the left and right side of the top flange, the values of strain on both sides (strain_5 and strain_6) are quite different.

Failure modes of twelve beams were shown in Figure 3.17 - Figure 3.28. It can be observed that one section C-1 of all beams failed by lateral torsional buckling while another section C-2 of all beams failed by distortional buckling. It is noted that distortional buckling of all beams occurred at the section located between two loading points. In addition, there is no failure of bolts for all twelve specimens.
Figure 3.17 Failure mode of IC10012-L/2
Figure 3.18 Failure mode of IC10012-L/3
Figure 3.19 Failure mode of IC10012-L/4
Figure 3.20 Failure mode of IC10012-L/6
Figure 3.21 Failure mode of IC10015-L/2
Figure 3.22 Failure mode of IC10015-L/3
Figure 3.23 Failure mode of IC10015-L/4
Figure 3.24 Failure mode of IC10015-L/6
Figure 3.25 Failure mode of IC15015-L/2
Figure 3.26 Failure mode of IC15015-L/3
Figure 3.27 Failure mode of IC15015-L/4
Figure 3.28 Failure mode of IC15015-L/6
From Figure 3.29, it can be seen that, when the thickness was increased, the maximum load increased. From the increase of thickness 25% (from 1.2 mm to 1.5 mm), the increase of maximum load is 3%, 40%, 19%, and 98%, in case of connection spacing L/2, L/3, L/4, and L/6, respectively.
Chapter 4

Numerical study

In the present study, ABAQUS program version 6.14-1 was used to simulate CFS steel C back-to-back beams in order to compare with the experimental test of the beams (Figure 4.1). The finite element program ABAQUS is a computational tool for modelling structures with material and geometric nonlinear behavior [Schafer and Moen 2010]. 12 beams specimens in the test were modelled. The loads were applied at two points of loading bearing plates whose width was 10 cm and those loading bearing plates were installed symmetrically with respect to the mid-span.

Figure 4.1 Model of CFS C back-to-back beam IC10012L/4
4.1 Modeling in Finite Element Analysis

4.1.1 Modelling details and element type

The element types that used in modelling C back-to-back beam were shell elements and solid elements. For Cold-formed steel C back-to-back beam, the thickness dimension which ranges from 1.2 mm to 1.5 mm, was very small compared to other two dimensions so that shell elements (S4R) were used. Solid elements (C3D8R) were used for loading bearing plates and support bearing plates because three dimensions are almost in the same order.

4.1.2 Contact, loading and boundary conditions

There were two types of contact conditions which were used in the present analysis: surface-to-surface contact and tie contact. Surface-to-surface contact was the interaction between bottom surface of loading bearing plate and top flange surfaces of CFS C back-to-back beam (Figure 4.2 (a)). Surface-to-surface contact was also applied between both webs of C-section. Moreover, two assumptions were introduced in this simulation for the contact properties. The first assumption is that frictionless was selected for tangential behavior. The second assumption is that hard contact was chosen for normal behavior. For tie contact, it was applied between the bottom surfaces of both C sections and the top surface of the support (Figure 4.2(b)). For the condition of tie contact, both surfaces were touched together. Therefore, when the beam failed, there is no gap between both surfaces. On the contrary, for the surface-to-surface contact, when the beam failed, some part of the surface could be separated from other parts.
In the test, the beam supports and the loading from load-transferring beam were applied on rigid plates attached to the beam in order to distribute the concentrated forces on the beam. In the numerical analysis, the controlled displacement was imposed vertically with Y direction on the bearing plate. The movement of the loading bearing plate in X and Z direction was constrained, so that the loading bearing plates can move vertically in Y direction shown in Figure 4.3. For roller support, the translations in Y direction of one line located at the middle bottom surface of the support were constrained while the translation in X direction of one node at the edge of the line was constrained (Figure 4.4 (a)). For pinned support, the
translations in Y and Z direction of one line at the middle bottom surface of the support were constrained while the translation of one node at the edge of the line was constrained in X direction (Figure 4.4 (b)). In addition, at both supports, the lateral movement of one line at the top lip was constrained in X direction to prevent torsional deformation at the support.

**Loading modelling**

![Support simulation diagram](image)

**Figure 4.3 Constraints of loading bearing plate**

**Figure 4.4 Support simulation**
4.1.3 Material modelling

Material properties for the thickness 1.2 mm and 1.5 mm were obtained from the coupon test. Properties of CFS is Young’s modulus \( E = 208 \) GPa and Poisson’s ratio \( \nu = 0.3 \). Stress-Strain relations are shown Figure 4.5. A loading and support bearing plate were modelled as a rigid elastic material which has 1000 times of steel Young’s modulus, equal to 208,000 GPa.

![Stress-Strain curve for thickness 1.2 mm](image)

![Stress-Strain curve for thickness 1.5 mm](image)

Figure 4.5 Stress and Strain relations for thickness 1.2 mm and 1.5 mm
4.1.4 Finite element mesh and connection

As shown in Figure 4.6, the size of finite element mesh of CFS C back-to-back beam was 7.5 x 7.5 mm. At the corner of C section, three segments were divided and the lip of C section was divided into two segments. For loading bearing plates and support bearing plates, the mesh sizes were 10 x 10 x 10 mm and 20 x 20 x 20 mm, respectively.

Through the observation in the test, there was no failure of the bolts. Therefore, simplified modelling called as fasteners function in Abaqus program was used for bolts to connect two C-sections, and all rotational and translational degrees of freedom at the bolt location were constrained. The radius of bolt, 6 mm was input to the program.
4.1.5 Analysis

In the analysis, the nonlinear geometric parameter (NLGEOM = ON) was set to consider nonlinear effects of large displacements throughout the calculation steps. It is noted that effects of initial imperfections are not included in the present analysis.

4.2 Comparison between numerical and test results

The comparison of maximum load, failure mode of CFS C back-to-back beam from experimental and numerical results were shown in Table 4.1. For the ultimate load comparison, the difference between experimental and numerical results is less than 21 percent for the section C10012, 27 percent for C10015, and 36 percent for C15015 and the failure modes from both results are the same.

Table 4.1 Comparison between test and FEM result

<table>
<thead>
<tr>
<th>No</th>
<th>Specimen</th>
<th>Experimental results</th>
<th>FEM result</th>
<th>( \frac{F_{FEM}}{F_{test}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( F_{test} ) (kN)</td>
<td>Failure mode</td>
<td>( F_{FEM} ) (kN)</td>
</tr>
<tr>
<td>1</td>
<td>IC10012L/2</td>
<td>8.89</td>
<td>LTB</td>
<td>7.32</td>
</tr>
<tr>
<td>2</td>
<td>IC10012L/3</td>
<td>8</td>
<td>LTB</td>
<td>7.88</td>
</tr>
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<td>LTB</td>
<td>7.72</td>
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<td>LTB</td>
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*Note: C-1: one C section side; C-2: the other C section side; LTB: lateral torsional buckling; DB: distorsional buckling

The load-vertical deflection graphs between FEM and test for IC10012L/2, IC10012L/3, IC10012L/4, IC10012L/6, IC10015L/2, IC10015L/3, IC10015L/4, IC10015L/6, IC15015L/2, IC15015L/3, IC15015L/4 and IC15015L/6 were illustrated
and compared in Figure 4.7, Figure 4.8, and Figure 4.9. It can be seen that the slopes of load-displacement curves from the test and FEM are almost the same. This indicates the validity of the present finite element analysis in comparison with the experimental results.

**IC10012-L/2**

![IC10012-L/2 graph](image)

(a): Load-Vertical Deflection curve of IC10012L/2

**IC10012-L/3**

![IC10012-L/3 graph](image)

(b): Load-Vertical Deflection curve of IC10012L/3
Figure 4.7 Load-Vertical Deflection curve of IC10012
(a): Load-Vertical Deflection curve of IC10015L/2

(b): Load-Vertical Deflection curve of IC10015L/3
Figure 4.8 Load-Vertical Deflection curve of IC10015

Figure 4.8 Load-Vertical Deflection curve of IC10015

(c): Load-Vertical Deflection curve of IC10015L/4

(d): Load-Vertical Deflection curve of IC10015L/6
(a). Load-Vertical Deflection Graph of IC15015L/2

(b). Load-Vertical Deflection Graph of IC15015L/3
Figure 4.9 Load-Vertical Deflection curve of IC15015

(c). Load-Vertical Deflection Graph of IC15015-L/4

(d). Load-Vertical Deflection Graph of IC15015-L/6
It was observed that the maximum load of IC10012L/6, IC10015L/6, and IC15015L/6 was larger than that of IC10012L/2, IC10015L/2, and IC15015L/6, respectively shown in Figure 4.10.

Figure 4.10 Load-Vertical Deflection Graph with all sections and spacings
(a). Failure mode near the loading plate

(b). Distortional Buckling of IC10015L/4
Figure 4.11 Failure mode of IC10015L/4 between Test and FEM

(c). Lateral Torsional Buckling of IC10015L/4
The failure modes of IC10015L/4 and IC10015L/2 in Abaqus program were compared with the test in Figure 4.11 and Figure 4.12. A good agreement between two results can be seen, i.e. distortional buckling occurred at two sections for IC10015L/4 in Figure 4.11, and distortional buckling occurred at one section for IC10015L/2 in Figure 4.12.
Chapter 5

Conclusions and Recommendations

Twelve specimens of cold-formed steel C back-to-back beams were tested under four-point loading, to investigate their flexural behavior. There were three different sections such as: C10012 (depth = 102 mm, thickness = 1.2 mm), C10015 (depth = 102 mm, thickness = 1.5 mm), and C15015 (depth = 152 mm, thickness = 1.5 mm). The connector spacing of each section were L/2 = 2,000 mm, L/3 = 1,333 mm, L/4 = 1000 mm, and L/6 = 667 mm which L was the overall length of the beam.

The following conclusions can be drawn:

1. For failure modes, all specimens failed by lateral torsional buckling (LTB) and distortional buckling (DB) and no failure of bolts was observed.

2. For the effect of thickness, when the increase of thickness is from 1.2 mm to 1.5 mm, the increase of maximum load is 3%, 40%, 19%, and 98%, in case of connection spacing L/2, L/3, L/4, and L/6, respectively.

3. Considering the effect of connection spacing, when the connection spacing decreased from L/2 to L/6, the maximum load of section IC10015 and IC15015 increases 40.72%, and 2.89%, respectively. However, for section IC10012, the maximum load in case of L/2 is larger than that of L/6. The reason for the larger maximum load in case of large connection spacing might be due to eccentricity of applied load on the beam during the test, and geometric imperfection of the beam specimen.

4. Finite element analysis considering geometric and material nonlinearity was performed by using ABAQUS program. The failure modes of CFS C back-to-back beams from numerical simulation comparatively agreed with the modes observed in the experiment. For the comparison of ultimate loads from experiment, and finite element analysis, the maximum difference is 21 percent for C10012, 27 percent for C10015, and 36 percent for C15015.

Further research is recommended to study about improvement of modeling of C back-to-back beams such as bolt connection and support conditions for restraint of
lateral displacement, and to investigate on the effects of span length, height-to-thickness ratio and width-to-thickness ratio of the section.
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